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FEUP FACULDADE DE ENGENHARIA
UNIVERSIDADE DO PORTO

SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS ACCORDING TO THE MODERN CODES

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A thesis submitted to the Faculty of Engineering of the University of Porto, in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Earthquake and Structural Engineering.

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DOCTORAL PROGRAM IN CIVIL ENGINEERING, 2024

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To my family, for their infinite patience

I have explained the phenomena
of the heavens and of our sea by the force of gravity,
but I have not yet assigned a cause to gravity.
Isaac Newton

ACKNOWLEDGEMENTS

Completing this PhD has been a transformative journey. I am grateful to acknowledge the support and guidance of those who have made this achievement possible.

First and foremost, I extend my most profound appreciation to my supervisor, Professor Humberto Varum, whose scholarly insight and steadfast mentorship have been invaluable. Professor Varum's expertise and meticulous attention to detail have profoundly shaped this research. Professor Varum's encouragement and patience during challenging times kept me motivated and focused.

I am equally indebted to my co-supervisor, Dr. José Melo, whose critiques and suggestions have greatly enriched my work. Dr. José Melo provided me with the tools to refine my thinking and broaden my perspective, which has been instrumental in shaping the intellectual rigour of my thesis.

My academic journey would not have been possible without the support of the faculty and staff in the Department of Civil Engineering at the Faculty of Engineering of the University of Porto. I am thankful for their generous assistance and a stimulating academic environment to learn and grow.

I want to mention a special thank you to my fellow PhD candidates and friends who have accompanied me on this journey. Their companionship, peer support, and shared insights have made this challenging process more enjoyable and enlightening.

My deepest gratitude goes to my family, who have provided unwavering support and encouragement throughout my life, especially during this PhD. To my wife Joana—your patience, understanding, and love have sustained me through the most demanding periods of my studies.

This thesis is a testament to my perseverance and the collective effort and support of everyone mentioned above. Thank you all for your contributions, which have been vital to my success.

ABSTRACT

The research aims to contribute to the seismic design by analysing reinforced concrete (RC) structures under Eurocode 8, mainly focusing on the transition between its first and second generations. The document spans several critical areas of earthquake engineering and structural dynamics, integrating substantial theoretical, comparative, and analytical research on seismic codes, detailing provisions and structural behaviour under seismic actions.

The introduction outlines the thesis's scope and objectives, explaining the necessity for advancing seismic-resistant design methodologies amidst evolving building codes. It establishes the context for seismic structural engineering's role in minimizing earthquake infrastructure damage. It highlights a shift towards ductility-based design approaches, detailing provisions and adopting newer, more sophisticated ductility classes in the updated Eurocode 8.

The thesis conducts a thorough review of international seismic design standards. It compares them, mainly focusing on changes and enhancements in the seismic behaviour factor, a key element in design calculations affecting structures' seismic load capacity. This section delves into various national standards, including those from the U.S., New Zealand, Chile, and Europe, and their evolution to integrate more robust and realistic structural performance measures during seismic events.

The core analytical chapters discuss the comparative impact of Eurocode 8's first and second generations on the design and detailing of RC moment-resisting frames and wall structures. It evaluates the implications of updated ductility requirements and new design methodologies that shift from force-based to displacement-based design approaches. Detailed simulations and structural analyses demonstrate how these changes affect building integrity and safety under seismic loads.

The findings from the comparative analyses are intended to highlight the practical implications of the updated Eurocode on structural design and safety. This includes a detailed examination of chord rotation requirements, behaviour factor components, and their influence on interstorey drift and overall seismic resilience. The document uses these findings to argue for optimized design strategies that better balance safety, cost, and performance in seismic zones.

Finally, the document synthesises the research findings and proposes future directions for continued advancements in seismic design standards and practices. It suggests areas where further research is needed to refine the design and detailing of concrete structures to withstand seismic forces more effectively, emphasizing the ongoing need for innovation and adaptation in structural engineering to meet the challenges posed by natural disasters.

KEYWORDS: Eurocode 8, 2nd Generation of Eurocodes, ductility classes, force-based design, displacement-based design, behaviour factor

RESUMO

A investigação visa contribuir para o desenho sísmico através da análise de estruturas de betão armado (RC) sob o Eurocódigo 8, com um foco particular na transição entre as suas primeira e segunda gerações. O documento abrange várias áreas críticas da engenharia sísmica e dinâmica estrutural, integrando uma pesquisa substancial teórica, comparativa e analítica sobre códigos sísmicos, disposições de detalhe e comportamento estrutural sob ações sísmicas.

A introdução delineia o âmbito e os objetivos da tese, explicando a necessidade de avançar metodologias de design resistente a sismos em meio à evolução dos códigos de construção. Estabelece o contexto para o papel da engenharia estrutural sísmica na minimização de danos à infraestrutura causados por sismos. Destaca uma mudança em direção a abordagens de design baseadas em ductilidade, detalhando disposições e adotando classes de ductilidade mais recentes e sofisticadas no Eurocódigo 8 de segunda geração.

A tese realiza uma revisão completa dos códigos internacionais de projeto sísmico. Compara-os, concentrando-se especialmente nas mudanças e melhorias no fator de comportamento sísmico, um elemento chave nos cálculos de design que afeta a capacidade de carga sísmica das estruturas. Esta seção explora diversos padrões nacionais, incluindo os dos EUA, Nova Zelândia, Chile e Europa, e a sua evolução para integrar medidas de desempenho estrutural mais robustas e realistas durante eventos sísmicos.

Os capítulos analíticos centrais discutem o impacto comparativo das primeiras e segundas gerações do Eurocódigo 8 no desenho e detalhamento de estruturas de enquadramento de momento resistente e paredes em RC. Avalia as implicações dos requisitos de ductilidade atualizados e novas metodologias de design que passam de abordagens baseadas em força para abordagens baseadas em deslocamentos. Simulações detalhadas e análises estruturais demonstram como essas mudanças afetam a integridade e segurança dos edifícios sob ações sísmicas.

Os resultados das análises comparativas pretendem destacar as implicações práticas do Eurocódigo atualizado no design estrutural e na segurança. Isso inclui um exame detalhado dos requisitos de rotação das cordas, componentes do fator de comportamento e a sua influência resiliência sísmica global. O documento usa esses resultados para argumentar a favor de estratégias de design otimizadas que equilibram melhor segurança, custo e desempenho em zonas sísmicas.

Por fim, o documento sintetiza os resultados da investigação e propõe direções futuras para avanços contínuos nos padrões e práticas de design sísmico. Sugere áreas onde mais pesquisa é necessária para refinar o design e o detalhamento de estruturas de betão para resistir de forma mais eficaz às forças sísmicas, enfatizando a necessidade contínua de inovação e adaptação na engenharia estrutural para enfrentar os desafios impostos por desastres naturais.

PALAVRAS-CHAVE: Eurocódigo 8, 2ª Geração de Eurocódigos, classes de ductilidade, design baseado em força, design baseado em deslocamento, fator de comportamento.

Table of contents

1 INTRODUCTION 1-1

1.1 MOTIVATION AND GOALS 1-5

1.2 DOCUMENT ORGANIZATION 1-10

1.3 REFERENCES 1-14

2 INTEGRATED LITERATURE REVIEW: INTERNATIONAL SEISMIC DESIGN CODES AND BEHAVIOUR FACTOR RESEARCH 2-1

2.1 REVIEW OF INTERNATIONAL SEISMIC DESIGN CODES LITERATURE 2-1

2.1.1 U.S. seismic and concrete codes 2-3

2.1.2 New Zealand seismic code 2-21

2.1.3 Chilean seismic and concrete standard 2-45

2.1.4 Mexico City Technical Standards 2-52

2.1.5 European seismic code 2-76

2.1.6 Comparison between international seismic codes 2-99

2.1.7 Research and development of the behaviour factor – Literature review 2-107

2.1.8 References 2-115

2.1.9 Symbology 2-120

3 COMPARATIVE ANALYSIS OF THE IMPACT OF DESIGN AND DETAILING PROVISIONS FOR RC MOMENT RESISTING FRAMES UNDER THE FIRST- AND SECOND-GENERATION OF EUROCODE 8 ... 3-1

3.1 ABSTRACT 3-1

3.2 INTRODUCTION 3-1

3.3 OBJECT OF RESEARCH AND ANALYSIS 3-3

3.3.1 Buildings geometry and structural system 3-4

3.3.2 Materials 3-4

3.4 DESIGN CRITERIA OPTIONS 3-5

3.4.1 Design and modelling options 3-5

3.4.2 Seismic action 3-7

3.4.3 Structural safety verifications to limit states 3-9

3.4.4 Vertical loads 3-12

3.4.5 Load combinations 3-12

3.4.6 Beam design and detailing for local ductility 3-15

3.4.7 Column design and detailing 3-16

3.4.8 Chord rotation 3-17

3.5 RESULTS AND DISCUSSION 3-18
3.5.1 Seismic response demands 3-18
3.5.2 Beam design and detailing for local ductility 3-24
3.5.3 Column design and detailing for local ductility 3-31
3.5.4 Structures inventory 3-37
3.6 SUMMARY AND CONCLUSIONS 3-40
3.7 REFERENCES 3-42
3.8 SYMBOLOGY 3-44
3.8.1 Upper-case latin symbols 3-44
3.8.2 Lower case latin symbols 3-44
3.8.3 Greek symbols 3-45
4 RC WALL BUILDING STRUCTURES: COMPARISON OF FIRST AND SECOND GENERATION EUROCODE 8 SEISMIC DESIGN AND DETAILING STRATEGIES 4-1
4.1 ABSTRACT 4-1
4.2 INTRODUCTION 4-1
4.3 OBJECT OF RESEARCH AND ANALYSIS 4-3
4.3.1 Buildings geometry and structural system 4-4
4.3.2 Materials 4-4
4.4 DESIGN CRITERIA OPTIONS 4-5
4.4.1 Design and modelling options 4-5
4.4.2 Seismic action 4-7
4.4.3 Structural safety verifications to limit states 4-9
4.4.4 Vertical loads 4-11
4.4.5 Design action effects 4-11
4.4.6 Loads combinations 4-15
4.4.7 Wall design and detailing 4-15
4.4.8 Chord rotation 4-20
4.5 RESULTS AND DISCUSSION 4-21
4.5.1 Seismic response demands 4-21
4.5.2 Wall design and detailing for local ductility 4-26
4.5.3 Bending resistance 4-32
4.5.4 Shear resistance in walls – Capacity design 4-36
4.5.5 Detailing of walls transverse reinforcement 4-37
4.5.6 Chord rotation 4-38

4.5.7 Structures inventory 4-39
4.6 SUMMARY AND CONCLUSIONS 4-41
4.7 REFERENCES 4-44
4.8 SYMBOLOGY 4-46
4.8.1 Upper case latin symbols 4-46
4.8.2 Lower case latin symbols 4-46
4.8.3 Greek symbols 4-47
5 EXPLORING BEHAVIOUR FACTORS AND CHORD ROTATION IN RC MRF STRUCTURES: NONLINEAR ANALYSIS BASED ON SECOND-GENERATION OF EUROCODE 8 5-1
5.1 ABSTRACT 5-1
5.2 INTRODUCTION 5-1
5.3 OBJECT OF RESEARCH AND ANALYSIS 5-3
5.4 MODELLING AND ANALYSIS 5-4
5.4.1 Pushover procedure 5-4
5.4.2 Material constitutive models 5-6
5.5 BEHAVIOUR FACTOR ANALYSIS 5-6
5.5.1 Estimation of the behaviour factor components 5-8
5.6 CHORD ROTATION ANALYSIS 5-13
5.6.1 Estimation of the chord rotation parameters 5-14
5.6.2 Relation between chord rotation and effective confinement at columns base 5-18
5.7 RESULTS AND DISCUSSION 5-21
5.7.1 Behaviour factor analysis 5-22
5.7.2 Estimation of the bilinear parameters 5-24
5.7.3 Estimation of the chord rotation parameters of columns and beams 5-26
5.7.4 Estimation of the relationship between effective concrete core confinement and chord rotation 5-35
5.8 SUMMARY AND CONCLUSION 5-36
5.9 REFERENCES 5-38
5.10 SYMBOLOGY 5-41
5.10.1 Upper case latin symbols 5-41
5.10.2 Lower case latin symbols 5-42
5.10.3 Greek letter 5-43

6 SEISMIC DESIGN CHALLENGES: INVESTIGATING BENDING MOMENTS, SHEAR ACTIONS, AND DESIGN PARAMETERS IN CONCRETE WALLS UNDER THE SECOND-GENERATION OF EUROCODE 8 6-1

6.1 ABSTRACT ... 6-1

6.2 INTRODUCTION ... 6-1

6.3 OBJECT OF RESEARCH AND ANALYSIS ... 6-3

6.4 MODELLING FOR NONLINEAR ANALYSIS ... 6-4

6.4.1 Nonlinear analysis method ... 6-4

6.4.2 Pushover procedure ... 6-5

6.4.3 Material constitutive models ... 6-7

6.5 BEHAVIOUR FACTOR ANALYSIS ... 6-8

6.5.1 Estimation of the behaviour factor components ... 6-8

6.6 COMPARISON BETWEEN THE CRITICAL REGION HEIGHT, TENSION SHIFT WITH PLASTIC HINGE LENGTH OF WALLS 6-10

6.7 PLASTIC HINGE LENGTH ... 6-10

6.7.1 Tension shift for wall bending design moment ... 6-15

6.8 CRITICAL HEIGHT DEFINITION ... 6-21

6.9 SHEAR MAGNIFICATION FACTOR ... 6-22

6.10 NUMERICAL RESULTS AND DISCUSSION ... 6-24

6.11 BEHAVIOUR FACTOR ANALYSIS ... 6-25

6.11.1 Comparison between the plastic hinge length obtained by nonlinear analysis with Eurocode 8 ... 6-29

6.11.2 Computational evaluation between the plastic hinge ($l_{pl,NLA}$), the critical and tension shift (a_l) and height (h_{cr}) ... 6-32

6.11.3 Computational evaluation of between the tension shift (a_l) and tension shift obtained by nonlinear analysis (d_{NLA}) ... 6-33

6.12 SHEAR MAGNIFICATION FACTOR COMPUTATIONAL ANALYSIS ... 6-34

6.13 SUMMARY AND CONCLUSIONS ... 6-36

6.14 REFERENCES ... 6-38

6.15 SYMBOLOGY ... 6-41

6.15.1 Upper case Latin symbols ... 6-41

6.15.2 Lower case Latin symbols ... 6-42

6.15.3 Greek letters ... 6-43

7 CLOSURE AND FUTURE RESEARCH ... 7-1

7.1 General conclusions ... 7-2

7.2 Future research ... 7-5

APPENDIX A

A.1 Moment Resisting Frames – 5-storey buildings……A-1

A.1.1 Beams……A-1

A.1.2 Columns……A-6

A.2 Moment Resisting Frames – 10-storey buildings……;……A-10

A.2.1 Beams……A-11

A.2.2 Columns……A-19

A.3 Wall structures – 5-storey buildings……A-23

A.3.1 Beams……A-23

A.3.2 Columns……A-33

A.3.3 Walls……A-38

A.4 Wall structures – 5-storey buildings……A-39

A.4.1 Beams……A-40

A.4.2 Columns……A-60

A.4.3 Walls……A-70

LIST OF FIGURES

Figure 1-1: Overview of the design of the concrete structure according to the current and second generation of Eurocode 8 1-4

Figure 1-2: Building parametric variables combination to set up the analysis scenarios 1-7

Figure 1-3: Computation flowchart for Structural analysis stage 1-9

Figure 1-4: Document organization 1-10

Figure 2-1: Mode shapes and inertial forces associated with free vibration 2-12

Figure 2-2: Storey drift (Adapted from Maranhão et al[15]) 2-13

Figure 2-3: (P-Δ) effects on a moment-resistant frame 2-14

Figure 2-4: Determination of shear demand for walls (Adapted from ACI 318-19[12]) 2-18

Figure 2-5: Configurations of boundary transverse reinforcement and web crossties (Adapted from ACI 318-19 [12]) 2-18

Figure 2-6: Summary of boundary element requirements for special walls (adapted from ACI 318-19 [12]) 2-19

Figure 2-7: Vertical irregularities (Adapted from NZS 1170.5:2004[19]) 2-24

Figure 2-8: Plant irregularities (adapted from NZS 1170.5:2004[19]) 2-25

Figure 2-9: Horizontal design earthquake action coefficients for soil type A and B (adapted from NZS 1170.5:2004[19]) 2-27

Figure 2-10: Horizontal design earthquake action coefficients for soil type C (adapted from NZS 1170.5:2004[19]) 2-28

Figure 2-11: Horizontal design earthquake action coefficients for soil type D (adapted from NZS 1170.5:2004[19]) 2-28

Figure 2-12: Horizontal design earthquake action coefficients for soil type E (adapted from NZS 1170.5:2004[19]) 2-28

Figure 2-13: P-Δ effects (adapted from NZS 1170.5:2004 [19]) 2-31

Figure 2-14: mechanisms of post-elastic deformation of moment resisting frames during severe seismic loading(a) Beam sway mechanism (b) Column sway mechanism (c) Mixed beam-column sway mechanism (adapted from NZS 3101: Part 2:2006 [21]) 2-33

Figure 2-15: Minimum dimensions of boundary elements of wall sections in plastic hinge regions (Adapted from NZS 3101: Part 2:2006[21]) 2-43

Figure 2-16: Seismic zonation maps for northern, central, and southern Chile (adapted from NCh433.Of1996[29]) 2-46

Figure 2-17: Seismic Characteristic shapes of the design spectra, both for pseudo-acceleration and displacement. Note the variation in the ordinates for (T>T_b) depending on the parameter (k) (Adapted from NTC-DS:2017 [33]) 2-59

Figure 2-18: Value of (Q') for Q=4 and various (k) values for a site with T_s=2 s. 2-62

Figure 2-19: Determination of the design shear force (*V*) for capacity design..... 2-69

Figure 2-20: Detailing of a beam (Adapted from NTC-DCEC [34]) 2-71

Figure 2-21: Detailing of column (Adapted from NTC-DCEC:2017[34])..... 2-72

Figure 2-22: Envelope of the design bending moment (Adapted from NTC-DCEC[34])..... 2-74

Figure 2-23: Detailing for local ductility of walls within the critical region for structures with medium ductility (Adapted from NTC-DCEC:2017[34])..... 2-75

Figure 2-24: Detailing for local ductility of walls within the critical region for structures with high ductility (Adapted from NTC-DCEC:2017[34])..... 2-75

Figure 2-25: Structural systems preconized in EN 1998-1:2004 [37] 2-77

Figure 2-26: Definition of compact shapes 2-79

Figure 2-27: Definition of mass-stiffness eccentricity (a) X direction (b) Y direction 2-81

Figure 2-28: Criteria for regularity of buildings with setbacks EN 1998-1 (2004)..... 2-82

Figure 2-29: Elastic response spectra (a) Type 1 for ground types A to E (5% damping) (b) Type for ground types A to E (5% damping) (Adapted from EN 1998-1:2004[37])..... 2-84

Figure 2-30: Flowchart to define the seismic action..... 2-85

Figure 2-31: Methods of analysis..... 2-86

Figure 2-32: (*P-Δ*) effect (a) Column (b) Moment- resisting frame 2-87

Figure 2-33: Capacity design against column hinging (a) Frame nodal flexure vectorial system (b) Beam-column nodal vectorial system 2-89

Figure 2-34: Capacity design values of shear forces on beams (Adapted from EN 1998-1:2004[37]) .. 2-90

Figure 2-35: Capacity design values of shear forces on columns (Adapted from EN 1998-1:2004[37]) 2-91

Figure 2-36: Envelope of bending moment (Adapted from EN 1998-1:2004 [37]) 2-94

Figure 2-37: Envelope of bending moment of a wall (a) Structure (b) Wall (c) Bending moment diagram from analysis 2-94

Figure 2-38: Envelope of bending moment of a wall (a) Structure (b) Definition of tension shift (a_l) (c) Envelope of bending moment diagram..... 2-95

Figure 2-39: Design envelope of the shear forces in the walls of a dual system (Adapted from EN 1998-1:2004[37])..... 2-96

Figure 2-40: Confined boundary element of free-edge wall end 2-97

Figure 2-41: Value Base-shear versus roof displacement relationship. Overstrength (R_Ω), redundancy (R_ρ), and ductility (R_μ) reduction factors. 2-107

Figure 3-1: Frame building system configuration: (a) 5 storeys, (b) 10 storeys and (c) plan view..... 3-4

Figure 3-2: Frame building 3-6

Figure 3-3: Vectorial system of the resistant moments for beams and columns at a node..... 3-11

Figure 3-4: Vectorial representation of the chord rotation 3-17

Figure 3-5: Example of modal shapes: (a) First vibration mode T_1, (b) Second vibration mode T_2 and (c) Third vibration mode T_3 3-18

Figure 3-6: Scheme for base shear (V_b) and roof displacement ($d_{e,roof}$) 3-19

Figure 3-7: Design ground acceleration at fundamental period $S_d(T_1)$, Base shear (V_b) and fundamental period (T_1) 3-20

Figure 3-8: Normalised Base shear (V_b/ $V_{b,max}$) and roof displacement ($d_{e,roof}$/ $d_{e,roof,max}$) 3-20

Figure 3-9: Interstorey drift ratio for *X* and *Y* directions: (a) 5-storey MRF buildings and (b) 10-storey MRF buildings 3-21

Figure 3-10: Torsional effects due to accidental eccentricities, e_{ai} (a) Illustration of the angle of torsion (b) 5-storey building angle of twist (c) 10-storey building angle of twist 3-22

Figure 3-11: Interstorey sensitivity coefficient i 3-23

Figure 3-12: Comparison of resistant bending moment (M_{Rd}) and acting bending moment (M_{Ed}) for (a) Hogging Bending Moment and (b) Sagging Bending Moment 3-24

Figure 3-13: Normalised reinforcement ratio of beams: (a) Top normalised reinforcement ratio and (b) Top minimum normalised reinforcement ratio 3-25

Figure 3-14: Normalised reinforcement ratio of (a) Bottom normalised reinforcement ratio and (b) Bottom minimum normalised reinforcement ratio 3-25

Figure 3-15: Illustration of the vectorial system of capacity design in shear for beams ($V_{i,d}$) 3-26

Figure 3-16: Normalised capacity design in shear ($V_{i,d}$) to shear force from the analysis for the seismic design situation ($V_{Ed,SEISM}$) 3-26

Figure 3-17: Transverse reinforcement in critical regions of beams 3-27

Figure 3-18: Hoop spacing (a) DCM (b) DC2, and (c) DC3 3-27

Figure 3-19: Illustration of bond stresses in beam-column joint 3-28

Figure 3-20: Beam rebar diameter d_{bL}: (a) Beams framing exterior columns and (b) Beams framing interior columns 3-28

Figure 3-21: Beam rebar diameter d_{bL}: (a) Beams framing exterior columns and (b) Beams framing interior columns 3-29

Figure 3-22: Verification of the chord rotation capacity of beams 3-30

Figure 3-23: Normalised axial force (ν_d) 3-31

Figure 3-24: Capacity design in flexure, strong column/weak beam: (a) External columns and (b) Internal columns 3-31

Figure 3-25: Normalised reinforcement ratio for columns: (a) Maximum reinforcement ratio and (b) Minimum reinforcement ratio 3-32

Figure 3-26: Illustration of the vectorial system of capacity design in shear ($V_{i,d}$) for columns 3-33

Figure 3-27: Normalised shear capacity design of columns (a) Exterior columns (b) Interior columns 3-33

Figure 3-28: Illustration of the transverse reinforcement in critical regions of columns 3-34

Figure 3-29: Hoop spacing (a) DCM, (b) DC2, and (c) DC3 3-34

Figure 3-30: Column confinement: (a) Confinement capacity versus demand and (b) Mechanical reinforcement ratio 3-36

Figure 3-31: Verification of the chord rotation capacity of column 3-36

Figure 3-32: Capacity design in flexure: (a) Beams and columns(b) Virtual work 3-39

Figure 4-1: Wall system configuration: (a) 5 storeys, (b) 10 storeys and (c) plan view 4-4

Figure 4-2: Schematic representations of the FEM model for 5 and 10-storey buildings 4-7

Figure 4-3: Design envelope for bending moments for wall systems EN 1998-1:2004[5] 4-12

Figure 4-4: Design envelope for bending moments for wall systems prEN 1998-1-2:2023[3] 4-12

Figure 4-5: Calculation model of the resistant bending moment $M_{Rdw,base}$ (a) Wall vectorial system (b) Stress block model 4-13

Figure 4-6: Wall components 4-16

Figure 4-7: Wall boundary elements 4-17

Figure 4-8: Wall boundary elements 4-19

Figure 4-9: Vectorial representation of the chord rotation 4-20

Figure 4-10: Scheme for base shear (V_b) and roof displacement ($d_{e,roof}$) 4-21

Figure 4-11: Design ground acceleration at fundamental period $S_d(T_1)$, Base shear (V_b) and fundamental period (T_1) 4-23

Figure 4-12: Normalised base shear ($V_b/V_{b,max}$) and roof displacement ($d_{s,roof}/d_{s,roof,max}$) 4-23

Figure 4-13: Interstorey drift ratio for X and Y directions: (a) 5-storey wall buildings and (b) 10-storey wall system buildings 4-24

Figure 4-14: Torsional effects due to accidental eccentricities, e_{ai} (a) Illustration of the angle of torsion (φ) (b) 5-storey building angle of twist (c) 10-storey building angle of twist (φ) 4-25

Figure 4-15: Interstorey sensitivity coefficient θ 4-26

Figure 4-16: Normalized axial force ν_d 4-27

Figure 4-17: Scheme of the different areas to analyse below al according to EN 1998-1:2004[5](a) bending moment diagram from the analysis area $\mathrm{A'MEdw}$ (b) Design bending moment diagram area AMEdw 4-28

Figure 4-18: Scheme of the different areas to analyse below max(h_{cr}, a_l) according to prEN 1998-1-2:2023[4](a) bending moment diagram from the analysis area $\mathrm{A'MEdw}$(b) Design bending moment diagram area AMEdw 4-28

Figure 4-19: Wall bending moment area (a) $\mathrm{A'MEdw}$ *vs* AMEdw for θ=22° (b) $\mathrm{A'MEdw}$ *vs* AMEdw for θ=45° 4-29

Figure 4-20: Scheme of the different areas to analyse above al according to EN 1998-1:2004[5](a) bending moment diagram from the analysis area $\mathrm{A'MEdw}$(b) Design bending moment diagram area AMEdw 4-30

Figure 4-21: Scheme of the different areas to analyse above max(h_{cr},a_l) according to EN 1998-1-2:2023[3](a) bending moment diagram from the analysis area A'MEdw(b) Design bending moment diagram area AMEdw 4-30

Figure 4-22: Wall bending moment area (a) A'MEdw *vs* AMEdw for θ=22° (b) A'MEdw *vs* AMEdw for θ=45° 4-31

Figure 4-23: Wall bending moment below tension shift (a_l) (a) M_{Edw} *vs* M_{Rdw} for θ=22° (b) M_{Edw} *vs* M_{Rdw} for θ=45° 4-32

Figure 4-24: Wall bending moment above tension shift (a_l) (a) M_{Edw} *vs* M_{Rdw} for θ=22° (b) M_{Edw} *vs* M_{Rdw} for θ=45° 4-33

Figure 4-25: Normalized reinforcement ratio of boundary elements: (a) Top normalized reinforcement ratio and (b) Top minimum normalized reinforcement ratio 4-34

Figure 4-26: Hoop spacing against the multiples: (a) DCM (b) DC2, and (c) DC3 4-35

Figure 4-27: Normalized shear capacity design of walls (a)Strut inclination of θ=22° (b) Strut inclination of θ=45° 4-36

Figure 4-28: Horizontal reinforcement (a) Strut inclination of θ=22° (b) Strut inclination of θ=45°. 4-37

Figure 4-29: Verification of the chord rotation capacity (a) Strut inclination of θ=22° (b) Strut inclination of θ=45° 4-38

Figure 4-30: Longitudinal (Long.) and transversal (transv.) reinforcement (a) Strut inclination of θ=22° (b) Strut inclination of θ=45° 4-40

Figure 5-1: Buildings geometry model [4], [7]: (a) 5-storey building; (b) 10-storey building; (c) Plan view 5-3

Figure 5-2: Schematic illustration of 3 integration sections and meshing for 150 fibres 5-4

Figure 5-3: Concrete stress-strain curve: (a) Proposed by Mander et al. [14] (b) Actual curve used in the FE modelling [10] 5-6

Figure 5-4: Steel stress-strain curve: (a) Proposed by Menegotto et al. [15]; (b) Actual curve used in finite element modelling [7] 5-6

Figure 5-5: Capacity curve and idealised elastic-perfectly plastic force-displacement relationship (adapted from Elnashai et al. [18]) 5-7

Figure 5-6: Capacity curve (adapted from Plumier [21]) 5-8

Figure 5-7: Structure damage states: (a) Elastic state (b) Attainment of the first plastic hinge; (c) Plastic mechanism 5-9

Figure 5-8: General definition of bilinear load-deformation relationship: (a) global plastic mechanism is observed; (b) strength drop of 20% relative to the maximum resistance. 5-10

Figure 5-9: Definition of the behaviour factor component q_R 5-10

Figure 5-10: Definition of the behaviour factor component q_S 5-11

Figure 5-11: Definition of the behaviour factor component q_D 5-12

Figure 5-12: Vectorial representation of the chord rotation (adapted from Maranhão et al.[4]) 5-13

Figure 5-13: Definition of the chord rotation at yield $\theta_{y,NLA}$ and ultimate chord rotation $\theta_{u,NLA}$ 5-15

Figure 5-14: Capacity curve reduction to an equivalent SDOF: (a) Capacity curve MDOF; (b) Capacity curve SDOF 5-16

Figure 5-15 Determination of the bilinear idealization of the force-deformation relationship in the case of (a) non-decreasing and (b) decreasing capacity curve (adapted from FprEN 1998-1-1:2024 [3]) 5-16

Figure 5-16: Determination of the target displacement for the equivalent SDOF system: (a) Structure with inelastic response (b) Structure with elastic response (adapted from Franchin [25]) 5-17

Figure 5-17: Geometrical properties and confinement at the base of a column 5-18

Figure 5-18: Columns confinement..... 5-19

Figure 5-19: Capacity curves(a) 5-storey buildings(b) 10-storey buildings..... 5-21

Figure 5-20: Ductility ratio (μ) for the different structures 5-21

Figure 5-21: Behaviour factor and its components: (a) $q_{R,}$ (b) $q_{S,}$ (c) q_D and (d) q 5-22

Figure 5-22: Determination of the bilinear idealization of the capacity curve: (a) 5-storey buildings; (b) 10-storey buildings..... 5-24

Figure 5-23: First hinges appearance (a) DC2.50.5 (b) DC3.50.5 5-25

Figure 5-24: Chord rotation corresponding to the attainment of SD limit state: (a) θ_{SD} vs $\theta_{SD,\ NLA}$ at of columns bottom end; (b) θ_{SD} vs $\theta_{SD,NLA}$ of columns at the top end; (c) θ_{SD} vs $\theta_{SD,\ NLA}$ at beam left end (d) θ_{SD} vs $\theta_{SD,\ NLA}$ at beam right end 5-27

Figure 5-25: Chord rotation at yield: (a) θ_y vs $\theta_{y,NLA}$ at bottom end (b)θy vs $\theta_{y,NLA}$ at top end (c) θ_y vs $\theta_{y,NLA}$ at beam left end (d) θ_y vs $\theta_{y,\ NLA}$ at beam right end 5-29

Figure 5-26: Ultimate chord rotation: (a)θ_u *vs* $\theta_{u,NLA}$ at bottom end(b) θ_u vs $\theta_{u,NLA}$ at top end(c) θ_u *vs* $\theta_{u,NLA}$ at beam left end(d) θ_u *vs* $\theta_{u,NLA}$ at beam right end 5-31

Figure 5-27: Plastic chord rotation capacity: (a) θ_u^{pl} vs $\theta_{u,NLA}^{pl}$ at bottom end(b) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at top end(c) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at beam left end (d) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at beam right end..... 5-33

Figure 5-28: Effective mechanical volumetric ratio ($\alpha\cdot\omega_{wd}$) *vs* Mechanical volumetric ratio of the hoops as a function of the plastic chord rotation capacity [$\alpha\cdot\omega_{wd}(\theta_u^{pl})$] 5-35

Figure 6-1: Buildings model [6] (a) 5-storey building (b) 10-storey building (c) Plan view..... 6-3

Figure 6-2: Schematic illustration of 3 integration sections and meshing for 150 fibres for beams and columns and 250 fibres for walls 6-6

Figure 6-3: Capacity curve (Adapted from Plumier[18] and Maranhão et al.[13])..... 6-8

Figure 6-4: Roof displacement from elastic analysis 6-9

Figure 6-5: Structure damage states (a) Attainment of the first plastic hinge (b) Plastic mechanism 6-9

Figure 6-6: General definition of bilinear load-deformation relationship: (a) global plastic mechanism is observed; (b) strength drop of 20% relative to the maximum resistance. 6-10

Figure 6-7: (a) Cantilevered wall bending moment diagram (b) Moment-curvature..... 6-11

Figure 6-8: Schematic representation of chord rotation (θ) and curvature (ϕ)..... 6-11

Figure 6-9: Theoretical curvature diagram (a) Distribution of the curvature along the wall (b) Curvature distribution at yield (c) Plastic part of the curvature 6-12

Figure 6-10: Plastic hinge length (a) Schematic representation of the plastic hinge field (b) Schematic representation (θ_u^{pl}) assuming deformation concentrated in plastic hinge length (l_{pl})..... 6-13

Figure 6-11: Definition of the curvature at yield (ϕ_y) and ultimate curvature(ϕ_u) (a) With verified 20% strength drop (b) Without strength drop..... 6-14

Figure 6-12: Definition of the chord rotation at yield (θ_y) and ultimate chord rotation (θ_u) (a) With verified 20% strength drop (b) Without strength drop..... 6-14

Figure 6-13: Design moment envelope proposed by Paulay et al.[23]..... 6-15

Figure 6-14: Wall envelope of bending moment (a) Design envelope for bending moments for wall systems EN 1998-1:2004[1] (b) Design envelope for bending moments for wall systems prEN 1998-1-2:2023[2] (Adapted from Maranhão et al.[6])..... 6-17

Figure 6-15: Wall strains and axial forces d tensile force and compression force resultant (Adapted from Maranhão et al.[6])..... 6-18

Figure 6-16: (a) Wall curvature (b) Tension shift..... 6-19

Figure 6-17: Geometrical definition of tension shift obtained by nonlinear analysis..... 6-20

Figure 6-18: Critical region at the base of a cantilevered wall (Adapted from Maranhão et al.[6])..... 6-21

Figure 6-19: Shear force diagrams..... 6-22

Figure 6-20: Capacity curves (a) 5-storey buildings (b) 10-storey buildings..... 6-24

Figure 6-21: Ductility ratio (μ) for the different structures for walls designed with θ=45° and θ=22°..... 6-24

Figure 6-22: Behaviour factor and its components (a) $q_{R,}$ (b) $q_{S,}$ (c) q_D, and (d) q..... 6-26

Figure 6-23: Moment vs chord rotation for (a) 5-storey buildings (b) 10-storey buildings..... 6-29

Figure 6-24: Moment vs curvature for (a) 5-storey buildings (b) 10-storey buildings..... 6-29

Figure 6-25: Height vs curvature (a) 5-storey buildings designed for strut θ=45° (b) 5-storey buildings designed for strut θ=22° (c) 10-storey buildings designed for strut θ=45° (d) 10-storey buildings designed for strut θ=22°..... 6-30

Figure 6-26: Plastic hinge length (l_{pl}) comparison (a) Structures designed with strut inclination =45° (b) Structures designed with strut inclination θ=22°..... 6-31

Figure 6-27: Comparison with walls plastic hinge length ($l_{pl,NLA}$) with tension shift (a_l) and critical height (h_{cr}) (a) Structures designed with strut inclination θ=45° (b) Structures designed with strut inclination θ=22°..... 6-32

Figure 6-28: Comparison with walls tension shift (d_{NLA}) with tension shift (a_l) (a) Structures designed with strut inclination =45° (b) Structures designed with strut inclination θ=22°..... 6-33

Figure 6-29: Shear force for linear analysis (V'_{Ed}) and shear force at shear failure at the base of the wall ($V_{R,NLA}$)..... 6-34

Figure 6-30: Shear magnification factor derived by nonlinear analysis (ε_{NLA}) and behaviour factor (q) *vs* shear magnification factor (ε) derived from Maranhão et al.[7] (a) Structures designed with a strut inclination θ=45° (b) Structures designed with a strut inclination θ=22°..... 6-35

LIST OF TABLES

Table 2-1: Characteristics/parameters of the structures analysed 2-2

Table 2-2: Horizontal Structural Irregularities (ASCE/SEI 7-22:2022[11])..... 2-7

Table 2-3: Vertical Structural Irregularities (ASCE/SEI 7-22:2022[11])..... 2-7

Table 2-4: Capacity design in shear (Adapted from ACI 318-19[12])..... 2-15

Table 2-5: Detailing provisions for beam (ACI 318-19[12])..... 2-16

Table 2-6: Detailing provisions for columns (ACI 318-19 [12]) 2-17

Table 2-7: Structural ductility category classification (NZS 1170.5:2004[19] and NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21]) 2-22

Table 2-8: Detailing provisions for beam (NZS 3101: Part 2:2006[21]) 2-39

Table 2-9: Detailing provisions for columns (NZS 3101: Part 2:2006[21])..... 2-40

Table 2-10: Detailing provisions for walls (NZS 3101: Part 2:2006[21]) 2-44

Table 2-11: Maximum values of seismic coefficient (C_{max}) based on (R) (NCh433.Of1996[29])..... 2-49

Table 2-12: Detailing provisions for beam (NTC-DCEC:2017[34])..... 2-72

Table 2-13: Detailing provisions of columns (NTC-DCEC:2017[34]) 2-73

Table 2-14: Detailing provisions for walls (NTC-DCEC:2017[34])..... 2-74

Table 2-15: Basic value of the behaviour factor (q_0) for systems regular in elevation (EN 1998-1:2004[37])..... 2-86

Table 2-16: Detailing provisions for columns (EN 1998-1:2004[37])..... 2-88

Table 2-17: Detailing provisions for local ductility for beams (EN 1998-1:2004[37])..... 2-92

Table 2-18: Rules for design detailing and dimensioning of columns (EN 1998-1:2004 [37])..... 2-92

Table 2-19: Detailing provisions for local ductility for beams (EN 1998-1:2004[37])..... 2-98

Table 2-20: Structural system comparison across seismic codes 2-99

Table 2-21: Structural type comparison across codes..... 2-100

Table 2-22: Structural type comparison across codes..... 2-101

Table 2-23: Seismic action comparison across codes..... 2-102

Table 2-24: Structural type comparison across codes..... 2-103

Table 2-25: Design and detailing provisions for different ductility classes in building codes 2-105

Table 2-26: Research on the basic value of the behaviour..... 2-114

Table 3-1: Characteristics/parameters of the structures analysed 3-3

Table 3-2: Adopted nomenclature for the structures..... 3-3

Table 3-3: Concrete mechanical properties..... 3-4

Table 3-4: Reinforcement steel mechanical properties 3-5

Table 3-5: Modelling criteria 3-6

Table 3-6: Default values of the behaviour factors for MRF systems regular in plan and in elevation – prEN 1998-1-2[3] 3-8

Table 3-7: Properties of the horizontal response spectra 3-9

Table 3-8: Limits of IDR in % in buildings with masonry infills - EN 1998-1:2004[2] 3-10

Table 3-9: Limits of IDR in buildings with masonry infills – prEN 1998-1-2:2023[3] 3-11

Table 3-10: Vertical loads 3-12

Table 3-11: Load combinations 3-13

Table 3-12: Rules for design detailing and dimensioning of primary beams 3-15

Table 3-13: Rules for design detailing and dimensioning of columns 3-16

Table 3-14: Seismic response parameters 3-19

Table 3-15: Limit to the beam rebar diameter d_{bLmax} prEN 19981-2:2023[3] 3-29

Table 3-16: Bill of quantities 3-37

Table 3-17: Quantities differences between structures 3-38

Table 4-1: Characteristics/parameters of the structures analysed 4-3

Table 4-2: Adopted nomenclature for the structures 4-3

Table 4-3: Geometry of the wall structures. 4-4

Table 4-4: Concrete mechanical properties 4-5

Table 4-5: Reinforcement steel mechanical properties 4-5

Table 4-6: Modelling criteria 4-6

Table 4-7: Default values of the behaviour factors wall systems regular in plan and in elevation according to prEN 1998-1-2:2023[3] 4-8

Table 4-8: Properties of the horizontal response spectra 4-9

Table 4-9: Vertical loads 4-11

Table 4-10: Load combinations 4-15

Table 4-11: Longitudinal and vertical reinforcement design 4-17

Table 4-12: Detailing boundary elements 4-18

Table 4-13: Detailing web elements 4-19

Table 4-14: Seismic response parameters 4-22

Table 4-15: Critical height (h_{cr}) and tension shift (a_l) for θ=22° and θ=45° 4-28

Table 4-16: Bill of quantities 4-39

Table 4-17: Quantities differences between structures 4-39

Table 5-1: Seismic response parameters (Adapted from Maranhão et al. [7]) 5-3

Table 5-2: Nonlinear analysis parameters 5-22

Table 5-3: Elastic properties, effective elastic period, and target displacements 5-24

Table 5-4: Paired *t*-test for the pair ($\theta_{y,NLA}-\theta_y$) for columns and beams ends [34] 5-30

Table 5-5: Paired *t*-test for the pair ($\theta_{u,NLA}-\theta_u$) for columns and beams ends [34] 5-32

Table 5-6: Paired *t*-test for the pair ($\theta_{u,NLA}^{pl}-\theta_u^{pl}$) for columns and beams ends[34] 5-34

Table 5-7: Paired *t*-test for the pair [$\alpha\cdot\omega_{wd}(\theta_u^{pl}),\alpha\cdot\omega_{wd}$] for columns base [34] 5-35

Table 6-1: Seismic response parameters (Adapted from Maranhão et al.[6]) 6-4

Table 6-2: Critical height (h_{cr}) and tension shift (al) for θ=22° and θ=45° (Adapted from Maranhão et. al.[6]) 6-22

Table 6-3: Nonlinear analysis parameters for structures designed with a strut of θ=45° 6-25

Table 6-4: Nonlinear analysis parameters for structures designed with a strut of θ=22° 6-25

1 Introduction

Seismic Structural Engineering is the branch of engineering aiming to mitigate structural damage induced by seismic events and predict strong earthquake consequences in physical infrastructures. It is also responsible for designing, developing methodologies, building and assessing existing structures that can withstand ground shakings, and complying with structural building codes.

In the past decades, structural designers have focused on designing safe and cost-effective buildings and infrastructures, such as power plants (especially nuclear power plants), that can successfully withstand seismic events. It should be noted that the seismic design philosophy differs substantially from the design approaches conventionally adopted for other types of actions. The conventional approach concerns designing structures that behave mostly elastically without damage (e.g., cracking) until the maximum loads are reached. In the case of seismic design, it is generally accepted that structures can experience damage because they should perform in the elastoplastic range for seismic events. The structural seismic design philosophy establishes that properly engineered structures' performance levels should prevent near-collapse in major seismic events and avoid damage to non-structural components, such as building partitions and electrical equipment, in minor ground-shaking events.

Due to seismic actions' unpredictable nature, damage control is challenging to quantify by code provisions, mainly related to acceptable risk levels. The challenge for efficient design of seismic resistant structures is balancing the seismic demand and the structural capacity to resist seismic induced effects without failure, recurring to several parameters associated with the structural behaviour.

EN 1998-1:2004[1] represents the state-of-the-art seismic design in Europe. However, some sections need to be updated in EN 1998-1:2004[1] regarding the last research developments and practitioners' feedback. A review of some fundamental concepts is required, such as the concept of ductility class. Allows state-of-the-art buildings yet provides clear and practical guidance for traditional buildings. Currently, three ductility classes are defined, namely the high (DCH) and medium (DCM), which are recommended for areas of high or medium seismicity and low (DCL) for low seismicity regions. These ductility classes allow the designer, within strictly prescribed limits, to balance the provision of strength against ductility. Thus, concrete structures can be designed to meet the sophisticated high ductility class (DCH) requirements or the more straightforward medium ductility class (DCM) requirements. However, the rules of DCH concrete structures are reported to be challenging in some cases.

In contrast, the DCM rules are sometimes complex (for example, for capacity design in shear of beams and columns). The DCH rules for concrete for concrete buildings often appear to be hard to satisfy in practice. In any case, there are two general reasons why providing high ductility is usually of no advantage, according to Booth[2].

One reason is that tall or flexible buildings, governed by deflection rather than strength, gain no advantage from high ductility levels. Another reason is that the high ductility option is becoming less attractive with the increasing emphasis on serviceability performance. The best choice will likely be to eliminate the DCM/DCH distinction. For concrete structures, a review of DCM provisions would be needed. The FprEN 1998-1-1:2024[3] overcame this issue by introducing new ductility classes known as DC1, DC2, and DC3, where DC1 and DC3 can be

related to the previous DCL and DCM regarding EN 1998-1:2004[1], letting this way, DCH out of the second-generation of Eurocode 8.

One of the significant innovations of the second-generation Eurocode 8 is the introduction of displacement-based design, complementing the traditional force-based method with reduced equivalent forces. The FprEN 1998-1-1:2024[3] provides the necessary resistance models for applying this approach to reinforced concrete structures. Displacement-based design involves verifying chord rotation according to prEN 1998-1-2:2023[4] for structural members, as described by Franchin et al.[5].

In the new generation of Eurocode 8, the structure of the parts was revised, concentrating in FprEN 1998-1-1:2024[3] the general aspects independent of the type of structures and materials, and in prEN 1998-1-2:2023[4] are included the specific rules for buildings.

The FprEN 1998-1-1:2024[3] contains general requirements for all types of structures for earthquake-resistant design, including the definition of the seismic action and the description of the analysis and verification methods in general terms. FprEN 1998-1-1:2024[3]should be applied to verifying buildings and civil engineering works in seismic regions. It is subdivided into 6 clauses. Clauses 4 to 6 give general rules relevant to all structures except for special ones. Clause 4 contains the basic performance requirements and compliance criteria applicable to buildings and civil engineering works in seismic regions. Clause 5 gives the rules for representing seismic actions and describing the seismic design situations. Clause 6 of FprEN 1998-1-1:2024[3] contains general structural analysis methods and verification under seismic actions, including base-isolated structures and structures with distributed dissipated systems.

The prEN 1998-1-2:2023[4] is applied to the design of new buildings and temporary structures in seismic regions, and it shall be applied to buildings of consequence classes CC1, CC2 and CC3. prEN 1998-1-2:2023[4] contains only those provisions that, in addition to the requirements of the other relevant Eurocodes, shall be observed for the design of new buildings in seismic regions. The prEN 1998-1-2[4] is subdivided into several clauses. Clauses 4 to 6 contain general rules for the verification of new buildings. Guidance for the analysis and the design of ancillary elements is given in 7. Specific rules for various structural materials and elements used in buildings are given in 10 to 15 in addition to the other relevant Eurocodes. Guidance for the design of base-isolated buildings is given in 8. Guidance for the design of buildings equipped with energy dissipation systems is given in 9.

The EN 1998-1:2004[1] allows the definition of the design seismic base shear force from elastic analysis (Equivalent lateral static force method), reduced by a constant factor to include the effects of the ductile, nonlinear behaviour of the structural system. This reduction factor is the behaviour factor (q) that accounts for the structure's capacity to dissipate hysteretic energy by ductile behaviour, overstrength, and redundancy of the structural system. Quantifying different parameters associated with buildings' structural behaviour, such as the behaviour factor (q), associated with the reference earthquake intensity and the definition of relevant design criteria, are still open issues among the structural researchers and engineering practitioners' community that need further studies.

To account for the nonlinear response of a structure associated with the material, the structural system, and the design procedures, the behaviour factor (q) is defined as the ratio between the peak force that would have developed if the single-degree-of-freedom system (SDOF) were linear-elastic, to the yield force of the system, according to Fardis[6]. According to EN 1998-1:2004[1], the behaviour factor (q) is an approximation of the ratio of the seismic

forces that the structure would experience if its response were completely elastic with 5% viscous damping to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure as referred to by EN 1998-1:2004. The influence of the behaviour factor on the shape of the design response spectrum, intrinsically on the spectral acceleration, and therefore the design force, is sensitive to the numerical value of the behaviour factor.

The FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4] introduce a behaviour factor composed by the product of the behaviour factors components (q_R), (q_S) and (q_D)where:

- q_R is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures.
- q_S is the behaviour factor component accounting for overstrength due to all other sources.
- q_D is the behaviour factor component accounting for the deformation and energy dissipation capacity.

The (q_R)value greater than unity is possible only if the structure is redundant, but (q_R) is not a redundancy measure, nor is “plastic deformation capacity.” Instead, it measures how much the internal distribution of forces calculated with a linear elastic analysis differs from the distribution, resulting in close-to-ultimate conditions that account for all sources of nonlinearity. In prEN 1998-1-2:2023[4], the behaviour factor component (q_S) accounts mainly for the material overstrength and excess resistance due to the unavoidable over-designed sections. The (q_D) component represents the building's deformation and energy dissipation capacity. This capacity depends on the building’s many characteristics, including the wall’s deformation capacity, the coupling of the walls through slabs and ring beams, wall-length distributions, and torsional effects. The behaviour factor components influence the displacement-based design under chord rotation analysis, interstorey drift, and (P-Δ) effects.

Regarding the detailing provisions, the second generation of the Eurocode 8, namely the prEN 1998-1-2:2023[4], offers some modifications and improvements for the reinforcement detailing of structural elements such as beams, columns, and walls. This includes specifications for rebar sizes, spacing, and lap lengths.

A notable novelty in the second generation of Eurocode 8 is that it no longer requires direct verification of concrete core confinement at the base of columns when detailing for local ductility.

Figure 1-1 shows the overview of the concrete structures design according to EN 1998-1:2004[1], FprEN 1998-1-1:2024[3]and prEN 1998-1-2:2023[4].

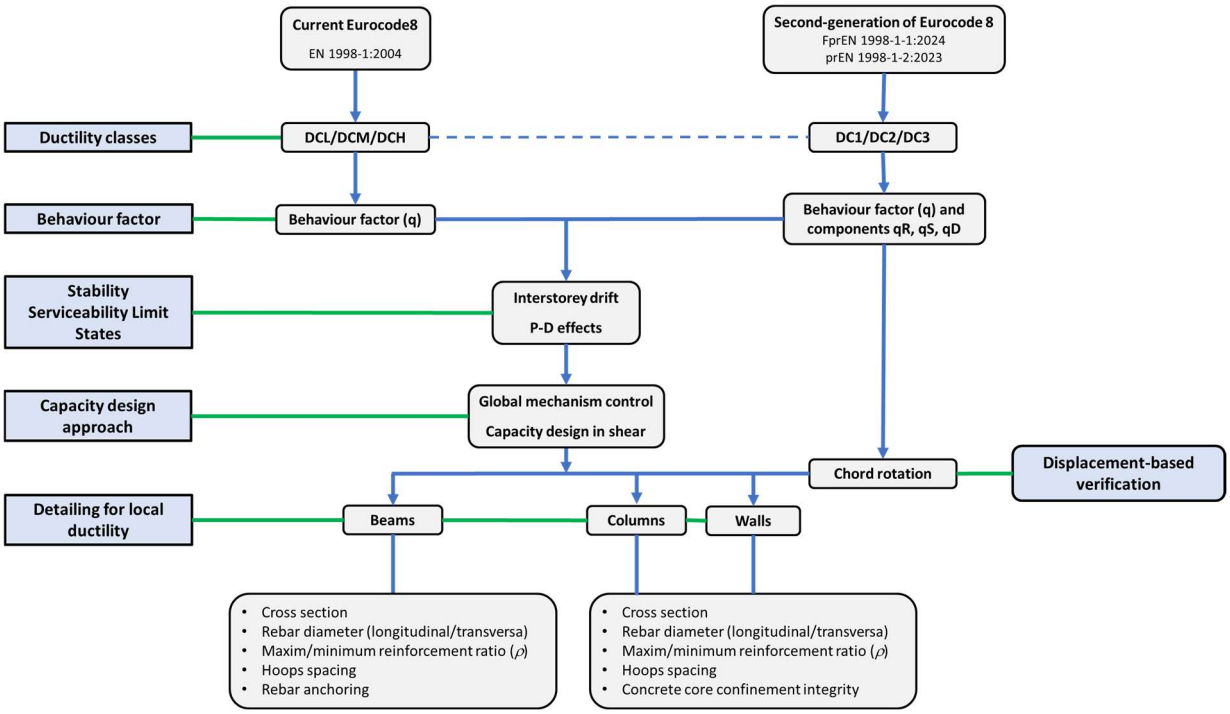


Figure 1-1: Overview of the design of the concrete structure according to the current and second generation of Eurocode 8

The prEN 1998-1-2:2023[4] introduces several innovative provisions for detailing shear walls and addressing seismic actions, marking significant advancements in seismic design standards. This includes comprehensive provisions for boundary elements, encompassing their confinement and reinforcement detailing, which is crucial for enhancing structural integrity under seismic loading. Notably, modifications to the bending moment envelope and shear force envelope are incorporated, reflecting a refined understanding of seismic behaviour. This work delves into these critical aspects, examining their implications and applications in seismic-resistant structural design.

All the above aspects are thoroughly analysed and discussed in the subsequent chapters. This discussion includes detailed evaluations, comparative analyses, and a synthesis of the findings to provide a comprehensive understanding of the implications for the field. Additionally, our findings' practical applications and potential limitations are explored to ensure a well-rounded examination of the topics.

1.1 MOTIVATION AND GOALS

This research addresses critical aspects of seismic design and detailing in reinforced concrete (RC) structures, particularly under the second generation of Eurocode 8. The study aims to advance the understanding and efficacy of design and detailing provisions for seismic-resilient structures by conducting a comprehensive investigation encompassing various chapters.

As explained before, the second generation of the Eurocode 8 under the form of FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4] introduces new ductility classes (DC1, DC2 and DC3), and consequently, new components in the behaviour factor, non-existing in the current version of the Eurocode 8, the EN 1998-1:2004[1]. The second generation of the Eurocode 8 introduces a new paradigm in some respects: displacement-based design approach and detailing for local ductility for moment-resisting frames and wall structures.

The second-generation Eurocode 8 introduces displacement-based design alongside the traditional force-based one with reduced equivalent forces; it provides resistance models needed to apply the approach to reinforced concrete RC moment-resistant frames (MRF) and wall structures: chord rotation for members. The EN 1998-1:2004[1] implicitly addressed displacement-based design, whereas the prEN 1998-1-2:2023[4] explicitly incorporates it.

In the second generation of Eurocode 8, the guidelines governing bending moment and shear actions for wall structures have been updated. The detailing requirements for local ductility, particularly in the boundary regions, have been revised. Due to these novelties, the present work proposes to compare and analyse the structural behaviour of reinforcement moment resisting frames and wall structures for the ductility classes DCM preconised in EN 1998-1:2004[1], DC2 and DC3 established by prEN 1998-1-2:2023[4]. It also aims to analyse the influence of the displacement-based design under the form of chord rotation analysis, the influence of the behaviour factor components on the interstorey drift and (P-Δ) effects introduced by the FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4]. The present analysis also intends to compare the influence envelope of the bending moment and shear actions calculation models preconized by EN 1998-1:2004[1] for DCM structures and prEN 1998-1-2:2023[4] for DC2 and DC3 structures. Lastly, the present research work also aims to compare and observe the influence and differences of provisions for local ductility between the current EN 1998-1:2004[1] and the second-generation FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4].

This analysis preconizes two different moments:

- Linear analysis of RC MRF and wall structures to compare the design between the current EN 1998-1:2004[1] and second generation FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4].
- Nonlinear analysis is used to analyse variables used in the design, such as the behaviour factor and its components, chord rotation, concrete core confinement, wall bending moment, and shear actions.

These analyses are thoroughly conducted and elaborately detailed across chapters 3 to 6, encompassing the overarching research purpose, the methodology employed, and the conclusions derived from the outcomes. Each chapter methodically presents the steps taken, the tools utilized, and the implications of the findings, offering a comprehensive view of the research trajectory and its significant results.

The primary objective is to evaluate their impact on the design, behaviour, and safety verification processes of various families of reinforced concrete (RC) structures. Through comprehensive analysis, this study aims to provide insights into structural performance and analyse the influence on the safety of reinforced concrete (RC) structures.

The study may be divided into two parts:

1) The design and analysis of the detailing provisions and seismic action are done using linear analysis.
2) The several parameters of the detailing provision and seismic action are analysed using nonlinear analysis.

To achieve these objectives, sixty reinforced concrete (RC) structures were meticulously modelled, designed and analysed. These sixty structures are divided as follows:

a) Ten-moment resisting frame structures (MRF) for linear analysis and design.
b) Twenty wall structures for linear analysis and design.
c) Ten-moment resisting frame structures for nonlinear analysis derived from the linear analysis and design.
d) Twenty wall structures for linear analysis and design for nonlinear analysis derived from the linear analysis and design.

These structures were intentionally crafted to adhere to regularity in plan and elevation, ensuring uniformity and consistency in their geometry. Moreover, they were designed to be non-torsionally flexible, prioritizing stability and predictability in their response to seismic forces. Each structure within this set was carefully crafted to replicate various scenarios, allowing for comprehensive exploration and analysis. These scenarios encompassed the variation of several vital aspects and parameters, described in Figure 1-2.

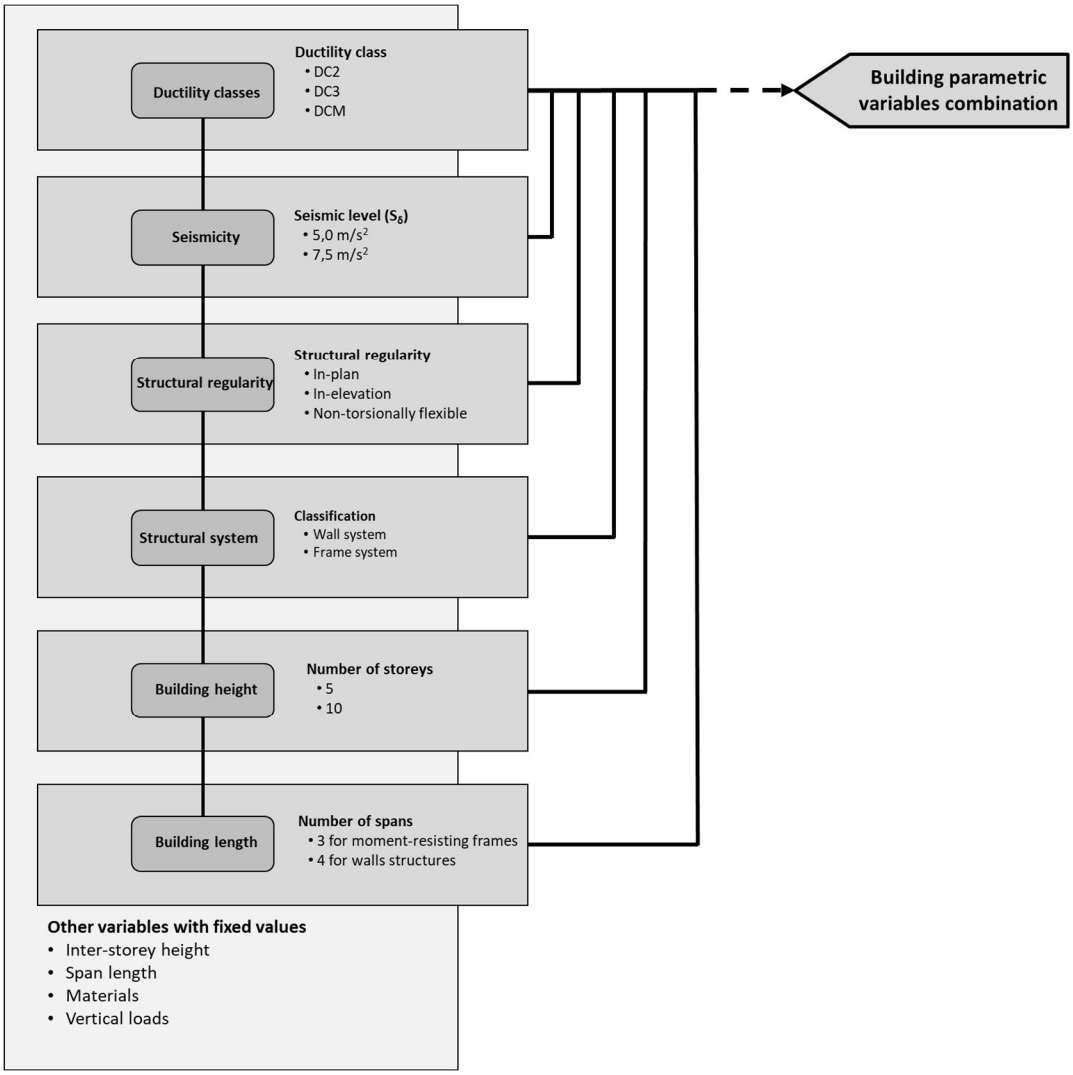


Figure 1-2: Building parametric variables combination to set up the analysis scenarios

The buildings are designed for the ductility class DCM and the force-based design concept (FBD). The FBD is an essential procedure for seismic design of building structures adopted in earthquake-resistant design practice EN 1998-1:2004[1]. This method calculates the design base shear by dividing the seismic forces by the behaviour factor(q). The buildings designed for the ductility class DC2 and DC3 were designed with the force-based design concept (FBD) alongside the displacement-based design concept (DBD) according to FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4]. The set of buildings structures to be analyzed have a consequence class CC2 according to EN 1990:2002[7].

It should be noted that all buildings are designed in a three-dimensional environment (3D). A 3D model allows a better idea of how the forces are transferred to the structure's elements and foundations, its behaviour and the detection of stability deficits relevant to safety reasons.

The computational process encompasses the various stages of analysis, detailing their functionality and how they interact. This involves the individual operations within each stage and how these stages communicate and work together to achieve the analytical objectives. The computation process comprehends three stages:

- Stage 1–Structural analysis–Linear elastic.
- Stage 2–Design verification, reinforcement design, and detailing.
- Stage 3–Nonlinear analysis.

The structural analysis stage (stage 1) is foreseen to determine loads' effects and calculate the structural elements' internal forces, deformations, nodal displacements, and the structure's horizontal seismic forces. Structural Analysis considers the material's properties, structure geometry, and applied loads.

At this stage, the vertical loads are already defined. The seismic loads are calculated by introducing the seismic spectra in the finite element (FE) software. Subsequently, the software calculates the natural frequency and chooses the spectral acceleration according to the building mass, geometry, and stiffness. The main outputs are the internal forces, structural deformation, nodal displacements, and eigenfrequency. This stage also comprises the verification of the inter-storey limitation and verifying the second-order effects, known as (P-Δ). The element's gross cross-sections also depend on this verification's outcome since the drift limitation may govern the design in some situations.

The reinforcement of structural elements is designed at stage 2. All detailing provisions preconized by the EN 1998-1:2004[1], FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4] and EN 1992-1:2004[8] are verified. The aim is to select concrete dimensions and reinforcement so that the member strength can resist forces resulting from the internal forces. The design concept is known as “strength design”. Based on the strength design concept, a member's nominal strength must be calculated based on the material's inelastic behaviour. This means reinforcing steel and concrete behave inelastically under ultimate strength conditions. The output of this stage is the structural elements reinforcement quantity and elements cross-section.

The third stage of the computational process is nonlinear parametric values, a task fundamental in the present research. Capacity curves are a critical analytical tool used across all our analyses. These curves are derived through static pushover analysis, a technique in which the structure's incremental loading is simulated to elicit its nonlinear behaviour. For this purpose, SeismoStruct[9] software is used, which facilitates the computation of the structural system's nonlinear response during static analysis and moves beyond the limitations of elastic analysis to provide a deeper insight into the structure's performance under stress.

Figure 1-3 illustrates the computational process used in the present research.

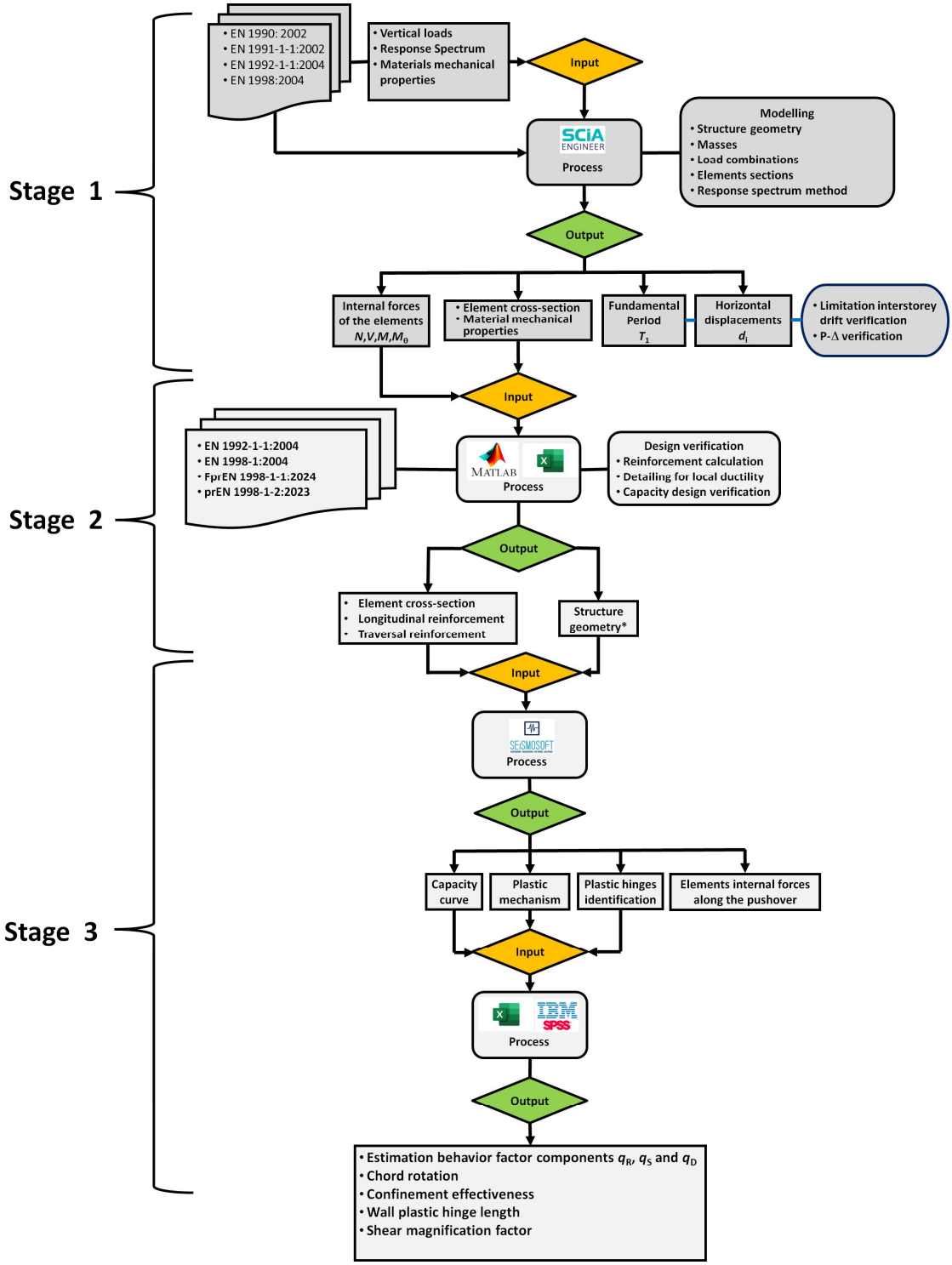


Figure 1-3: Computation flowchart for Structural analysis stage

1.2 DOCUMENT ORGANIZATION

Figure 1-4 provides a structured overview of the document's organization, mapping the logical flow and interconnections between the chapters. This visualization aids in navigating the content and understanding the progression of themes and topics within the present document. Additionally, it highlights the chapters' hierarchical relationship and offers readers a clear and coherent framework to follow.

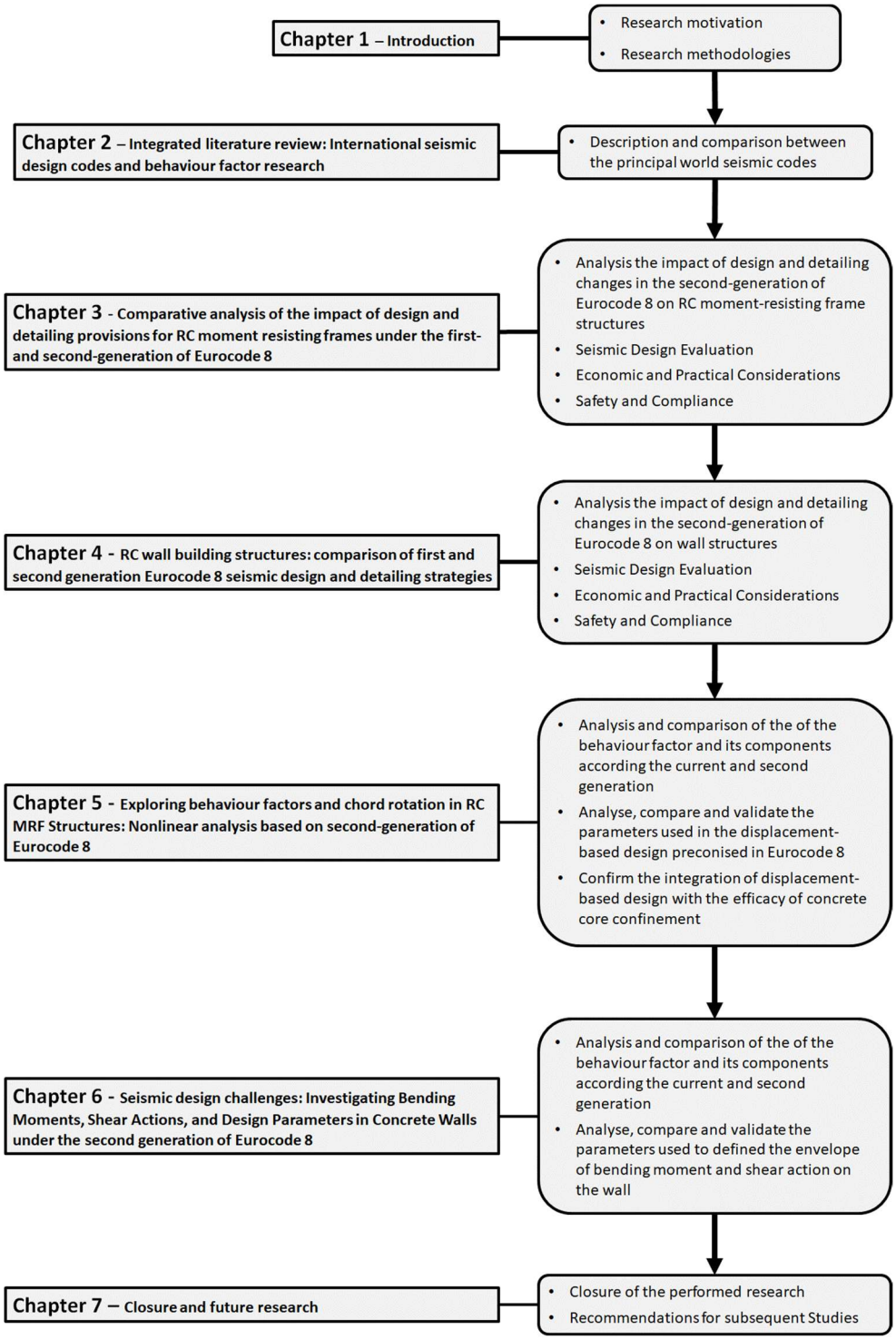


Figure 1-4: Document organization

Chapter 1," *Introduction*", concerns that the introduction serves as a gateway to the research theme, providing context and rationale behind the study. In this section, the motivations and goals of the current research are elucidated, laying the groundwork for understanding the significance and objectives of the study. By articulating the driving factors that prompted the investigation, the introduction sets the stage for delving into the intricacies of the research topic. It highlights the gaps in existing knowledge or practices, identifies potential areas for improvement or exploration, and outlines the specific objectives the research aims to achieve. Additionally, the introduction may provide insights into the broader implications of the research findings, indicating how they contribute to advancing understanding or addressing practical challenges within the field. Overall, the introduction serves as a roadmap, guiding the reader through the journey of discovery and inquiry the research endeavours to undertake.

Chapter 2, "*Integrated literature review: International seismic design codes and behaviour factor research*", describes and compares the principal world seismic codes such as:

- United States of America.
- New Zealand.
- Chile.
- Mexico.
- Europe.

Regarding the definition and provisions of the behaviour factor, it briefly compares the ductility classification between codes, detailing requirements, the associated base shear, and drift control. It also concerns state-of-the-art background concepts that led to the consciousness of the importance of the behaviour factor on the seismic design and the formulation proposed by various researchers.

In Chapter 3, "*Comparative analysis of the impact of design and detailing provisions for RC moment resisting frames under the first- and second-generation of Eurocode 8*", a study is conducted concerning a comparative analysis of the impact of design and detailing provisions for reinforced concrete (RC) moment-resisting frames (MRFs) under the first and second-generation of Eurocode 8. The research aims to evaluate the implications of novel ductility classes, drift limits for damage limitation, and local ductility conditions introduced in the second-generation Eurocode 8. By comparing these changes to the current provisions, the study assesses the effectiveness of the new design and detailing provisions.

An extensive examination was conducted on ten symmetrical Moment-resistant frame (MRF) buildings, regular in plan and elevation and non-torsionally flexible, using the current and the upcoming Eurocode 8 provisions. The results indicated variations in detailing demands for local ductility and capacity design among different ductility classes and building configurations.

The study also incorporates a comprehensive review of methodologies for seismic design and the performance of RC buildings, considering recent seismic events and their implications for structural integrity. It employs modern design procedures intending to deliver structural systems capable of withstanding seismic events, leveraging the force-based approach for analysis.

Through meticulous finite element (FE) modelling and seismic action analysis, the study provides insights into the structural performance and safety requirements, particularly under seismic combinations. The findings offer valuable information on economic considerations for structural configurations under strong seismic actions, aiding in the evolution of design practices and code provisions for earthquake-resistant structures.

Chapter 4, "*RC wall building structures: comparison of first and second generation Eurocode 8 seismic design and detailing strategies*", discusses modern design procedures to create robust and resilient structural wall systems capable of withstanding seismic events while maintaining acceptable performance levels. It emphasizes the importance of appropriately configured, sized, and meticulously detailed wall systems within the overall building framework to achieve this objective, particularly under challenging earthquake loading conditions.

The study compares wall structures across the new ductility classes, examining the impact of displacement-based design on structural behaviour, including interstorey drift and (P-Δ) effects. It also evaluates the new approach to calculating the bending moment and shear action envelopes and the implications of local ductility provisions on structural design.

This research contrasts structures that comply with the original and second generation of Eurocode 8. It conducts meticulous design calculations for each ductility class to highlight the nuanced differences introduced by the second generation of the code on wall structures.

Chapter 5, "*Exploring behaviour factors and chord rotation in RC MRF Structures: Nonlinear analysis based on second-generation of Eurocode 8*", EN 1998-1:2004[1] defines three ductility classes for seismic design in Europe: high (DCH), medium (DCM), and low (DCL), tailored to regions of varying seismic activity. Feedback suggests that an update is needed to align with modern developments and practical applications.

The new generation of Eurocode 8, particularly in its documents FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4], introduces refinements such as new ductility classes (DC1, DC2, and DC3). These changes address issues like serviceability performance and the limited benefits of high ductility for tall or flexible structures. Additionally, the second-generation Eurocode 8 revises the behaviour factor (q), introducing components (q_R, q_S, and q_D) reflecting a structure's ability to dissipate energy and maintain serviceability. The current study aims to analyze each behaviour factor component using nonlinear analysis within a 3D modelling environment and examine capacity design's impact on these components.

The second generation of Eurocode 8 code also shifts towards displacement-based design methods for concrete structures, focusing on chord rotation control to ensure ductility in critical regions, thereby improving the performance and safety of the structure. The study also analyses these chord rotation components through nonlinear analysis, comparing them with expressions preconised in FprEN 1998-1-1:2024[3]. This study also explores the implications of the revised approach to concrete core confinement in columns by examining displacement-based design alone, ensuring safety without directly verifying the column's core confinement at the base. The analysis builds on Maranhão et al.' 's[9] work. It assesses RC MRF structures designed for regularity in plan and elevation without torsional flexibility.

Chapter 6, "*Seismic design challenges: Investigating Bending Moments, Shear Actions, and Design Parameters in Concrete Walls under the second-generation of Eurocode 8*", discussed the significance of bending moments and shear actions in the seismic safety and design of wall structures, alongside various parameters influencing their design envelope. It highlights the importance of Eurocode 8's requirements for maintaining wall elasticity above the plastic hinge region and covering potential increases in bending moments due to inelastic responses. The wall's critical height (h_{cr}) is a crucial factor intrinsically linked to the level of detailing for local ductility, essential for ensuring the wall's integrity and energy dissipation capabilities.

The research aims to comprehensively analyse plastic hinge lengths derived from nonlinear analysis with critical tension shift height (a_l) and critical height (h_{cr}). The objective is to investigate the influence of plastic hinge length on tension shift height, critical height, and bending moment distribution envelope, which is crucial for effective energy dissipation. Additionally, the study aims to compare the tension shift height(a_l), delineating the envelope of bending moment actions with the tension shift height derived from nonlinear analysis, and to discern the optimal strut angle for structural performance.

Furthermore, the research aims to evaluate and compare the magnification factor of shear action (ε) as specified in Eurocode 8, aiming to enhance seismic structures' safety, reliability, and performance by incorporating uncertainties and dynamic effects into the design process.

The analysis builds on Maranhão et al.' 's [10] work. It assesses RC wall structures designed for regularity in plan and elevation without torsional flexibility.

Finally, chapter 7," *Closure and future research*", draws general conclusions about the research and discusses future research that could explore advancements in structural design methodologies and detailing practices for reinforced concrete (RC) structures, mainly focusing on their performance under seismic loading conditions. This could involve further investigation into the effectiveness of the design and detailing provisions introduced in the second generation of Eurocode 8, compared to previous versions, through comprehensive comparative analyses of various structural configurations.

1.3 REFERENCES

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[7] EN 1990, *"Eurocode 0 - Basis of structural design"*, CEN-European Committee for Standardization, Brussels, 2002.

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

[9] H. Maranhão, H. Varum, J. Melo, and A. A. Correia, *"Comparative analysis of the impact of design and detailing provisions for RC moment resisting frames under the first- and second-generation of Eurocode 8"*, Engineering Structures, vol. 306, p. 117809, May 2024, doi: 10.1016/j.engstruct.2024.117809.

[10] H. Maranhão, H. Varum, J. Melo, and A. Correia, 4 *"RC wall building structures: comparison of first and second generation Eurocode 8 seismic design and detailing strategies"*, Engineering Structures, 2024, doi: Not yet published.

2 Integrated literature review: International seismic design codes and behaviour factor research

This chapter presents the principal seismic codes regarding seismic design actions and associated criteria, the capacity of members and joints, the definition and provisions of the behaviour factor, and a brief comparison of the ductility classification between codes. The focus of the codes and the comparison between them is tailored explicitly to reinforced concrete buildings, which are the subject of this research.

This section also describes the state-of-the-art evolution regarding behaviour factors, such as the difficulties of parameterizing its value and the various proposals to define it for different design situations. A description of the background of the behaviour factor is made in the second part, realizing its importance in seismic design and its formulation, which various researchers have proposed.

The terms behaviour factor, response modification factor, force reduction factor, and ductility factor are used interchangeably between the provisions of different seismic design codes. It should, however, be realized that they have the same purpose and influence on the seismic design procedure. For conformity and to avoid confusion in this study, the term behaviour factor shall be used throughout the current chapter.

2.1 REVIEW OF INTERNATIONAL SEISMIC DESIGN CODES LITERATURE

The national seismic design codes converge on the issue of design methodology. These are built up using force-based design methodology (FB), where the design is performed using a linear elastic analysis through a behaviour factor. This factor governs the seismic design forces since it directly impacts the response spectra draw up and, therefore, the design seismic forces. Different national codes diverge significantly on account of various specifications which govern the design force level. As considered in the seismic design codes, the behaviour factor depends on the structure's ductility and overstrength. Building codes define different ductility classes and specify corresponding response reduction factors in the form of behaviour factor based on the structural material, geometrical configuration, and detailing. Another main issue that governs the design and expected seismic performance of a building is drift control. Drift is an important control parameter, and the different codes differ regarding the effective flexural stiffness of reinforced concrete members. The procedures to estimate drift and the allowable limits on drift also diverge considerably from code to code, according to Khose et al.[1].

The seismic codes differ in the design, load, and material or strength behaviour factors for members' design and detailing for local ductility. Hence, the provided strength in different codes does not follow the same pattern. This concept directly impacts the expected performance of buildings designed using other codes.

A comprehensive discussion and comparison of specific issues and requirements for reinforced concrete buildings accompanies an overview of prominent international seismic codes. It covers critical aspects, including structural systems, design coefficients, irregularities, seismic loads, drift and stability analysis, capacity design approach, and detailing for local ductility.

This chapter aims to concisely overview international seismic standards without delving into exhaustive detail. Instead of providing an in-depth analysis of each standard, it offers a broad understanding of the fundamental principles and approaches these standards follow

worldwide. By presenting a summary of international seismic standards, readers can gain insights into common practices, methodologies, and considerations employed in seismic design and analysis across different regions and countries. The focus is on capturing the essence of these standards rather than providing comprehensive coverage of every aspect. This approach allows readers to grasp the fundamental concepts and requirements governing seismic-resistant design without being overwhelmed by technical intricacies. Moreover, it serves as a valuable reference for professionals and researchers seeking an overview of seismic standards from a global perspective. The seismic codes analysed in this study are presented in Table 2-1.

Table 2-1: Characteristics/parameters of the structures analysed

Country	Standard	Title	Year
U.S.A	ASCE/SEI 7-22	Minimum Design Loads and Associated Criteria for Buildings and Other Structures	2022
	ACI 318 -19	Building Code Requirements for Structural Concrete	2019
New Zealand	NZS 4219:2009	Seismic Performance of Engineering Systems In Buildings	2009
	NZS 1170.5 Supp 1:2004	Structural Design Actions Part 5: Earthquake actions New Zealand - Commentary	2004
	NZS 3101:Part 1:2006	Concrete Structures Standard	2006
	NZS 3101:Part 2:2006	Concrete Structures Standard -Commentary	2006
Chile	NCh 433.Of1996 Modificada en 2009	Diseño sísmico de edificios/ Earthquake resistant design of buildings	2009
	NCh430.Of2008	Hormigón armado - Requisitos de diseño y cálculo/ Reinforced concrete - Design and calculation requirements	2008
Mexico	NTCDS	Norma Técnica Complementaria para Diseño por Sismo/ Mexican Standard for Design and Construction of Concrete Structures	2017
	NMX-C-061-ONNCCE	Norma Mexicana de Diseño y Construcción de Estructuras de Concreto/ Mexican Standard for Design and Construction of Concrete Structures	2019
Europe	EN 1998-1	Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings	2004
	EN 1992-1-1	Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings	2004

Through this chapter, readers can appreciate the diversity of approaches to seismic design and understand how various standards address seismic hazards based on regional seismicity, building codes, and construction practices. Ultimately, the goal is to offer a starting point for further exploration and study of seismic standards applicable in different contexts worldwide.

2.1.1 U.S. SEISMIC AND CONCRETE CODES

Two design standards represent the most current and widely used documents applicable to the seismic design of concrete structures in the United States. ASCE/SEI 7-10[2] specifies all structure types' design requirements, including seismic design ground accelerations or equivalent static forces, seismic design coefficients, load combinations, analysis procedures, and story drift limitations ACI 318-08[3] presents provisions specific to reinforced concrete. Both of these standards are referenced by the International Building IBC-2006[4], the governing code in most of the United States, as referred by FIB-FIP-CEB 69[5].

Seismic design provisions in U.S. building codes followed a specific pattern. Provisions were first proposed by the Structural Engineers Association of California in its Recommended Lateral Force Requirements SEAOC:1999[6]. The UBC provisions were then adopted by the American National Standards Institute ANSI A58.1:1982[7].

The 2000 IBC-2000[8] had seismic design provisions based on the NEHRP 1997[9] provisions. The entire text of the provisions was in the code ASCE 7-98[10], which also had seismic design provisions based on the 1997 NEHRP[9] Provisions and was not referenced.

While any group of manufacturers, associations, consumers, users, or agencies can cooperatively develop a standard for its purposes and reasons, only when a standard is designed following definitive rules of procedure and consensus does the standard acquire the stature appropriate and necessary for regulating use in building codes The ACI 318 "*Building Code Requirements for Structural Concrete*" and the ASCE/SEI 7 "*Minimum Design Loads for Buildings, and Other Structures"* are the best-known standards adopted into all model codes.

A code typically references a given standard, identifying when and how the standard is to be utilized. Standards are commonly adopted into model codes without amendments; however, modifications are uncommon. The standard development process varies from one standard-writing body to another. However, all ANSI-accredited methods have certain essential elements in common. The ACI 318 process is described here as reasonably representative of an ANSI-accredited process.

According to FIB-FIP-CEB 69[5], the design philosophy of the United States standards is that every structure should have a "lateral and vertical force-resisting system capable of providing adequate strength, stiffness, and energy dissipation to withstand the design ground motions within the prescribed limits of deformation and strength demand. In the present subchapter it will present structural systems, design coefficients, irregularities, seismic loads, drift and stability analysis regarding ANSI/SEI 7-22[11], capacity design approach, and detailing for local ductility concerning ACI 318-19[12].

2.1.1.1 STRUCTURAL SYSTEM

Choosing a structural system type is a pivotal and initial step in establishing building seismic design criteria. Structural systems are classified based on three fundamental characteristics:

- The construction materials.
- The method by which the structure resists lateral forces induced by earthquake loads.
- The relative degree of earthquake-resistant design and detailing.

According to Metwally[13], ductility denotes the capacity of specific structural systems to undergo deformations beyond those leading to their peak strength while still sustaining the

load. Brittle structural systems lack ductility, undergoing elastic deformation until reaching their ultimate strength, followed by sudden failure and loss of load-carrying ability. Structures with limited ductility may maintain load-carrying capacity up to a deformation 50% larger than the one triggering peak strength. Highly ductile structures can withstand deformations up to 4 or 5 times those at which peak strength is reached without losing load-carrying ability. In practice, most structural systems exhibit some degree of ductility.

ASCE/SEI 7-22[11] classifies systems with excellent ductility as special, those with restricted ductility as ordinary, and those with intermediate ductility as intermediate. ASCE/SEI 7-22[11] defines the applicable structural systems for seismic resistance, associated height restrictions, applicability to different structural ductility classes, and applicable design coefficients.

The designated portion of a structure intended to furnish the necessary earthquake resistance is called the seismic force-resisting system (SFRS). Structures allocated to Seismic Design Category (SDC) A have the flexibility to employ any SFRS provided the system is comprehensive and meets the minimum specified strength requirements. Conversely, buildings assigned to SDC B or higher must adhere to one of the specific seismic force-resisting systems or combinations outlined in Table 12.2-1 of ASCE/SEI 7-22[11] and comply with all associated design regulations. While the SFRS of a building also addresses other lateral loads, such as wind, this discussion primarily focuses on seismic considerations, as referred to by Metwally[13].

ASCE/SEI 7-22[11] enumerates over 90 structural systems, affording designers diverse options. These systems are broadly categorized for buildings into six distinct groups:

- Bearing wall systems.
- Building frame systems.
- Moment-resisting frame systems.
- Dual systems and shear wall frame interactive systems.
- Cantilever column systems.
- Systems not explicitly designed for seismic resistance.

The wall systems can be classified as bearing wall systems or building frame systems, depending on whether the walls carry a substantial portion of the gravity loading of the building or rely on them only to resist lateral loads. The building code requires that wall systems that carry significant portions of the vertical load of a building be designed with higher strength than those that do not so that they will experience less damage in response to strong shaking.

The ASCE/SEI 7-22[11] and ACI 318-19[12] introduced a new class of special concrete shear wall system called the ductile coupled wall system. The coupled wall system must incorporate coupling beams, meeting specific dimensional criteria, over openings. These coupling beams serve as a benign means of dissipating earthquake energy and permit the use of reduced design forces relative to other concrete wall systems.

Walls of light-frame construction are categorized according to their ability to resist inelastic response, primarily based on the type of sheathing used to provide lateral resistance. Traditional plaster and gypsum board sheathing systems have limited ability to provide repeated resistance to lateral deformation once the plaster or gypsum product cracks. Walls with these

sheathing materials are limited to SDC (B), (C) and (D), and SDC (D) are only permitted for low-rise structures.

Regarding the Braced Frame Systems, the most common braced frame systems are constructed from steel. Steel-braced frames are a common type of structural steel building frame system. Braced frame systems detailed explicitly for seismic resistance must meet the criteria of ANSI/AISC 341-22[14]. This is required for braced frames in SDC D, E, or F and permitted for other SDCs. ANSI/AISC 341-22[14] does not permit single diagonal braced frames with more than 50% of the braces in a story and a frame line aligned in one direction because if the braces are overloaded and buckle, the frame will lose lateral resistance.

Concentrically braced frames are configured so that the beams, columns, and braces intersect at common points. Eccentrically braced frames are configured such that brace-to-beam connections are offset from each other or offset from beam-column intersections, such that nonlinear behaviour is accommodated through ductile flexural or shear yielding of the beams rather than the braces themselves. Buckling-restrained braced frames use braces consisting of a central steel core that can yield tension or compression, braced by an outer sleeve.

Moment-resisting frame systems can be constructed of structural steel, reinforced concrete, or a combination of steel and reinforced concrete called composite construction. Moment frames derive their lateral resistance through the rigid or semi-rigid connection of their beams and columns. This results in the lateral deformation pattern. That coincides with the development of shearing forces, bending moments in the beams and columns, and axial forces associated with overturning in the columns. ACI 318-19[12] and ANSI/AISC 341-22[14] describe the detailing required of special, intermediate, and ordinary moment frames of reinforced concrete, steel, and composite construction.

Special moment frames detailed following ACI 318-19[12]and ANSI/AISC 341-22[14] can provide significant inelastic response, permitting them to be designed for low lateral forces relative to other systems and allowing the use of these systems without height restrictions, even in SDC D, E, and F. Both ACI 318-19[12] and ANSI/AISC 341-22[14] require design of beams and columns such that flexural yielding of the frame will occur mainly in the beams, rather than the columns, to minimize damage to columns. Both specifications also restrict the materials used in special moment frames and the locations of splices in members to ensure that nonlinear behaviour is achievable.

ACI 318-19[12] and ANSI/AISC 341-22[14] recognize intermediate and ordinary moment frame systems. These systems have relaxed detailing criteria relative to special moment-resisting frames. However, they must have greater strength than special moment frames to limit the nonlinear behaviour experienced in design shaking.

Dual systems combine elements of both moment-resisting frame systems, crafted from concrete, steel, or composite materials, and either concrete or masonry walls or braced steel frame systems. In seismic design categories C, D, E, and F, the engineering of dual systems requires a structural analysis that considers the interplay between the moment frame and other structural elements. The moment frame must be engineered to withstand at least 25% of the required seismic forces and possess adequate strength to endure the forces predicted by the analysis. This prerequisite ensures that the moment frame is a backup system capable of withstanding earthquake forces in case of extensive damage to the primary system (walls or braced frames). Like special moment frame systems, dual systems incorporating a special moment-resisting frame can be constructed without height restrictions in seismic design

categories D, E, and F. In seismic design category B, ASCE/SEI 7-22[11] allows for a dual system called a frame-wall interactive system, where the moment frame is not required to resist 25% of the total seismic design forces.

Cantilever column systems represent a unique form of moment-resisting frame in which no beams are attached to the tops of the columns to prevent rotation. These systems solely rely on the fixity against rotation at the column base for lateral resistance. The simple single degree of freedom (SDOF) structure exemplifies a cantilever column system. Both concrete and steel cantilever column systems are permissible under the code. The detailing of cantilevered columns may conform to the criteria for special, intermediate, or ordinary moment-resisting frame systems. However, height restrictions are stringent, and these structures must be engineered to remain nearly elastic in response to seismic forces. This is because these systems often lack redundancy, resulting in the formation of a single hinge at the column base, which can lead to the formation of a plastic mechanism. These highly flexible systems can quickly develop significant (P-Δ) effects and instability.

2.1.1.2 DESIGN COEFFICIENTS

In ASCE/SEI 7-22[11], table 12.2-1 delineates the values of three design coefficients crucial for determining the requisite strength and stiffness of a structure's seismic force-resisting system.

The response modification coefficient, denoted as (R), encompasses the capacity of specific seismic force-resisting systems to withstand earthquake shaking in a ductile manner without sacrificing load-carrying capacity. The (R) values span from 1 for systems lacking ductile response capability to 8 for systems with a highly ductile response. They are employed to diminish the required design strength for a structure.

The deflection amplification coefficient (C_d) adjusts computed lateral displacements for the structure derived through linear analysis methods to the expected inelastic lateral displacement anticipated during design earthquake shaking. The (C_d) factors attributed to various structural systems are typically akin but marginally smaller than the (R) coefficients, approximating the effective damping and energy dissipation mobilized during the inelastic response of highly ductile systems. The discrepancy between the (R) and (C_d) values amplifies with increased system ductility.

The coefficient (Ω_0) is an overstrength coefficient utilized to accommodate the phenomenon where actual seismic forces exerted on certain structural elements can notably surpass those estimated by analysis using the designated seismic forces for design. For most structural systems, the (Ω_0) coefficient typically ranges between 2 and 3, as referred by Metwally[13].

2.1.1.3 REGULARITY OF STRUCTURES

The design coefficients (R, C_d, and Ωo) specified in ASCE/SEI 7-22[11] are initially formulated for systems exhibiting regular configurations, where deformation and nonlinear response are evenly distributed throughout the structure and limited torsion about the vertical axis. However, specific analysis procedures employed to determine the required strength of structural systems may lack reliability when applied to structures that deviate from this regularity. Non-uniform distribution of strength or stiffness and discontinuous structural systems can invalidate the design procedures' assumptions. Such deviations from regularity are termed irregularities, and structures featuring one or more irregularities are classified as irregular structures.

Irregularities necessitate adjustments to the analysis procedures to determine required strength and story drift. Some irregularities mandate the strengthening of specific portions of the structure to counteract the adverse effects of the irregularity. Moreover, certain irregularities have exhibited poor performance during past earthquakes, leading to their prohibition in structures designated to seismic design categories E or F. ASCE/SEI 7-22[11] identifies two primary categories of structural irregularity: horizontal and vertical. Table 2-2 describes horizontal irregularities, according to ASCE/SEI 7-22[11].

Table 2-2: Horizontal Structural Irregularities (ASCE/SEI 7-22:2022[11])

Type	Description
1a	Torsional Irregularity
2	**Reentrant Corner Irregularity**: Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given
3	**Diaphragm Discontinuity Irregularity**: Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.
4	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined as a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.
5	**Nonparallel System Irregularity**: Nonparallel system irregularity is defined as the existence of vertical, lateral force-resisting elements that are not parallel to the major orthogonal axes of the seismic force-resisting system.

Table 2-3 describes the vertical irregularities:

Table 2-3: Vertical Structural Irregularities (ASCE/SEI 7-22:2022[11])

Type	Description
1a	**Stiffness-Soft Story Irregularity:** Stiffness-soft story irregularity is defined as a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
1b	**Stiffness-Extreme Soft Story Irregularity:** Stiffness-extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.
2	Weight (Mass) Irregularity: Weight (mass) irregularity is defined as the existence of an effective mass of any story that is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
3	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined as the horizontal dimension of the seismic force-resisting system in any story, more than 130% of that in an adjacent story.
4	In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity: In-plane discontinuity in vertical, lateral force-resisting elements irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on a supporting beam, column, truss, or slab.
5a	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story's lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.
5b	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity**: Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the storey's lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

It should be noted that ASCE/SEI 7-22[11] removes the extreme torsional irregularity, a type of horizontal irregularity included in previous ASCE/SEI 7 editions.

2.1.1.4 SEISMIC ACTIONS

ASCE/SEI 7-22[11] mandates that structures possess sufficient strength to withstand designated design earthquake forces in conjunction with other loads. Earthquake shaking generates horizontal and vertical forces within structures, which can fluctuate significantly during seismic events, occasionally reaching substantial levels for short durations ranging from a fraction of a second to a few seconds. In structures allocated to seismic design categories D, E, or F, these forces can surpass those associated with supporting the building's weight and contents. Aligning with the fundamental design principle of tolerating damage while striving to prevent collapse, the seismic forces prescribed by ASCE/SEI 7-22[11] are deliberately set lower than those required to ensure a structure remains undamaged during design earthquake shaking.

Typically, engineers engineer structures so that only specific structural elements (e.g., beams, columns, walls) and their connections furnish the necessary seismic resistance. Consequently, the braces, beams, and columns connected to the braces are designed to withstand seismic forces, while the remaining beams and columns are not. The ASCE/SEI 7-22[11] standard delineates the magnitude of earthquake design forces and the requisite combinations of seismic forces with other loads, encompassing dead and live loads, necessary for designing the seismic force-resisting system (SFRS).

The determination of specified earthquake forces and their calculation methodology varies based on the seismic design category (SDC), the structural system employed, the structure's configuration, and the type of element or connection being designed. Succinctly, for structures categorized as SDC A, ASCE/SEI 7-22[11] necessitates that structures be engineered with adequate strength to withstand lateral forces equivalent to 1% of the structure's weight applied in each direction at every level. Chapter 2 of the ASCE/SEI 7-22[11] specifies the required combinations of seismic loads with other design loadings, including dead and live.

2.1.1.4.1 Horizontal seismic loads

ASCE/SEI 7-22[11] requires the design of structures for a minimum base shear force (V), given by the expression (2-1):

$$V = C_s \cdot W \tag{2-1}$$

(C_s) is the seismic response coefficient, and (W) is the seismic weight of the structure.

The seismic response coefficient (C_s) depends on the structure's fundamental period of vibration (T), the risk category, and the type of SFRS used. For structures with fundamental periods of vibration less than the mapped value of (T_L) at their site, the seismic response coefficient (C_s) is taken as the lesser of the value given by expression (2-2) :

$$C_s = \frac{S_{DS}}{\frac{R}{I_e} \cdot T} \tag{2-2}$$

and equation (2-3):

$$C_s = \frac{S_{DS}}{\frac{R}{I_e} \cdot T} \tag{2-3}$$

where (S_{DS}) is the spectral response acceleration parameter obtained from the USGS database, (S_{D1}) is the spectral response acceleration parameter obtained from the USGS database, (R) is the response modification coefficient, (I_e) is an importance factor, the value of which depends on the risk category, and (T) is the fundamental period of vibration of the structure.

The seismic weight encompasses the structure's total weight and all permanent loads. The seismic weight (W) also incorporates 25% of the design storage load in storage and warehouse occupancies. Furthermore, the seismic weight includes 20% of the uniform design snow load for buildings with flat roofs in areas prone to ground snow loads.

The quantity (R/I_e) in the equations represents the allowable level of inelastic structural response or ductility. The value of (R) is derived from ASCE/SEI 7-22[11] based on the chosen seismic force-resisting system. In buildings classified under Risk Category I or II, the importance factor (I_e) is assigned a value of 1,0. The importance factors for structures falling within Risk Categories III and IV are elevated to 1,25 and 1,5, respectively. Consequently, structures in higher-risk categories have a stricter limitation on inelastic behaviour, aligning to minimize the risk of damage.

For structures with a fundamental vibration period exceeding (T_L), the (C_s) value is calculated using the specified equation (2-4):

$$C_s = \frac{S_{D1} \cdot T_L}{\frac{R}{I_e} \cdot T^2} \tag{2-4}$$

Irrespective of the fundamental period, the base shear coefficient for any structure cannot fall below the value derived from the subsequent equation (2-5):

$$C_s = 0{,}044 \cdot S_{DS} \cdot I_e \tag{2-5}$$

In areas adjacent to major active faults, where ground motion may display significant impulsive characteristics, the base shear coefficient must adhere to the minimum value specified in equation (2-6):

$$C_s = 0{,}5 \frac{S_1 \cdot T_L}{\frac{R}{I_e} \cdot T^2} \tag{2-6}$$

Regarding equation (2-6), the parameter (S_1) represents the spectral response acceleration parameter at a period of 1,0 seconds. It quantifies the expected Peak Ground Acceleration (PGA) at the site for a specific seismic hazard level under conditions corresponding to Site Class BC.

Designing the strength of structures categorized under SDC D, E, and F involves considering redundancy. A structure is deemed adequately redundant if removing any single element within the Seismic Force-Resisting System (SFRS), like a shear wall or brace, does not diminish the structure's lateral strength by over one-third and avoids causing extreme torsional irregularity. A thorough redundancy assessment is unnecessary if the SFRS configuration satisfies specific preset conditions. However, if a structure fails to meet these conditions or the minimum requirements for strength and irregularity, the redundancy factor (ρ) comes into play. In such cases, the required strength of all SFRS elements and their connections, excluding diaphragms, must be augmented by 30%.

2.1.1.4.2 Vertical seismic loads

Following ASCE/SEI 7-22[11], structures in Seismic Design Categories (SDC) C, D, E, and F must consider the impacts of vertical shaking. This mandate applies to all structural elements within these categories, irrespective of their inclusion in the designated Special Seismic Force-Resisting System (SFRS). Engineers must design these members to withstand vertical seismic forces, denoted as (E_v), for which two equations are available to calculate. Either equation (2-7):

$$E_V = 0{,}2 \cdot S_{DS} \cdot D \tag{2-7}$$

Or equation (2-8):

$$E_V = 0{,}3 \cdot S_{av} \cdot D \tag{2-8}$$

In the equations (2-7) and (2-8), (S_{DS}) is the horizontal design spectral acceleration at short periods, and (S_{av}) is the vertical spectral response acceleration at short periods, derived following Section 11.9.2 of ASCE/SEI 7-22[11]. The parament (D) is defined as the dead load.

2.1.1.5 STRUCTURAL ANALYSIS

As specified in ASCE/SEI 7-22[11], the seismic design forces acting on the structure are determined by one of the following methods:

- Equivalent static forces from acceleration response spectra for the horizontal earthquake motions
  i. Equivalent Lateral Force Analysis (ELF)
  ii. Simplified Equivalent Lateral Force (SELF)
- Dynamic analysis
  i. Modal Response Spectrum Analysis
  ii. Linear Response History Analysis
  iii. Nonlinear Response History Analysis method

The permitted analysis method depends on the building's Seismic Design Category (SDC), which varies from A to F and is determined from a combination of the structure's Seismic Use Group and the acceleration response spectrum. A Seismic Design Category of A corresponds to a low acceleration level, and F corresponds to a high acceleration level. The structure's geometry, such as height and regularity, limits the analysis method.

2.1.1.5.1 Equivalent static forces

The predominant method employed is the Equivalent Lateral Force (ELF) procedure. This approach distributes vertically the seismic base shear force (V) as a series of static forces across a mathematical structure representation. Each element's seismic demand (E) is then assessed and integrated with other specified forces, such as dead and live loads, to ascertain the necessary strength. The equation governing the static seismic design force applied at each level is expressed as depicted in (2-9):

$$F_i = \frac{w_i \cdot h_i^k}{\sum_{j=i}^{n} w_j \cdot h_j^k} \quad (2\text{-}9)$$

The exponent (k) in the above equation is assigned specific values based on the structure's fundamental period (T). For structures with a ($T \leq 0{,}5$ s), k=1; for ($T \geq 2{,}5$s; k=2); and for periods falling between these extremes, (k) is interpolated linearly. The fundamental period can be determined through approximate formulas related to the Special Seismic Force-Resisting System (SFRS) type or directly using structural dynamics methods accounting for mass and stiffness distribution. This equation aims to depict the inertial force distribution during free vibration across the structure's natural modes. The term within the equation approximates each dominant mode shape's relative amplitude and contribution at every level.

Seismic design procedures according to ASCE/SEI 7-22[11] necessitate independent computation and application of fundamental period (T), seismic base shear force (V), and individual story forces (F_i) in two primary orthogonal directions of response. While major vertical elements of the Special Seismic Force-Resisting System (SFRS) are typically aligned in these directions, structures may align them differently or in non-orthogonal axes. In Seismic Design Categories (SDC) C, D, E, and F, structures with vertical seismic force-resisting elements must account for significant seismic forces resulting from shaking in the central orthogonal building axis. For instance, structures with intersecting braced frames or moment frames aligned differently or with vertical elements aligned in multiple non-orthogonal directions require considering forces incident from any direction.

Accidental torsion, resulting from live load placement imbalances or differences in strength and stiffness of lateral elements, must be addressed to ensure adequate torsional resistance in structures. This analysis is crucial for torsionally irregular structures in (SDC) B and all structures in SDC C, D, E, and F lacking flexible diaphragms. In the Equivalent Lateral Force (ELF) method, lateral forces (F_i) are applied at a location displaced from the centre of mass to account for potential unbalanced loading during seismic events. These eccentric loads ensure structures have the minimum resistance to withstand torsional effects, maintaining structural integrity and safety.

In structures categorized under (SDC) B and those under (SDC) C, D, E, and F lacking flexible diaphragms, the analysis must consider the effects of accidental torsion. Within the Equivalent Lateral Force (ELF) method, accounting for accidental eccentricity involves applying lateral forces (F_i) at each level, displaced from the centre of mass by a distance equal to 5 per cent of the width perpendicular to the force's direction. This displacement ensures structures accommodate potential unbalanced loading scenarios during seismic events, such as occupancy imbalances between building sides. By requiring this eccentric application, the standard guarantees that all structures possess sufficient resistance to torsional effects, enhancing overall structural stability and safety.

As described earlier, the design seismic forces on some elements in irregular structures must be amplified by the over-strength coefficient (Ω_0). The purpose of design using these amplified forces is to avoid damage to elements whose failure could result in widespread damage and collapse of the structure. An example of such an element is the column beneath a discontinuous shear wall.

The simplified (ELF) method applies to low-rise structures with stiff seismic force-resisting systems using walls or braced frames. The base shear force equations, such as the

vertical force distribution formula, are simplified relative to those in the standard (SELF) method. Further, it is not required to compute story drift. However, the simplified method requires design for larger forces to ensure that assumptions used to create the simple procedure do not compromise safety.

2.1.1.5.2 Dynamic analysis

Modal response spectrum analysis bridges the gap between Equivalent Lateral Force (ELF) and dynamic methods, particularly for Multi-Degree-of-Freedom (MDOF) structures. MDOF structures possess as many natural vibration modes as they have dynamic degrees of freedom, each representing a unique direction of motion associated with an individual mass. In a two-dimensional MDOF structure like the one illustrated, with three masses, there are three dynamic degrees of freedom, each allowing independent lateral translation of the masses. Expanding this concept to a three-dimensional structure would entail nine dynamic degrees of freedom, including lateral displacement in two orthogonal directions and twisting about a vertical axis for each mass, as shown in Figure 2-1.

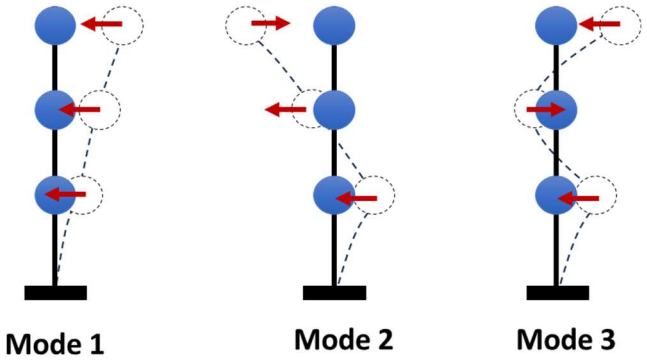


Figure 2-1: Mode shapes and inertial forces associated with free vibration

During seismic events, each mode of vibration in the structure is excited, resulting in inertial forces on the masses. Modal response spectrum analysis involves creating a mathematical model of the structure incorporating its geometry, stiffness, and mass distribution. This model determines the natural periods and mode shapes, which, together with the design response spectrum for the site, are used to calculate modal forces on the structure. These forces are then combined, typically using a square root sum of squares approach, to estimate maximum combined forces, considering that the maximum forces in each mode may not coincide. However, despite its potential for less conservative design forces, modal response spectrum analysis has drawbacks. The root sum of squares combination results in a loss of sign for forces and displacements, which some engineers find disadvantageous. Nevertheless, this method remains valuable in seismic design, ensuring structures meet safety requirements by scaling up forces to match those obtained from (ELF) analysis, thus ensuring structural integrity during seismic events.

Another approach ASCE/SEI 7-22[11] permitted is the Linear Response History Analysis (LRHZA), which involves constructing a mathematical model of the structure and subjecting it to a set of ground motions scaled or matched to the site's design response spectrum. Computer software then performs numerical integration of the structure's equation of motion, determining maximum forces in each member and connection for each ground motion. These forces are averaged and scaled to match the total base shear force obtained from the ELF method. Like

ELF methods, these forces are combined with other loads after scaling. Linear response history analysis (LRHZA) offers advantages over modal response spectrum analysis as it retains the sign of earthquake forces and displacements, which is beneficial for connection design. Additionally, it can reduce conservatism associated with the root sum of squares summation approach used in modal response spectrum analysis. However, it is computationally complex and requires manipulation of results from a suite of ground motions, demanding more effort in analysis and interpretation.

The Nonlinear Response History Analysis (NLRHA) is the fourth permitted analysis method according to ASCE/SEI 7-22[11]. It shares similarities with linear response history analysis. However, with a crucial distinction: throughout the analysis, the stiffness of members and connections is adjusted to simulate various types of damage, such as cracking, yielding, and buckling. (NLRHA) is a sophisticated technique that calculates the forces and deformations induced in a structure by subjecting it to a suite of earthquake records. It explicitly considers the dynamic properties of the structure and the damage caused by earthquake response.

During (NLRHA), members and connections are assessed in two categories. Those with inherent ductility and the ability to yield while still carrying load are evaluated based on the level of nonlinear deformation predicted by the analysis. On the other hand, elements with limited or no ductility are assessed based on the force predicted by the ground motions. This method is commonly employed in performance-based design approaches, especially for high-rise buildings and structures equipped with energy dissipation systems or seismic isolation, as described by Metually[13].

2.1.1.6 DRIFT AND STABILITY

It is essential to assess structures to ensure that their expected lateral deflection during earthquake shaking remains within acceptable limits and does not lead to (P-Δ) instability. This evaluation comprises two critical assessments:

- Analysis of the adequacy of the interstorey drift at each level.
- Evaluation of stability (P-Δ) effects.

Interstorey drift refers to the displacement of one floor or roof level about the floor level directly below it under load. Usually quantified as a ratio, it is calculated by dividing the difference in deflection between two adjacent floors by the height of the story that separates them. This concept is depicted in Figure 2-2, where the quantity di represents the drift induced by applying design seismic forces.

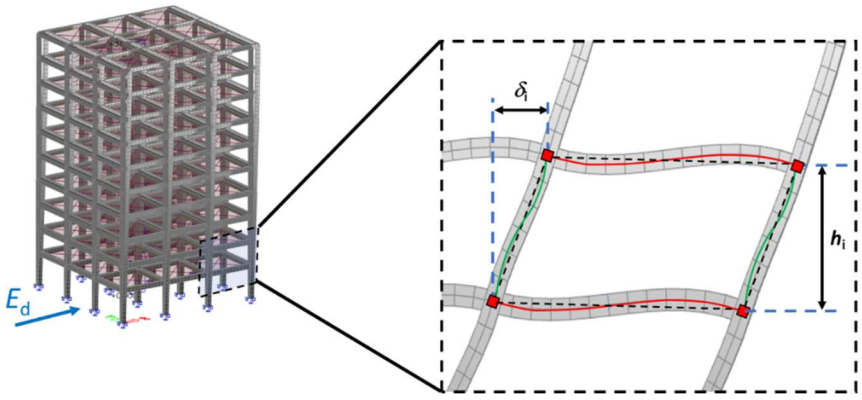


Figure 2-2: Storey drift (Adapted from Maranhão et al.[15])

ASCE/SEI 7-22[11] establishes maximum allowable story drift limits depending on the risk category and construction type. The suitability of a structure in this regard is assessed by computing the design story drift (Δ) utilizing the equation (2-10):

$$\Delta = \frac{C_d \cdot \delta_i}{I_e} \leq \Delta_a \cdot h_i \tag{2-10}$$

where (δ_i) is the computed story drift under the influence of the design seismic forces, (C_d) is the deflection amplification coefficient described above, and (I_e) is the occupancy importance factor. The acceptable drift ratio (Δ_a) varies from 0,007 to 0,025 depending on the risk category of the building and construction type.

In addition, ASCE/SEI 7[11] requires an evaluation of structural stability under the anticipated lateral deflection by calculating the quantity (θ) for each storey. To determine the severity of (P-Δ) effects on the structure, the coefficient (θ) is calculated at each story according to the equation (2-11):

$$\Delta = \frac{P_x \cdot \Delta}{V_x \cdot h_x \cdot C_d} \tag{2-11}$$

where (P_x) is the total vertical design load at and above the level, (Δ) is the design story drift, (V_x) is the story shear, and (h_x) is the story height of the level below, shown in Figure 2-3.

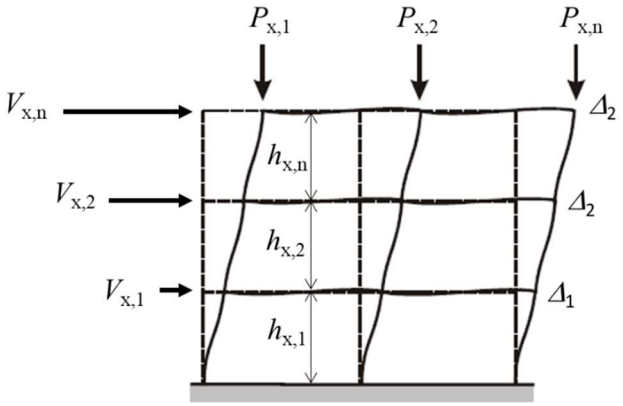


Figure 2-3: (P-Δ) effects on a moment-resistant frame

If ($\theta \leq 0{,}10$) at each storey, the structure is considered to have adequate stiffness and strength to provide stability. The structure must be redesigned if ($\theta \geq \theta_{max}$) where ($\theta_{max}=0{,}5/\beta \cdot C_d$) and where ($\beta$) is the ratio between the story shear and story capacity of the level below. In the equation for(θ_{max}), (β) is calculated as the ratio of the story shear demand under the design seismic forces to the story shear strength. For values between $\theta=0{,}10$ and θ_{max}, the design story drift should be amplified by $1{,}0/(1{,}0-\theta)$.

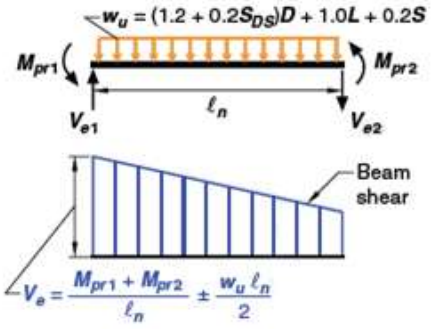
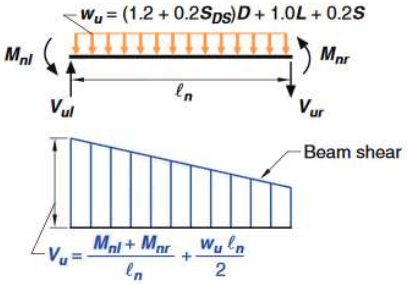
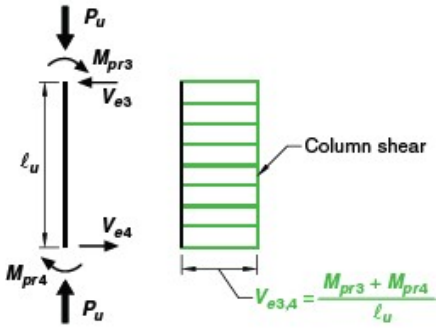
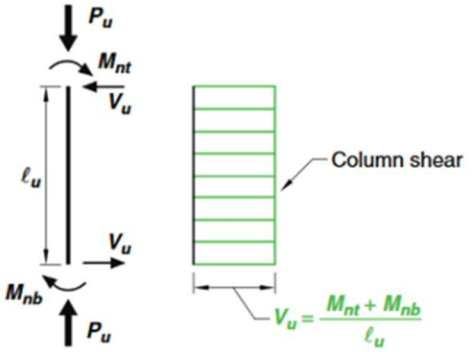
2.1.1.7 THE CAPACITY DESIGN APPROACH

The United States codes for concrete do not have a complete capacity design approach, but they do require some form of capacity design for the following areas of moment frames:

- Determining beam and column shear forces.
- Requiring strong column/weak beam behaviour.

The ACI 318-19[12] provisions attempt to ensure that beams and columns of moment-resisting frames yield a ductile, flexural behaviour at member ends rather than a brittle, shear behaviour. The provisions also attempt to reduce the likelihood of yielding in the columns above the base of the moment frame, which could lead to a story mechanism. The design shear force of beams in special and intermediate moment-resisting frames is calculated assuming that the beams form flexural plastic hinges at each end, as shown in Table 2-4. The shear forces corresponding to the formation of the plastic hinges are added to the gravity shears.

Table 2-4: Capacity design in shear (Adapted from ACI 318-19[12])

Element	Special moment-resisting frames	Intermediate moment-resisting frames
Beams		
Columns		

For beams, special moment-resisting frames (M_{pr1}) and (M_{pr2}) are the flexural strengths of the beam at each end based on a steel tensile strength of ($1{,}25 \cdot f_y$) to account for steel overstrength and strain hardening, l_n is the clear span of the beam, and (w_u) is the shear due to gravity loads. The parameters (M_{pr3}) and (M_{pr4}) are the flexural strengths of the column at the top and bottom based on a steel tensile strength of ($1{,}25 \cdot f_y$) to account for steel overstrength, strain hardening, and (l_u) is the clear span of the column.

To promote strong column/weak beam behaviour and reduce the likelihood of a story mechanism, the flexural strengths of columns of special moment resisting frames must satisfy the following expression (2-12):

$$\sum M_{nc} = \frac{6}{5} \cdot \sum M_{nb} \qquad (2\text{-}12)$$

where (ΣM_{nc}) is the sum of the nominal flexural strengths of columns at the faces of the joint.

Flexural strengths shall be calculated at the factored axial force, and (ΣM_{nb}) is the sum of nominal flexural strengths of beams at the faces of the joint. For ordinary moment resisting

frames there are no capacity design requirements for the design of ordinary moment-resisting frames.

2.1.1.8 DETAILING FOR LOCAL DUCTILITY OF BEAMS AND COLUMNS OF CONCRETE STRUCTURES

Concerning the concrete structures, ACI 318-19[12] provides detailed requirements for the reinforcement and detailing of seismic structures to ensure adequate local ductility. Specifically, it includes provisions related to the development length, splice length, anchorage, and confinement of reinforcement in critical regions such as beam-column joints, beam ends, and column ends. These provisions aim to enhance the ductility and energy dissipation capacity of reinforced concrete members subjected to seismic forces. Additionally, ACI 318-19[12] emphasizes the importance of proper detailing to mitigate potential brittle failure modes and ensure structures' overall seismic performance and resilience.

2.1.1.8.1 Materials

In the United States, the minimum compressive strength of concrete (f_c), generally at age 28 days, is 21 MPa. There is no upper limit on the strength of normal-weight concrete ACI 318-19[12]. However, the maximum strength of lightweight concrete is limited to 35 MPa due to the lack of experimental data on the performance of members made with lightweight concrete subjected to cyclic behaviour in the non-linear range.

Reinforcement for frame members or wall boundary elements subject to axial or flexural loads is required to have a specified yield strength of 276 MPa or 414 MPa. An actual yield strength that does not exceed its specified yield strength by more than 124 MPa per ACI 318-19[12]. The ultimate tensile strength shall not exceed 1,25 times the actual yield strength. There is no upper limit on the actual yield strength in other structural elements.

2.1.1.8.2 Design provisions for beams and columns - Detailing

This subchapter presents the main detailing exigences for seismic detailing according to ACI 318-19[12] for beams and columns regarding the special and intermediate moment-resisting frames for the present work. The following table summarizes the detailing exigencies for the beams according to the type of frame structure specified in 18.4 and 18.6, part 5, “Earthquake Resistance” of the ACI 318-19[12].

Table 2-5: Detailing provisions for beam (ACI 318-19[12])

Beams		Special moment-resisting frames	Intermediate moment resisting frame
Clear Span		$4 \cdot d$	-
Minimum width	$b_w >$	$0{,}3h$	-
height	$h <$	(l/21) both end continuous	
Minimum reinforcement ratio	$A_{s,min}$	$max\left(1{,}4\frac{b_w \cdot d}{f_y}; 0{,}25 \cdot \frac{\sqrt{f'_c} \cdot b_w \cdot d}{f_y}\right)$	
Maximum longitudinal reinforcement ratio	ρ_{max}	0,025	-
Maximum stirrup spacing within the plastic hinge zone	$s_w \leq$	min(;d/4;6·d_b; 6 in)	min(2·h;d/4,8·d_{bL};24·d_{bw})
Maximum stirrup spacing out plastic hinge zone	$s_w \leq$	$d/2$	$d/2$

The following table summarizes the detailing exigences for columns according to the frame structure type.

Table 2-6: Detailing provisions for columns (ACI 318-19[12])

Columns		Special moment-resisting frame	Intermediate moment resisting frame
Plastic hinge length	l_0	(h_c, l_u/6; 450mm)	(h_c, l_u/6; 450mm)
Minimum longitudinal reinforcement ratio	ρ_{min}	1%	1%
Maximum longitudinal reinforcement ratio	ρ_{max}	6%	6%
Minimum bars per side		3	3
Spacing between restrained bars	s	200	200
Distance of unrestrained bar from nearest restrained bar		150	150
Spacing	$s_w \leq$	min[0,25·h_w;6·$d_{bL;}$(100·(350-hx/3)]	min[0,5·h_w;8·$d_{bL,;}$24·d_{bw};300mm

The transverse reinforcement in columns of ductile frames must consist of hoops with 135-degree hooks. Where cross ties are required, the cross tie must engage a longitudinal bar at both ends and have a 135-degree hook at least at one end. Where one end of the crosstie has a 90-degree hook, the location of the 90-degree hook should alternate.

2.1.1.9 DESIGN AND DETAILING PROVISIONS FOR CONCRETE WALL STRUCTURES

In ACI 318-19[12], the design and detailing of concrete shear walls are extensively covered to ensure adequate strength, stiffness, and ductility under lateral loads, particularly seismic forces.

The design of shear walls is obtained from lateral load analysis with appropriate load factors increased to account for:

- Flexural overstrength at critical sections, where yielding of longitudinal reinforcement is expected.
- Dynamic amplification due to higher mode effects.

Because (M_n) and (M_{pr}) depend on axial force, which varies for different load combinations, and loading direction for flanged and coupled walls, the condition producing the largest value of (Ω_v) should be used as depicted in Figure 2-4. Dynamic amplification is not significant in walls with (h_w/l_w<2). A limit of (0,007·$h_{w,CS}$)is imposed on ns to account for buildings with large story heights. The application of (Ω_v) to (V_u) does not preclude the application of a redundancy factor if required by ACI 318-19[12].

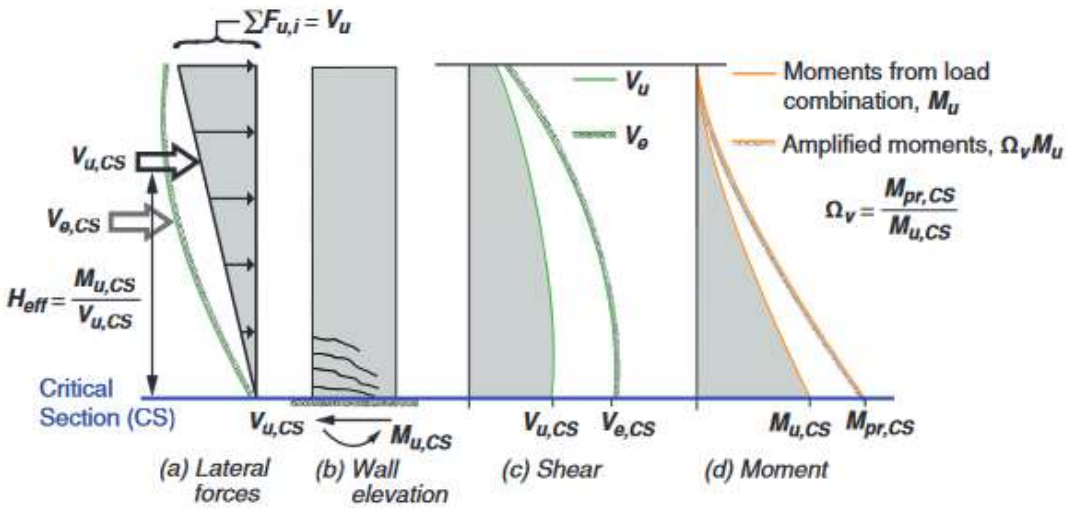


Figure 2-4: Determination of shear demand for walls (Adapted from ACI 318-19[12])

The flexural strength of a wall or wall segment is determined according to procedures commonly used for columns. Strength should be determined considering the applied axial and lateral forces. Reinforcement concentrated in boundary elements and distributed in flanges and webs should be included in the strength calculations based on a strain compatibility analysis.

The ACI 318-19[12] specifies requirements for the minimum amount of reinforcement, spacing, and detailing of reinforcement in shear walls. This includes detailing for vertical and horizontal reinforcement, boundary elements, and special detailing considerations for boundary zones and wall openings.

Design provisions for vertical wall segments depend on the aspect ratio of the wall segment in the plane of the wall (h_w/l_w) and the aspect ratio of the horizontal cross-section (l_w/b_w), where h_w is the height of the wall, (l_w) is the length and (b_w) is the width. The distributed web reinforcement ratios, (ρ_l) and (ρ_t), shall be at least 0,0025 for structural walls. Reinforcement spacing each way in structural walls shall not exceed 457,2 mm. Reinforcement contributing to (V_n) shall be continuous and distributed across the shear plane.

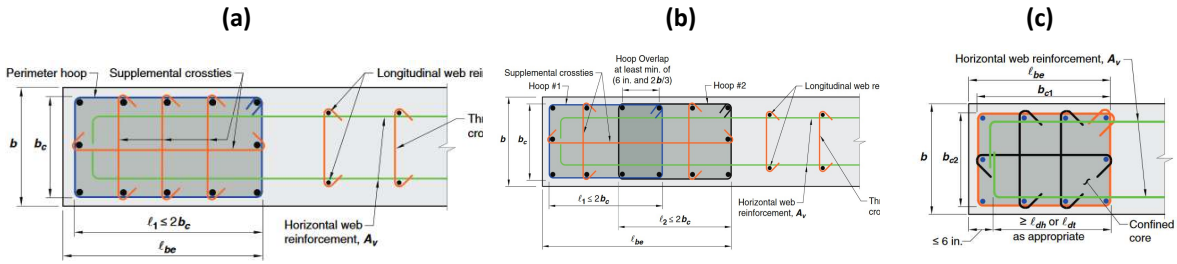


Figure 2-5: Configurations of boundary transverse reinforcement and web crossties (Adapted from ACI 318-19[12])

ACI 318-19[12] provides guidelines for using shear reinforcement, such as stirrups or inclined bars, to enhance the shear capacity and ductility of concrete shear walls. The detailing requirements ensure that the shear reinforcement is anchored and spaced to provide effective confinement and prevent brittle shear failure. The requirement for distributed shear reinforcement is related to the intent to control the width of inclined cracks.

Proper anchorage of reinforcement is critical to ensure ductile behaviour and prevent premature failure of shear walls under lateral loads. ACI 318-19[12] specifies requirements for development length, lap splicing, and anchorage detailing to achieve the required strength and ductility of the reinforcement.

Special detailing provisions are included for boundary elements of shear walls, such as coupling beams, chord reinforcement, and boundary columns. These elements are crucial in distributing lateral forces and maintaining overall structural stability. This considers that concrete spalling is likely to occur only on the exposed faces of the confined boundary element. The limits on the spacing between laterally supported longitudinal bars are intended to provide a more uniform spacing of hoops and crossties for thin walls. Configuration requirements for boundary element transverse reinforcement and crossties for web longitudinal reinforcement are illustrated in Figure 2-5.

According to Segura et al.[16], a limit is placed on the relative lengths of boundary element hoop legs because of tests that a single perimeter hoop with supplemental crossties with alternating 90-degree and 135-degree hooks is not as effective as overlapping hoops and crossties with seismic hooks at both ends if it exceeds approximately ($2 \cdot b_w$). These tests also show that loss of axial load-carrying capacity of a wall can occur immediately following damage to the wall boundary elements if web vertical reinforcement within the plastic hinge region is not restrained. Using web crossties outside boundary elements also results in a less abrupt transition in transverse reinforcement used to provide concrete confinement and restrain buckling of longitudinal reinforcement, which addresses potential increases in the neutral axis depth due to shear (diagonal compression) and uncertainties in axial load, according to Mohele et al.[17]. The horizontal reinforcement in a structural wall with a low shear-to-moment ratio resists shear through truss action, with the horizontal bars acting like the stirrups in a beam, according to ACI 318-19[12].

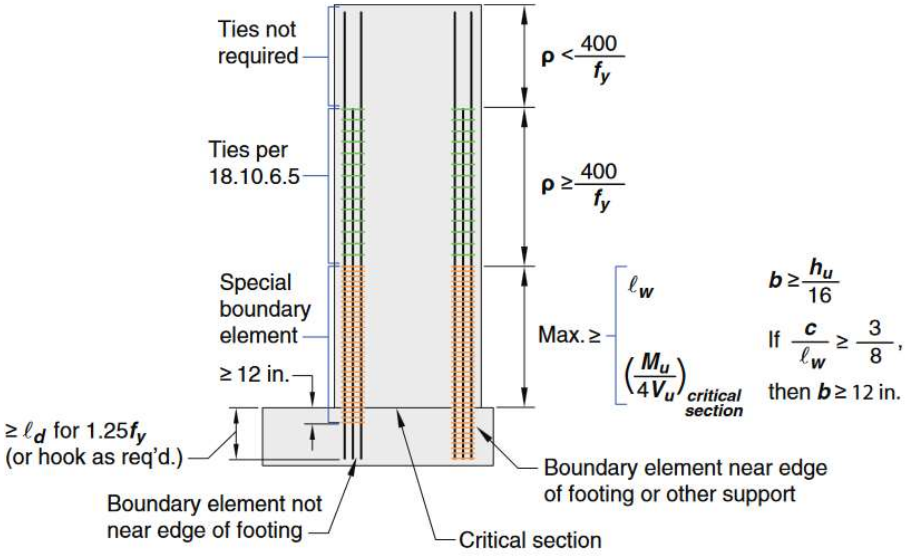


Figure 2-6: Summary of boundary element requirements for special walls (adapted from ACI 318-19[12])

Thus, the horizontal bars provided for shear reinforcement must be developed within the confined core of the boundary element and extended as close to the end of the wall as cover requirements and proximity of other reinforcement permits. The horizontal web reinforcement

must be anchored within the confined core of the boundary element and extended to within 6 in. from the end of the wall, which applies to all horizontal bars, whether straight, hooked, or headed, as illustrated in Figure 2-6.

2.1.2 NEW ZEALAND SEISMIC CODE

During the last four decades, much attention has been given to earthquake engineering, and seismic provisions now dominate most structural design procedures in New Zealand.

The early developments in New Zealand standards for the seismic design of reinforced concrete buildings were greatly influenced by the Structural Engineers Association of California's recommendations and the American Concrete Institute. However, research and development in New Zealand have led to several differences between the United States and New Zealand standards. This has been due to different interpretations of the test date, models of behaviour, and performance criteria. For example, research and development in New Zealand have led to the capacity design approach and provisions for the seismic design of moment-resisting frames and structural walls. In addition, since 1992, seismic design forces have been reduced below those of most other countries of similar seismic activity. As a result, secondary actions, such as P-Δ effects, are much more significant than previously. Recently, New Zealand standards have emphasised designing for local deformation in structures. Park[18] has previously summarised the development of seismic design standards for reinforced concrete in New Zealand.

The NZS 1170.5:2004[19] is a New Zealand Standard titled "Structural Design Actions - Part 5: Earthquake Actions - New Zealand. It is a comprehensive document that provides guidelines and specifications for determining the earthquake actions that structures in New Zealand should be designed to withstand. Key aspects covered by NZS 1170.5:2004[19] include:

- **Seismic Hazard Assessment**: The standard outlines methods for assessing the seismic hazard at a particular location in New Zealand. This involves considerations such as the region's seismicity, ground shaking characteristics, and the likelihood of earthquakes of various magnitudes occurring.
- **Design Spectra**: It provides spectra that define the acceleration response of the ground motion for different seismic hazard levels. These spectra determine the seismic loads that structures need to resist.
- **Site Characteristics**: NZS 1170.5:2004[19] considers the influence of soil conditions on seismic response. Different soil types can significantly affect the amplification of ground motion and, therefore, the seismic demands on structures.
- **Structural Design Requirements**: The standard specifies design actions and load combinations for various types of structures, considering factors such as building occupancy, structural materials, and construction methods.

For the design of concrete structures, there is a more specific New Zealand Concrete Structures Standard, the NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21]. In New Zealand, as in other seismic countries, buildings are generally designed for a seismic action less than that corresponding to elastic response in the design earthquake. This requires the structure to dissipate seismic energy by ductile post-elastic deformation. Buildings should have “adequate ductility”, which can be achieved through “capacity design”. The capacity design aims to impart the structure features, resulting in the most desirable behaviour in a severe earthquake. This is achieved by assuring an appropriate hierarchy of strength of members and joints. The capacity design approach was developed in New Zealand. It was recognised that uncertainty exists in selecting analytical models of the structure and in the characteristics of the earthquake ground shaking. Through capacity design, the significance of these uncertainties is

considerably reduced. The capacity design approach was first introduced in New Zealand in 1976.

In the Structural Concrete Standard NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21], the maximum permissible structural ductility factor depends upon the structural form. This value determines the design forces and the expected ultimate deformations. The inelastic regions (plastic hinge zones) are assessed to determine the expected level of deformation in each inelastic area. This value determines the required level of detailing, as referred to in FIB-FIP-CEB 69[5].

2.1.2.1 STRUCTURAL SYSTEM

The NZS 1170.5:2004[19] standard provides guidelines for structural design actions due to earthquakes in New Zealand. It preconizes various structural systems to ensure buildings can withstand seismic forces effectively. Some of the structural systems recommended in NZS 1170.5:2004[19] include:

- Moment-resisting frames (MRF): These frames primarily resist lateral forces through flexural resistance at beam-to-column connections.
- Braced frames: These frames include diagonal or cross-bracing elements that resist lateral loads through tension and compression in the braces.
- Shear walls: Vertical elements designed to resist lateral loads through their shear capacity. Shear walls can be reinforced concrete, masonry, or structural steel.
- Dual systems: Combining two or more structural systems to enhance overall seismic performance. For example, a building might utilize both moment-resisting frames and braced frames.

It should be noted that these structural systems are also referred to intrinsically in NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21].

2.1.2.2 STRUCTURAL TYPES - CATEGORIES OF REINFORCED CONCRETE STRUCTURES

The Standard for Earthquake Actions NZS 1170.5:2004[19], NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21] classify, define the structural ductility and specify values for the structural ductility factor and design procedures for the following three categories of reinforced concrete structures, as present in Table 2-7.

Table 2-7: Structural ductility category classification (NZS 1170.5:2004[19] and NZS 3101:Part 1:2006[20] and NZS 3101:Part 2:2006[21])

Ductility category	Anacronym	Description
Nominally Ductile Structures	NDS	Structures expected to respond essentially in the elastic range at the ultimate limit-state are exempt from special seismic design requirements, providing that appropriate energy dissipating mechanisms may form under seismic actions that are more significant than assumed.
Structures of Limited Ductility	SLD	Structures that are expected to respond with limited ductility demand. These are intermediate between nominally ductile structures and ductile structures. They are expected to sustain some inelastic deformation under an ultimate limit-state earthquake.
Ductile Structures	DS	Structures expected to respond in a ductile manner at the ultimate limit- state are designed for a high level of ductility.

The structural ductility factor determines the seismic design forces and the design deflections in the structure. However, the regions where inelastic deformation is intended to develop in the event of a significant earthquake are assessed in terms of the predicted deformation each must sustain. On this basis, the level of detailing, namely nominally ductile, limited ductile or ductile detailing, needed for that region is selected as referred to in FIB-FIP-CEB 69[5].

2.1.2.3 STRUCTURAL REGULARITY

NZS 1170.5:2004[19] defines structural irregularities as deviations from regularity in a building or structure's configuration, stiffness, strength, and mass distribution. These irregularities can significantly affect the structure's response to seismic forces and may lead to unexpected behaviour during earthquakes.

According to NZS 1170.5:2004[19], a structure's configuration can significantly affect its performance during a strong earthquake. Configuration can be divided into two aspects: vertical configuration and plan configuration. Past earthquakes have repeatedly shown that structures with irregular configurations suffer greater damage than structures with regular configurations. This situation prevails even with good design and construction. These provisions encourage the designer to design structures with regular configurations. The NZS 1170.5:2004[19] defines two types of irregularities: vertical regularities and plan irregularities.

According to NZS 1170.5:2004[19], the vertical configuration irregularities affect the responses at the various levels and induce loads at these levels significantly different from the predominantly first mode distribution assumed in the equivalent static analysis method. Examples of vertical irregularities are shown in Figure 2-7 and are specified as follows:

- A moment resisting frame structure might be classified as having a vertical irregularity if one storey were much taller than the adjoining storeys and the resulting decrease in stiffness that would typically occur was not, or could not be, compensated for as illustrated in as depicted in Figure 2-7(a).
- A structure would be classified as irregular if the ratio of mass to stiffness in adjoining storeys differs significantly. This might occur when a heavy mass, as Figure 2-7(b) depicts.
- Vertical irregularity is also created by unsymmetrical geometry concerning the vertical axis of the structure, as depicted in Figure 2-7(c). The structure would also be considered irregular if the smaller dimension were below the larger one, creating an inverted pyramid effect.
- The structure may have a symmetrical geometry about the vertical axis and still be classified as irregular because of significant horizontal offsets in the vertical elements of the horizontal force-resisting system at one or more levels.
- The problem of concentration of energy demand in the resisting elements in a storey due to abrupt changes in strength capacity between storeys has been noted in past earthquakes.

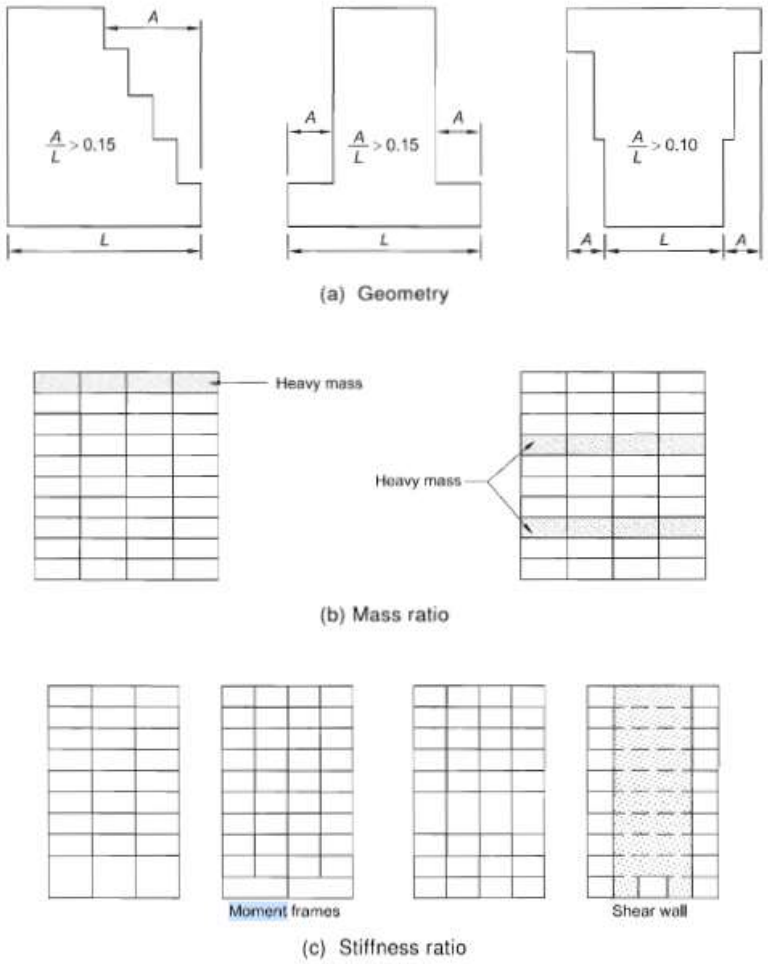


Figure 2-7: Vertical irregularities (Adapted from NZS 1170.5:2004[19])

According to NZS 1170.5:2004[19], variation in plan dimensions, stiffness, or strength along the horizontal plane leads to torsional effects or uneven distribution of seismic forces. Examples of plan irregularities are shown in Figure 2-8 and specified as follows:

- A structure may have a symmetrical geometric shape without re-entrant corners or wings. However, it can still be classified as irregular in the plan because of the distribution of mass or vertical earthquake-resisting elements.
- A regular configuration structure can be square, rectangular, or circular. A square or rectangular structure with minor re-entrant corners would still be considered regular, but large re-entrant corners creating a crucifix form would be classified as irregular. The response of the wings of this type of structure is generally different from the response of the structure as a whole, and this produces higher local forces than would be determined by the application of the Standard without modification. Other plan configurations, such as H shapes with geometrical symmetry, would also be used.
- Significant differences in stiffness between portions of a diaphragm at a particular level are classified as irregularities since they may cause a change in the distribution of horizontal earthquake forces to the vertical components and create torsional forces not accounted for in the standard distribution considered for a regular structure.

- The structure can no longer be considered regular where discontinuities exist in the horizontal force resistance path. The most critical discontinuities to be considered is the out-of-plane offset of vertical elements of the horizontal earthquake force-resisting elements. Such offsets impose vertical and horizontal load effects on horizontal elements that are, at the least, challenging to provide adequately.
- Where vertical elements of the horizontal force resisting system are not parallel to, or symmetrical to, principal orthogonal axes, the static horizontal force procedures cannot be applied as given, and, thus, the structure should be considered irregular.

There is a type of distribution of vertical force-resisting components that, while not classified as irregular, do not perform well in strong earthquakes. This arrangement is a core-type building with the vertical components of the horizontal earthquake-resisting system concentrated near the centre of the structure. Better performance has been observed when these vertical components are distributed near the structure's perimeter.

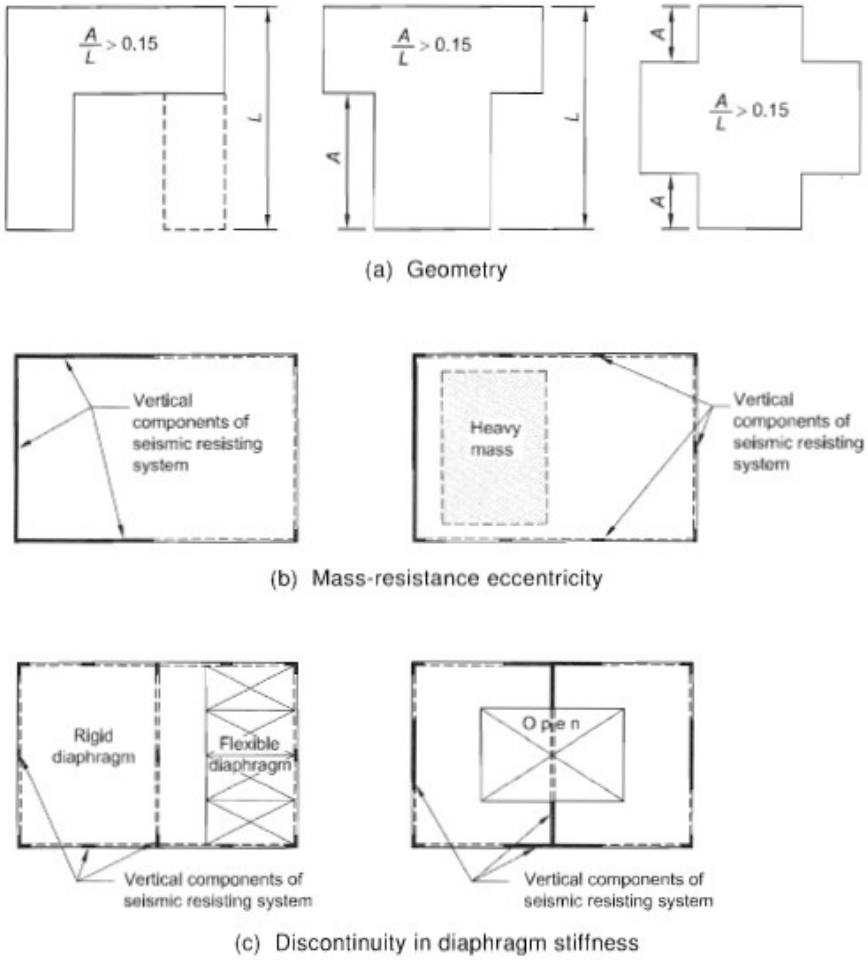


Figure 2-8: Plant irregularities (adapted from NZS 1170.5:2004[19])

2.1.2.4 SEISMIC ACTIONS

According to NZS 1170.5:2004[19], the elastic site spectra for horizontal loading in New Zealand are derived from a probabilistic seismic hazard model developed by the Institute of Geological and Nuclear Sciences (GNS). This model incorporates 305 active faults and a grid of distributed seismicity sources. The hazard analysis uses a "magnitude-weighting" approach to account for the increased damage potential with higher magnitude earthquakes. The elastic site spectrum, denoted as $C(T)$, is a product of various factors, including spectral shape factor, hazard factor, return period factor, and near-fault factor. The hazard factor is calculated using the GNS probabilistic seismic hazard model. Return period factors adjust the spectrum for different return periods, while the near-fault factor accounts for effects near active faults. This factor varies from 1 when the site is far from active faults to different values when the site is within 20 km of active faults for periods greater than 1,5 seconds.

The spectral shape factors, $C_h(T)$, are tailored differently for the equivalent static method compared to the modal response spectrum (MRS) or numerical integration time history (NITH) methods. For the MRS and NITH methods, $C_h(T)$ is defined based on smooth approximations of the shapes of estimated hazard spectra for various site classes. These approximations are provided in equation form. However, for the equivalent static method, the coefficients for periods less than 0,4 s are set as the 0,4 s values across the entire period band from 0 s to 0,4 s. This adjustment is made to address difficulties in accurately estimating short fundamental periods. The corner period (T_c) at the long-period end of the plateau at the spectrum's peak varies based on the site class. For site classes A (strong rock), B (rock), and C (shallow soil), the corner period is 0,3 s for the MRS and NITH methods. However, for site class D (deep or soft soil), the corner period is 0,56 s; for site class E (very soft soil), it is 1,0 s.

In order to simplify the standard, a single spectral shape has been chosen for all rock sites in New Zealand. The attenuation model that derives these spectra primarily relies on data from class B rock sites. Sites with very low-velocity soils (V_s<150 m/s) were excluded from the analysis during the development of the attenuation models. As such, the amplifications for class D may not be suitable for such sites. The corner period for class D sites has been extended to 1 second to address this. This adjustment increases the long-period branches (beyond 1,0 second) for class D sites by a ratio of 1,55 compared to class D, consistent with the ratio of long-period site factors for classes D and E in the NEHRP 2000 code.

Initially, consideration was given to scaling up the plateau of the spectrum from class D to class E using factors. However, comparisons with recorded motions suggested that amplification at class E sites was mainly significant in longer-period ranges. Therefore, the amplitude of the short-period plateau and peak ground accelerations are considered the same for classes D and E.

2.1.2.4.1 Horizontal design action coefficients and design spectra

For the serviceability limit state, seismic actions are determined from scaled elastic design spectra or elastic seismic acceleration coefficients. Similarly, a scaled elastic design spectrum is employed for the ultimate limit state when using modal response spectrum or numerical integration time history methods. The design action coefficient is defined according to the expression (2-13):

$$C_d(T)/S_p \tag{2-13}$$

In the equivalent static method, actions and displacements for the ultimate limit state may be derived by scaling those for the serviceability limit state. However, it is noted that for fundamental periods exceeding 0,7 seconds and structural ductility factors of 6, this scale factor typically equals unity. This indicates that, provided P-Δ actions are minimal, the serviceability limit state actions may govern minimum strength levels for periods above 0,7 seconds and structural ductility factors above 6. Inelastic lateral design action coefficients are obtained by factoring elastic coefficients (μ) and assuming equal displacement theory for all structural periods. While this approach may underestimate inelastic response for structural periods under 0,7 seconds, it is deemed acceptable given other inaccuracies in deriving inelastic response and, for simplification, a common practice in international standards.

For the ultimate limit state, a transition zone between equal energy and equal displacement is utilized for site subsoil classes A, B, C, and D up to a period of 0,7 seconds. However, for class E site subsoil, the corner point is extended to 1,0 seconds to maintain spectral ordinates greater than those for other site subsoil classes at periods less than 1,0 seconds.

Regarding the serviceability limit state, (S_p) is set at 0,7, where (S_p) is the structural performance factor, which accounts for the enhanced stiffness and strength of actual buildings compared to structural models generally limited to primary structural members. While this approach differs from the basis used elsewhere in the standard, it provides a practical means of recognizing these additional effects.

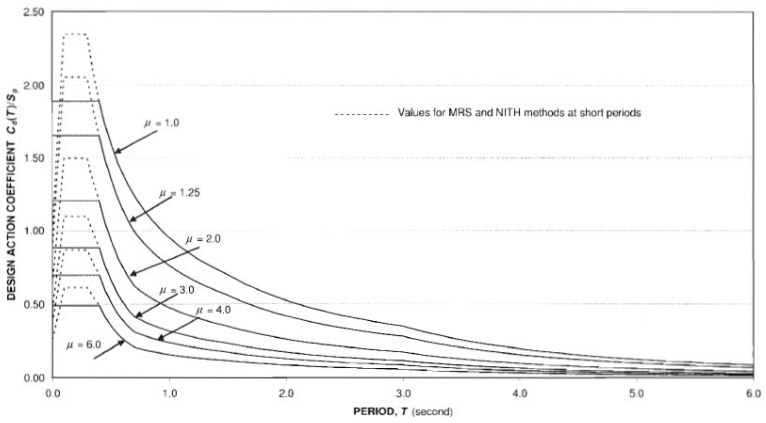


Figure 2-9: Horizontal design earthquake action coefficients for soil type A and B (adapted from NZS 1170.5:2004[19])

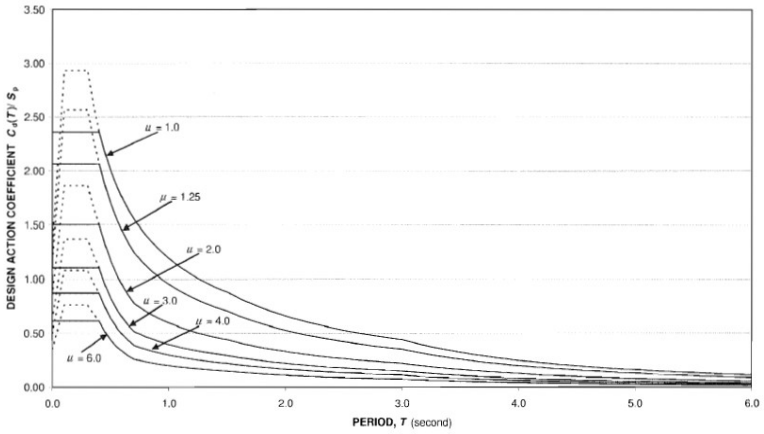


Figure 2-10: Horizontal design earthquake action coefficients for soil type C (adapted from NZS 1170.5:2004[19])

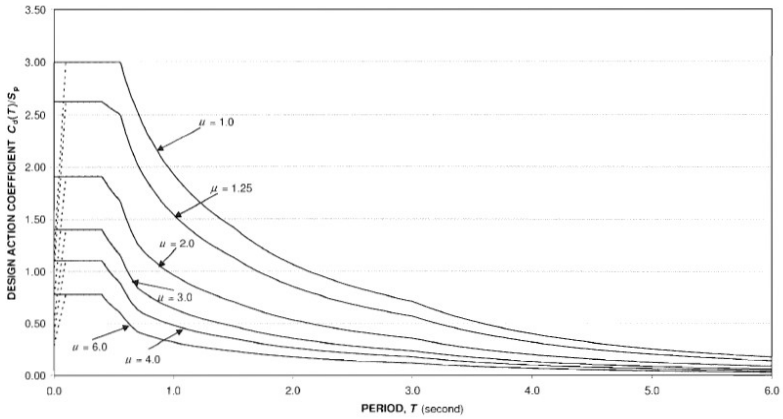


Figure 2-11: Horizontal design earthquake action coefficients for soil type D (adapted from NZS 1170.5:2004[19])

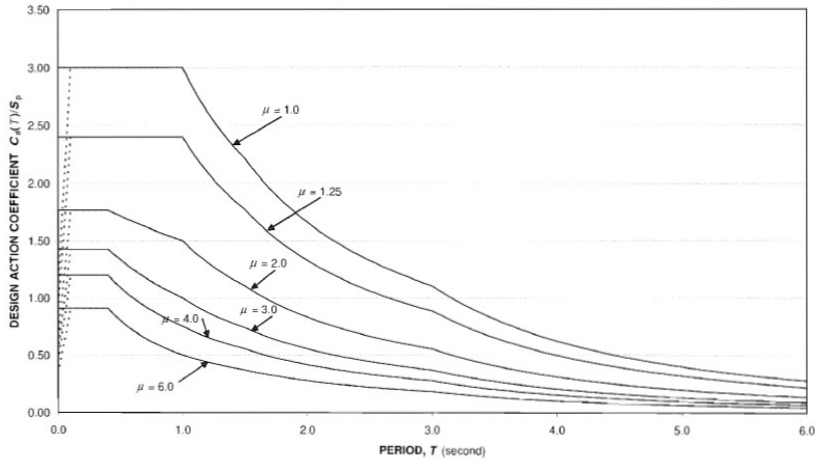


Figure 2-12: Horizontal design earthquake action coefficients for soil type E (adapted from NZS 1170.5:2004[19])

The occurrence of post-elastic deformations in a reinforced concrete structure during a severe earthquake implies a degree of damage will occur in the yielding regions. The main emphasis of seismic design standards in New Zealand in the past has been life safety. The performance criteria specified by the NZS 1170.5:2004[19] for seismic design are:

- Serviceability Limit State (SLS)
- Ultimate Limit State (ULS)

The serviceability limit state is concerned with avoiding damage to structural and non-structural components that would prevent the structure from being used as intended initially without repair in a serviceability level earthquake. The required strength and stiffness requirements for ordinary buildings to achieve this are based on earthquakes with a return period of 25 years. In Ultimate Limit States, the structure of normal-use buildings should have sufficient stiffness, strength and ductility to provide a high margin of safety and protection against loss of life in or around a building subjected to an earthquake with a return period of

500 years. In addition, the strength and inelastic deformation capacity are intended to be sufficient to provide a margin of safety against collapse in the event of an earthquake with a return period of 2500 years.

2.1.2.5 STRUCTURAL ANALYSIS

As specified in NZS 1170.5:2004[19], the seismic design forces acting on the structure are determined by one of the following methods:

- Equivalent Static Method (ESM)
- Modal Response Spectrum Analysis (MRSA)
- Numerical integration time History Analysis (NITH)

2.1.2.5.1 Equivalent Static Method

Regarding the Equivalent Static Method (ESM), the Earthquake loads skew to two orthogonal principal axes may be assessed using the formula:

$$V_\theta = V_x \cdot \cos^2\theta + V_y \cdot \sin^2\theta \tag{2-14}$$

Where θ is s the angle of skew measured from the X-axis, and V_x and V_y are the base shears in two principal directions, x and y.

In assessing horizontal seismic shear, the structure's mass at or below the base level is assumed not to contribute to the horizontal seismic shear force at the base or any levels above it. However, these lower parts should be designed to resist inertial forces resulting from their masses, ground acceleration, and reactions from levels above the base.

A lateral force of $(0{,}08 \cdot V)$ is applied at the top of the structure to compensate for the effects of higher modes in the upper few storeys. Although ideally, this force should increase with the fundamental period, maintaining a constant value of $(0{,}08 \cdot V)$ simplifies the process and yields sufficiently accurate results for periods up to 2,0 seconds. For buildings with lightweight construction or penthouses, the $(0{,}08 \cdot V)$ lateral force may be distributed between upper levels or treated as part of the structure. Regarding the points of application of equivalent static forces, the 10% of the building width eccentricity allows for variations in structural properties, mass distribution, participation of cladding and partitions in lateral resistance, and rotation of the ground about a vertical axis. For arbitrary loading directions, the load may be assumed to lie on an ellipse with semi-major axes equal to $(0{,}1 \cdot b)$, where (b) is the building width calculated for each orthogonal axis. In scaling deflections, comparative analyses have shown that the equivalent static method tends to over-predict both inter-storey and lateral displacements. Multiplying by scale factors provided in the standard reduces the difference between the two methods. However, scaled equivalent static displacements are still typically larger than those from the model response spectrum method.

2.1.2.5.2 Modal Response Spectrum Analysis

In the modal response spectrum method, while actions in each mode are in equilibrium, combining modal quantities yields envelopes of design actions, each potentially occurring at different times during the passage of the design earthquake. NZS 1170.5:2004[19] states that this approach has consequences, such as the Envelope values not being in equilibrium. The

Envelopes cannot be manipulated similarly as simultaneously occurring actions, leading to difficulties in determining a simple set of static forces that satisfies all combined quantities.

Combined modal values are unsuitable for "capacity design" due to the lack of equilibrium. Instead, capacity design procedures for reinforced concrete and structural steel frames typically scale the results of an equivalent static analysis. These equivalent static results closely match first mode behaviour, and dynamic magnification factors are applied where necessary to account for higher mode effects, including elastic and complex modes resulting from plastic hinges forming in certain structure regions.

2.1.2.5.3 Numerical integration time history analysis

Numerical integration time history analyses (NITH) are valuable for assessing various properties of a structure's response to earthquake loads. These analyses can validate design or response assumptions and precede more general prescriptive requirements. NITH can assess compliance with strength requirements, deflection limits, ductility demands, capacity design assumptions, accelerations, deformations, or any combination thereof. NITH requires a three-dimensional model of the building for accurate representation, but it can be applied to two-dimensional analyses for regularly planned buildings with adjustments for torsional effects.

Vertical excitation significantly influences response parameters such as horizontal cantilevers and vertically sensitive parts of buildings, mainly when supported on long-spanning floor systems. This method realistically simulates earthquake effects by introducing input motions at the structure's base, generating displacements throughout the structure that produce action effects. It allows the influence of inelastic action in selected elements and different seismic input motions (e.g., soil structure or near-fault effects) on structural response to be realistically determined. However, NITH requires considerable designer input across various structural and earthquake modelling areas. Designers need to understand the limits of the chosen computer program, the mathematical concepts behind NITH, structural properties, seismic conditions, and foundation effects. Visualization of input parameters and their physical representation is crucial for appropriate selection.

According to Haymes et al.[22], the design of parts or components to resist seismic demands is currently prescribed in Section Eight Requirements for Parts and Components of the New Zealand Standard NZS 1170.5:2004[19] with 2016 amendments using a floor response spectrum approach. This approach separates the ground motion intensity, amplification of demands with building height, part or component period, and the effects of the nonlinear response of the part or component into individually approximated parameters. The horizontal design earthquake action on the part or component (F_{ph}) is determined using Equation (2-15):

$$F_{ph} = C_p(T_p) \cdot C_{ph} \cdot R_p \cdot W_p \leq 3{,}6W_p \tag{2-15}$$

Where $C_p(\underline{T}_p)$ is the horizontal design coefficient of the part, which varies as a function of the period of the part (T_p), (C_{ph}) is the part horizontal response coefficient; (R_p) is the part risk factor, given as 1,0 for all cases except for where the consequential damage caused by its failure is disproportionately significant; and (W_p) is the weight of the part. $C_p(T_p)$ is calculated using Equation (2-16):

$$C_p(T_p) = C(0) \cdot C_{Hi} \cdot C_i(T_p) \tag{2-16}$$

$C(0)$ is the peak ground acceleration, (C_{Hi}) is the floor height coefficient for level i, and $C_i(T_p)$ is the part spectral shape factor.

2.1.2.6 DRIFT AND STABILITY

The inter-storey drift limits specified in Clause 7.5 of NZS 1170.5:2004[19] are generally consistent with other principal overseas codes of practice. The maximum inter-storey deformation limit of 2,5% serves as the upper bound limit for ultimate limit state considerations of all buildings. This limit is imposed to minimize the probability of instability through the development of soft-storey mechanisms. The reinforced concrete and structural steel frame buildings, detailed according to their respective material standards, are expected to have a low probability of experiencing significant loss of strength or stiffness at this deflection. They are anticipated to sustain inter-storey deflections up to 50% above this limit without collapsing. However, specific structural systems, such as those relying on masonry walls or rotationally sensitive heavy secondary components like hollow core floor units, may require tighter inter-storey deformation limits to prevent premature collapse.

These limits should be specified within such systems' material standards or related trade design literature. When employing time history analysis methods to assess deflections, each record should be scaled to match the elastic response spectra of the design spectra prescribed for the ultimate limit state, typically corresponding to a 1/500 annual exceedance probability for ordinary buildings. In near-fault zones, one in three records should include forward directivity effects. At the same time, these records are still scaled to match the ultimate limit state design spectra, including forward directivity results in actions more consistent with much lower annual probabilities of exceedance (e.g., for ordinary buildings with a 1/2500 annual probability of exceedance). In such cases, the inter-storey deflection limit is increased to 3,75%, representing near-collapse limits for common structural forms.

In several seismic design codes, such as the UBC 1997[23], the base shear found in a modal analysis cannot fall below a proportion of the base shear found from the corresponding value in an equivalent static analysis. A conservative empirical equation is given to determine the fundamental period for the equivalent static method. A consequence is that the empirical equation gives a lower fundamental period and a higher base shear than the corresponding value found in modal analysis for relatively flexible buildings. This results in scaling up design actions in these structures, which are generally sensitive to second-order effects, known as (P-Δ) effects. This scaling up of design actions reduces the sensitivity of buildings designed to these codes to (P-Δ) effects.

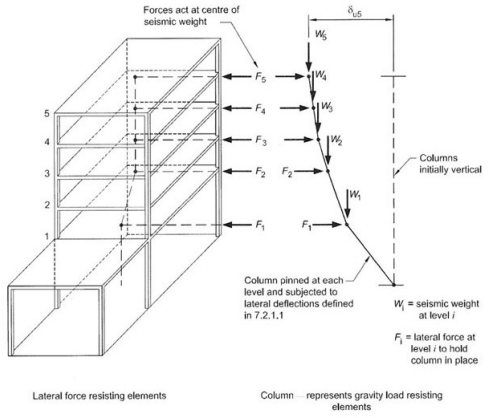


Figure 2-13: P-Δ effects (adapted from NZS 1170.5:2004[19])

NZS 1170.5:2004[19] has no corresponding criteria, and the design base shear and related seismic actions tend to be considerably smaller. As a result, secondary effects, such as (P-Δ) effects, are much more important when designing New Zealand standards than they are to codes such as UBC or IBC. NZS 1170.5:2004[19] requires additional design strength to be added to the structure and additional deformations to be calculated to cope with (P-Δ) effects actions where the value of (θ), given by the expression (2-17), exceeds 0,1.

$$\theta = \frac{\sum_{i=1}^{n} W_i \cdot \delta_{\mu i}}{V_i \cdot \Delta h_i} \tag{2-17}$$

where (ΣW_i) is the seismic weight resisted in storey i, (V_i) is the design storey shear force in the storey, (h_i) is the inter-storey height, and ($\delta_{\mu i}$) is the inter-storey displacement found by multiplying the value found from the displacement envelope by the equivalent static or modal methods of analysis by the structural ductility factor. The value of ($\delta_{\mu i}$) does not include displacements due to (P-Δ) effects. If the value (θ) exceeds 0,3, the structure must be redesigned.

2.1.2.7 CAPACITY DESIGN

The design method in which elements of the primary horizontal earthquake action resisting system are chosen, suitably designed, and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength to maintain the chosen means of energy dissipation.

Appropriate material Standard design and detailing provisions that enable zones where the post-elastic response is acceptable to be identified and detailed to ensure these zones can accept the inelastic demands placed upon them. All other zones are designed to ensure that all other undesirable inelastic response mechanisms are suppressed and detailed in a manner that the ultimate limit state horizontal deformations that they are expected to be subjected to can be sustained without significant (e.g. > 20%) loss of load carrying capacity after four complete cycles of loading as refereed in NZS 1170.5:2004[19].

Overstrength in flexure in some regions of the structure may lead to yielding in the non-ductile areas if the likely increase in strength is not considered in the design. The reasons for flexural overstrength being greater than design strength for seismic actions include:

a) Variations in the steel and concrete strengths from the lower characteristic strengths are used to determine design strengths.

b) Use of strength reduction factors (ϕ) in the design of cross-sections.

c) Cross-sectional sizes can be larger than assumed when determining design actions. In particular, the width of the overhanging flange thought to participate in resisting negative moment design actions is considerably smaller than the width testing has shown participates in overstrength actions where high strains are developed in the beam.

Additional reinforcement, beyond that it is required to satisfy seismic design actions, is placed in members for:

- Construction purposes.
- To satisfy minimum reinforcement requirements.
- To enable a member to sustain gravity loads simultaneously with seismic actions.

- To meet the provisions of a more critical non-seismic load case.

New Zealand seismic design standards require using the capacity design approach for ductile structures and structures of limited ductility. The intention is to protect non-ductile regions of the structure by restricting flexural yielding to selected areas, which is suitably detailed by the designer as mentioned in FIB-FIP-CEB 69[5].

For moment-resisting frames of buildings, the best means of achieving ductile post-elastic deformations is by flexural yielding at selected plastic hinge positions. With appropriate detailing of plastic hinges, they can be made adequately ductile so that the required deformation can be sustained without loss of strength, according to Park et al.[25].

The preferred mechanism for moment-resisting frames is a beam sideways mechanism based on the strong column/weak beam concept, Figure 2-14(a). The ductility demand at the plastic hinges in the beams and column bases is moderate for this mechanism and can easily be provided in the design. A column sideways mechanism with plastic hinges occurring at both ends of one storey's columns is not permitted in NZS 1170.5:2004[19], Figure 2-14(b). In these situations, the deformation imposed on plastic hinges in the columns is moderate and can be provided in the design. Column sideways mechanisms in tall frames can considerably demand ductility at the plastic hinges in the critical storey columns, as Park et al.[26] mentioned. Soft-storey failures have often led to the collapse of buildings during earthquakes.

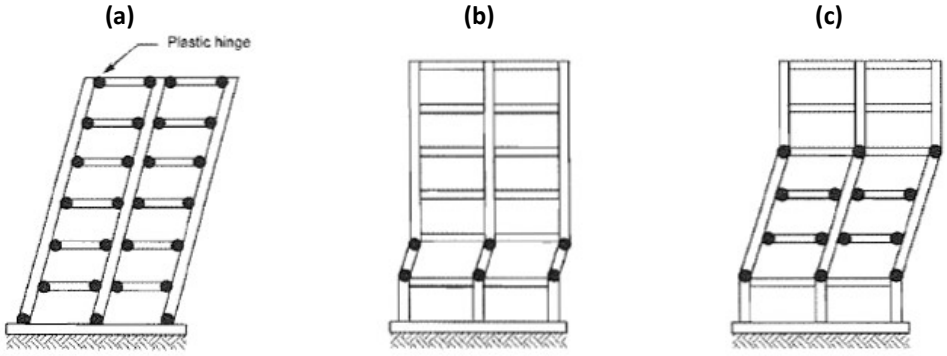


Figure 2-14: mechanisms of post-elastic deformation of moment resisting frames during severe seismic loading(a) Beam sway mechanism (b) Column sway mechanism (c) Mixed beam-column sway mechanism (adapted from NZS 3101: Part 2:2006[21])

A capacity design procedure should determine the design shear forces in beams when the flexural overstrength is reached at the most probable plastic hinge locations within the span and when the design gravity load is present. For example, for a beam A-B, with a clear span of length, the design shear force at end B, (V^*_B) is given by equation (2-18):

$$V_B^* = \frac{M_{0A} + M_{0B}}{L_{AB}} + \frac{w \cdot L_{AB}}{2} \tag{2-18}$$

where M_{0A} and M_{0B} are the flexural overstrength of the beam sections for the positive moment at A and negative moment at B, respectively, and w is the uniform dead and live load considered to be present at the ultimate limit state per unit length. The flexural overstrength moments are calculated using upper characteristic strengths and making allowance, where appropriate, for the contribution that precast prestressed floor units can make to the strength of a beam, according to Fenwick et al.[26].

NZS 3101: Part 1:2006[20] and NZS 3101: Part 2:2006[21] give two methods for designing columns in moment-resisting frames to ensure that a ductile beam sway failure mode will develop in preference to other possible non-ductile failure modes in the event of a significant earthquake. Both methods apply to ductile or limited ductile frames with more than two storeys and are consistent with capacity design concepts. Hence, the design actions for columns correspond to the minimum required nominal. In these methods, it is assumed that plastic hinges will form at the base of the columns and in the beams above the foundation beam in the event of a significant earthquake. However, it is recognised that elongation of the beams can cause plastic hinges to form in the columns immediately adjacent to beams at the top of the first storey. An allowance must be made for this action in detailing the columns.

NZS 3101: Part 1:2006[20] and NZS 3101: Part 2:2006[21] apply two methods. Method A applies to regular frames where the relative stiffness of beams to columns satisfies a given limit. Method B can be used for any ductile or limited ductile moment-resisting frame.

Method A provides a high level of protection against the formation of plastic hinges in columns below the uppermost storey and above the mid-height of the second storey. When this design approach is used, confinement reinforcement in columns may be reduced, and lap splices in longitudinal reinforcement may be located anywhere in the region of a column defined as having a high level of protection against the formation of plastic hinges. Essentially, Method A can be applied to any frame in which points of inflexion occur in each storey when lateral forces are applied to the frame. The steps involved with this method are outlined in the following paragraphs.

The design bending moments of columns at the centre of the beam-column joint are given in equation (2-19):

$$M_{col}^{*} = \omega \cdot \phi_0 \cdot M_E \tag{2-19}$$

where (M_E) is the column bending moment at the centre of the beam-centre joint derived by elastic structural analysis from the equivalent static design forces or modal response spectral analysis in the case of modal response. The ratio (ϕ_0) is equal to the ratio given by dividing the sum of the flexural moments sustained at the centre of the joint when overstrength moments are sustained in the critical sections of the plastic regions divided by the corresponding sum of the seismic analysis bending moments in the beams at the same point. The absolute minimum value of (ϕ_0) when Grade 300 reinforcement is used is 1,25/0,85, which is 1,47.

These include:

- Increased design strength allows the beam to sustain gravity loads simultaneously with seismic actions.
- Generally, more reinforcement is used than the minimum required for the design action.
- Greater widths of the overhanging flange are assumed to act with the beam when overstrength actions are determined compared to design actions.

The dynamic magnification factor varies with the height of the joint zone considered in the frame. The maximum value (ω_{max}) is given by equation (2-20):

$$\omega_{max} = 0{,}6 \cdot T_1 + 0{,}85 \tag{2-20}$$

where (T_1) is the fundamental period of the structure. The value (ω) is equal to (ω_{max}) for levels above 30% of the height of the frame and at or below the third highest level in the frame. The column base and top levels (ω) are taken as 1,0. At the level above the first storey, the value (ω) equals the larger of 1,3 or the value found by linear interpolation between the value at the base and 30% of the height. Likewise, the value (ω) at the second to the highest level is taken as the larger of 1,3 or the value found by interpolation between the third highest and top levels. For cases where the ratio of ($\Sigma M_0/\Sigma M_n$) is high, some reduction in the value (ω) may be made.

The critical column sections are assumed to be at the top and bottom of the clear storey height. Hence, the column moments at the intersection of the beam and column centre lines need to be reduced by the shear force acting in the columns and the distance from the intersection points and the critical sections. This reduction in bending moment is based on the assumption that the shear force acting in the column is equal to 0,6 of the capacity design shear force when the maximum bending moments act in the columns. Hence, the reduction in bending moment between the intersection point of the member centre lines and the critical section is ($0,3 \cdot h_b \cdot V^*_{col}$), where h_b is the depth of the beam.

The recommended amplification of column moments above those found in an elastic analysis by this procedure can be significant, with the combined factor ($\phi_0 \cdot \omega$) being at least 1,91 and 2,14 for frames with Grade 300 and Grade 500 reinforcing steel, respectively, in beams, where ω it is equal to (ω_{max}). For two-way frames, the columns are designed for biaxial bending, which is determined as described above for the frames in one direction and (ω) taken as 1,0 for beams in a second or subsequent direction.

The capacity design axial loads in the columns (N^*_{col}) to be used with the design moments in the column sections are found by summing three axial load components, as set out below.

- The first is the self-weight of the column and any fixings attached to it.
- The second is the shear force at the column faces due to gravity loads, but it neglects any shear associated with end moments (assuming the beams are pinned at the column faces to calculate these values).
- The third is the shear forces acting on the column due to the end moments when overstrength moments act in the critical sections of the plastic hinges in the beams. An adjustment may be made to this component NZS 3101: Part 2:2006[21], which considers the probability that not all beam plastic hinges reach their flexural overstrength simultaneously over the whole height of the frame.

The design shear force in columns above the first storey of the building, (V^*_{col}), is taken as ($1,3 \cdot \phi'_0 \cdot V_E$) in one direction and 60% of that value in the other direction. The value of (ϕ'_0) is taken as the average value of (ϕ'_0) at the two ends of the column. The parameter (V_E) is the column shear force derived from the equivalent static design seismic forces or model analysis's first mode bending moments. These forces were estimated from potential critical moment gradients along columns.

For the columns of the first storey, the design shear forces are calculated, assuming that overstrength moments act at critical sections located at the top and bottom levels of the first storey. The flexural overstrength column capacities should include the effects of the steel and the concrete overstrength and confinement of the concrete.

Method B protects against the premature formation of a column sideways mechanism, but with a lower level of protection than Method A against the formation of plastic hinges above the mid-height of the second storey. With this approach, some column plastic hinge formation may be anticipated in the storeys in the event of a significant earthquake. Consequently, lap splices in longitudinal reinforcement should be in the mid-height of a storey so they are outside potential plastic hinge regions. In addition, full confinement reinforcement is required in all potential plastic regions in the columns.

Columns are proportional so that the storey shear strength corresponding to a column sideway mechanism in a single storey is equal to or greater than a dynamic magnification factor, ω, times the storey shear corresponding to the beam sideways mechanism when overstrength bending moments act in the primary plastic hinge regions. The steps involved in determining the column actions are set below.

a) The locations of primary plastic hinges in the members are identified. These are selected based on the chosen ductile failure mechanism, which is generally a beam sway mechanism.

b) The locations of points of inflexion are chosen in all the columns. Generally, these are located at the mid-height of each storey. However, in some cases, calculations may be required to show that a column has sufficient ductility to enable the moments to be redistributed to accommodate the selected location of the inflexion point.

c) Each level of the structure is considered, in turn, with the lateral force acting in an equivalent static or first mode analysis, being scaled up to correspond to the actions acting on the level when overstrength moments act in the locations of the primary plastic hinges.

d) The shear forces induced in each column due to overstrength moments acting in the primary plastic hinges are calculated from statics and added to the shear forces induced in the columns by the scaled lateral force acting at the level being considered. There is considerable freedom regarding how the scaled lateral force is distributed to the individual columns, though equilibrium requirements must be satisfied.

e) The resultant column shears are multiplied by the dynamic magnification factor 1,3 in all storeys except the two highest. In these, some reduction in dynamic magnification is permitted. For cases where the ratio of ($\Sigma M_0/\Sigma M_n$) is high, some reduction in the value ω may be made. The design moments are found from the product of the shear force and the distance to the critical sections of the columns from the assumed point of inflexion. Each column requires minimum shear strength is calculated, assuming that overstrength moments develop in the critical sections of the column.

f) Where a column is part of more than one frame, it is designed for biaxial actions. As for Method A, moments from the first frame are calculated assuming the dynamic magnification factor (ω) as defined above. However, the dynamic magnification factor is 1,0 for a second or subsequent frame.

g) Method A's approach calculates the axial load acting on the column.

Elastically responding moment resisting frames for which the seismic design forces are found assuming a structural (displacement) ductility factor (μ) equal to or less than 1,25 is expected to respond essentially in the elastic range during the ultimate limit state earthquake.

The structures should be designed to satisfy the following criteria:

a) When the structural system is such that under seismic actions larger than anticipated, the energy dissipating mechanism that develops is as required for ductile structures or structures of limited ductility, the structure is exempt from other seismic requirements. This means that in the event of a larger earthquake, plastic hinges should not form in the columns, except for the top storey and the base, if the frame has more than three storeys. Under these conditions, ordinary reinforcement detailing, as used for non-seismic design, can be used. This requirement is to ensure that the frame does not deform inelastically in a mode which imposes significant ductility demands on some regions of the structure if a larger earthquake than expected occurs. Hence, the members' appropriate regularity and relative strengths are required for this design, as mentioned in FIB-FIP-CEB 69[5].

b) When energy dissipation would be possible in a form not permitted for ductile structures or structures of limited ductility, the relevant mode of inelastic deformation should be identified and the structure detailed appropriately following the additional seismic design provisions of NZS 3101: Part 1:2006[20] and NZS 3101: Part 2:2006[21].

2.1.2.8 DETAILING FOR LOCAL DUCTILITY OF CONCRETE STRUCTURES – BEAMS AND COLUMNS

NZS 3101: Part 1:2006[20] and NZS 3101: Part 2:2006[21] are the New Zealand standard titled "Concrete Structures Standard: The Design of Concrete Structures". It provides guidelines and requirements for designing and constructing concrete structures in New Zealand. This standard covers various aspects of concrete construction, including materials, structural design principles, and detailing requirements.

2.1.2.8.1 Materials

In New Zealand, the specified compressive strength of concrete, (f'_c), generally at age 28 days, is required to be equal to or greater than 25 MPa and not greater than 100 MPa according to NZS 3101: Part 1:2006[20] and NZS 3101: Part 2:2006[21]. Very little concrete is used with a design strength greater than 50MPa. In seismic design, (f'_c) cannot exceed 70MPa for ductile members or members of limited ductility.

Reinforcing steel should satisfy the Standard for reinforcing materials (Australia/New Zealand, 2001). The lower characteristic yield strength (f_y) determines nominal and design strengths. This yield strength corresponds to a value below which fewer than 5% of the test results fall. In the past, the lower characteristic yield strength of reinforcing steel used in New Zealand was either 300 or 430 MPa (i.e. Grade 300 or Grade 430 steel). Grade 430 reinforcement has been withdrawn and replaced with Grade 500 reinforcement. This Grade has disadvantages: it is more challenging to achieve anchorage, greater displacement occurs at first yield, members have a lower stiffness as less reinforcement is used, and greater care is required than for the lower grades where reinforcement is welded or re-bent.

For longitudinal reinforcement, the lower characteristic yield strength is required to be equal to or less than 500 MPa. For transverse reinforcement, the lower characteristic yield strength can be greater than 500 MPa, but where such reinforcement is used, in calculations for transverse reinforcement for shear or torsion, the yield strength, which may be assumed, is limited to 500MPa. Where the design yield stress is 800 MPa or more, the limit that may be considered in confinement calculations is set as 800 MPa.

In calculations for flexural overstrength, it is necessary to recognise the different levels of control on the yield strength and the strain-hardening characteristics of the two reinforcement grades. With Grade 300, the strain-hardened upper characteristic strength bar is assumed to reach 1,25 times the design yield stress (1,25 times the lower characteristic strength). With Grade 500 reinforcement, the corresponding stress level is 1,4 times the design yield stress. Where concrete strength makes a significant difference in strength, a value of (f'_c+15 MPa) should be used to allow for strength to be higher than the minimum design value and for an increase in strength with age beyond 28 days.

2.1.2.8.2 Design provisions for beams and columns

This subchapter presents the main detailing exigences for seismic detailing according to NZS 3101: Part 2:2006[21] for beams and columns regarding the Ductile structures (DS) type. In NZS 3101:Part 2:2006[21], ductile and nominally ductile refer to different seismic performance levels and detailing requirements for concrete structures.

Nominally ductile structures, on the other hand, are designed to provide a basic level of seismic resistance but may not exhibit the same level of energy dissipation and deformation capacity as ductile structures. While nominally ductile structures are expected to withstand seismic events without collapsing, they may experience more limited deformations and energy dissipation than ductile structures. Detailing requirements for nominally ductile structures may be less stringent than ductile structures, focusing on providing basic reinforcement configurations and detailing to ensure structural integrity under seismic loading. Nominally ductile structures may be appropriate for buildings with lower seismic hazards or where full ductile detailing is prohibitive.

Ductile structures are designed to exhibit significant energy dissipation and deformation capacity during seismic events. These structures are expected to undergo large deformations and redistribute loads effectively, thereby maintaining overall stability and preventing collapse even under severe seismic loading. Ductile detailing requirements include the use of specific reinforcement configurations, detailing of plastic hinge regions, and provisions for ensuring adequate ductility and strength in critical elements such as beams and columns. Ductile structures are subjected to rigorous design and detailing criteria to ensure superior seismic performance.

Table 2-8 summarizes the main detailing exigencies for beams according to the frame structure type.

Table 2-8: Detailing provisions for beam (NZS 3101: Part 2:2006[21])

Beams		Nominally Ductile Structures	Ductile Structures
Clear Span	l_n	$4 \cdot h_w$	
Minimum width	$b>$	250 mm	
width-to-depth ratio	$h/b_w<$	0,3	
Minimum longitudinal reinforcement		$A_s = \frac{\sqrt{f'_c}}{4 \cdot f_y} \cdot b_w \cdot d$	$p_{min} = \frac{\sqrt{f'_c}}{4 \cdot f_y}$
Maximum reinforcement ratio	p_{max}	Must limit neutral axis depth to $0{,}75 \cdot c_b$	$\frac{f'_c + 10}{6 \cdot f_y}$
Bar diameters through the interior beam-column joints	$d_b/h_c \leq$	$4 \cdot \alpha_t \cdot \frac{\sqrt{f'_c}}{f_y}$	$3{,}3 \cdot \alpha_f \cdot \frac{\sqrt{f'_c}}{\phi_{0,f_y} \cdot f_y}$
Plastic hinge zone length	l_p		$2 \cdot h_w$
Minimum shear reinforcement outside plastic hinge regions	A_v	$\frac{1}{16} \cdot \sqrt{f'_c} \cdot \frac{b_w \cdot s}{f_{yt}}$	$\frac{1}{12} \cdot \sqrt{f'_c} \cdot \frac{b_w \cdot s}{f_{yt}}$
Minimum shear reinforcement outside plastic hinge regions			$\frac{\sqrt{f'_c}}{12 \cdot f_y} \cdot b_w \cdot s$
Maximum stirrup spacing outside the plastic hinge zone	$s \leq$	0,5·d, or 600 mm, or where b_w exceeds 0,5·d, larger of 0,5·d or 250 mm. Half this spacing when V_s exceeds $0{,}33\ (f'_c)^{1/2}\ b_w \cdot d$	
Maximum stirrup spacing within the plastic hinge zone			

Table 2-9 summarizes the main detailing exigences for columns according to the frame structure type.

Table 2-9: Detailing provisions for columns (NZS 3101: Part 2:2006[21])

Beams		**Nominally Ductile Structures**	**Ductile Structures**	
Minimum width	$h>$	Refer to slender column requirements	$b_w \geq L_n/25$ $b_w \geq (L_n \cdot h/100)$ for cantilevered columns	
Extent of ductile detailing length for detailing purposes	l_y	Not applicable	$N_0^* \leq 0{,}25 \cdot \phi f'_c A_g$	Greater of h, diameter or where moment exceeds 0,6 M_{max}
			$0{,}25 \cdot \phi f'_c A_g < N_0^* \leq 0{,}5 \cdot \phi f'_c A_g$	
			$N_0^* > 0{,}5 \cdot \phi f'_c A_g$	Greater of 3 times (h,b_w diameter) or where moment exceeds 0,7 M_{max}
Minimum longitudinal reinforcement		$0{,}008 \cdot A_g$		
Maximum longitudinal reinforcement ratio		$0{,}08 \cdot A_g$	$18 \cdot A_g/f_y$	
Maximum longitudinal column bar diameter	$d_b/h_b \leq$	No limitations	$3{,}2\frac{\sqrt{f'_c}}{f_y}$	
Minimum Shear reinforcement outside plastic hinge regions	A_v	$\frac{1}{16} \cdot \sqrt{f'_c} \cdot \frac{b_w \cdot s}{f_{yt}}$	$\frac{1}{12} \cdot \sqrt{f'_c} \cdot \frac{b_w \cdot s}{f_{yt}}$	
Minimum shear reinforcement outside plastic hinge regions			$\frac{\sqrt{f'_c}}{12 \cdot f_y} \cdot b_w \cdot s$	
Maximum stirrup spacing outside the plastic hinge zone	$s \leq$	$\min(h/3, b_w/3, \text{diameter}/3, d_b \cdot 10)$		
Maximum stirrup spacing within the plastic hinge zone				
Confinement reinforcement outside the potential plastic hinge	A_{sh}	$\frac{(1{,}0 - p_t \cdot m) \cdot s_h \cdot h"}{3{,}3} \cdot \frac{A_g}{A_c} \cdot \frac{f'_c}{f_{yt}} \cdot \frac{N^*}{\phi \cdot f'_c \cdot A_g} - 0.0065 \cdot s_h \cdot h"$		
Confinement reinforcement inside the potential plastic hinge		$\frac{(1{,}0 - p_t \cdot m) \cdot s_h \cdot h"}{3{,}3} \cdot \frac{A_g}{A_c} \cdot \frac{f'_c}{f_{yt}} \cdot \frac{N^*}{\phi \cdot f'_c \cdot A_g} - 0.0060 \cdot s_h \cdot h"$		

2.1.2.9 DESIGN AND DETAILING PROVISIONS FOR REINFORCED CONCRETE STRUCTURAL WALLS

According to NZS 3101: Part 2:2006[21], walls must be designed to resist loads to which they are subjected, including eccentric gravity loads and lateral forces due to wind or earthquake, which may result in actions in the plane of or transverse to the wall. In general, this section applies to walls spanning vertically between horizontal supports. Walls must be designed for combined flexure, axial load and shear accordingly, considering the wall to be a member subject to axial force and flexure while also satisfying the vertical and horizontal reinforcement requirements.

The simplified method for designing moments and considering (P-Δ) effects is based on the requirements specified in ACI 318-02[27] and experimental research. Panels with large openings, such as windows, are treated differently as they do not have a constant cross-section over their height, and their design needs to account for the effects of these openings. Equation (2-21) is derived under several assumptions:

- The panel is simply supported at the top and bottom.
- The deflected shape of the panel approximates that of a parabola.
- The wall is sufficiently cracked so that the effective moment of inertia approaches that of the cracked section.

$$\Delta_u = \frac{5 \cdot M_u^* \cdot L_n^2}{0{,}75 \cdot 48 \cdot E_c I_{cr}} \tag{2-21}$$

While some walls may exhibit fixity at the base, assuming simple fixity in the analysis is conservative. However, factors like foundation flexibility or yielding of reinforcement under seismic action can reduce the base restraint to that of a pin. In regions with low seismic activity, where the design force load moment is significantly lower than the moment required to cause cracking, a rational analysis incorporating (*P-Δ*) effects may be more suitable for calculating the design moment.

2.1.2.9.1 Design for shear in planar of walls

The design for shear in the plane of a wall is primarily essential for walls with a small height-to-length ratio. For taller walls, flexural considerations typically govern the design. NZS 3101: Part 1:2006[20] outlines several provisions for the shear design of wall elements. The maximum shear stress in a wall section is limited based on the compression strength of concrete. The thinnest section of the wall must be considered when computing shear stress.

Values of shear strength computed at a specific section apply to that section and all sections between it and the base. Minimum shear reinforcement is expected for all wall elements. Sufficient horizontal shear reinforcement is required to carry shear exceeding the provided shear strength. The minimum reinforcement ratio should not be less than 0,7 times the yield strength of the reinforcement material. These provisions ensure adequate shear capacity and reinforcement in wall elements to withstand applied loads and resist shear forces effectively.

The maximum nominal shear stress in wall sections is limited by the compression strength of concrete, as specified in NZS 3101: Part 1:2006[20]. This limitation applies even though the wall length-to-thickness ratio is smaller than that of ordinary beams. The thinnest section of the wall, which may occur where horizontal recesses reduce thickness, must be considered when calculating shear stress. The total nominal shear stress, denoted as (V_n), at any section, including the base of the wall, is constrained following the relevant standard provisions. For sections near the base of walls, the values of (V_c) at a specified distance apply to that section and all sections between it and the base. Regardless of the concrete contribution to shear resistance, a minimum amount of shear reinforcement is mandated in all wall elements, unlike slabs. Horizontal shear reinforcement must be sufficient to carry shear exceeding (V_c).

2.1.2.9.2 Bending moment action of walls

For cantilever structural walls of buildings, the best means of achieving ductile post-elastic deformation is generally recognised by flexural yielding at the base of the walls (primary plastic region).

The Standard requires the design bending moment for the wall obtained from an equivalent static or modal analysis to be:

a) Scaled up by the ratio (ϕ_w), which is the ratio of the overstrength moment at the critical section of the primary plastic hinge in the wall to the corresponding moment found in the analysis for the design actions (M_0/M_n)

b) The distribution of design moments over the height of the wall is modified to allow for higher mode actions associated with higher elastic modes and modes related to the formation of the primary plastic hinge in the wall.

c) Flexural reinforcement has to be extended for a distance of a wall-length plus a development length beyond the level indicated by the moment diagram. This extension is required to allow for tension lag associated with diagonal cracking.

The commentary indicates that for uniform walls, an appropriate moment envelope, corresponding to step (b), is obtained by assuming the bending moment envelope is linear between the nominal flexural strength at the base of the wall and zero moments at the top of the wall. The design shear envelope in the wall is strongly influenced by higher mode actions associated with the plastic hinge formation, and the Standard requires the envelope obtained from an equivalent static analysis to be modified for higher mode actions. Again, the commentary guides changing the equivalent static shear force envelope to allow this effect. For regular walls, the shear force envelope is multiplied by the ratio (ϕ_w) and a dynamic magnification factor (ω_w), which is given by:

$$\begin{cases} \omega_w = 0{,}9 + \dfrac{n}{10} \; for\ building\ up\ to\ 6\ storeys \\ \omega_w = 1{,}3 + \dfrac{n}{30} \; for\ building\ over\ to\ 6\ storeys \end{cases} \tag{2-22}$$

where (n) is the number of storeys in the structure. The commentary notes that the distribution of shear forces between walls can depart markedly from analytical values found in elastic analyses where walls of different lengths are used.

When coupling beams exist between structural walls, the walls act as individual cantilevers connected by pin-ended links if the coupling is weak. Suppose the coupling beams are stiffer and have significant flexural strength but insufficient to cause the shear failure of the walls at ultimate conditions. In that case, plastic hinging will develop at the base of the walls and in the coupling beams. According to the NZS 3101: Part 2:2006[21], the maximum structural ductility factor, which may be used to determine design actions, depends on the wall's height and the coupling beams' stiffness and strength. In no case is the structural ductility factor allowed to exceed 6.

With dual-wall frame systems, the walls control the distribution of seismic deformation in the frames. This prevents the development of storey sway mechanisms. Consequently, the dynamic magnification factor used for the capacity design actions in the columns can be reduced well below that used in the columns of moment-resisting frames. When proportioned

by capacity design principles, the wall remains essentially in the elastic range above the plastic hinge in the base.

2.1.2.9.3 Dimensions of boundary elements

The NZS 3101: Part 2:2006[21] addresses the design of ductile structural walls at the ultimate limit state, requiring compliance with general requirements for analysis, design forces, ductilities, and capacity design procedures. The nominal moment capacity (M_n) for walls with returns should be based on a 1/2 spread of flange action for tension reinforcement. Walls experiencing overstrength flexural action engage more of the flange in tension, determined by reinforcement over a 45° spread from the top of the wall. Dimensional limitations consider potential out-of-plane buckling in the plastic hinge region after large inelastic tensile strains occur. Parameters affecting wall instability include maximum steel tensile strains, wall thickness, reinforcement arrangement, vertical reinforcement quantity, and probable buckling length. Simplifications are introduced to facilitate practical design implementation.

When stability criteria govern the geometry of the wall section, thickening the wall in boundary regions will be necessary. This is readily achieved by providing flange elements with sufficient dimensions to provide adequate flexural rigidity at the end of the wall section. Figure 2-15 summarises the nomenclature of the dimensions for possible applications.

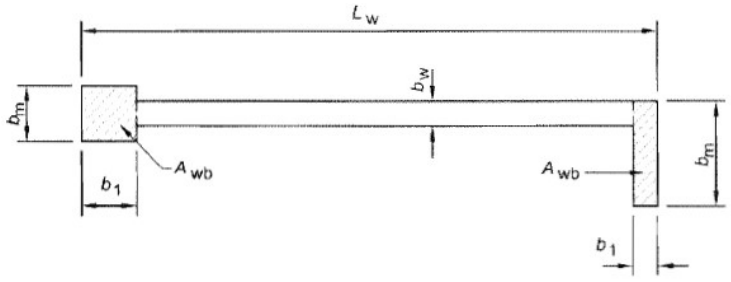


Figure 2-15: Minimum dimensions of boundary elements of wall sections in plastic hinge regions (Adapted from NZS 3101: Part 2:2006[21])

For the end regions, where the contribution of the concrete towards shear resistance is evaluated, modifications for walls relative to columns are introduced to consider the relative dimensions of cantilever or coupled walls, which may differ from those of columns. The end region typically extends from the wall level at which the critical base moment can develop.

2.1.2.9.4 Transverse reinforcement of walls

Transverse reinforcement also referred to as lateral reinforcement, is usually placed horizontally in walls to resist horizontal shear forces, controlling shrinkage strains or giving restraint to NZS 3101: Part 2:2006[21] vertical compression bars or confining the concrete in areas of large compression strains. The principles also apply to coupling beams of walls. The requirements concerning spacing, quantity and bar sizes in wall webs are similar for both the vertical and horizontal reinforcement. The principles of the use for (horizontal) shear reinforcement in walls are the same as those for stirrups in flexural members. In particular, the same principles apply to allocating shear resistance to concrete and steel mechanisms. The region of a wall where the formation of a plastic hinge may affect the shear strength is generally at its base.

Transverse reinforcement for lateral restraint in plastic hinge regions is intended to ensure that the principal longitudinal reinforcement, usually placed near the edges of walls, receives

adequate lateral support, taking the Bauschinger effect into account to enable it to be strained beyond compression yield. The requirements extend to areas, both horizontally and vertically, where yielding of the longitudinal reinforcement could occur. In most walls, every vertical bar should be assumed to be subjected to alternating yielding in tension and compression.

2.1.2.9.5 Confinement requirements in the plastic hinge region of walls

Confinement of the compressed concrete, usually at the base of a wall, is required if the compression strain at the ultimate limit state is expected to be excessive. The special hoop requirements need not be satisfied, provided the designer ensures that the flexural wall reinforcement yielding will not occur outside the potential yield region. This may be achieved if the flexural reinforcement is curtailed following a linear bending moment envelope rather than the bending moment diagram derived for the lateral static forces.

Diagonal shear reinforcement, except for squat walls, is seldom required in potential plastic hinge regions. Transverse reinforcement, placed horizontally in walls, serves various purposes, including resisting horizontal shear forces and controlling shrinkage strains. Requirements for transverse reinforcement spacing, quantity, and bar sizes are similar for vertical and horizontal reinforcement.

2.1.2.9.6 Detailing provisions for local ductility of walls

Table 2-10 summarizes the main detailing provisions for walls according to NZS 3101: Part 2:2006[21].

Table 2-10: Detailing provisions for walls (NZS 3101: Part 2:2006[21])

Beams		Nominally Ductile Structures	Ductile Structures
Minimum structural wall thickness-genera l	t	100 mm	
Limitations on the dimensions of the compression zone within plastic hinge region		Not applicable	$\frac{\alpha_r \cdot k_m \cdot \beta \cdot (A_r + 2) \cdot L_w}{1700 \cdot \sqrt{\xi}}$
Minimum longitudinal reinforcement	A_s	$\frac{\sqrt{f'_c}}{4 \cdot f_y} \cdot b_w \cdot d$	
Maximum longitudinal reinforcement ratio		$P_l < 16/f_y$ and ensure neutral axis depth is less than 0,75 c_b	
Maximum longitudinal Wall bar diameter	$d_b \leq$	No limitations	b/10 for full ductility b/8 for limited ductility
Minimum Shear reinforcement outside plastic hinge regions	A_v	$0{,}7 \cdot \frac{b_w \cdot s_2}{f_{yt}}$	
Minimum Shear reinforcement within plastic hinge regions			
Maximum stirrup spacing outside the plastic hinge zone	$s \leq$	Larger of $L_w/5{,}3\ t$, or 450 mm	
Maximum stirrup spacing within the plastic hinge zone			
Confinement reinforcement outside the potential plastic hinge	A_{sh}	Not required	
Confinement reinforcement inside the potential plastic hinge		Not required	$\alpha \cdot s_h \cdot h'' \frac{A_g^*}{A_c^*} \cdot \frac{f'_c}{f_{yh}} \left(\frac{c}{L_w} - 0{,}07 \right)$

2.1.3 CHILEAN SEISMIC AND CONCRETE STANDARD

NCh433.Of1996[28] is a seismic design code specific to Chile, established by the Chilean National Institute of Standards (INN), known in Spanish as "*Instituto Nacional de Normalización*". This code provides guidelines and regulations for the seismic design of structures in Chile, considering the seismic hazard and building practices prevalent in the country.

NCh 433:2012[29] outlines procedures for evaluating seismic hazards in different regions of Chile, considering factors such as seismicity, soil types, and historical earthquake data. It defines seismic design parameters such as seismic zones, design acceleration values, and response spectra based on the calculated seismic hazard NCh433.Of1996[28] prescribes requirements for the seismic design and detailing of various structural systems, including reinforced concrete, steel, masonry, and timber structures. It covers lateral load-resisting systems, ductility considerations, and detailing requirements to ensure adequate seismic performance.

2.1.3.1 STRUCTURAL SYSTEM

According to NIST GCR 12-917-18[29], NCh433.Of1996[28] recognises three general types of seismic force-resisting systems:

- Shear wall and other braced systems (SWBS).
- Moment-resisting space frame systems (MRF).
- Dual systems containing a combination of the above two systems (DS).

The shear wall systems consist of reinforced concrete or masonry walls designed to resist lateral forces induced by seismic activity. These walls are typically located along the length or perimeter of a building and are capable of carrying both gravity and lateral loads. Shear walls primarily resist lateral loads through their stiffness and strength, which prevents excessive building sway during earthquakes. They work by transferring lateral forces to the foundation, providing stability to the structure. Other braced systems, such as diagonal bracing or moment-resisting frames with supplementary bracing, also fall under this category. These systems employ additional bracing elements, such as diagonal braces or shear links, to enhance lateral stability and energy dissipation.

Moment-resisting frames (MRF) consist of vertical and horizontal members interconnected to form a rigid framework capable of resisting lateral forces. These frames are typically constructed using steel or reinforced concrete and rely on beam-column joints that can develop significant bending resistance. The (MRF) systems primarily resist lateral loads by flexural action in beams and columns, allowing the structure to deform without excessive displacement. They are designed to develop plastic hinges at predetermined locations, dissipating seismic energy through inelastic deformation while maintaining overall structural integrity. The moment-resisting frames can vary in configuration, including concentric or eccentric bracing, special moment frames (SMF), and intermediate moment frames (IMF), each offering different levels of ductility and energy dissipation capacity.

Dual systems combine elements of both shear wall and moment-resisting frame systems to capitalize on their respective strengths and mitigate their weaknesses. These systems typically incorporate shear walls or braced frames in conjunction with moment-resisting frames to achieve enhanced seismic performance. Dual systems leverage the stiffness and strength of

shear walls for lateral stability while utilizing the ductility and energy dissipation capabilities of moment-resisting frames to mitigate structural damage. By combining these systems, dual systems aim to provide a robust and resilient structural response to seismic loading. Dual systems offer a balanced approach to seismic design, combining the benefits of both shear walls and moment-resisting frames to achieve optimal strength, stiffness, and ductility performance. They are often preferred for tall or irregular buildings where a single system may not effectively address all seismic demands.

These systems are further classified according to the material of construction. Structural response modification factors, designated as (R) or (R_0), are assigned based on system type and used to determine the design base shear.

2.1.3.2 REGULARITY OF STRUCTURES

The NCh433.Of1996[29] does not provide a specific classification for structures based on their regularities or irregularities. However, it does address buildings with irregular floor plans, such as H-shaped, L-shaped, T-shaped, and U-shaped. According to the standard, these buildings can only be designed as a single structure if the diaphragms are calculated and constructed to ensure that the building behaves as a unified whole during earthquakes. Additionally, if the irregularly shaped building is designed as a single structure, particular attention must be paid to connecting the various parts comprising the floor plan.

2.1.3.3 SEISMIC ACTIONS

According to NIST GCR 12-917-18[30], NCh433.Of1996[29], similar to the system used under the Uniform Building Code in the United States, NCh433.Of1996[29] uses seismic zonation to establish design shaking intensities. Seismic zonation maps depict three seismic zones designating three levels of maximum effective soil acceleration, A_0 (Peak Ground Acceleration). Zone 3 represents the highest design shaking intensity, with maximum effective soil acceleration, A_0 = 0,4 g. Zone 2 represents a more moderate shaking intensity, with A_0 = 0,3 g. Zone 1 represents the lowest design shaking intensity, with A_0 = 0,2 g.

Figure 2-16 shows the seismic zonation maps contained in NCh433.Of1996[29]. Zone 3 encompasses the coastal region, Zone 2 generally encompasses the central plain, and Zone 1 encompasses the eastern portion of the country along the western flank of the Andes.

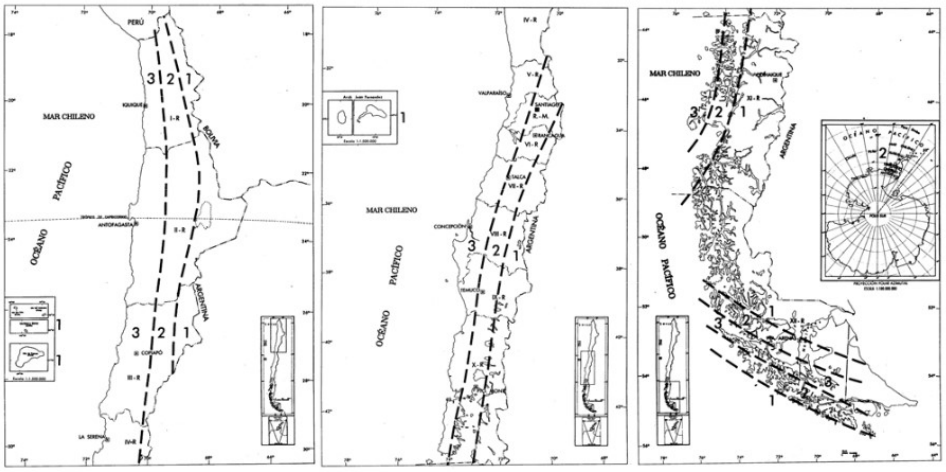


Figure 2-16: Seismic zonation maps for northern, central, and southern Chile (adapted from NCh433.Of1996[29])

2.1.3.3.1 Design Spectrum

The design spectrum of pseudo-accelerations that determines the seismic resistance of the structure is defined by equation (2-23):

$$S_a = \frac{S \cdot A_0 \cdot \alpha}{\frac{R^*}{I}} \tag{2-23}$$

where (S) is the parameter dependent on the soil type; (A_0) represents the maximum effective soil acceleration determined by the seismic zone. The parameters (I) are occupancy importance factors, (α) are amplification factors, and (R^*) are reduction factors.

2.1.3.4 STRUCTURAL ANALYSIS

According to NIST GCR 12-917-18[30], NCh433.Of1996[29] recognizes two analytical procedures for determining seismic design forces:

- Static Analysis.
- Modal response spectrum analysis.

The static analysis method is permissible for buildings up to 15 stories tall; it is rarely utilized for such structures.

The modal response spectrum procedure is typically favoured for designing larger structures. Irrespective of the analysis method, seismic forces must be distributed to seismic force-resisting elements using a three-dimensional model with two translational and one rotational degree of freedom at each level. Buildings must be assessed for seismic forces acting in two approximately orthogonal directions. Cantilevered components., must be designed to withstand an effective vertical earthquake force equivalent to 30% of the combined dead and live loads. This adjustment is achieved by increasing the design dead and live loads by 30% for these elements. The modal response spectrum method is applicable for designing any building, while the static analysis method is restricted to specific scenarios:

- It can be used for ordinary structures (Category C) or uninhabited structures (Category D) in Seismic Zone 1.
- It applies to structures with a height not exceeding 5 stories or 20 meters.
- Static analysis is permitted for structures ranging from 6 to 15 stories high if they meet specific regularity criteria.

2.1.3.4.1 Static analysis

The total seismic base shear in each direction, denoted as (Q_0), is determined using expression (2-24):

$$Q_0 = C \cdot I \cdot P \tag{2-24}$$

Where (C) is the seismic coefficient. The parameter (I), aforementioned, is the occupancy importance factor:

- For Categories A and B, I=1,2.
- For Category C, I=1,0.
- For Category D, I=0,6.

Parameter (P) is the effective seismic weight of the building above the base, calculated as the weight of permanent elements plus a specified fraction of live loads, excluding the roof's live load. For typical public and private buildings, 25% of the specified live load is used in calculating the seismic weight. However, at least 50% of the specified live load must be included in the seismic weight for storage occupancies. The seismic coefficient (C) is calculated according to the equation (2-25):

$$C = \frac{2{,}75 \cdot A_0}{g \cdot R} \cdot \left(\frac{T'}{T^*}\right)^n \tag{2-25}$$

In equation (2-25), (A_0) represents the maximum effective soil acceleration determined by the seismic zone, (g) denotes the acceleration due to gravity, (R)stands for a system-dependent structural response modification coefficient; (T') denotes a characteristic site period dependent on soil type, while (T^*) represents the period of the mode with the highest translational mass participation in the considered direction; and (n) is a coefficient reliant on soil type.

For linear static analysis, the response modification coefficient (R) varies from 2 for undefined systems to 7, applicable to moment-resisting frames and specific shear wall and braced frame systems. Meanwhile, the value of (T') ranges from 0,2 seconds on rock sites (Class I) to 1,35 seconds on saturated clay sites. The exponent n varies from 1.0 on rock sites to 1,8 on saturated clay sites. The value of (C) cannot be lower than (A_0/6g) and should not exceed a maximum value correlated with (R).

According to NIST GCR 12-917-18[30], NCh433.Of1996[29], buildings equipped with seismic force-resisting systems comprising reinforced concrete walls, either alone or in combination with frames or reinforced masonry walls, are eligible for a reduction in required seismic design forces. This reduction is facilitated by a factor denoted as (f_q) and computed according to expression (2-26):

$$f_q = 1{,}25 - 0{,}5 \cdot q \tag{2-26}$$

where (q) represents the smallest fraction of the total shear resisted by the reinforced concrete walls in the lower half of the building, considering both analysis directions. The value of (q) must fall between 0,5 and 1,0 inclusively.

For buildings predominantly constructed with reinforced concrete walls, the maximum base shear coefficient is decreased to 75% of the value presented in Table 2-11, as per NCh433.Of1996[29]. In single-story buildings with a rigid diaphragm at the roof, a reduced seismic force coefficient equals 80% of the value determined by NCh433.Of1996[29], specified in expression (2-25).

Table 2-11: Maximum values of seismic coefficient (C_{max}) based on (R) (NCh433.Of1996[29])

Response Modification Coefficient (R)	Maximum Seismic Coefficient (C_{max})
2	$0{,}90 \cdot A_0/g$
3	$0{,}60 \cdot A_0/g$
4	$0{,}55 \cdot A_0/g$
5,5	$0{,}40 \cdot A_0/g$
6	$0{,}35 \cdot A_0/g$
7	$0{,}35 \cdot A_0/g$

For multi-storey structures, 5 storeys or less, the base shear force obtained from equation (2-24) is distributed vertically to each diaphragm level by equation (2-27):

$$F_k = \frac{A_k \cdot P_k}{\sum_{j=1}^{n} A_j \cdot P_j} \cdot Q_0 \tag{2-27}$$

where (Q_0) is the total base shear force, (P_k)and (P_j) are the effective seismic weight at levels k and j, respectively, and the coefficients (A_k) and (A_j) are defined by the equation (2-28):

$$A_k = \sqrt{1 - \frac{Z_{k-1}}{H}} - \sqrt{1 - \frac{Z_k}{H}} \tag{2-28}$$

where (Z_k) and (Z_{k-1}) are the height of levels k and (k-1) above grade, respectively, and (H) is the total height of the structure above grade. For structures more than 5 stories but less than 16 stories in height, Equation (2-28), or other rational procedures, are used to distribute forces vertically within the structure.

According to NIST GCR 12-917-18[30], apart from the story forces derived from equation (2-27), buildings must also be designed to withstand the effects of an accidental torsional moment applied at each story. This moment is calculated as the static force (F_k) at each level, multiplied by an accidental eccentricity determined by the expression (2-29):

$$\pm 0{,}1 \cdot \frac{b_k \cdot Z_k}{H} \tag{2-29}$$

which represents the largest dimension of the structure at level (k) perpendicular to the direction of analysis, and other terms retain their previous definitions. Two scenarios should be considered: one where the accidental torsional moment is applied positively and another where it is applied negatively. However, these torsional moments may be disregarded for structures exhibiting limited torsional displacement under load.

2.1.3.4.2 Modal Response Spectrum Analysis

Modal response spectrum analysis should encompass adequate natural modes to ensure that at least 90% of the structure's total mass and torsional inertia are represented in the two principal horizontal directions. Additionally, the impact of accidental torsion needs to be addressed by

either displacing the centre of mass at each level by an amount equivalent to 5% of the diaphragm dimension perpendicular to the analysis direction or by applying an accidental torsional moment at each level. This torsional moment is calculated as the product of the incremental story shear obtained from the analysis and the specified accidental torsional eccentricity for static analysis. Similar to static analysis, two scenarios of accidental torsion must be considered in response spectrum analysis: one with positive and negative eccentricity, as referred to in NIST GCR 12-917-18[30].

2.1.3.5 DRIFT AND STABILITY

According to NCh433.Of1996[28], the horizontal displacements and rotations of floor diaphragms must be calculated for the seismic design actions specified in clause 6, including the effect of accidental torsion. The maximum relative displacement between two consecutive floors, measured at the centre of mass in each analysis direction, must not exceed the floor height multiplied by 0,002. The maximum relative displacement between two consecutive floors, measured at any point on the floor plate in each analysis direction, must not exceed by more than (0,001h) the corresponding relative displacement measured at the centre of mass, where h is the floor height. In floors without rigid diaphragms, the maximum value of the transverse floor displacement of the chains, induced by solicitations acting perpendicular to the wall plane on which the chain is located, must be equal to or less than the floor height multiplied by 0,002. For the design of reinforced concrete structures, the lateral design displacement at the roof, (δ_u), should be considered equal to the ordinate of the elastic displacement spectrum (S_{de}) for 5% damping relative to critical damping, corresponding to the period (T) of the mode with the highest translational mass in the direction of analysis, multiplied by a factor of 1,3.

2.1.3.6 DETAILING FOR LOCAL DUCTILITY

According to Nch430.Of2008[30] The official Chilean standards for reinforced concrete, NCh429.EOf1957 "*Reinforced Concrete - Part 1*" and NCh430.EOf1961 "*Reinforced Concrete - Part 2*" was initially based on the German standard DIN 1045[31], "*Concrete and Reinforced Concrete*" from the 1950s. However, due to their age, these standards were not used extensively. In 1983, after extensive public consultation, structural engineering professionals selected the ACI 318 "*Building Code Requirements for Reinforced Concrete and Commentary*" from the United States as the new Chilean standard, choosing it over the DIN 1045[31], NZS 3101 (New Zealand), and CEB (Europe) standards.

After several years of informal use, the NCh433.Of1993, the standard "Seismic Design of Buildings" officially adopted ACI 318-89 to design reinforced concrete structures. Subsequently, in 1996, the NCh433 standard adopted ACI 318-95. This normative process is sanctioned in a separate standard, which is required in NCh433.Of1996[28], Annex B, and updated the 1995 version of ACI 318 to the 2005 version.

Based on the experience gained after several years of using this code, it became necessary to introduce some complements to this adoption to adjust it to the national design conditions. These complements were generated from observations on the text of ACI 318 itself received in the public consultation in 1983 and the satisfactory behaviour exhibited by reinforced concrete wall buildings during the March 1985 earthquake, as indicated in NCh433.Of96[28]. These modifications constitute the body of this standard, which must be used in conjunction with ACI 318-05[32]. The ACI 318M-05 corresponds to the version in the metric system of units and complements ACI 318-05. The ACI 318S-05 corresponds to the Spanish translation of ACI 318M-05[32].

The ISO 19338 standard "Performance and Assessment Requirements for Design Standards on Structural Concrete," and its corrigendum 1, recognizes ACI 318-05[32] as a national standard that meets the requirements of this international standard. Therefore, one can parallelise chapters 2.1.7 and 2.1.8 of the present work.

2.1.4 MEXICO CITY TECHNICAL STANDARDS

The Complementary Technical Standards of the Construction Regulations for the Federal District (NTC) are the technical provisions that establish the minimum technical requirements for the design and construction of all types of buildings with the specifications and exceptions indicated therein to ensure proper functioning, accessibility, habitability, hygiene, services, environmental conditioning, communication, evaluation, and prevention of emergencies, emergency safety, integration into the urban context and urban image, installations, and structural safety of constructions in Mexico City.

In 2017, the "Agreement to Update the Complementary Technical Standards" was published in the former Official Gazette of the Federal District, corresponding to:

- Design and Construction of Foundations.
- Seismic Design.
- Wind Design.
- Design and Construction of Steel Structures.
- Design and Construction of Concrete Structures.
- Design and Construction of Wood Structures.
- Design and Construction of Masonry Structures.
- Criteria and Actions for the Structural Design of Buildings.

For the present work, this subchapter will present the “Seismic Design” standard, known as NTC-DS:2020[33] “Design and Construction of Concrete Structures”, known as NTC-DCEC:2017[34].

2.1.4.1 STRUCTURAL SYSTEM

According to NTC-DS:2020[33], every building must have a structural system capable of transmitting to the soil-foundation system the combined effects of the lateral inertia forces generated during the earthquake and the gravitational forces.

The choice of the structural system is one of the most critical decisions made during seismic design. Post-earthquake building damage assessment indicates that regular and well-detailed structural systems exhibit better seismic performance than those with structural irregularities and ordinary detailing. Additionally, the better structural performance of dual systems composed of frames and stiffening structural elements, such as walls and bracings, has been noticeable compared to those observed in systems structured exclusively with frames. Seismic response control systems are now a viable alternative to promote better seismic performance by controlling the lateral deformation of the structural system.

NTC-DS:2020[33] considers three types of ductility structures:

- High ductility.
- Medium ductility.
- Low ductility.

The specific requirements that must be met for a structure to be considered as having high, medium, or low ductility are specified in the corresponding Technical Standards for the material in question. A structural system is considered to exhibit high ductility when the specific requirements outlined by the respective technical standard for detailing members and connections corresponding to that designation are met. Within the same context, medium and low ductility levels are assigned to detailing corresponding to medium and low ductility, respectively.

2.1.4.2 STRUCTURAL TYPES

The structural systems, according to NTC-DS:2020[33], are defined as follows:

- Frames.
- Prefabricated element frames.
- Dual system formed by concrete frames and walls.
- System formed by concrete walls.
- Dual system comprising coupled concrete frames and walls.
- Dual system comprising concrete frames and steel bracing.
- System comprising uncoupled frames and wall diaphragms.
- System based on cantilever concrete columns.
- Suspended system supported by concrete core comprising walls or frames.
- Outer frames and inner columns interconnected by rigid horizontal diaphragms.
- System comprising columns and interconnected frames or walls with flat slabs
- System of interconnected concrete columns with flat slabs.

The Frames are monolithically cast, consisting of beams and columns connected by rigid joints. Their lateral stiffness and ability to resist seismic actions depend essentially on the flexural stiffness of beams and columns and the shear stiffness of the connections. Monolithic frames can achieve high levels of ductility. To achieve this, it is necessary to ensure that the possible failure of structural elements is due to bending and not brittle failures such as shear or bond failures. If these conditions are met, frame-based structures can be designed with seismic behaviour factor values (Q) of up to 4 and allowable interstorey drifts (γ_{max}) of up to 0,03. Reviewing the NTC-DCEC:2017[34] provisions for beams, columns, and joints is necessary.

The Prefabricated Element Frames, given the difficulty of achieving high ductility in connections between prefabricated elements or in frame joints, only structures of medium or low ductility are allowed in this type of structuring, with maximum drifts of 0,020 and 0,015, respectively. To use medium ductility, joints must be monolithically cast, and connections between prefabricated elements must be designed to ductile behaviour and located away from critical sections of the structure, such as the joints themselves, where plastic hinges can form or where internal actions reach their maximum value. If these conditions are not met, low ductility must be considered.

For a dual system formed by concrete frames and walls, seismic actions are jointly resisted by concrete frames and concrete walls. There can be two configurations in this type of structuring. In one of them, some axes of the building's plan have only walls; in other axes, there are only frames. In the other configuration of this type of structural system, some axes have both walls and frames simultaneously. Since rigid horizontal diaphragms connect the walls and frames in both configurations, deformations must be compatible with both systems. That is, they must have the same configuration of horizontal deformations. This circumstance is essential because frames deform in horizontal shear forces, while walls deform in a bending configuration. However, by acting together, the stiffness and strength of the system increase, especially in the upper floors.

For this reason, this type of structuring is more convenient in taller buildings than those that are effective when frames or walls are used separately. The system can be of high, medium, or low ductility. Depending on the selected level, the beams, columns, joints of the frames, and the shear walls must meet the requirements for that level of structuring 1 and 4, respectively. In any case, frames and walls must be detailed to have the same level of ductility. Verifying that the frames alone can resist at least 30% of the horizontal shear force is essential. Note that the maximum allowable distortion (γ_{max}) is the lower of both types of structuring, namely, 0,020, which corresponds to concrete walls.

In systems Formed by Concrete Walls, there are reinforced concrete walls throughout the height of the buildings that work as vertical cantilevers subject to bending or flexural compression with great stiffness in their plane. They are also called structural walls, although the term is not entirely correct because their behaviour is not always governed by shear but rather by bending. It is a structuring suitable for tall structures where rigid frame systems (structuring 1) become inefficient and result in columns and beams of such size that they hinder the efficient use of interior spaces. The walls can be flat or form boxes or three-dimensional tubes around stairwells and elevator shafts. They can also have shoes at their ends; in this case, their functioning is similar to that of T or L beams. In any case, for them to work effectively, they must be appropriately embedded in the foundation. Suppose this condition is met, and other design details are specified in the NTC-DCEC:2017[34]. In that case, the concrete walls

can withstand large deformations due to bending, so they are allowed in structures with low, medium, and high ductility.

Dual System Comprising Coupled Concrete Frames and Walls: The difference between structures corresponding to structuring 3 is that the walls on the same axis are connected by beams called coupling beams. The effect of these beams is that the walls work together and flex around the centroidal axis of the assembly. Therefore, their stiffness and resistance to bending are much greater than the sum of the stiffnesses or resistances of independent walls. This structural system is effective in tall buildings. The coupling beams, whose design is specified in section 10.3.7 of the NTC-DCEC:2017[34], predominantly work in compression and tension, so they cannot achieve high ductility levels. For this reason, these systems are only allowed with medium and low ductility and maximum distortions of 0,015, lower than the system of uncoupled frames and walls.

Dual System Comprising Concrete Frames and Metal Bracing: Combining frames with bracing diagonals is standard in tall buildings. The combined action of the frames and the diagonals is similar to that of a vertically placed truss, where the frame columns are the top and bottom chords, the bracings are the diagonals of the truss web, and the beams are the members perpendicular to the top and bottom chords. The system is very effective in resisting lateral forces since these produce tension or compression forces in the truss diagonals, depending on the direction of these forces. The actions in the beams are small, and in the columns, they result mainly in axial loads. The diagonals can be made of concrete or steel, but since they can work in tension or compression, steel diagonals are much more commonly used since concrete ones are not efficient under tension. These systems can achieve high ductility, so a seismic behaviour factor, Q, of 4 is allowed in Table 4.2.1. The maximum distortion is limited to 0,020. The design of concrete frames must comply with the provisions of the NTC-DCEC:2017[34] and that of bracings with those of the NTC-DCEC:2017[34]. These provisions apply to bracings whose height is of a single story or multiple stories.

System Comprising Uncoupled Frames and Wall Diaphragms: In some cases, it is considered convenient for the interior or exterior walls of buildings to remain uncoupled from the frames so that the frames can deform under seismic or wind actions without affecting the walls, which are typically constructed of brittle materials. These walls are called partition walls. When the walls, whether concrete or masonry, are connected to the frames, they are called wall diaphragms, and being linked to the frames, they contribute to the stiffness and resistance of the assembly. Their structural function is similar to compression diagonals that go from one node to another of the frame. The frames must withstand at least 70% of the horizontal shear forces in this system. These systems can be designed with seismic behaviour factor values and maximum distortions equal to those of systems with concrete frames and metal bracings. The design of frames and walls must comply with the applicable provisions of the Complementary Technical Standards according to the selected ductility level.

System Based on Cantilever Concrete Columns: This type of structuring refers to columns that support beams or floor systems without continuity with the supports. They are found, for example, in industrial roofs based on frameworks freely supported on columns. The essential characteristic is that they do not form frames with the beams, so they do not have the advantages of hyperstatic or redundant systems. Therefore, the structural system must have low ductility, and the maximum distortion is limited to 0,010. However, the columns must still be designed as medium or high ductility elements to avoid an utterly fragile assembly failure.

Suspended System Supported by a Concrete Core Comprising Walls or Frames: In this system, a central core consisting of concrete walls or frames of great rigidity is used, which works as a cantilever in the form of a tube embedded in the foundation and resists lateral actions from earthquakes or wind. Because of this characteristic, the system is known as a central core system. A horizontal structural system comprising beams and slabs protrudes at each floor level, resisting gravitational loads. This horizontal system also works as a cantilever embedded in the central core. Although the central core comprises several walls or frames, it remains a structural element resisting lateral actions. Therefore, the system lacks the advantages of hyperstaticity or structural redundancy, although not as critically as in the previous case. For this reason, seismic behaviour factors corresponding to high ductility are not allowed in this system.

Outer Frames and Inner Columns Interconnected by Rigid Horizontal Diaphragms: Buildings using this structural system have a very rigid frame with small bays around their outer perimeter, i.e., with columns close to each other. Since this configuration cannot be used on the ground floor because it would prevent wide entrances, a very rigid beam, called a transfer beam, is usually placed on the first floor, serving as a transition between the upper part of frames with small bays and the ground floor with greater height and bays. This system has often been used in tall buildings. The system is also known as a tube structure because the perimeter frame has small bays resembling a tube with perforations. Buildings with this system also tend to have a rigid central core where elevators and stairs are located; thus, they are known as tube-in-tube structures. Concrete frames are used between the outer and inner tubes, which can be designed to resist gravitational loads, with rigid horizontal diaphragms to connect the outer and inner tubes. This structural system allows seismic behaviour factors and maximum distortions corresponding to medium and high ductility.

System Comprising Columns and Interconnected Frames or Walls with Flat Slabs: It is a dual system where rigid frames or concrete walls, usually placed on the outer perimeter, resist most lateral actions because they are stiffer than the flat slab and column system. This second system is designed to resist gravitational loads and the portion of the corresponding lateral actions. Flat slabs are usually post-tensioned when this system is used, allowing for large spans and reducing vertical deflections. Specifications for the design of these post-tensioned flat slab systems are presented in section 11.7 of the NTC-DCEC:2017[34]. These systems can be designed with seismic behaviour factors and maximum distortions corresponding to medium and low ductility, as the flat slab and column system cannot achieve high ductility, especially in the connections of flat slabs and columns.

System of Interconnected Concrete Columns with Flat Slabs: Unlike the previous system, the columns and flat slabs must resist gravitational and lateral actions without the aid of rigid walls or frames. Low ductility is allowed since this system cannot accept large deformations, especially when connecting slabs and columns.

2.1.4.3 STRUCTURAL REGULARITY

According to NTC-DS:2020[33], experiences from the increased damage caused by the 1985 earthquake in buildings with irregular structuring led to subsequent editions of these standards of more significant resistance requirements for irregular buildings. This is through a limitation of the reduction factor of spectral ordinates (Q') when specific requirements are not met for a structure to be considered regular. The requirements refer, in part, to the shape of the building, which should ideally be symmetrical in plan and without discontinuities in plan and elevation. For the most part, they refer to the structural system of the building, which must allow a clear

and compelling path for inertial forces to be transmitted between the structural elements of different floors and the foundation and between those of the same floor in order to avoid discontinuities, concentrations, or local amplifications of deformations and acting mechanically.

NTC-DS:2020[33] and NTC-DCEC:2017[34], some changes have been made to the text and the limits of the various requirements regarding the shape and configuration of the structural system, which, if exceeded, lead to classification as irregular or highly irregular, as well as the weight that each requirement carries in the penalty that the Standards apply for irregularity. Regular and well-detailed structural systems exhibit better seismic performance than those with structural irregularities and ordinary detailing. Regardless of whether the design requirements of these Standards are followed regarding the consideration of structural irregularity, it will always be desirable to limit it in such a way as to provide greater certainty and level of structural safety to the design.

2.1.4.3.1 Criteria for regularity

According to NTC-DS:2020[33], for a structure to be considered regular, it must satisfy several requirements. In the present work, it will be presented the main criteria to classify a seismic resistant structure as regular as follows:

a) The different walls, frames, and other vertical seismic-resistant systems are essentially parallel to the principal orthogonal axes of the building. A seismic-resistant plane or element is considered essentially parallel to one of the orthogonal axes when the angle it forms in the plan concerning that axis does not exceed 15 degrees

b) The ratio of its height to the smaller dimension of its base is not greater than four.

c) The ratio of length to width of the base is not greater than four.

d) The floor plan has no protrusions or recesses larger than 20% of the plan dimension measured parallel to the direction in which the protrusion or recess is considered.

e) Each level has a floor system whose stiffness and resistance in its plane satisfy what is specified in section 2.7 of NTC-DS:2020[33] for a rigid diaphragm.

f) The floor system has no openings on any level that exceed 20% of its area on that level, and the hollow areas do not differ in position from one floor to another. The roof of the building is exempt from this requirement.

g) The weight of each level, including the live loads that must be considered for seismic design, is not greater than 120% of that of the immediate lower floor.

h) In each direction, no floor has a plan dimension greater than 110% of that of the immediate lower floor. Additionally, no floor has a plan dimension greater than 125% of the smaller dimensions of the lower floors in the same direction.

i) All columns are restrained on all floors in both analysis directions by horizontal diaphragms or beams. Consequently, no column passes through a floor without being connected to it.

j) All columns on each floor have the same height, although this may vary from floor to floor. The last floor of the building is exempt from this requirement.

k) The lateral stiffness of no floor differs by more than 20% from that of the immediately lower floor. The last floor is excluded from this requirement.

l) On no floor does the lateral displacement of any plan point exceed by more than 20 per cent the average lateral displacement of its ends

m) In systems designed for (Q=4), on no floor should the ratio of the resistant capacity to lateral load between the design action be less than 85% of the average of such ratios for all floors. In systems designed for ($Q \leq 3$), on no floor should the ratio above be less than 75% of the average of such ratios for all floors. To verify compliance with this requirement, the resistant capacity of each floor will be calculated, considering all elements that may contribute significantly to it. The last floor is exempt from this requirement.

2.1.4.3.2 Criteria for irregularity

According to NTC-DS:2020[33], any structure failing to meet one of the requirements described in 2.1.4.3.1 is considered irregular. Irregularity in a building is primarily determined by requirements affecting the structure's ability to dissipate energy, mainly if it is concentrated in certain parts of the structure. The shape of the building has a lesser impact. Penalties aim to discourage the design of irregular structures.

Moreover, according to NTC-DS:2020[33], structures are classified as highly irregular if they fail to satisfy two or more of the requirements e) to m) described in 2.1.4.3.1 or if any of the following conditions occur:

1) The lateral displacement of a point on a floor exceeds 30% of the average displacements of its ends.

2) Any floor's lateral stiffness or shear resistance exceeds 40 per cent of the immediately lower floor's. To verify compliance with this requirement, each floor's resistant capacity and lateral stiffness will be calculated, considering all the elements contributing significantly to them.

3) For highly irregular structures, penalties are more severe due to the increased impact of factors affecting seismic response. Structures are classified as highly irregular when they fail to meet two or more of the most critical irregularity requirements.

2.1.4.4 SEISMIC ACTIONS

According to NTC-DS:2020[33], when employing the modal dynamic analysis method defined, seismic design actions will be determined based on the design spectra contained in the Seismic Action Design System (SASID), referred to as SASID, for the specific location. Within this database, the elastic spectrum for the construction site and the spectrum affected by the reduction factors for seismic behaviour (Q') and over-strength (R) are available.

When employing the static analysis method, design actions for evaluating the collapse prevention limit state will be derived using parameters obtained from SASID. To assess the damage limitation limit state, the maximum inter-story drifts must be analysed using the design spectrum adjusted according to seismic behaviour and over-strength factors and then multiplied by ($Q' \cdot R$) and the factor (K).

2.1.4.4.1 Response spectra

According to NTC-DS:2020[33], the design spectra in the SASID follow the format described below, which will also be used to construct the site spectra and those required when considering soil-structure interaction. The ordinates of the elastic pseudo-acceleration spectrum as a fraction of gravity, a, will be determined based on the structure's vibration period (T) and basic parameters, according to the system of equations (2-64):

$$a = \begin{cases} a_0 + (\beta_c - a_0) \cdot \dfrac{T}{T_a} \; if \; T < T_a \\ \beta_c \qquad\qquad if \; T_a \le T < T_b \\ \beta_{cp} \cdot \left(\dfrac{T_b}{T}\right)^2 \qquad if \; T \ge T_b \end{cases} \tag{2-30}$$

The ground acceleration coefficient (a_0), coefficients k, and characteristic periods (T_a) and (T_b) of the spectral plateau, as well as the dominant period of the site (T), will be taken from the SASID when geotechnical studies do not indicate anomalies in the subsurface characteristics compared to the surrounding area. The reduction factor (β) considers supplementary damping (greater than 0,05) due to soil-structure interaction effects or viscous-type dissipators.

The elastic design spectra are the starting point for calculating design lateral forces and determining lateral deformations in structures. Therefore, they should provide clear indications to the designer about the acceleration levels that may occur on-site and the maximum demands, both in acceleration and displacement, that the structures there would experience. Given their simplified and parametric form, design spectra do not pose a uniform hazard. For the collapse prevention limit state, their ordinates are associated with return periods of at least 250 years; for the serviceability limit state, the return period is at least 20 years. The spectral shapes for ($T<T_b$) are the same as those used in the Standards for many years. However, for ($T>T_b$), the shape is new, which provides a more accurate description of displacement spectra in that period interval. Figure 2-17 illustrates the shape of the displacement spectra implicit in the proposed design spectra.

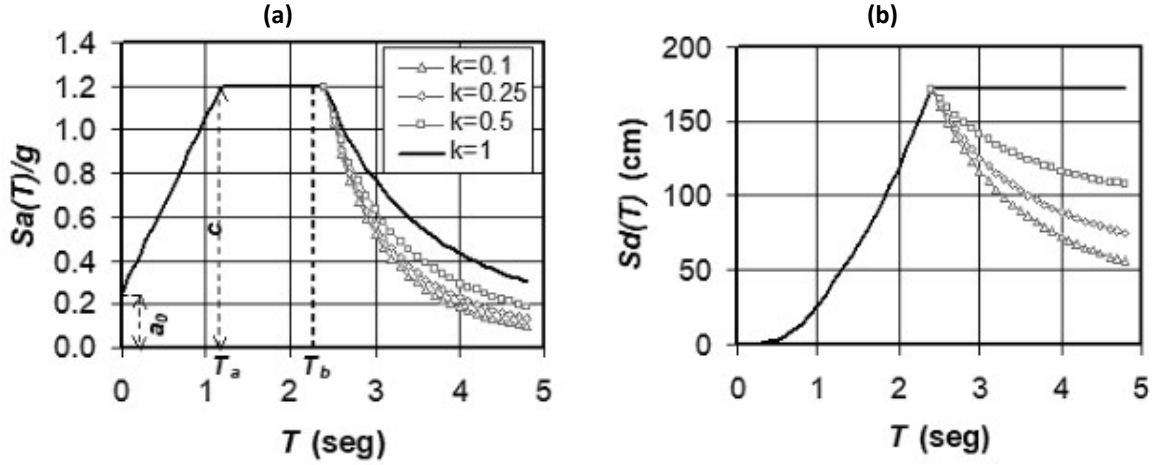


Figure 2-17: Seismic Characteristic shapes of the design spectra, both for pseudo-acceleration and displacement. Note the variation in the ordinates for ($T>T_b$) depending on the parameter (k) (Adapted from NTC-DS:2017[33])

2.1.4.4.2 Site-specific spectra

Site-specific design spectra should only be determined when geotechnical studies indicate anomalies in the subsurface characteristics compared to the surrounding area. An anomaly in soil characteristics is considered to exist when the predominant period of the site differs by more than 25 per cent from that determined using SASID for the same site. The site-specific study will consider all variables that may significantly influence the relationship between uniform hazard spectra characteristics in the "Hilly Zone" and those corresponding to the site-specific conditions. These include site stratigraphic and topographic characteristics, wave propagation and modification mechanisms from the firm ground base to the surface, soil-structure kinematic interaction, and the cyclic behaviour properties of the materials that waves propagate.

The design spectra should be obtained from SASID. However, the regulations acknowledge the possibility of misclassified sites in Mexico City regarding soil type. In such cases, a study is authorized to determine the site spectrum to use.

2.1.4.4.3 Soil-structure interaction effects

Both for cases where the spectrum is obtained from SASID and those where it is determined by a study based on specific soil properties, seismic design response calculation will consider the effects of soil-structure interaction, kinematic and inertial.

The soil-structure interaction effects occur when deformations between the foundation system and the surrounding base terrain are sufficiently large, relative to the expected deformation in the structure, to be considered. These effects can modify fundamental dynamic parameters of the soil-structure system, especially those associated with the fundamental mode of vibration in each orthogonal direction of the structure.

2.1.4.4.4 Importance Factor

According to the Mexico City Building Regulations, the spectral ordinates shall be multiplied by 1,5 for buildings belonging to Subgroup A1, which are identified as having extreme consequences in the event of their failure, either due to the size of the losses in the building itself or those they can cause in their surroundings. Those important to remain operational in an emergency are also included, and 1,3 are for those of Subgroup A2. The same factors must be applied to design actions using the static method. The importance factors will be considered when using the nonlinear step-by-step dynamic analysis method.

2.1.4.4.5 Spectral Ordinate Reduction Factors

Spectral ordinates corresponding to the elastic spectrum can be divided for design purposes between the over-strength factor (R) and the seismic behaviour reduction factor (Q') calculated according to the system of equations (2-31):

$$Q' = \begin{cases} 1 + (Q-1) \cdot \sqrt{\frac{\beta}{k}} \cdot \frac{T}{T_a} & if\ T < T_a \\ 1 + (Q-1) \cdot \sqrt{\frac{\beta}{k}} & if\ T_a \le T < T_b \\ 1 + (Q-1) \cdot \sqrt{\frac{\beta \cdot p}{k}} & if\ T \ge T_b \end{cases} \quad (2\text{-}31)$$

where (Q) is the seismic behaviour factor for different structures. For the design of irregular structures, the value of (Q') will be affected.

The regulations foresee two separate reduction factors. The first accounts for force reduction due to ductile behaviour. The standard accepts that the structure exhibits nonlinear behaviour for the earthquake used for the collapse prevention limit state review. This allows for the limiting force demands on structural elements and, therefore, uses lower design strengths in exchange for limited ductility demands and a certain level of damage caused by the cracking and yielding of some structure sections. To model nonlinear behaviour, virtually all building codes worldwide are based on analysing a single-degree-of-freedom system with elastoplastic behaviour. This model determines the necessary strength to limit ductility demands to a specified value, denoted as (Q). It is common to express the necessary strength to achieve a given ductility demand, C(T,Q), as a fraction of the necessary strength for unit ductility demand (elastic behaviour), C(T,1). It should be noted that both strengths depend on the structural period (T).

Note from Figure 2-18 that the value of (Q') can be greater than (Q). This happens only when (k<1), which in turn occurs for structural systems founded on soft soil. In other words, (Q') can be greater than (Q) for soft soils. In certain periods, seismic design demands for systems exhibiting degrading behaviour can be significantly higher than those corresponding to elastoplastic behaviour systems. The regulations do not consider a corrective factor for degrading behaviour because it was deemed that the requirements considered to establish the simplified and parametric form of design spectra result in reasonably conservative spectral ordinates for the design of concrete, steel, or composite structures, and masonry, detailed following the requirements posed by the corresponding Complementary Technical Standards for the material in question.

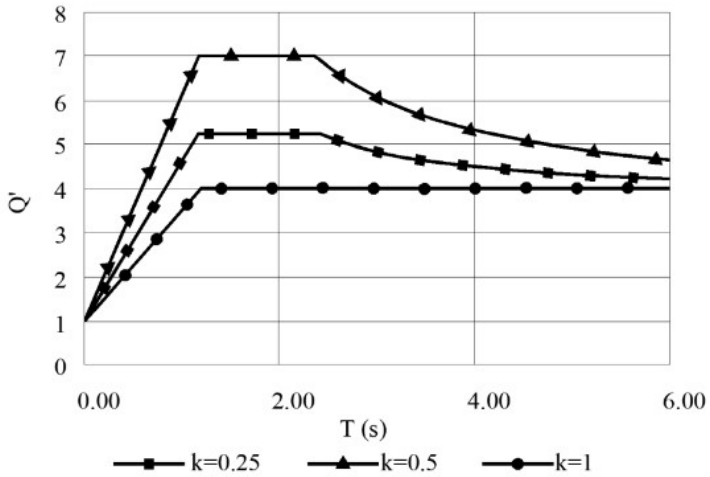


Figure 2-18: Value of (Q') for Q=4 and various (k) values for a site with T_s=2 s.

2.1.4.5 BEHAVIOUR FACTOR Q

For the seismic behaviour factor (Q) and the maximum inter-story drift (γ_{max}), the specified values in the NTC-DS:2017[33] are adopted, depending on the material composition of the structures: concrete, steel, composite materials and level of ductility. The corresponding Technical Standards for each material outline specific requirements for categorizing a structure as exhibiting high, medium, or low ductility. High ductility is achieved when a structural system meets the detailed requirements of the relevant technical standard for member and connection design. Medium and low ductility levels are similarly categorized within this framework.

The values provided for the seismic behaviour factor and maximum inter-story drift in these Standards apply to structures utilizing integrated structural systems as defined in NTC-DS:2017[33]. When combining two or more structural systems listed in the NTC-DS:2017[33], the lowest (Q) value among them contributing to seismic resistance in the direction of analysis is utilized for the combined system. The behaviour factor (Q) values may differ between the two orthogonal directions analysed. If analysis tools permit, the appropriate factor may be applied in each direction, provided interstorey distortions and three-dimensional effects are considered. Alternatively, the system may be designed based on the smaller (Q) values of the two analysed directions. For structures relying on elements or materials not specified in the tables, (Q=1) is used for lateral force resistance design. Structural systems may exhibit maximum distortions significantly below the prescribed (γ_{max}) value in some cases. This should not be seen as undesirable or abnormal but rather as a scenario where the seismic demand for lateral deformation is lower than the structural system's capacity to accommodate it.

2.1.4.6 STRUCTURAL ANALYSIS

The NTC-DS:2020[33] considers seismic design forces acting on the structure are determined by one of the following methods:

a) Static analysis.

b) Dynamic analysis.

   i. Modal Dynamic Analysis.

   ii. Nonlinear Time History Analysis.

The following subsections present the different methods to calculate the seismic actions.

2.1.4.6.1 Static analysis

NTC-DS:2017[33] outlines how to apply the seismic analysis requirements to structures. It begins by establishing that the structure's base level, where its movements relative to the surrounding terrain become noticeable, serves as the starting point. Different types of structural behaviour, such as torsion, bidirectional effects, and asymmetric behaviour, are considered based on specific guidelines provided in the document.

Additional rules must be followed when designing structural elements like inverted pendulums and diaphragms. NTC-DS:2020[33] also emphasizes that dynamic analysis is preferred for seismic design due to its accuracy, especially for tall or irregular structures. Static analysis may be used for simpler and regular structures, but it has limitations. These limitations include the type of structure and its height and adherence to specific requirements regarding diaphragm rigidity and plan symmetry.

The procedure to calculate the design shear forces on the floors of a structure involves assuming a set of horizontal forces acting on each point where the masses of the floors are concentrated. Each of these forces is determined by multiplying the weight of the corresponding mass by a coefficient proportional to hi, which represents the height of the mass above the base of the structure. The coefficient is chosen such that the ratio (V_0/W_0) is equal to [$c/(Q'\cdot R)$] but not less than (a_0/R), where (a_0) is the spectral ordinate corresponding to (T=0 s), and (c) is the coefficient specified in the section of the code where the spectra are defined. According to this requirement, the lateral force acting at the i-th level (F_i) is determined as follows:

$$F_i = \frac{c}{Q' \cdot R} \cdot W_i \cdot h_i \cdot \frac{\sum_i^n W_i}{\sum_i^n W_i \cdot h_i} \tag{2-32}$$

where (W_i) is the weight of the i-th mass, (h_i) is the height of the i-th mass above the base, and (Q') is the seismic behaviour reduction factor defined. To estimate the lateral forces using equation (2-32), the value of (Q') must be evaluated using equation (2-31) for the case where (T) is within the interval from (T_a) to (T_b). The coefficient (c) will be obtained from the SASID.

NTC-DS:2020[33] states that the static analysis methods are based on simplified models and may not provide precise results for complex structures. Therefore, they are unsuitable for high-risk structures or those with significant irregularities. This highlights the importance of considering dynamic analysis methods for accurate seismic design, such as Modal Dynamic Analysis (MDA) and Nonlinear Time History Analysis (NTHA).

2.1.4.6.2 Dynamic analysis

The dynamic modal analysis and the nonlinear dynamic step-by-step analysis are accepted as dynamic analysis methods by the NTC-DS:2020[33]. For the application of the requirements of this chapter, the level from which its displacements concerning the surrounding terrain begin to be significant will be taken as the base of the structure. The dead and live loads corresponding to dead and live loads corresponding to the different floors are considered when calculating the weight of the different floors, according to the Supplementary Technical Standards on Criteria and Actions for the Structural Design of Buildings.

The effects of torsion, second-order effects, bidirectional effects, and asymmetric behaviour are considered according to the code section where the torsion effects are defined. Modal dynamic analysis can be used to design any structural system. The code section methods for seismic analysis indicate the cases in which verifying the structural design with a Nonlinear Time History Analysis (NTHA) will be necessary.

2.1.4.6.3 Modal dynamic analysis

According to NTC-DS:2020[33], the Modal Dynamic Analysis (MDA) is an elastic analysis method that estimates the contribution of each mode of vibration. The total response of the system is determined by overlaying the modal responses of those modes that are considered to contribute appreciably to that response. It is useful during design because it relates the properties of the structural system to its response and seismic demands. Its use requires a detailed three-dimensional structural system model that considers the participation of all construction elements that may significantly influence its seismic response.

According to NTC-DS:2020[33], the modal dynamic analysis must utilize a three-dimensional elastic model and include the effect of natural modes of vibration, ordered according to decreasing values of their periods of vibration, which are necessary for the sum of the effective weights in each analysis direction to be greater than or equal to 90% of the total weight of the structure. The effective modal weights are calculated using equation (2-33):

$$W_{ei} = \frac{(\{\varphi_i\}^T [W] \{J\})^2}{\{\varphi_i\}^T [W] \{\varphi_i\}} \quad (2\text{-}33)$$

where (φ_i) is the amplitude vector of the i-th natural mode of vibration of the structure; (W_{ei}) is the mass weight matrix of the structure; and (J) is a vector formed with "ones" in the positions corresponding to the degrees of freedom of translation in the analysis direction and "zeros" in the other positions.

To determine how each natural mode contributes to the lateral forces on the structure, utilize the design spectra, adjusted according to (Q) and (R) factors. The modal responses, encompassing shear force, lateral displacement, overturning moment, and other factors, are then aggregated to compute the overall responses (S) using the following expression (2-34), provided that the periods of the natural modes in question differ by at least 10 % from each other:

$$S = \sqrt{\sum_{i=1}^{n} S_i^2} \quad (2\text{-}34)$$

otherwise, the complete quadratic combination (CQC) must be used as depicted in expression (2-35):

$$S = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n} \rho_{ij} \cdot S_i \cdot S_j} \tag{2-35}$$

where (S_i) and (S_j) can be positive or negative, and (ρ_{ij}) is a correlation coefficient between the *i*-th and *j*-th modes of vibration, which is greater than zero

2.1.4.6.4 Nonlinear time history analysis

When employing the nonlinear step-by-step dynamic analysis method, pairs of accelerograms recorded during real earthquakes, simulated motions, or combinations thereof must be used in the two principal analysis directions. When the site period (T) is less than 1,0 seconds, at least eight pairs of representative motions must be used. At least twelve pairs must be used for (T_s) equal to or greater than 1,0 s.

The representative motions should be independent and have intensities, durations, and frequency content similar to ground motions recorded during earthquakes with intensities assumed for design. These correspond to uniform hazard spectra determined as specified in Section A.1., considering the most unfavourable condition that may occur, taking into account the variation in natural periods of the ground during the expected life of the structure due to the desiccation process that occurs in Transition Zones and the Lake area.

The time variation forms of amplitudes and frequency contents of the accelerograms used, and their effective durations must be consistent with those observed in real records of motions corresponding to the most probable combinations of magnitude, source-to-site distance, and focal mechanism leading to the considered intensity. For this purpose, the intensity will be measured by the ordinate of the elastic uniform hazard spectrum of pseudo-accelerations for the structure's natural period corresponding to the dominant mode of vibration, which generates the greatest contribution to the base shear force.

To determine the natural periods of vibration of the structure, mechanical properties and gravitational loads will be taken as expected values instead of nominal values resulting from the design process. For Group A structures, the ordinate of this spectrum must be multiplied by the importance factor. The nonlinear step-by-step dynamic analysis must be performed with a three-dimensional model considering the response to two simultaneous orthogonal horizontal components. Therefore, each seismic excitation will be represented by a pair of orthogonal horizontal components with similar individual intensities. Individual intensities obtained from field records should be considered for real motions for the components. The intensity of each seismic excitation will be measured with a spectrum whose ordinates are established by the following expression:

$$a_{es}(T) = \sqrt{\frac{a_{c1}^2\,(T) + a_{c1}^2(T)}{2}} \tag{2-36}$$

where (a_{es}) is the ordinate of the spectrum characterizing the intensity of the seismic excitation, and (a_{c1}) and (a_{c2}) are the spectral ordinates, corresponding to elastic spectra of pseudo-acceleration as a fraction of gravity obtained for the two individual horizontal components and

damping of 5% of critical. The parameter (T) is the period corresponding to the three spectral ordinates under consideration in equation (2-36).

Both real and simulated pairs of accelerograms will be affected by scaling factors leading to reasonable coverage of individual spectrum ordinates measuring the intensity of seismic excitations considered within elastic hazard spectra provided by SASID. This is considered to occur when in a range of periods between 0,1 and 1,5 times the natural period of the first mode of vibration in the structural system in the analysis direction, the average spectrum per family and epoch should not systematically underestimate its corresponding uniform hazard spectrum.

The same scaling factor must scale both components of each pair. For structures with unidirectional structural elements, whose failure is brittle and significantly affects the stability of the structural system in the analysis direction (e.g., buckling of a bracing element), it will be necessary to perform a series of additional nonlinear dynamic analyses. For the additional analyses, it will be considered that the higher-intensity component, scaled by a factor of 1,3, acts in the direction of interest. The component with lower intensity, orthogonal to the first, will be scaled by a factor of 0,6. For this purpose, the intensity of each component will be measured with the spectral ordinate corresponding to the first mode of vibration of the structural system in the analysis direction."

NTC-DS:2020[33] outlines requirements for structural system modelling in seismic analysis. It mandates three-dimensional models with at least three degrees of freedom per floor, including translations in two orthogonal horizontal directions and rotation about a vertical axis, accurately representing spatial mass distributions and lateral stiffnesses. Plasticity-concentrated models may be used for structural members predominantly damaged in flexion, provided the distortion demands remain within specified limits. Otherwise, hysteresis behaviour must be modelled based on laboratory tests, considering all significant characteristics related to yield, stiffness and strength degradation, and cyclic response characteristics.

The properties of the foundation should be established based on hypotheses consistent with specific site soil properties and supported structural mechanics and material behaviour models. Explicit modelling of structural system connections is not required, though rigid zones at the ends of structural members may be removed to account for flexibility and nonlinear behaviour. Damping should be modelled with a Rayleigh damping matrix, and the critical damping percentage for each vibration mode should not exceed 2,5%. Non-rigid floor diaphragms must be adequately modelled, accounting for their flexibility and required degrees of freedom to include deformations. Second-order effects should be considered if significant.

2.1.4.7 DRIFT AND STABILITY

NTC-DS:2020[33] According to the interstorey, drift (γ_i) is defined as the difference between the lateral displacements (δ_i) of consecutive floors divided by the corresponding difference in elevations (h_i).

$$\gamma_i = \frac{\delta_i}{h_i} \tag{2-37}$$

For review purposes, the lateral displacements are obtained from the analysis conducted with seismic design forces, and the highest interstorey drift calculated for each vertical element or subsystem contained within the floor (frames, walls, or any other vertical element) must be

considered. The lateral displacements are to be reviewed for the following two design conditions:

a) To meet the ultimate collapse limit state, it will be verified that the interstorey drifts obtained with the design spectrum, multiplied by ($Q \cdot R$), do not exceed the specified values for the maximum interstorey drift (γ_{max}), depending on the adopted structural system. (Q) is the seismic behaviour factor. The value of (R) will be calculated for the fundamental period of vibration of the structure. The lateral displacements and interstorey drift for this condition will also be used to review the requirements for separating neighbouring buildings and calculating second-order effects.

b) To meet the limit state of damage limitation for frequent earthquakes, it will be reviewed that the floor interstorey drifts determined as indicated in subsection 3.1.1 do not exceed 0,002 unless all non-structural elements can support significant deformations or are separated from the main structure in a way that they do not suffer damage from their deformations. In such a case, the limit in question will be 0,004. When calculating the interstorey drifts mentioned in this paragraph, those due to the overall bending of the structure can be discounted.

In previous editions of this standard, the review of lateral floor displacements served only to meet the requirement that non-structural elements did not suffer damage during frequent earthquakes. In this new edition, both the collapse prevention and damage limitation limit states must be reviewed.

For both limit states, the maximum floor interstorey drift is reviewed. The first aims to ensure that, for the design earthquake, the interstorey drift of no floor exceeds the lateral deformation capacity of the floor, which would cause a collapse of it or the entire building. The floor interstorey drift capacity must consider both the floor's linear and nonlinear behaviour; that is, it depends on the ductility of the structural system. Therefore, the results of the structure analysis under the design earthquake must be multiplied by (Q) and by (R) since this analysis assumes linear behaviour until failure and does not consider that the inelastic deformations are (Q) times greater than calculated; it also does not take into account that the strength that the structure can reach is (R) times greater than assumed in the analysis.

The maximum interstorey drifts thus obtained must not exceed those corresponding to the exhaustion of the floor deformation capacity, provided in Chapter 4, depending on the material and the structural system.

To make possible the design of the structural system under these circumstances, dual systems consisting of frames and stiffening structural elements, such as walls and bracing, and systems for controlling seismic response can be considered. The second review of lateral displacements refers to the limit state of damage limitation, which consists of ensuring that no floor exceeds the interstorey drift that would cause damage to non-structural elements or incipient damage to the structure itself. This review should be carried out for a spectrum with a much lower exceedance probability than the spectrum for which the collapse prevention limit state is reviewed. However, calculations performed for a return period of around 20 years indicate that the corresponding spectrum has a very similar shape to collapse prevention, but with its ordinates reduced by an amount that differs relatively little at the sites of the three geotechnical zones. For this reason, it is admitted that the interstorey drifts for reviewing the damage limitation limit state are taken from those obtained from the analysis to review the

collapse prevention limit state, but with the results reduced by a factor and depending on the geotechnical zone.

In practice, limiting floor interstorey drifts in the structural system for both limit states implies imposing lateral stiffness requirements during its design. The elements susceptible to damage are generally masonry walls. However, this specification must also apply to gypsum board or similar partition walls, ceilings, and especially facade elements such as glazing, glass, and prefabricated elements.

2.1.4.8 CAPACITY DESIGN APPROACH

The capacity for global mechanism control (flexion) and the capacity in shear is only demanded for the medium and high ductility structures.

Concerning the global mechanism control, according to NTC-DCEC:2017[34], the bending resistances of the columns at a joint must satisfy equation (2-38):

$$\sum M_e = \sum 1{,}2 \cdot M_g \tag{2-38}$$

where (ΣM_e) is the sum at the joint of the resistant moments in the plane of analysis calculated with a resistance factor equal to one from the columns that reach that joint; the resistant moment will be the one that corresponds to the factored axial load that, in a column interaction diagram, produces the lowest resistant moment.

When calculating the resistant moments in the plane of analysis, the moments acting in the perpendicular plane will not be considered, and(ΣM_g) is the sum at the joint of the resistant moments calculated with a resistance factor equal to one from the beams that reach the joint. The above sums must be performed so that the moments of the columns oppose those of the beams. The condition must be met for both directions in which the earthquake can act.

NTC-DCEC:2017[34] defines the capacity design in shear for beams, considering that the elements primarily subjected to bending will be designed so that shear failure does not occur before plastic hinges can form at their ends due to bending. For this, the design shear force (V) will be obtained from the equilibrium of the member between support faces, as depicted in expression(2-39):

$$V = \frac{M^+ + M^-}{l} + \frac{w_u}{2} \tag{2-39}$$

It will be assumed that moments of the same direction act at the ends (M^+ and M^-), as illustrated in Figure 2-19.

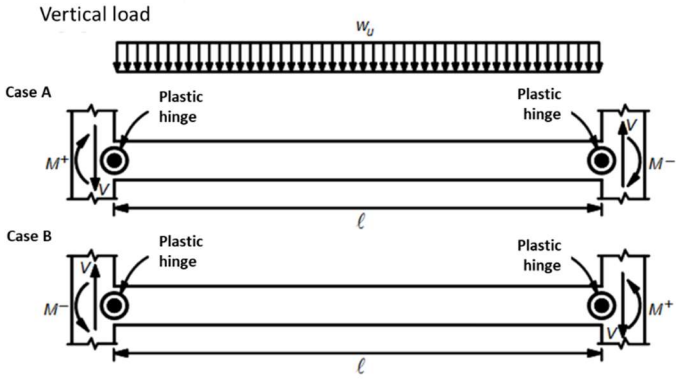


Figure 2-19: Determination of the design shear force (V) for capacity design

These moments represent an approximation of the bending resistance. They are valued with the element's properties at those sections, with a unit resistance factor, and with the stress in the tension steel at least equal to (1,25 f_y).

Regarding the capacity design in shear for columns, the NTC-DCEC:2017[34] states that flexural compression elements will ensure they do not fail due to shear force (V) before plastic bending hinges can form at their ends. For this, the design shear force (V) is calculated from the equilibrium of the element at its free height, assuming that at its ends act bending moments of the same direction, numerically equal to the moments that approximate the actual bending resistance of those sections, with a resistance factor equal to one, and obtained with the design axial load that leads to the highest resistant bending moment.

2.1.4.9 DETAILING FOR LOCAL DUCTILITY FOR BEAMS AND COLUMNS

NTC-DCEC[34] offers guidelines and regulations for designing and constructing concrete structures specifically tailored for Mexico City. This standard encompasses a range of critical elements in concrete construction, encompassing materials, structural design principles, and detailed construction specifications.

It should be noted that the Mexican codes define distinct performance criteria for local ductility: (i) Low ductility, (ii) medium ductility, and (iii) High ductility. This ensures that the structural elements are specifically tailored to meet the seismic requirements prevalent in the region, thereby enhancing the safety and resilience of buildings under earthquake conditions.

2.1.4.9.1 Materials

According to NTC-DCE[34], the concrete used for structural purposes may be of two classes: Class 1, with a fresh state volumetric weight greater than 22 kN/m^3 and Class 2, with a fresh state volumetric weight between 19 and 22 kN/m^3. Class 1 concrete shall be used for group A or B1 works. Class 2 concrete may be used in smaller structures, with spans not exceeding 5,0 m and heights of no more than two levels. Additional requirements for high-strength concrete with specified compressive strength (f'_c), equal to or greater than 40 MPa.

In the manufacture of concretes, any type of cement consistent with the purpose and characteristics of the structure shall be used, resistant class 30 or 40, complying with the requirements specified in standard NMX-C-414-ONNCCE[35],

Class 1 concretes shall have a specified compressive strength (f'_c) equal to or greater than 25 MPa. The specified strength of Class 2 concretes shall not be less than 20 MPa. In both classes, it shall be verified that the structural concrete level of the entire construction complies

with the specified strength. It shall be admitted that a concrete complies with the specified strength if it meets the requirements. Class 2 concrete with a specified strength of 15 MPa shall be allowed if the minimum covers are guaranteed in reinforced concrete walls of housing up to two levels. All structural concrete shall be mixed by mechanical means and dosed by weight.

The mean tensile strength of concrete is considered the average of the resisting tensile stresses obtained for Class 1 concrete $0{,}47 \cdot \sqrt{f'_c}$, in MPa and Class 2 concrete $0{,}38 \cdot \sqrt{f'_c}$, in MPa. For Class 1 concretes, the modulus of elasticity (E_c) shall be assumed equal to $4\,400\sqrt{fc'}$, in MPa for concretes with limestone coarse aggregate, and $3500 \cdot \sqrt{f'_c}$, in MPa for concretes with basalt coarse aggregate. For Class 2 concretes, they shall be assumed equal to $2500 \cdot \sqrt{f'_c}$, in MPa.

2.1.4.9.2 Design provision for beams and columns

NTC-DCEC[34] outlines special requirements for the three levels of ductility structural elements. These elements must comply with specific provisions detailed in the section and adhere to the general requirements outlined in preceding sections. This includes using Class 1 concrete, which must have a specified strength of at least 25 MPa. High-strength concretes must meet further requirements specified in another standard section.

The standards specify that reinforcing bars used in structural elements that experience axial and bending forces due to seismic activities must be corrugated and made from normal or low alloy steel. These bars should comply with the mechanical properties specified by Mexican standards NMX-C-407[35]. Furthermore, the yield strength for transverse reinforcement, including helical reinforcements, is capped at 420 MPa.

In NTC-DCEC:2017[34], beams and columns are discussed extensively, particularly focusing on their design load limits, sizing, and reinforcement requirements. The present work will summarise the main aspects of detailing for local ductility, such as the beam ratios for the cross-sectional dimensions of beams. The guidelines stipulate minimum and maximum reinforcement levels to ensure stability and safety, addressing flexural and transverse reinforcement needs.

Figure 2-20 illustrates the configuration of the transverse reinforcement in beams, highlighting the placement and spacing of the stirrups for medium and high ductility MRF structures in compliance with NTC-DCEC:2017[34].

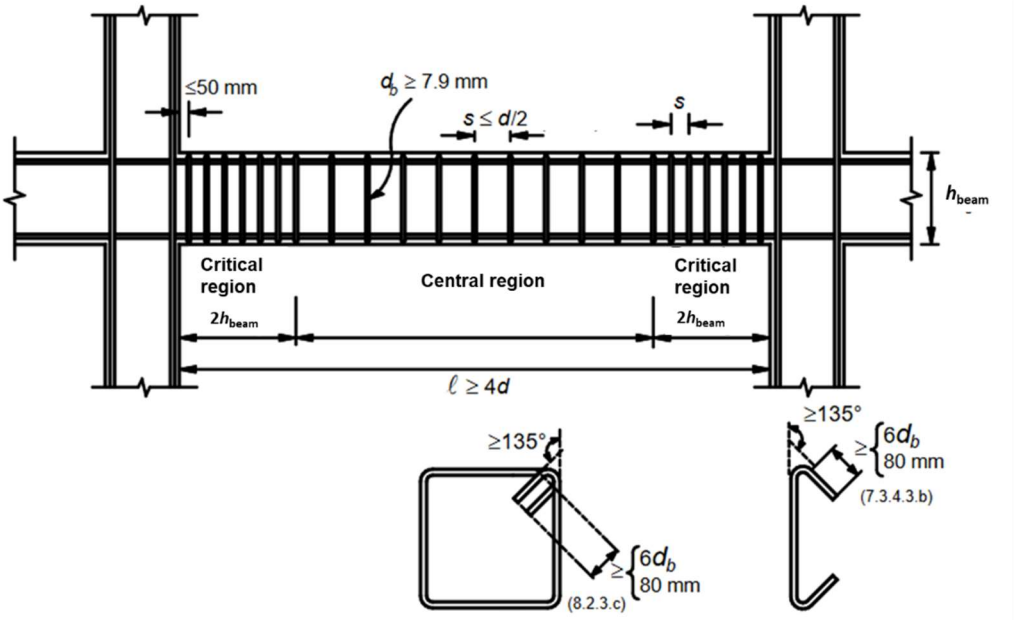


Figure 2-20: Detailing of a beam (Adapted from NTC-DCEC[34])

Table 2-12 summarises the key detailing requirements for beams, categorized according to the frame structure type.

Table 2-12: Detailing provisions for beam (NTC-DCEC:2017[34])

Beams		Low ductility	Medium ductility	High ductility
Critical region length		-	$2 \cdot h_{beam}$	
Detailing for local ductility – Longitudinal bars				
Minimum longitudinal reinforcement ratio	p_{min}	$\frac{0{,}22 \cdot \sqrt{f'_c}}{f_y}$		
Maximum longitudinal reinforcement ratio	p_{max}	$0{,}9 \cdot \left(\frac{f''_c}{f_y} \cdot \frac{600 \cdot \beta_1}{f_y + 600} \cdot b \cdot d\right)$	$0{,}75 \cdot \left(\frac{f''_c}{f_y} \cdot \frac{600 \cdot \beta_1}{f_y + 600} \cdot b \cdot d\right)$	0,025
Detailing for local ductility – Transverse reinforcement				
Outside critical regions				
Maximum hoop spacing	s	d/2		
In critical regions				
Minimum hoop diameter	d_{bw}	-	7,9 mm	
Maximum hoop spacing	s	-	$\min(d_{beam}/4; 8 \cdot d_b; 24 \cdot d_{b,hoop}; 300mm)$	

Figure 2-21 illustrates the configuration of the transverse reinforcement of columns, highlighting the placement and spacing of the stirrups for medium and high ductility MRF structures in compliance with NTC-DCEC[34].

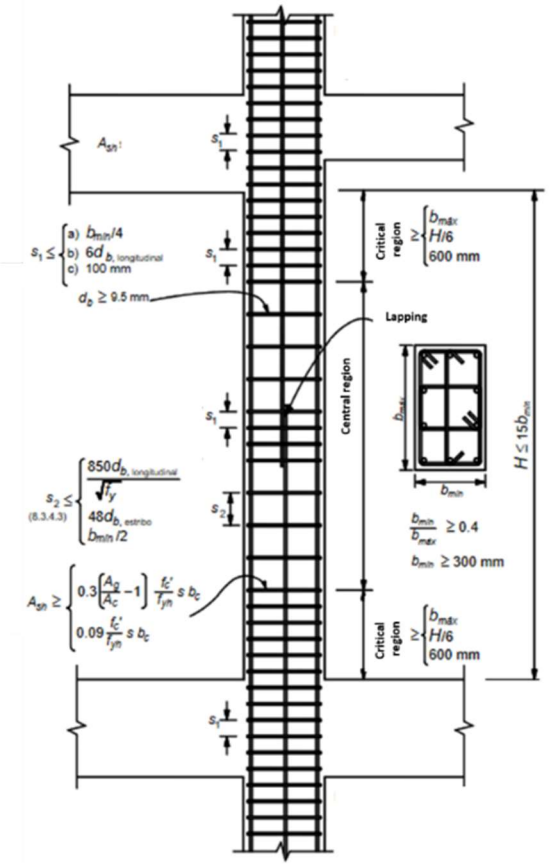


Figure 2-21: Detailing of column (Adapted from NTC-DCEC:2017[34])

Table 2-13 summarises the key detailing requirements for beams, categorized according to the frame structure type.

Table 2-13: Detailing provisions of columns (NTC-DCEC:2017[34])

Columns		Low ductility	Medium ductility	High ductility
Critical region length		-	max(b_{max};H/6;600mm)	
Minimum width	b_{min}		250mm	
Detailing for local ductility – Longitudinal bars				
Minimum longitudinal reinforcement ratio	$A_{s,min}$	0,01·A_g		
Maximum longitudinal reinforcement ratio	ρ_{max}	0,06·A_g		
Detailing for local ductility – Transverse bars				
Outside critical regions				
Minimum hoop bar diameter	d_{bw}	7,9mm	9,5mm	
Maximum hoop spacing	s	$min\left(\frac{850 \cdot d_{b,longitudinal}}{\sqrt{f_y}}; 48 \cdot d_{b,hoop}; \frac{b_{min}}{2}\right)$		
Within critical regions				
Minimum hoop bar diameter	d_{bw}	-	9,5mm	
Maximum hoop spacing	$s_1 \leq$	-	min($bmin$/4;6·d_b;100mm)	
Minimum traverse reinforcement area	$A_{sh} \geq$	-	$min\left(0{,}3 \cdot \left(\frac{A_g}{A_C} - 1\right) \cdot \frac{f'_c}{f_{yh}} \cdot s \cdot b_c; 0{,}09 \cdot \frac{f'_c}{f_{yh}} \cdot s \cdot b_c\right)$	

2.1.4.10 DESIGN AND DETAILING PROVISIONS FOR REINFORCED CONCRETE STRUCTURAL WALLS

The NTC-DCEC:2017[34] offers comprehensive guidelines that address the critical wall design and detailing requirements to ensure safety and functionality. The NTC-DCEC:2017[34] meticulously outlines how walls should be constructed to withstand vertical and horizontal loads, emphasising seismic resistance effectively. The standards delineate specific reinforcement placement, sizing, and buckling prevention measures, vital for maintaining structural integrity during seismic events. The following summary encapsulates the key aspects of these guidelines, focusing on the structural demands, safety considerations, and seismic resilience necessary for effective wall design in contemporary construction scenarios.

The provisions presented in this section apply to walls with low, medium and high ductility. In buildings with concrete walls in the foundation that are much stiffer than those above and with basement slabs that behave as rigid diaphragms in their plane, the total wall height (H_m) and the critical height (H_{cr}) are measured from the ground floor level.

This subchapter intends to summarize the mythologies of design and detailing for local ductility for the different classes of ductility for wall structures.

2.1.4.10.1 Bending moment and shear action in walls

According to NTC-DCEC:2017[34], the envelope bending moment for walls is calculated according to specific criteria for the three different ductility classes.

In walls where ($H_m/L \geq 2$), where (L) is the length of the wall, the design bending moment along (H_{cr}) will be considered with a constant value equal to the moment (M_u) obtained from the analysis at the base of the wall. The critical height (H_{cr}) is equal to the greater of (L) or ($M_u/4V_u$). From the wall height (H_{cr}), a linear bending moment diagram will be used that is parallel to the line connecting the moments calculated at the base and the top of the wall (Figure 2-22).

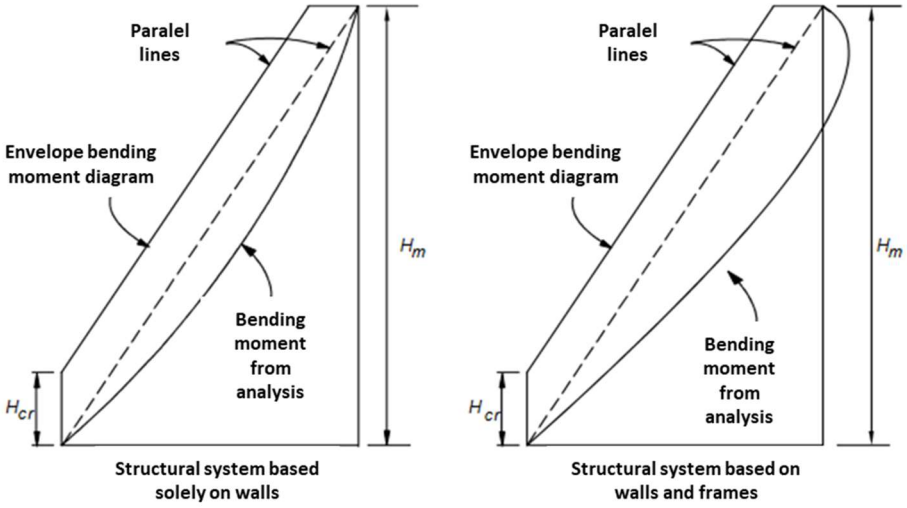


Figure 2-22: Envelope of the design bending moment (Adapted from NTC-DCEC[34])

Concerning the shear force, the NTC-DCEC:2017[34] does not foresee any magnification of shear force, though it provides guidelines for the shear reinforcement.

2.1.4.10.2 Detailing for local ductility

Table 2-14 summarizes the main detailing provisions for walls according to NTC-DCEC:2017[34].

Table 2-14: Detailing provisions for walls (NTC-DCEC:2017[34])

Detailing rules		Low ductility	Medium ductility	High ductility
In critical height region:				
Length from wall edge	$l \geq$	-	max(c/2;c-0,125·L_m)	max(c/2;c-0,1·L_m)
Length over thickness	L/t	If vertical loads less than 0,3·f'_c·A_g then L/t<70		
		If vertical loads less than 0,3·f'_c·A_g then L/t<40		
Thickness	b_{wc}>	130 mm	180 mm	250mm
Vertical reinforcement within the critical region				
Minimum longitudinal reinforcement ratio		$a_{sl} = \frac{660x_1}{f_y \cdot (x_1 + 100)}$		
Maximum longitudinal reinforcement ratio	ρ_{max}	0,008		
Confining hoops within critical regions				
Minimum hoop rebar diameter	d_{bw}	6mm	8 mm	
Maximum hoop spacing	s_w	-	min(8d_b,24$d_{b,hoop}$;t/2)	min(6d_b,100mm;t/4)

Figure 2-23 illustrates the detailing of local ductility for structures with medium ductility.

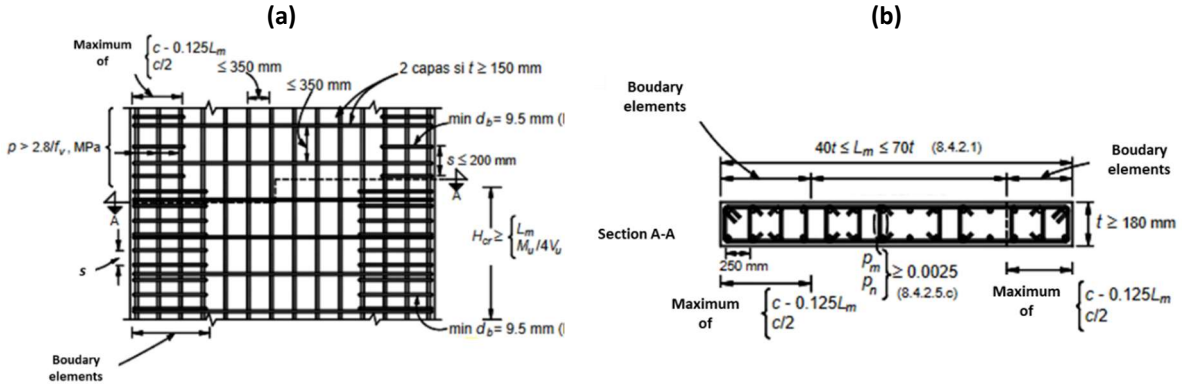


Figure 2-23: Detailing for local ductility of walls within the critical region for structures with medium ductility (Adapted from NTC-DCEC:2017[34])

Figure 2-24 illustrates the detailing for local ductility for structures with medium ductility.

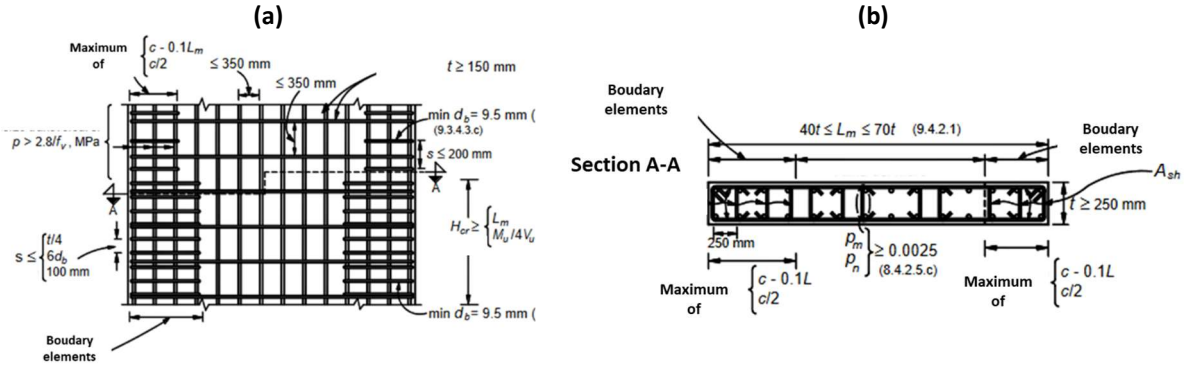


Figure 2-24: Detailing for local ductility of walls within the critical region for structures with high ductility (Adapted from NTC-DCEC:2017[34])

2.1.5 EUROPEAN SEISMIC CODE

This subchapter describes the features of EN 1998-1:2004[37], EN 1992-1-1:2004[63] and the new generation of Eurocode 8 regarding the clauses in the study in this research work. This chapter will run through the clauses of Eurocode 8 and Eurocode 2 concerning the design of concrete buildings.

Eurocode 8 applies to the design and construction of buildings and other civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes

- Human lives are protected
- Damage is limited
- Structures important for civil protection remain operational

Eurocode 8 is divided into six parts:

- Part 1 (EN 1998-1): “General rules, seismic actions, rules for buildings”.
- Part 2 (EN1998-2): Bridges.
- Part 3 (EN 1998-3): “Assessment and retrofitting of buildings”.
- Part 4 (EN1998-4): “Silos, tanks, pipelines”.
- Part 5 (EN 1998-5): “Foundations, retaining structures, geotechnical aspects”.
- Part 6 (EN 1998-6): “Towers, masts, chimneys”.

The European Commission considers the EN-Eurocodes as tools for integrating the construction market and all related services in the European Union (EU) and enhancing the competitiveness of European designers, contractors, consultants and construction products within and outside the EU.

The “Eurocode 2”, EN 1992-1-1:2004[63] deals with designing reinforced concrete structures of buildings, bridges and other civil engineering works. Eurocode 2 allows the calculation of action effects and resistances of concrete structures submitted to specific actions and contains all the prescriptions and good practices for adequately detailing the reinforcement, according to Biasioli et al.[64].

Eurocode 2 consists of three parts:

- EN 1992-1 Design of concrete structures - Part 1-1 General rules and rules for buildings,
- EN 1992-1 Part 1-2 Structural fire design
- EN 1992-2 Design of concrete structures - Part 2: Concrete Bridges – Design and detailing rules
- EN 1992-3 Design of concrete structures - Part 3: Liquid retaining and containment

The background to Eurocode 2 can be traced back to 1975 and the Treaty of Rome. The European Commission asked CEN (Centre European de Normalisation) to draft structural design standards that could be used across the Common Market.

EN 1992-1-1:2004[63] considers the plurality of drafting in addition to technical considerations. Safety factors, deflection rules, detailing and crack control, column design and the EN 206-1[65] concrete acceptance rules are assessed from technical and practical points of view. Some technical issues are satisfactory, and improvements on structural concrete national

codes, such as the old Portuguese code REBAP[66], are identified in several areas but severe inadequacies and anomalies.

2.1.5.1 STRUCTURAL SYSTEM

EN 1998-1:2004[37] in section 5 identifies the following types of structural systems for concrete buildings, depending on how the system responds to the horizontal components of the seismic action:

- Inverted pendulum systems.
- Torsionally flexible systems.
- Frame systems.
- Dual systems of frames and walls.
- Wall systems (coupled or uncoupled walls).
- Systems of large lightly reinforced walls.

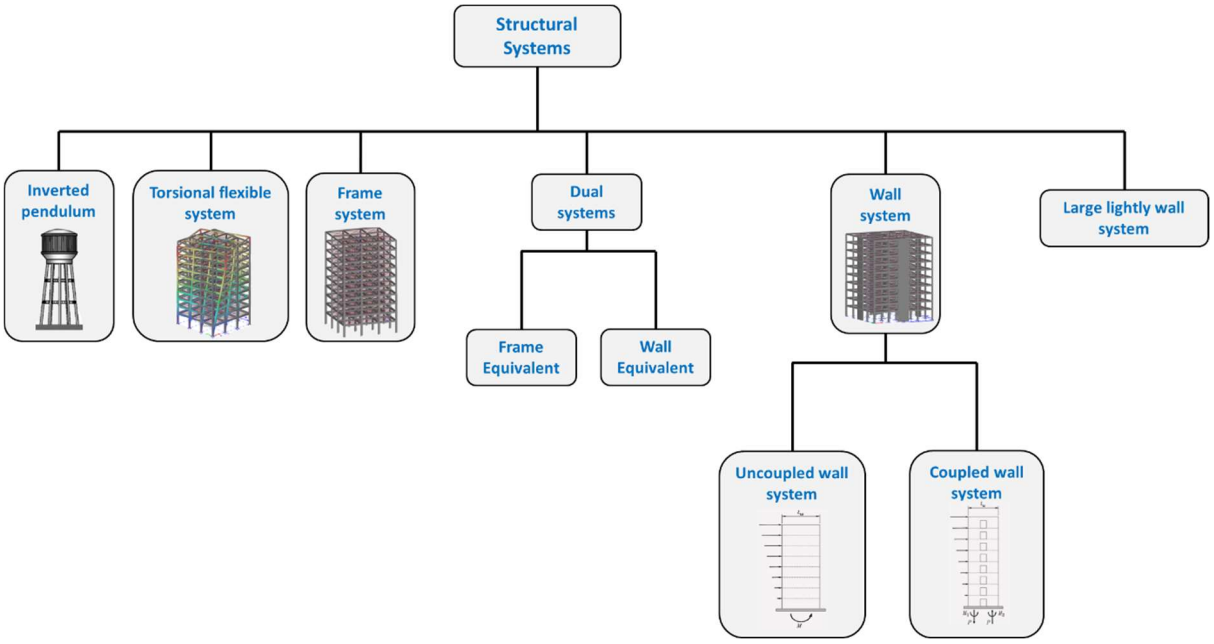


Figure 2-25: Structural systems preconized in EN 1998-1:2004[37]

2.1.5.2 STRUCTURAL TYPES – DUCTILITY CLASSES

EN 1998-1:2004[37] allows the design of concrete buildings for more strength and less ductility, or vice versa. This option is exercised through the ductility classification of concrete buildings: EN 1998-1:2004[37] permits trading ductility and dissipation capacity for strength by providing three alternative ductility classes:

- Ductility Class Low (DCL).
- Ductility Class Medium (DCM).
- Ductility High (DCH).

EN 1998-1:2004[37] commends using DCL (low ductility) only for "low seismicity cases," for which it is expected to be cost-effective and easier to apply. EN 1998-1:2004[37]

recommends that a “low seismicity case” be one where the design ground acceleration on rock does not exceed 0,08 g or that at the ground surface, the site ($a_g \cdot S$) is not more than 0,1 g.

Buildings of DCL are not designed for any ductility but only for strength. Except for certain minimum conditions for the ductility of reinforcing steel, such buildings have to follow only the dimensioning and detailing rules specified in EN 1992-1-1:2004[63]. They are designed to protect against earthquakes exactly as they are against other lateral actions. Although they are expected to respond elastically to the combination of its design seismic action with the concurrent gravity loads, they are entitled to a behaviour factor value of (q=1,5) attributed only to member overstrength over the seismic internal forces they are dimensioned for.

Buildings of DCM or DCH have q-factors higher than the value of 1,5 considered to be available owing to overstrength alone. DCH buildings are allowed to have higher values of q than DCM ones. To ensure ductile global behaviour, they must also meet more stringent detailing requirements for members and provide higher safety margins in capacity design calculations. The two upper ductility classes represent two possible combinations of strength and ductility, approximately equivalent in total material cost and achieved performance under the design seismic action Fardis[67].

The class of ductility DCM is slightly easier to design for and achieve at the construction site and may provide better performance in moderate earthquakes. Concrete buildings classified as ductility class DCM can provide energy dissipation capacity and overall ductile behaviour. General ductile behaviour is ensured if the ductility demand involves a large volume of the structure spread to all its stories’ different elements and locations. To this end, ductile failure modes should precede brittle failure modes with sufficient reliability. DCM corresponds to buildings designed, dimensioned, and detailed according to specific earthquake-resistant provisions. This enables the structure to develop stable mechanisms associated with significant hysteretic energy dissipation under repeated reversed loading without suffering brittle failure EN 1998-1:2004[37].

EN 1998-1:2004[37] does not link selection between the two higher ductility classes to the site's seismicity or the structure's importance, nor does it limit their application. It is up to countries to choose for the various parts of their territory and types of construction.

Buildings designed with class of ductility DCH are believed to provide higher safety margins against local or global collapse under earthquakes (much) stronger than the design seismic action.

2.1.5.3 STRUCTURAL REGULARITY

In the present research, the geometrical type in plan to be dealt with is the “rectangular.” As previously presented, the current study focuses on numerically simulating buildings with regularity (in plan and elevation) and non-torsionally flexibility. This chapter aims to describe the regularity concepts in the plan, elevation, and torsional flexibility in general, as well as the structures under analysis.

Section 4 of EN 1998-1:2004[37] presents general rules for earthquake-resistant structures, paying attention to structural regularity and appropriate foundation concepts. In particular, designers should provide direct paths for transmitting seismic forces through the structure to the ground, as stated by Meskouris et al.[68]. Regarding clause 4.2.3.2 of EN 1998-1:2004[37], the criteria for building regularity in plan is approximating lateral stiffness and

mass symmetry to two orthogonal horizontal directions. A building is considered regular in plan when its layout is compact and the in-plan stiffness of its floors is sufficiently large compared to the lateral stiffness of its vertical structural elements, according to Meskouris et al.[68].

EN 1998-1:2004[37] establishes that the structure of a building to be considered as regular in-plan if five conditions are be fulfilled at all storey levels:

a) With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes. Usually, the horizontal components of the seismic action are applied along these two axes.

b) The plan configuration shall be compact, i.e., a polygonal convex line shall delimit each floor. Suppose in-plan set-backs (re-entrant corners or edge recesses) exist. In that case, regularity in plan may still be considered as being satisfied, provided that these setbacks do not affect the floor n-plan stiffness and that, for each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5 % of the floor area.

Therefore, it may still consider the structure as regular in plan under the following conditions:

- These setbacks do not affect the floor in-plane stiffness.
- For each setback, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5% of the floor area.

A plan configuration illustrating the application of in-plan regularity concerning compactness is presented; they show that edge recesses are more severely penalised by this condition than re-entrant corners. This is so because edge recesses disturb the horizontal force paths of the diaphragmatic effect more than re-entrant corners Fardis et al.[69].

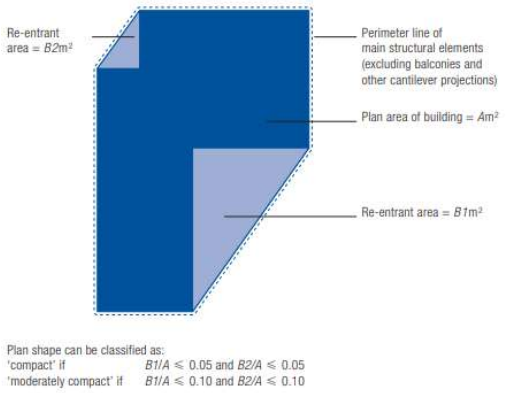


Figure 2-26: Definition of compact shapes

It should also be emphasised that this condition of EN 1998-1:2004[37] includes two conditions to be checked, meaning that, even in cases where the 5% ratio of areas is fulfilled, the condition that the setbacks do not affect the in-plane stiffness may be controlling.

The in-plane stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements. In this respect, the L, C, H, I, and X plan shapes should be carefully examined, notably concerning the stiffness of the lateral branches, which should be comparable to that of the central part, to satisfy

the rigid diaphragm condition. The application of this paragraph should be considered for the global behaviour of the building.

According to EN 1998-1:2004[37] section 4.3, a rigid diaphragm is defined as one in which, when it is modelled with its actual in-plane flexibility, its horizontal displacements due to the seismic action do not exceed anywhere by more than 10%, the corresponding absolute horizontal displacements that would result from a rigid diaphragm assumption.

This definition of a rigid diaphragm in the context of checking the in-plan regularity of building structures indicates that a more straightforward modelling approach is acceptable, provided that it does not induce differences in the distribution of the seismic load into the various lateral resisting systems larger than 10%. In this case, the rigid diaphragm assumption is justified. Still, it may be noted that, for instance, a solid reinforced concrete slab is considered a rigid diaphragm if its thickness and reinforcement (in both horizontal directions) are above the minimum thickness of 150 mm. It should also be noted that for the present work for a diaphragm to be considered rigid, the models are free of openings.

According to EN 1998-1:2004[37], the slenderness $\lambda = L_{max}/L_{min}$ of the building in plan shall be not higher than 4, where L_{max} and L_{min} are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions according to EN 1998-1:2004[37] clause 4.2.3.2(5). This limit is complementary to condition 3), which requires the in-plane rigidity of the diaphragm and is intended to ensure that, independently of the result of condition 3). In the case of very slender floor plans, its deformability is explicitly considered in the structural model and conditions the distribution of the seismic forces among the vertical structural elements, as stated by Fardis et al.[69].

At each level and for each direction of analysis X and Y, the structural eccentricity (e_o) and the torsional radius (r) shall follow the two conditions below, which are expressed for the direction of analysis:

$$e_{0x} \leq 0{,}3 \cdot r_x \text{ and } e_{0y} \leq 0{,}3 \cdot r_y \quad (2\text{-}40)$$

Torsion will not develop about a vertical axis if every structural and mass distribution plan storey is symmetric for the orthogonal horizontal axes. The absence of symmetry in plan is calculated in terms of the static eccentricity (e_i) between the centre of mass of a storey (CM) and the centre of rigidity (CR).

In symmetrical buildings, in each of the two orthogonal horizontal directions, X and Y, the static eccentricity (e_{0i}) between the floor centre of mass (CM) and the storey centre of stiffness (CR), as illustrated in, is not greater than 30% of the corresponding storey torsional radius r, as described in expression (2-40). The centre of lateral rigidity (CR) is defined as the point in plan through which application to the elastic structure of lateral forces produces only translation of the individual floors. The centre of rigidity (X_{CR}, Y_{CR}) is determined at each floor as the resultant of the forces applied on the floor in the direction of the applied lateral loads to the vertical members above and below. Any set of storey torques produces only rotation of the floors about the vertical axis that passes through the centre rigidity without the translation of that point in the horizontal plane at any floor. Each floor's centre of twist (CT) is geometrically determined due to these storey torsion moments (M_θ).

The horizontal projection of the centre of twist at a vertical elevation from the base about equal to 80% of the total building height, from this analysis, may be taken as the centre of

rigidity of the building because a set of horizontal forces, proportional to the mass of each storey multiplied its inter-storey height and applied at floor levels through that point produces a translation of the individual storeys with a minimum twist about the vertical axis. The elastic analysis in 3D, mainly used in modern design for buildings that are asymmetric and irregular in plan, captures the inelastic torsional seismic response sufficiently and provides reliable results for seismic design against its effects Kosmopoulos et al.[70] and Fardis[67].

EN 1998-1:2004[37] allows determining the centre of lateral stiffness and the torsional radius by considering the translational stiffness represented by the second-order moment (I_x) and (I_y)of the cross-sections of the vertical elements, neglecting the effect of beams. The position of the centre of lateral rigidity (X_{CR}, Y_{CR}) may be calculated according to equation (2-41), with (x_i) and (y_i) defining the position of each vertical element per storey, measured from the origin of any arbitrary plan reference.

$$X_{CR} = \frac{\sum_{i=1}^{n} x_i \cdot E \cdot I_{y,i}}{\sum_{i=1}^{n} E \cdot I_{y,i}} \text{ and } Y_{CR} = \frac{\sum_{i=1}^{n} y_i \cdot E \cdot I_{x,i}}{\sum_{i=1}^{n} E \cdot I_{x,i}} \tag{2-41}$$

and the centre of mass (CM) is defined as

$$X_{CM} = \frac{\sum_{i=1}^{n} x_i \cdot A_i}{\sum_{i=1}^{n} A_i} \text{ and } Y_{CM} = \frac{\sum_{i=1}^{n} y_i \cdot A_i}{\sum_{i=1}^{n} A_i} \tag{2-42}$$

Therefore, the static eccentricities along (X) and (Y) per storey are calculated as follows:

$$e_{0x} = |X_{CR} - X_{CM}| \text{and } e_{0y} = |Y_{CR} - Y_{CM}| \tag{2-43}$$

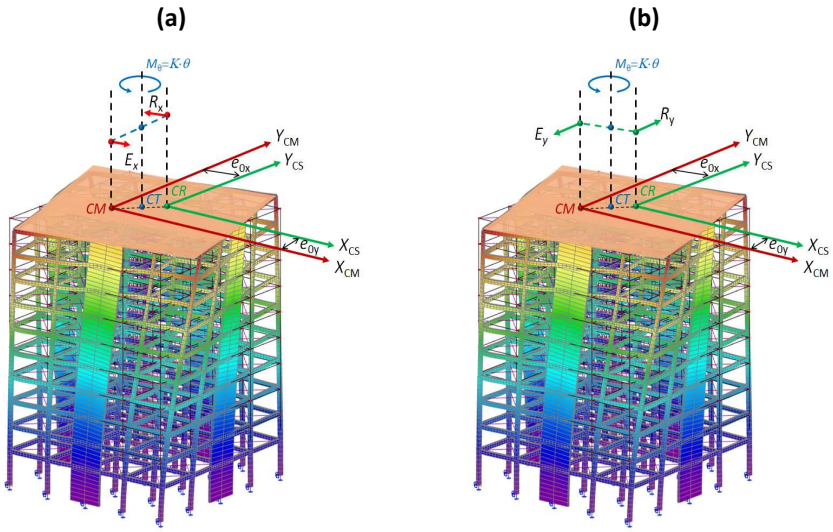


Figure 2-27: Definition of mass-stiffness eccentricity (a) X direction (b) Y direction

The torsional radius (r) in equations (2-44) is defined as the square root of the ratio of the torsional stiffness ($K_{\theta,i}$) of the i-th storey concerning the centre of lateral stiffness to the storey lateral stiffness ($K_{x,i}$) in direction X, and ($K_{y,i}$) in direction Y, similarly for the torsional radius.

$$r_{x,i} = \sqrt{\frac{K_{\theta,i}}{K_{y,i}}} \text{ and } r_{y,i} = \sqrt{\frac{K_{\theta,i}}{K_{x,i}}} \tag{2-44}$$

The torsional radius in the X direction is given by equation (2-45):

$$r_{x,i} = \sqrt{\frac{\sum_{j=1}^{n}\left(x_{j,i}-X_{CR}\right)^2 \cdot E \cdot I_{x,j,i} + \sum_{j=1}^{n}\left(y_{j,i}-Y_{CR}\right)^2 \cdot E \cdot I_{y,j,i}}{\sum_{j=1}^{n} E \cdot I_{y,j,i}}} \quad (2\text{-}45)$$

Moreover, the *Y* direction is given by equation as depicted in equation (2-46):

$$r_{y,i} = \sqrt{\frac{\sum_{j=1}^{n}\left(x_{j,i}-X_{CR}\right)^2 \cdot E \cdot I_{x,j,i} + \sum_{j=1}^{n}\left(y_{j,i}-Y_{CR}\right)^2 \cdot E \cdot I_{y,j,i}}{\sum_{j=1}^{n} E \cdot I_{x,j,i}}} \quad (2\text{-}46)$$

In equations (2-45) and (2-46), ($E \cdot I_x$) and ($E \cdot I_y$) denote the section flexural rigidities for bending within a vertical plane parallel to horizontal directions *X* and *Y*, respectively, and with ($x_{j,i}$) and ($y_{j,i}$) defining the position of each vertical element (j) of the *i*-th storey, measured from the origin of any arbitrary plan reference.

Concerning the regularity in elevation according to EN 1998-1:2004[37], when setbacks are present, the following additional conditions apply:

For gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20 % of the previous plan dimension in the direction of the setback, shown in Figure 2-28.

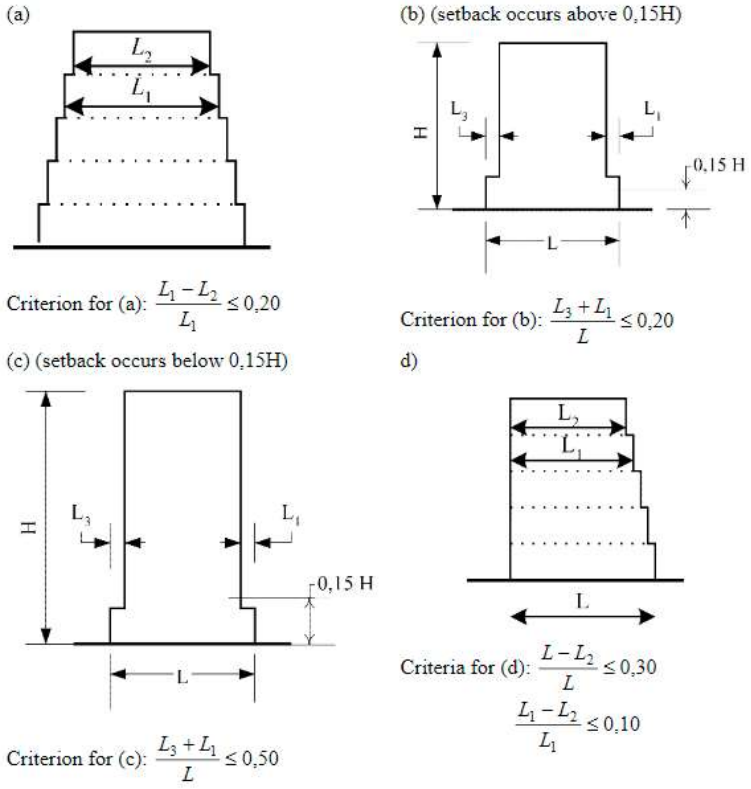


Figure 2-28: Criteria for regularity of buildings with setbacks EN 1998-1 (2004)

For a single setback within the lower 15 % of the total height of the main structural system, the setback shall be not greater than 50 % of the previous plan dimension. In this case, the structure of the base zone within the vertically projected perimeter of the upper storeys

should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement.

Suppose the setbacks do not preserve symmetry in each face. In that case, the sum of the setbacks at all storeys shall be not greater than 30 % of the plan dimension at the ground floor above the foundation or the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previous plan dimension.

From Figure 2-28 b) and c), it can be said that if there is a single setback within the bottom 15% of the total height of the building, H, this setback should not exceed 50% of the parallel dimension at the base of the building. In this case, there should be no undue reliance on enlarging the structure at the bottom to transfer the seismic shears developed in that part of the building to the ground. EN 1998-1 requires that the vertical continuation of the upper part of the building to the ground is designed for a seismic shear at least equal to 75% of the shear force that would develop in that zone in a similar building without the base enlargement Fardis et al.[67].

Regarding the torsional flexibility observations of buildings subjected to strong earthquakes, it was shown that the excessive torsional response is one of the most critical factors that severely damage the structures, even if they collapse. The disproportionate torsional response is often due to structural asymmetries.

Torsional stiffness and resistance are characteristics of building structures that significantly influence their response to seismic actions. Responses in which translational motion is dominant are preferable to those in which torsional motion is significant because they tend to stress the different structural elements more uniformly. To counteract the torsional response of buildings, the fundamental modes of vibration of the structure should be translational. To this end, the torsional stiffness of the structure must be sufficiently large to ensure that the first torsional vibration mode has a frequency higher than the translational modes, as stated by Fardis et al.[67].

Suppose the building has a natural mode purely torsional about the vertical axis and has a natural period longer than the lowest translational mode. In that case, accidental reasons may trigger twisting about the vertical axis, with vibration energy transfer from the lowest translational mode's response to the torsional one. The torsional response may induce significant horizontal displacements along the perimeter of the building. EN 1998-1 (2004) preconizes that the torsional radius of the storey in each of the two orthogonal horizontal directions, (X) and (Y) of symmetry being not less than the radius of gyration of the floor mass, as a criterion for regularity in plan.

EN 1998-1:2004[37] 4.2.3.2 (6) establishes a relation between the torsional radius of the storey in each of the two orthogonal horizontal directions, x and y, and the radius of gyration l_s of the floor mass defined. It defines a building as non-torsional flexible if the condition of the torsional radius of the storey in each of the two orthogonal horizontal directions, x and y, is higher than the radius of gyration, as defined in Equation (2-47):

$$r_x \geq l_s \text{ and } r_y \geq l_s \tag{2-47}$$

Both (r_x) and (r_y) must exceed the radius of gyration (l_s); otherwise, the building will be classified as torsionally flexible. The radius of gyration of the floor mass in the plan is defined as the square root of the ratio of the polar moment of inertia of the floor mass in the plan

concerning the centre of mass. If the mass is uniformly distributed over a rectangular floor area with dimensions L and B, the radius of gyration l_s is defined by equation (2-48):

$$l_s = \sqrt{\frac{L^2 + B^2}{12}} \tag{2-48}$$

2.1.5.4 SEISMIC ACTIONS

Resistance to extreme seismic actions is pursued by control of the inelastic response mechanism through the systematic and across-the-board application of capacity design. Within the national authority's philosophy on safety and economic issues, hazard levels corresponding to the two performance levels above are left for national determination.

According to EN 1998-1:2004[37], seismic action is defined in terms of its elastic response spectrum, which includes period ranges of constant spectral pseudo-acceleration, pseudo-velocity and displacement, as illustrated in Figure 2-29. The elastic response spectrum of the horizontal components of the seismic action is anchored to a single ground motion parameter, namely the design ground acceleration on rock (type A ground), a_g, given in national zoning maps.

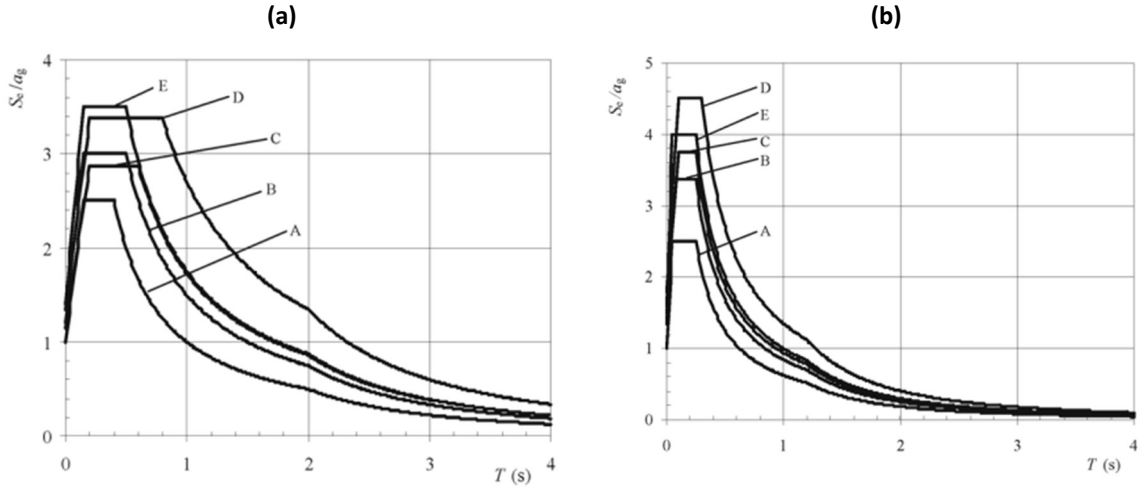


Figure 2-29: Elastic response spectra (a) Type 1 for ground types A to E (5% damping) (b) Type for ground types A to E (5% damping) (Adapted from EN 1998-1:2004[37])

A design response spectrum for elastic analysis may be used to design a building structure derived from the elastic response spectra by applying the behaviour factor (q). The behaviour factor (q) considers the global structure ductility and transforms the elastic response spectrum to an inelastic design spectrum. This procedure is a radical simplification as the tacit assumption is made that global ductility results from the combination of the existing local dissipative reserves due to ductility, as stated by Meskouris et al.[68].

Behaviour factors are a pragmatic approach in building practice, adequately estimating buildings' non-linear dynamic behaviour under seismic actions.

The design spectra for elastic analysis are calculated using the mathematical EN 1998-1:2004[37]. To define design spectra for elastic analysis, it is necessary to choose the behaviour

factor (q). The following are required to consider: ductility classes, structural regularity, structural types, and system, as shown in Figure 2-30.

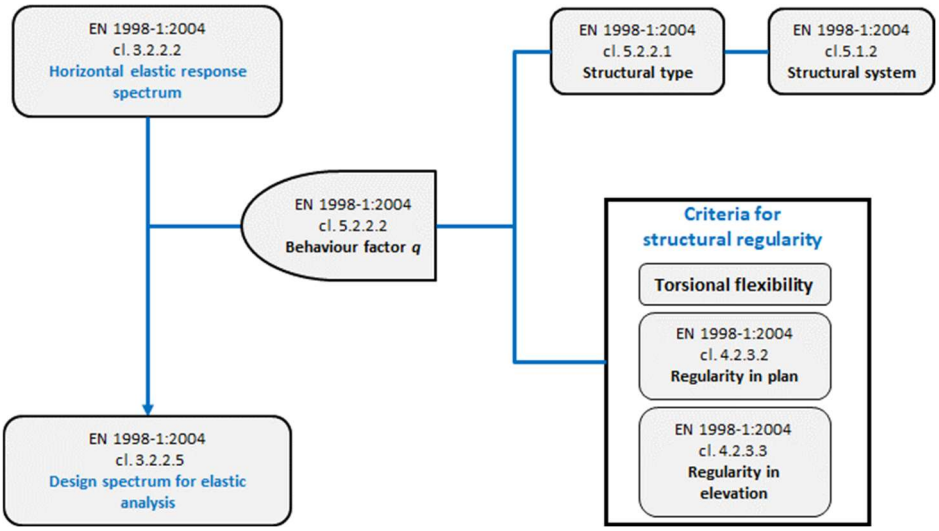


Figure 2-30: Flowchart to define the seismic action

The response spectra for elastic design are the elastic response spectra scaled down by behaviour factors (q) in EN 1998-1:2004[37], for both horizontal and vertical $S_d(T)$ is the design spectrum is the behaviour factor and (β) is the lower bound factor for the horizontal design spectrum. value proposed in EN 1998-1:2004[37] is 0,2.

The value of the accelerations (a_g) of the soil factor (S) and the corner periods (T_C) and (T_D) are those specified in EN 1998-1:2004[37] for the horizontal and vertical components of ground motions, respectively. Upper bounds for the q-factors, which also account for the influence of viscous damping (ξ) other than 5% and are to be used for horizontal and vertical components of ground motions, are given for the various materials and structural systems and according to the relevant class of ductility in the relevant parts of in EN 1998-1:2004[37].

2.1.5.5 BEHAVIOUR FACTOR

According to EN 1998-1:2004[37], to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced concerning the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behaviour factor (q).

The upper limit value of the behaviour factor (q), to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_0 \cdot k_w \geq 1{,}5 \tag{2-49}$$

where (q_0) is the basic value of the behaviour factor, dependent on the type of the structural system and its regularity in elevation and (k_w) is the factor reflecting the prevailing failure mode in structural systems with walls.

For buildings that are regular in elevation, the basic values of (q_0) for the various structural types are given in Table 2-15.

Table 2-15: Basic value of the behaviour factor (q_0) for systems regular in elevation (EN 1998-1:2004[37])

Structural type	DCM	DCH
Frame system, dual system, coupled wall system	$3{,}0 \cdot \alpha_u/\alpha_1$	$4{,}5 \cdot \alpha_u/\alpha_1$
Uncoupled wall system	3,0	$4{,}0 \cdot \alpha_u/\alpha_1$
Torsionally flexible system	2,0	3,0
Inverted pendulum system	1,5	2,0

Where (α_1) is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant; (α_u) is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor (α_u) may be obtained from a nonlinear static (pushover) global analysis.

For buildings which are not regular in elevation, the value of (q_0) should be reduced by 20%. For DCL structures, a behaviour factor (q) of up to 1,5 may be used to derive the seismic actions, regardless of the structural system and the regularity in elevation.

2.1.5.6 STRUCTURAL ANALYSIS

EN 1998-1:2004[37] provides the following analysis options for the design of buildings and the evaluation of their seismic performance:

- Lateral force method, linear static analysis.
- Modal response spectrum analysis.
- Linear Time-History analysis.
- Non-linear static analysis (pushover analysis) and or adaptive analysis.
- Non-linear dynamic analysis (time-history or response-history analysis).

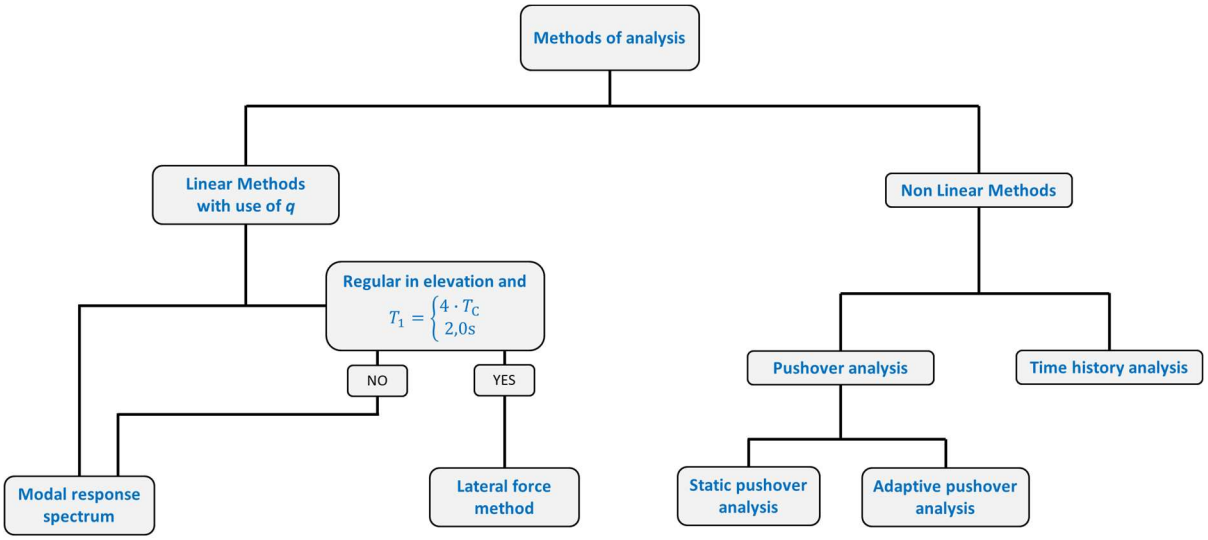


Figure 2-31: Methods of analysis

The linear analysis methods use the design response spectrum, essentially the elastic response spectrum with 5% damping divided by the behaviour factor (q). Internal forces due to the seismic action are taken to be equal to those estimated from the linear analysis; however, consistent with the equal displacement rule and the concept and use of the behaviour factor (q),

displacements due to the seismic action are taken as equal to those derived from the linear analysis, multiplied by the behaviour factor (q).

In contrast, when a non-linear analysis method is used, internal forces and displacements due to the seismic action are equal to those derived from the non-linear analysis. The use of a linear method of analysis does not imply that the seismic response will be linear elastic; it is simply a device for the simplification of practical design within the framework of force-based seismic design with the elastic spectrum divided by the behaviour factor (q), according to Fardis et al.[69].

2.1.5.7 DRIFT AND STABILITY

EN 1992-1-1:2004[63] requires considering a minimum moment based on the likelihood that the axial load cannot be fully concentric see Clause 6.1(4).

The (P-Δ) effects are associated with member end displacements. The (P-Δ) effects are important for overall structure behaviour under significant axial load. As indicated intuitively by Figure 2-32, gravity loading will influence structural response under significant lateral displacement.

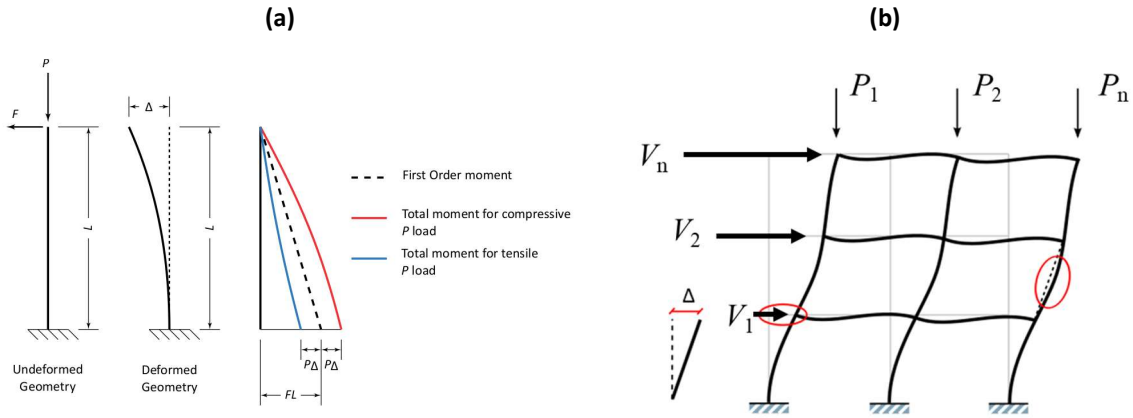


Figure 2-32: (P-Δ) effect (a) Column (b) Moment- resisting frame

EN 1998-1:2004[37] section 4 requires considering second-order (P-Δ) effects in buildings when these exceed 10% of the first-order effects in the aggregate for the vertical members of the storey. The criterion is the value of the interstorey drift sensitivity coefficient, (θ), defined for storey i as the ratio of the total second-order moment in the storey i to the change in the first-order overturning moment in that storey, as depicted in Equation (2-50):

$$\theta_i = \frac{P_{tot,i} \cdot d_{r,i}}{V_{tot,i} \cdot h_i} \tag{2-50}$$

where θ is the interstorey drift sensitivity coefficient; ($P_{tot,i}$) is the total gravity load at and above the storey considered in the seismic design situation; (d_r) is the design interstorey drift, and (V_{tot})is the total seismic storey shear; and h is the interstorey height.

According to EN 1998-1:2004[37], Section 4, the second-order effects may be neglected, provided that the value of (θ_i) does not exceed 0,1. Regarding EN 1998-1:2004[37] in any storey, however, they should be considered for the entire structure if, at any storey, the value of (θ_i) exceeds 0,1. Suppose the value of (θ_i) does not exceed 0,2 at any storey. Section 4 of EN 1998-1:2004[37] allows (P-Δ) effects to be taken into account approximately without a second-order

analysis by multiplying all first-order action effects due to the horizontal component of the seismic action by (1/(1-θ_i)). Although the value (θ_i) of the individual storeys can be used in this amplification, the use for the entire structure of the maximum value of (θ_i) in any storey is safe-sided. It maintains force equilibrium in the framework of first-order analysis. In the unlikely case that a value of (θ_i) exceeds 0,2 in any of the storeys, an exact second-order analysis is required.

EN 1998-1:2004[37] sets forth constraints on interstorey drift to limit damage during seismic events. The verification of interstorey drift is conducted to ensure compliance with the Damage Limitation (DL) limit state as specified by EN 1998-1:2004[37].

The interstorey drift ratio demand should be determined under the frequent (serviceability) seismic action, which is defined by multiplying the entire elastic response spectrum of the design seismic action for 5% damping by the same factor ν that reflects the effect of the mean return periods of these two seismic actions. If the analysis for the design seismic action is linear-elastic based on the design response spectrum, the elastic spectrum with 5% damping divided by the behaviour factor(q), then the values of the displacements to be used for ($d_{s,i+1}$) and ($d_{s,i}$) are those from that analysis multiplied by the behaviour factor (q_d) as depicted in equation (2-51):

$$d_s = q_d \cdot d_e \tag{2-51}$$

The design interstorey drift (d_r) is calculated as the difference of the average lateral induced by the reduced seismic action displacements d_s at the top and bottom of the storey under consideration.

$$d_{r,i,} = d_{s,i+1} - d_{s,i} \tag{2-52}$$

The displacements induced by the reduced seismic action d_s should be calculated by using expression (2-53):

$$d_s = q_{disp} \cdot d_e \tag{2-53}$$

Table 2-16, a summary of the interstorey drift limitation for different situations according

Table 2-16: Detailing provisions for columns (EN 1998-1:2004[37])

Type	LS	$d_r \cdot h/\nu$
Buildings having non-structural elements of brittle materials attached to the structure	DL	1,0%
Buildings having ductile non-structural elements:		1,50%
Buildings having non-structural elements fixed in a way so as not to interfere with structural deformations or without non-structural elements:		2,0%

2.1.5.8 CAPACITY DESIGN APPROACH

A prime target of seismic design is to apportion the given total seismic displacement demand to the various elements of the building, entrusting inelastic deformations only to those that can reliably withstand them while keeping those that cannot in the elastic range. The tool for controlling the inelastic seismic response is 'capacity design'.

Especially in frames, capacity design introduces strong interdependence between various phases of a building's detailed seismic design for ductility, within or between members.

Dimensioning a column in flexure depends on the amount and layout of the longitudinal reinforcement of the beams it is connected to in any horizontal direction. The design of a column or a beam in a shear depends on the amount and detail of its longitudinal reinforcement and the members framing it at either end Fardis[67].

The objective of the EN 1998-1:2004[37] rules for the design of concrete moment-resisting frames is to force plastic hinges out of the columns and into the beams so that a beam-sway mechanism develops and a soft storey is prevented.

A soft-storey mechanism develops in a frame system when the top and bottom ends of (all) the columns in a storey yield opposite bending and start undergoing unrestrained flexural rotations without a notable increase of their bending moments beyond the corresponding moment resistance (M_{Rc}). The way to prevent soft storeys in frames is by forcing flexural plastic hinges out of the columns and into the beams to develop a beam-sway mechanism. To this end, within any vertical plane in which a soft storey is to be prevented, the two columns framing into a beam-column joint from above and below are dimensioned to be jointly stronger by 30% than the beams connected to the same joint from any side.

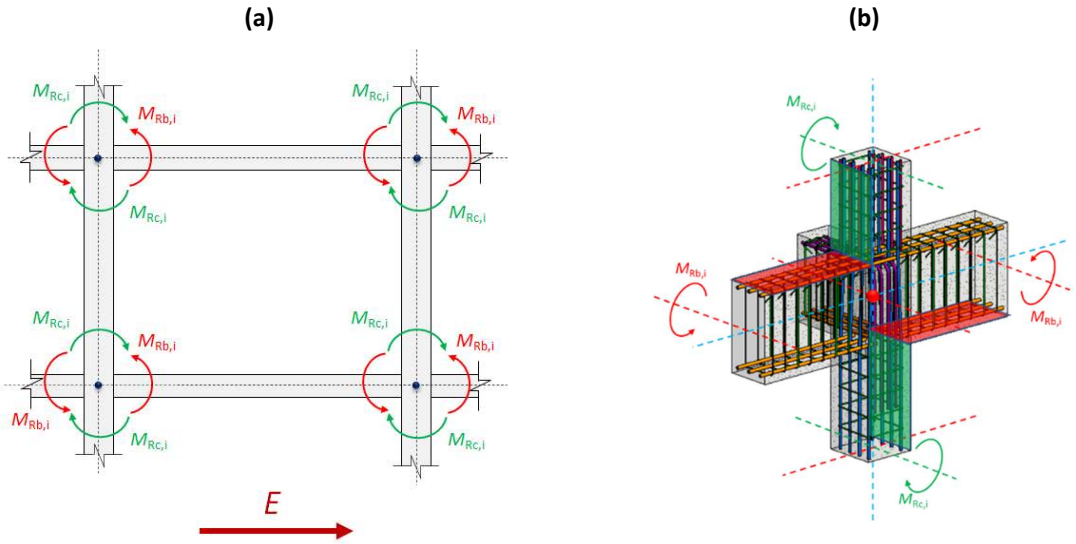


Figure 2-33: Capacity design against column hinging (a) Frame nodal flexure vectorial system (b) Beam-column nodal vectorial system

To this end, at their joints with beams, primary seismic columns are capacity designed to be stronger than the beams, with an overstrength factor of 1,3 on beam design flexural capacities (2-54):

$$\sum_{i=1}^{j} M_{Rc,i} \geq 1{,}3 \cdot \sum_{i=1}^{j} M_{Rb,i} \tag{2-54}$$

where ($\Sigma M_{\mathrm{Rc,i}}$) is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in the expression (2-54). The parameter ($\Sigma M_{\mathrm{Rb,i}}$) is the sum of the design values of the moments of resistance of the beams framing the joint.

Figure 2-33(a) shows the vectorial system of ($M_{\mathrm{Rc,i}}$) and ($M_{\mathrm{Rb,i}}$)in a moment-resisting frame, and Figure 2-33(b) shows the vectorial system in a node. This rule applies to DCM and DCH structures according to EN 1998-1:2004[37].

Regarding the capacity design of shear for the beam according to EN 1998-1:2004[37] in primary seismic beams, the design shear forces shall be determined following the capacity design rule based on the equilibrium of the beam under (i) the transverse load acting on it in the seismic design situation and (ii) end moments ($M_{\mathrm{i,d}}$) with i=1,2 denoting the end sections of the beam, corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams or (if they form there first) in the vertical elements connected to the joints into which the beam ends frame, as illustrated in Figure 2-34.

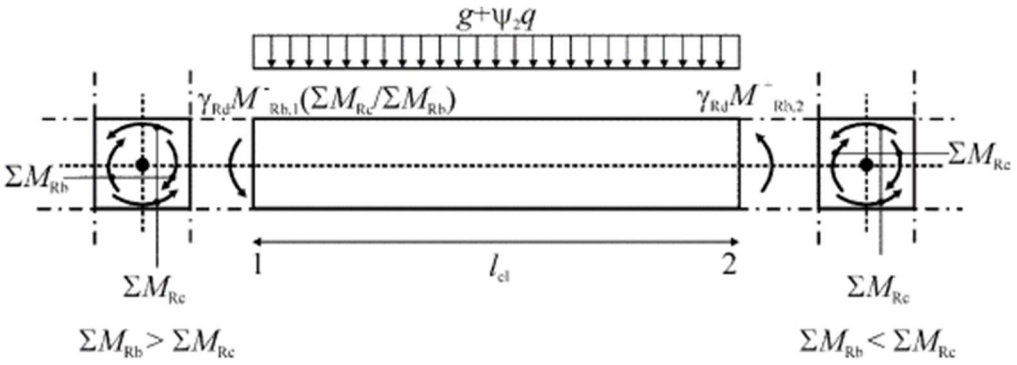


Figure 2-34: Capacity design values of shear forces on beams (Adapted from EN 1998-1:2004[37])

According to EN 1998-1:2004[37], at the end of section *i*, two values of the acting shear force should be calculated, i.e. the maximum ($V_{\mathrm{Ed,max,i}}$) and the minimum ($V_{\mathrm{Ed,min,i}}$) corresponding to the maximum positive and the maximum negative end moments $M_{\mathrm{i,d}}$ that can develop at ends 1 and 2 of the beam. The end moments ($M_{\mathrm{i,d}}$) may be determined according to Equation (2-55):

$$M_{i,d} = \gamma_{Rd} \cdot M_{Rb,i} \cdot \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right) \tag{2-55}$$

Where (γ_{Rd}) is the factor accounting for possible overstrength due to steel strain hardening, which in the case of DCM beams may be taken as being equal to 1,0; ($M_{\mathrm{Rb,i}}$) is the design

value of the beam moment of resistance at the end i in the sense of the seismic bending moment under the considered sense of the seismic action; (ΣM_{Rc}) and (ΣM_{Rb}) are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively. The value of (ΣM_{Rc}) should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

Concerning the columns, the design values of shear forces shall be determined following the capacity design rule, based on the equilibrium of the column under end moments ($M_{i,d}$) with (i=1,2) denoting the end sections of the column), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams connected to the joints into which the column end frames, or (if they form there first) at the ends of the columns, as depicted in Figure 2-35.

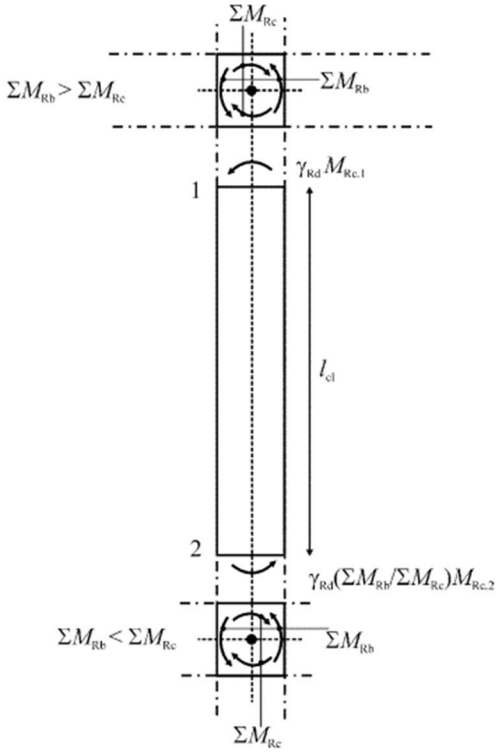


Figure 2-35: Capacity design values of shear forces on columns (Adapted from EN 1998-1:2004[37])

The end moments ($M_{i,d}$) may be determined from the following expression (2-56):

$$M_{i,d} = \gamma_{Rd} \cdot M_{Rb,i} \cdot \min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right) \quad (2\text{-}56)$$

Where (γ_{Rd}) is the factor accounting for overstrength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1,1 (for DCM); ($M_{Rc,i}$) is the design value of the column moment of resistance at the end i in the sense of the seismic bending moment under the considered sense of the seismic action. The values of ($M_{Rc,i}$) and (ΣM_{Rc})should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

2.1.5.9 DETAILING FOR LOCAL DUCTILITY FOR BEAMS AND COLUMNS

The detailing rules according to EN 1998-1:2004[37] for beams and columns are summarised in Table 2-17 and Table 2-18, respectively.

Table 2-17: Detailing provisions for local ductility for beams (EN 1998-1:2004[37])

Beams		DCL	DCM	DCH
Critical region length		h_w		1,5·h_w
Longitudinal bars				
Minimum longitudinal reinforcement ratio	ρ_{min}	max(0,26f_{ctm}/f_{yk};0,13%)	0,5f_{ctm}/f_{yk}	
Maximum longitudinal reinforcement ratio	ρ_{max}	0,04	$\rho' + \frac{0{,}0018}{\mu_\phi \cdot \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$	
Maximum bar diameter crossing interior joints	d_{bL}	-	$\frac{7{,}5 \cdot f_{ctm}}{f_{yd}} \cdot \frac{(1+0{,}8 \cdot \nu_d)}{1+0{,}50 \cdot \frac{\rho'}{\rho_{max}}}$	$\frac{6{,}25 \cdot f_{ctm}}{f_{yd}} \cdot \frac{(1+0{,}8 \cdot \nu_d)}{1+0{,}75 \cdot \frac{\rho'}{\rho_{max}}}$
Maximum bar diameter crossing exterior joints	d_{bL}	-	$\leq 7{,}5 \cdot (1+0{,}8 \cdot \nu_d) \cdot \frac{f_{ctm}}{f_{yd}}$	$\leq 6{,}25 \cdot (1+0{,}8 \cdot \nu_d) \cdot \frac{f_{ctm}}{f_{yd}}$
Transverse reinforcement				
Outside critical regions				
Maximum hoop spacing	s_w	0,75·d		
Minimum traversal reinforcement ratio	ρ_w	$0{,}08 \cdot \sqrt{\frac{f_{ck}}{f_{yk}}}$		
In critical regions				
Minimum hoop diameter	d_{bw}	6 mm		
Maximum hoop spacing	s_w	-	min(h_w/4,24·d_{bw}; 225mm; 8·d_{bL})	min(h_w/4,24·d_{bw}; 175; 6·d_{bL})

Table 2-18: Rules for design detailing and dimensioning of columns (EN 1998-1:2004[37])

Columns		DCL	DCM	DCH
Critical region length	l_{cr}		max(h_c; l_c/5; 0,45m)	max(1,5·h_c; l_{cl}/5; 0,60m)
Maximum admissible normalised axial force	ν_d	-	0,65	0,55
Detailing for local ductility – Longitudinal bars				
Minimum longitudinal reinforcement ratio	ρ_{min}	$\frac{0{,}1 \cdot N_{Ed}}{A_c \cdot f_{yd}}$; 0,20%	1%	
Maximum longitudinal reinforcement ratio	ρ_{max}	4%		
Minimum reinforcement diameter	d_{bL}≥	8 mm		
Minimum bars per side		3		2
Maximum distance between restrained bars		-	200mm	150 mm
Maximum distance of the unrestrained bar from the nearest restrained bar		150 mm		
Detailing for local ductility – Transverse bars				
Outside critical regions				
Minimum hoop bar diameter	d_{bw}	max (6mm; d_{bL}/4)		
Maximum hoop spacing	s	20d_{bL}; h_c, b_c; 400		
Within critical regions				
Minimum hoop bar diameter	d_{bw}	max (6, d_{bL}/4)		
Maximum hoop spacing	s	min (b_{oc}/2; 175; 8d_{bL})	min (b_{oc}/2; 200; 9·d_{bL})	min (b_{oc}/2; 175; 8d_{bL})
In critical region at the column base				
Minimum mechanical percentage of reinforcement	ω_{wd}	-	0,05	0,12
Confinement integrity verification	$\alpha \cdot \omega_{wd}$	-	30·μ_ϕ·ν_d·$\varepsilon_{sy,d}$·b_c/b_o-0,035	

2.1.5.10 DESIGN AND DETAILING OF WALLS

The walls of a wall system are arranged in two orthogonal horizontal directions with as much two-way symmetry as possible. Suppose the individual walls are all similar and symmetrically placed. In that case, they will be subjected to uniform seismic force and deformation demands at every storey, minimising the uncertainty about the seismic response.

Walls tend to deflect as vertical cantilevers. In that case, besides the increased uncertainty of the post-elastic response, the floor diaphragms will be stressed hard to iron out the differences in height-wise deflection patterns between the stiffer walls, which have gone inelastic, and the more flexible ones, which remain elastic.

The walls are conceived as a two-way-symmetric rectangular section with an aspect ratio of less than 4. Such walls are modelled and dimensioned as prismatic elements with an axis through the centroid of the section. Lacking a better alternative, the same practice is applied when a rectangular wall runs into or crosses another wall at right angles to create a wall with a composite cross-section of more than one rectangular part. The walls have high stiffness and strength in both horizontal directions and, hence, are subjected to biaxial bending and bi-directional shears during the earthquake.

Structural systems dominated by ductile walls have many advantages for earthquake resistance, such as the high lateral stiffness of walls reducing inter-storey drifts and structural or non-structural damage and preventing soft-storey mechanisms, according to Fardis et al.[69].

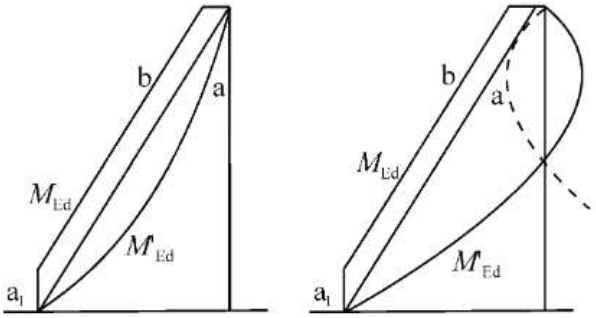
Design codes define a wall as a vertical element with an elongated cross-section: a lower limit of 4 for the aspect ratio (long-to-short dimension) of a rectangular cross-section is used in EN 1992-1-1:2004[63] for a vertical element to be considered as a wall.

If the cross-section consists of rectangular parts, one with an aspect ratio greater than 4, the element is also classified as a wall. With this definition based on the cross-sectional shape alone, a wall differs from a column in that it resists lateral forces mainly in one direction (parallel to the long side of the section) and can be designed for such a unidirectional resistance by assigning the flexural resistance to the two edges of the section (‘flanges’, or ‘tension and compression chords’) and the shear resistance to the ‘web’ between them, as in a beam. So, for moment resistance and deformation capacity, the designer may concentrate on the vertical reinforcement and provide concrete confinement only at the two edges of the section.

2.1.5.10.1 Bending moment actions of walls

According to EN 1998-1:2004[37], to ensure the localisation of the post-elastic behaviour at the base of the wall within the critical region, the diagram of the design bending moments on the height of the wall is given by an envelope of the diagram of the bending moments resulting from the analysis, vertically displaced due to ‘tension shift’.

EN 1998-1:2004[37] states that the design of the bending moment diagram along the height of the wall should be given by an envelope of the bending moment diagram from the analysis, vertically displaced (tension shift). The envelope may be assumed linear if the structure does not exhibit significant discontinuities of mass, stiffness or resistance over its height. The tension shift should be consistent with the strut inclination taken in the ULS verification for shear, with a possible fan-type pattern of struts near the base and with the floors acting as ties.



Key

a	moment diagram from analysis
b	design envelope
a_l	tension shift

Figure 2-36: Envelope of bending moment (Adapted from EN 1998-1:2004[37])

The tension shift should be consistent with the strut inclination taken in the ULS verification for shear, with a possible fan-type pattern of struts near the base and with the floors acting as ties.

The "tension shift" is defined by EN 1998-1:2004[37], taking into account the following steps:

Step 1: The envelope may be assumed linear, Figure 2-37(c), on the condition that the absolute value of the linear value is not lower than that of the initial diagram (M'_{Ed}).

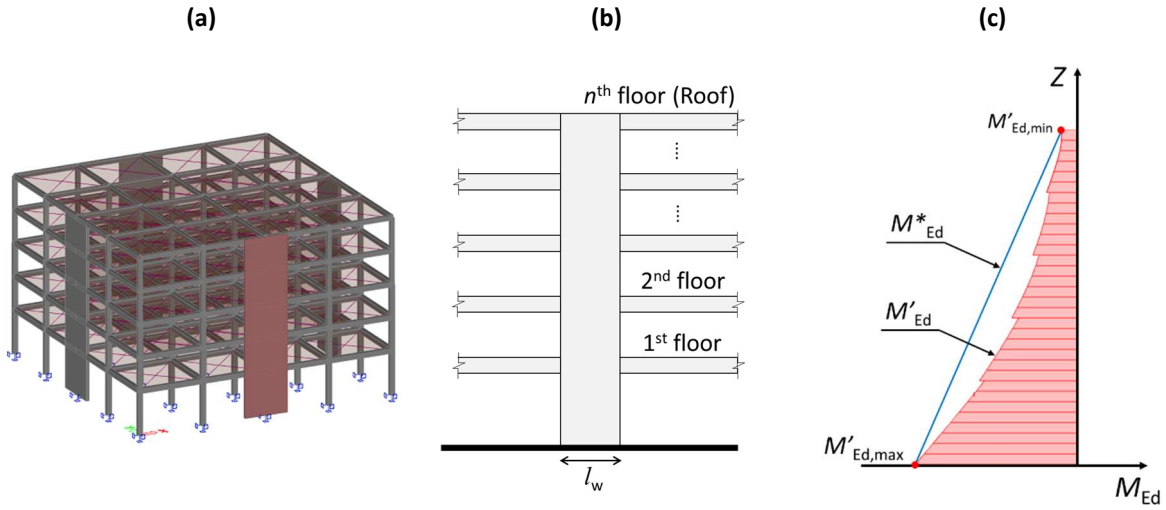


Figure 2-37: Envelope of bending moment of a wall (a) Structure (b) Wall (c) Bending moment diagram from analysis

The linear envelope (M^*_{Ed}) depicted in Figure 2-37(c) may be defined by expression (2-57):

$$M^*_{Ed}(z) = m \cdot z - M'_{Ed,max} \tag{2-57}$$

where (m) is the slope of the linear envelope and (z) is the height development.

Step 2: The tension shift in Figure 2-38(b) and (c) should be consistent with the strut inclination taken in the ULS verification for shear, with a possible fan-type pattern of struts near the base and the floors acting as ties.

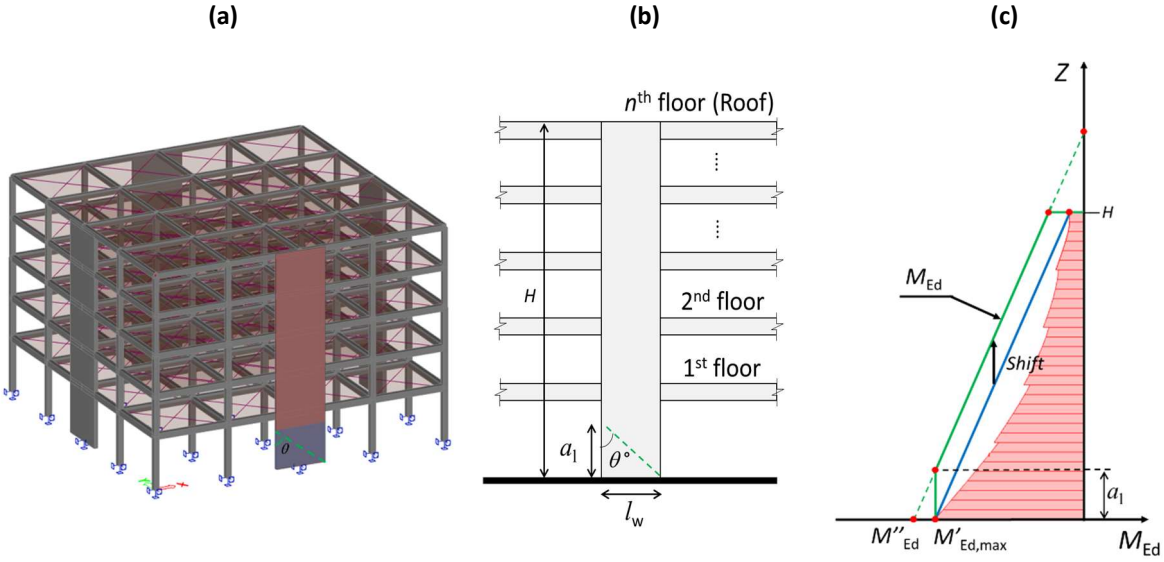


Figure 2-38: Envelope of bending moment of a wall (a) Structure (b) Definition of tension shift (a_l) (c) Envelope of bending moment diagram

The value of (a_l) depends on the shear strut angle (θ), as depicted in Figure 2-38 (b) and is defined as shown in expression (2-58):

$$a_l = \frac{l_w}{tan(\theta)} \tag{2-58}$$

Finally, the design envelope for bending moments may be defined according to Figure 2-38 (c) and may be written according to the equation system (2-59):

$$M_{Ed}(z) = \begin{cases} M'_{Ed,max} & if\ 0 < z \leq a_l \\ m \cdot z - M''_{Ed} & if\ a_l < z \leq H \end{cases} \tag{2-59}$$

Although it is advisable to take advantage of the presence of floors to place horizontal ties therein, which controls the value of (θ), it observed that according to EC2 Part 1-1 clause 6.2.3, the value θ may be assumed to be between 22° and 45°. The θ=45° is the lower acceptable and more conservative value. In that case, the tension shift is equal to (l_w).

2.1.5.10.2 Shear action

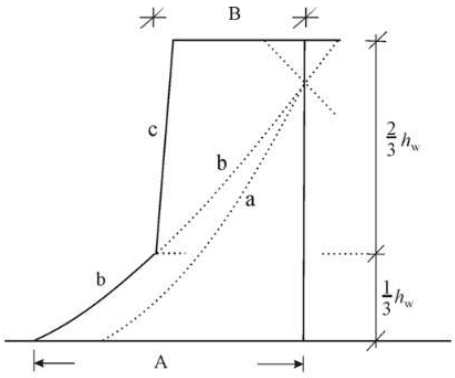
According to EN 1998-1:2004[37] 5.4.2.4 Special provisions for ductile walls, the possible increase in shear forces after yielding at the base of a primary seismic wall shall be considered. The requirement specified in EN 1998-1:2004[37] (6)P of 5.4.2.4 may be satisfied if the design shear forces are 50% higher than those obtained from the analysis. Consequently, a magnification factor (ε) of 1,5 is assumed for DCM.

$$V_{Ed}(z) = 1{,}5 \cdot V'_{Ed}(z) \tag{2-60}$$

The shear magnification factor (ε) for DCH preconized in EN 1998-1:2004[37] follows Equation (2-61):

$$\varepsilon(z) = \sqrt{\left(\frac{1{,}2 \cdot M_{Rd}}{M'_{Ed}}\right)^2 + 0{,}1 \cdot \left(\frac{q \cdot S_e(T_C)}{S_e(T_1)}\right)^2} \leq q \tag{2-61}$$

In dual systems containing slender walls, the design envelope of shear forces following Figure 2-39 should be used to account for uncertainties in higher mode effects.



Key

a	shear diagram from analysis
b	magnified shear diagram
c	design envelope
A	$V_{wall,base}$
B	$V_{wall,top} \geq V_{wall,base}/2$

Figure 2-39: Design envelope of the shear forces in the walls of a dual system (Adapted from EN 1998-1:2004[37])

2.1.5.10.3 Confinement requirements of walls

The wall boundary elements confinement should extend vertically over the height (h_{cr}) of the critical region as defined in 5.4.3.4.2(1) and horizontally along a length (l_c) measured from the extreme compression fibre of the wall up to the point where unconfined concrete may spall due to large compressive strains. If more precise data is unavailable, the compressive strain at which spalling is expected may be considered equal to (ε_{cu2}=0,0035). The confined boundary element may be limited to a distance (lc) as depicted in Equation (2-62):

$$l_c \geq x_u \cdot \left(1 - \frac{\varepsilon_{cu2}}{\varepsilon_{cu2,c}}\right) \tag{2-62}$$

from the hoop centreline near the extreme compression fibre, with the depth of the confined compression zone (x_u) at ultimate curvature estimated from equilibrium for a constant width (b_0) of the confined compression zone). Moreover, the ultimate strain ($\varepsilon_{cu2,c}$) of confined concrete estimated based on EN 1992-1-1:2004[63], as shown in equation (2-63) and depicted in Figure 2-40:

$$\varepsilon_{cu2,c} = 0{,}0035 + 0{,}1 \cdot \alpha \cdot \omega_{wd} \tag{2-63}$$

As a minimum, the length lc of the confined boundary element should not be taken as being smaller than $(0{,}15 \cdot l_w)$ or $(1{,}50 \cdot b_w)$.

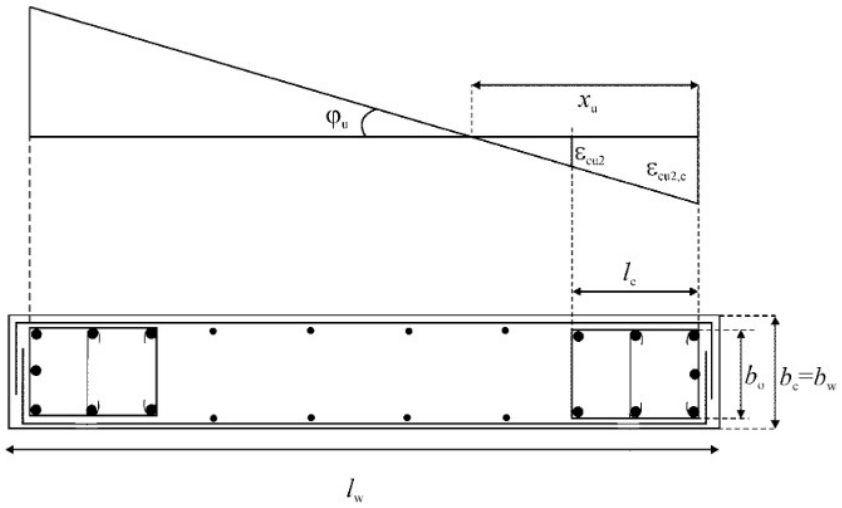


Figure 2-40: Confined boundary element of free-edge wall end

2.1.5.10.4 Detailing provisions for local ductility of walls

The detailing rules according to EN 1998-1:2004[37] for walls are summarised in Table 2-19.

Table 2-19: Detailing provisions for local ductility for beams (EN 1998-1:2004[37])

Walls		DCL	DCM	DCH
Critical region height	h_{cr}	-	≥max(l_w,H_w/6) ≤min(2·l_w,h_{storey}) if wall ≤ 6 storeys ≤min(2·l_w,2·h_{storey}) if wall ≥ 7 storeys	
Boundary elements				
a) In critical height region				
Length l_c from wall edge ≥	l_c	-	0,15·l_w; 1,5·b_w part of the section where ε_c>0,0035	
l_c≥		x_u·(1-ε_{cu2}/ $\varepsilon_{cu2,c}$)		
Thickness b_w over l_c ≥	b_w/l_c	-	(0,2m;h_{st}/15) if l_c≤max(2·b_w;0,2·l_w)	
			(0,2m;h_{st}/10) if l_c>max(2·b_w;0,2·l_w)	
Vertical reinforcement:				
Minimum reinforcement ratio over A_c=l_c·b_w	ρ_{min}	0,2%	0,5%	
Maximum reinforcement ratio over A_c	ρ_{max}	4,0%		
Spacing along the perimeter of bars restrained by tie corner or cross-tie hook	s	≤200mm		
Confining hoops				
Diameter	d_{bw} ≥	6mm	8mm	8mm
Spacing	s_w≤	min(b_0/2;175mm;8·d_{bL})		
Mechanical reinforcement ratio	ω_{wd} ≥	-	0,08	0,12
Confinement integrity verification	$\alpha\cdot\omega_{wd}$	-	30·μ_ϕ·ν_d·$\varepsilon_{sy,d}$·b_c/b_o-0,035	
b) Over the rest of the wall				
Minimum reinforcement ratio	ρ_{min}	0,5 % wherever in the section ε_c>0,2%		
		0,2 % elsewhere		
Distance of unrestrained bar in compression zone from nearest restrained bar		≤150mm		
Diameter,	d_{bw} ≥	max(6mm, d_{bL}/4)		
Hoops up to a distance of 4·b_w above or below floor beams or slabs	s_w≤	min(12·d_{bL};0,6·b_{wo};240mm)		
Hoops beyond a distance of 4·b_w above or below floor beams or slabs		min(20·d_{bL};b_{wo} ;400mm)		
Web				
Web thickness	b_{wo}≥	max(150mm;h_{storey}/20)		
Vertical bars				
Minimum vertical reinforcement ratio	$\rho_{v,min}$=A_{sv}/(b_{wo}·s_v)	0,2%	Wherever ε_c>0,2%:0,5% elsewhere 0,2%	
Maximum vertical reinforcement ratio	$\rho_{v,max}$=A_{sv}/(b_{wo}·s_v)	4,0%		
Spacing	s_v≤	min(3·b_{wo} ;400mm)		min(25·d_{bv} ;250mm)
Horizontal bars				
Minimum horizontal reinforcement ratio	$\rho_{h,min}$=A_s/(b_{wo}·s_h)	max(0,1%;0;25·ρ_v)		0,2%
Spacing of horizontal rebars	s_h≤	400mm		min(25·d_{bv} ;250mm)

2.1.6 COMPARISON BETWEEN INTERNATIONAL SEISMIC CODES

Seismic codes are crucial for ensuring the structural safety of buildings and infrastructure in earthquake-prone regions. This chapter provides a detailed comparison of the seismic codes from the United States, New Zealand, Chile, Mexico City, and Europe, examining their methodologies, ductility requirements, behaviour factors, and drift control and stability analysis provisions. The analysis is based on the most relevant sections of these codes.

Analyzing structural system types across various seismic design codes reveals intricate differences and shared principles in their approaches to seismic resistance. These differences and similarities are highlighted in the comparison between ASCE/SEI 7-22[11], various NZS 1170.5[19] and NZS 3101:part 1[20] (New Zealand standards), NTC-DS:2017[33] (Mexico City Seismic standard), NCh433.Of1996[28] (Chilean Standard), European standard EN 1998-1:2004[37].

Both ASCE/SEI 7-22[11] and the NZS 3101:part 1:2006[20] standards categorize systems with labels such as "special," "ordinary," and "intermediate" or similar terms to indicate varying levels of ductility and corresponding seismic performance.

However, the specifics of how structural systems are classified and the requirements for each category can vary significantly. The ASCE/SEI 7-22:2022[11] standard offers a detailed classification of over 90 structural system types, organizing them into distinct groups such as moment-resisting frames, dual systems, and bearing wall systems, each with defined applications based on seismic design categories (SDC). In contrast, the NZS 1170.5[19] and NZS 3101:part 1:2006[20] standards provide a more generalized approach, often blending structural elements like frames and walls within broader system categories such as dual systems.

Distinctive differences are also evident in the seismic design frameworks of the NTC-DS:2017[33] and NCh433.Of1996[28]. The NTC-DS:2017[33] emphasizes adaptability and detailed criteria for structural systems such as frames and wall systems, specifying requirements for ensuring ductility and resilience at different seismic intensities. Table 2-20 compares the structural systems across the seismic codes.

Table 2-20: Structural system comparison across seismic codes

Seismic Design Code	Structural System
ASCE/SEI 7-22 (U.S.A)	Over 90 types are categorized into moment-resisting frames, dual systems, and bearing wall systems, organized by seismic design categories and ductility requirements.
NZS 1170.5:2004 & NZS 3101(New Zealand Standards)	Combination of frames and walls, dual systems, with broad categorization rather than specific types, emphasizing the blending of structural elements for seismic resilience.
NCh433.Of1996(Chile)	Emphasis on the shear wall and braced systems, moment-resisting space frames, with performance-based criteria for seismic design.
NTC-DS:2017(Mexico City)	Includes various systems such as frames, dual systems with concrete frames and walls, systems using cantilever concrete columns, and specialized systems like suspended systems supported by a concrete core.
EN 1998-1:2004 (Europe)	Reinforced Concrete Moment Resisting Frames, Wall system, Dual Systems,

When comparing the structural types across different seismic design codes, it is important to note how each code integrates ductility classes with specific structural systems to enhance earthquake resilience.

In the United States, ASCE/SEI 7-22:2022[11], although not explicitly classifying structures by ductility as Eurocode 8, also considers ductility in its seismic design categories. It provides detailed requirements for various structural systems, including special moment

frames and shear walls, which are expected to exhibit higher ductility and thus are suitable for areas expecting more intense seismic activity.

New Zealand's standards, such as NZS 1170.5:2004[19], address ductility through designations like Nominally Ductile, Limited Ductility, and Ductile Structures. Each designation correlates with a specific structural framing system capable of achieving the ductility required to withstand local seismic forces. This approach ensures that structures are designed to resist seismic forces and to deform in a controlled manner that enhances safety during earthquakes. Chile's NCh433.Of1996[28] and Mexico City's NTC-DS:2017[33] incorporate ductility considerations into their structural system recommendations, albeit tailored to their unique seismic challenges. Chile's code emphasizes the cohesion between components in complex floor plans to ensure uniform behaviour across the structure during seismic events. Mexico City's code uses systems that can adapt to the local soil conditions, significantly amplifying seismic effects, thereby necessitating systems with higher ductility. EN 1998-1:2004[37] distinctly classifies structural systems into Low, Medium and High ductility classes. These classes guide the selection of systems, like moment-resisting frames and shear walls, designed to meet specific deformability and energy absorption criteria under seismic loading. This clear categorization helps engineers choose structural systems that align with both the seismic intensity expected in the region and the building's criticality. Table 2-21 shows the structural types across codes.

Table 2-21: Structural type comparison across codes

Seismic Design Code	Structural type
U.S. (ASCE/SEI 7-22 (U.S)	Ordinary (limited ductility) Intermediate (medium ductility) Special (high ductility)
NZS 1170.5:2004 (New Zealand Standards)	Nominally Ductile Structures (NDS) Structures of Limited Ductility (SLD) Ductile Structures (DS)
NCh433.Of1996 (Chile)	The Chilean code does not declare ductility classes specifically
Mexico City (NTC-DS:2017)	Low ductility Medium ductility High ductility
Eurocode 8 (EN 1998-1:2004)	Ductility Class Low (DCL) Ductility Class Medium (DCM) Ductility High (DCH)

When comparing the seismic design codes from NZS 1170.5:2004[19] and ASCE/SEI 7-22[11] concerning structural irregularities and their impact on seismic response, both similarities and differences emerge, reflecting the nuances in addressing seismic resilience across different geographic and seismic conditions. Both codes recognize the significant impact of structural irregularities on a building's seismic performance, identifying variations in mass, stiffness, and geometric configuration that deviate from idealized, regular structures. Both codes categorize these irregularities into vertical and plan (horizontal) types, acknowledging that such deviations can lead to unexpected behaviour during seismic events due to complex dynamic responses. Recognizing these factors in both codes underscores a common understanding that irregular structures pose a higher risk during earthquakes and thus require specific attention in the design process. NZS 1170.5:2004[19] focuses on detailed descriptions of possible irregular configurations, such as vertical irregularities due to uneven story heights or plan irregularities from non-symmetrical layouts, providing specific examples and outlining the consequences on load distribution and dynamic response.

ASCE/SEI 7-22:2022[11], on the other hand, categorizes irregularities more systematically, providing a structured table format that classifies types of irregularities (both plan and vertical) and describes them in a way that directly ties to their implications for seismic analysis and design. This includes specifying percentages for changes in stiffness or mass that qualify as irregular, thereby offering quantitative benchmarks for engineers. ASCE/SEI 7-22[11] also discusses the need for enhanced analysis and potential strengthening of structures identified with these irregularities, particularly in higher seismic design categories (E or F), indicating a proactive approach to ensure safety despite these complexities. This systematic approach reflects the vast and varied terrain of the United States and is likely influenced by the extensive experience with different earthquake scenarios across the country.

When considering other standards like Chile's NCh433.Of1996[28] and Mexico NTC-DS:2017[33], further distinctions and commonalities highlight each region's approach to managing structural irregularities under seismic forces, emphasizing the importance of unified building behaviour and detailed planning to enhance structural resilience. EN 1998-1:2004[37] addresses structural irregularities with considerable detail, underscoring their potential to significantly influence a building's response to seismic forces. The code identifies both plan and vertical irregularities that might affect the seismic behaviour of structures. Plan irregularities are concerned with the layout and geometry of the building, such as T-shaped or L-shaped plans, which can lead to torsional effects during seismic events. Vertical irregularities involve stiffness, mass, or strength variations from one floor to another, potentially making weak or soft stories more susceptible to damage under earthquake loading.

Table 2-22: Structural type comparison across codes

Seismic Design Code	Structural regularities/irregularities
ASCE/SEI 7-22 5 (U.S.A)	▪ Defines horizontal and vertical irregularities, such as torsional, reentrant corners, diaphragm discontinuity, out-of-plane offsets, and nonparallel systems. ▪ Vertical irregularities include stiffness soft story, weight (mass), vertical geometric, and in-plane discontinuity in vertical lateral force-resisting elements.
NZS 1170.5:2004 (New Zealand)	▪ Discuss configuration irregularities affecting a building's seismic response, including stiffness, strength, and mass distribution issues.
NCh433.Of1996 (Chile)	▪ Focuses on buildings with irregular floor plans, such as H-shaped, L-shaped, T-shaped, and U-shaped, and emphasizes the unified behaviour of diaphragms during earthquakes.
NTC-DS:2017 (Mexico City)	▪ Lists criteria for regularity in orientations of walls and frames, ratios of height and base dimensions. ▪ Floor plan configurations to ensure uniform stiffness and resistance across levels.
EN 1998-1:2004 (Europe)	Categorizes irregularities into two main types: plan and vertical irregularities: ▪ Plan irregularities include re-entrant corners, diaphragm discontinuity, and torsional irregularity. ▪ Vertical irregularities encompass soft or weak stories, abrupt changes in stiffness, and mass irregularity.

Comparing seismic design codes such as ASCE/SEI 7-22:2022[11], NZS 1170.5:2004[19], NTC-DS:2017[33], and NCh433.Of1996[28], each code's nuanced approaches to ensure structural safety in seismic conditions become evident, reflecting their respective regions' distinct seismic risks and engineering practices. ASCE/SEI 7-22[11], widely used in the United States, adopts a highly prescriptive method, offering a structured methodology for determining seismic forces. This involves calculating the base shear based on the seismic design category (SDC), the building's fundamental period of vibration, and additional factors such as the importance factor and the response modification coefficient. The

code mandates meticulous calculations that consider the building's weight and operational loads, and it outlines detailed procedures for assessing vertical seismic loads, especially in higher (SDC) categories, accommodating the diverse geological and seismic challenges found across the country.

In contrast, NZS 1170.5:2004[19] and NZS 3101:part 1[20], which governs seismic design in New Zealand, addresses the unique challenges of a region with high seismic activity by emphasizing the calculation of elastic site spectra through a probabilistic model that accounts for numerous active faults. This standard incorporates specific factors such as spectral shape and near-fault factors, tailoring its approach to the local geological conditions and the prevalent seismic activity. NZS 1170.5:2004[19] and NZS 3101:part 1[20] consider soil-structure interaction effects vital in a country with varied soil types and topographical features that influence seismic wave behaviour. Meanwhile, NTC-DS:2017[33] from Mexico and NCh433.Of1996[28] from Chile each brought their historical and regional contexts into their standards, focusing on detailed local conditions and seismic zonation, respectively, to define design spectra and seismic design parameters that reflect their specific seismic risk profiles. These varied methodologies underscore each code's strategic adaptations to local conditions, from highly prescriptive to probabilistic and adaptive approaches, highlighting a comprehensive range of strategies to enhance structural resilience against earthquakes. Table 2-23 summarises the seismic actions design across the seismic codes.

Table 2-23: Seismic action comparison across codes

Seismic Design Code	Seismic Actions Description
ASCE/SEI 7-22 (U.S.A)	▪ The response spectrum method and seismic design criteria vary depending on the structure's seismic design category. ▪ Definition of parameters such as spectral acceleration values, site coefficients, and the importance factor, which adjust the design response spectrum for different types of soil and building importance
NZS 1170.5:2004 and NZS 3101:part 1 (New Zealand)	▪ Provides seismic hazard assessments and design spectra, determining seismic loads based on probabilistic models considering active faults and site conditions.
NCh433Of1996(Chile)	▪ It uses seismic zonation to establish design shaking intensities and defines the design spectrum of pseudo-accelerations based on maximum effective soil acceleration, soil type, and occupancy importance factors.
NTC-DS:2017(Mexico City)	▪ It specifies that design actions for seismic loads should be based on the SASID design spectra and adjusted for seismic behaviour and over-strength factors. ▪ Addresses the use of modal dynamic analysis for determining seismic forces
EN 1998-1:2004(Europe)	▪ Emphasizes the control of inelastic response mechanisms through capacity design and defines seismic actions in terms of elastic response spectra. ▪ Specifies design ground acceleration and provides the formulas for calculating seismic forces based on the behaviour factor and structural ductility.

When comparing the approaches to structural analysis across various seismic design codes such as ASCE/SEI 7-22:2022[11], EN 1998-1:2004[37], NZS 1170.5:2004[19], NCh433.Significant methodological differences are evident in 1996[28] from Chile and NTC-DS:2017 from Mexico City. ASCE/SEI 7-22:2022[11] and EN 1998-1:2004[37] offer comprehensive frameworks that include linear and nonlinear analysis methods, catering to a broad spectrum of structural complexities and seismic intensities. ASCE/SEI 7-22[11] utilizes

the Equivalent Lateral Force procedure and various modal response spectrum analyses. At the same time, Eurocode 8 facilitates detailed analyses by incorporating advanced dynamic and static procedures tailored to European seismic conditions. NZS 1170.5:2004[19] aligns with these by supporting a blend of linear and nonlinear techniques, emphasizing capacity design critical for New Zealand's high seismic activity.

In contrast, NCh433.Of1996[28] and NTC-DS:2017[33] focus heavily on adapting their analysis strategies to Chile and Mexico's specific seismic risks and geological features, respectively. This includes using site-specific seismic response evaluations and detailed dynamic analyses to ensure the structural systems are robust and capable of withstanding localized seismic challenges. These codes reflect a global consensus on the need for versatility in seismic structural analysis. The structural analysis performed by the seismic codes under analysis is summarized in Table 2-24.

Table 2-24: Structural type comparison across codes

Seismic Design Code	Structural Analysis Description
ASCE/SEI 7-22 (U.S.A.)	Employs both linear and nonlinear analysis methods: ▪ Equivalent Lateral Force procedure ▪ Modal Response Spectrum Analysis, while nonlinear methods include ▪ Nonlinear Static Procedure (pushover analysis) ▪ Nonlinear Response History Analysis.
NZS 1170.5:2004 (New Zealand)	Prescribes both linear and nonlinear methods: ▪ Modal Response Spectrum Analysis, ▪ Capacity Spectrum Method ▪ Nonlinear Time-History Analysis
NCh433Of1996 (Chile)	Focuses on the linear analysis and nonlinear analysis ▪ Modal Response Spectrum Analysis for linear analysis and allows for ▪ Nonlinear Static and Dynamic Analyses
NTC-DS:2017 (Mexico City)	Includes detailed provisions for both linear and nonlinear ▪ Modal Response Spectrum Analysis) and nonlinear analysis ▪ Nonlinear Static and Dynamic Analysis
EN 1998-1:2004 (Europe)	Offers multiple analysis options, including: ▪ Lateral Force Method (linear static analysis) ▪ Modal Response Spectrum Analysis ▪ Linear Time-History Analysis ▪ Non-linear Static Analysis (pushover) ▪ Non-linear dynamic Analysis (time-history)

Regarding the behaviour factor, ASCE/SEI 7-22[11] does not explicitly define a behaviour factor like EN 1998-1:2004[37] but focuses on detailing provisions to ensure ductility in seismic design. The design relies on a strength design methodology where force reductions are implicit through ductility-related design and detailing requirements.

NZS 1170.5:2004[19] provides specific behaviour factors based on ductility and structural performance, adjusting seismic loads accordingly. Emphasizes the capacity design approach, where key elements are designed to remain elastic by providing energy dissipation through specific failure mechanisms in other designated ductile zones. The Chilean Seismic Design Code NCh433.Of1996[28] utilizes a behaviour factor similar to other international

standards aimed at reducing seismic forces based on the ductility and overstrength of the structure. The Chilean code strongly emphasises detailing and confinement of reinforcement to enhance ductility, similar to NZS 1170.5:2004[19] and ASCE/SEI 7-22[11]. However, it typically specifies higher behaviour factor values, reflecting the high seismicity of the region. The behaviour factor in Mexican standard NTC-DS:2017[33] is used to modify the design seismic forces considering the expected ductility and overstrength.

The NTC-DS:2017[33] provides tables with specific behaviour factors depending on the type of structural system and its ductility characteristics. It should be noted that the definition of behaviour factor is similar to EN 1998-1:2004[37]. EN 1998-1:2004[37] uses a behaviour factor (q), which is defined as the ratio between the peak force that would have developed if the single-degree-of-freedom system (SDOF) were linear-elastic to the yield force of the system. The behaviour factor in EN 1998-1:2004[37] reduces the seismic force by accounting for the structure's energy dissipation capability, ductility, and overstrength.

ASCE/SEI 7-22[11] focus on ensuring ductility through design and detailing provisions rather than explicitly using a behaviour factor, embedding these principles within the design methodology. The NZS 1170.5:2004[19] mix explicit behaviour factor usage with capacity design principles, focusing on global and local ductility to ensure seismic resilience. Chilean Code NCh433.Of1996[28] have a similar philosophy to NZS 1170.5:2004[19], emphasizing high behaviour factors and stringent detailing requirements due to the region's high seismic activity. Mexican Code NTC-DS:2017[33] also utilizes behaviour factors to tailor seismic design forces, with a clear focus on ductility and overstrength, providing specific values and requirements for various structural systems reflecting the seismic challenges in the region. The European code EN 1998-1:2004[37] is comprehensive, introducing behaviour factors that address different aspects of structural response, making it highly adaptable and detailed for varied structural conditions.

Regarding the capacity design, similar to EN 1998-1:2004[37], ACI 318-19[12] focus on the strong column-weak beam design but does not explicitly call this approach "capacity design." Instead, the design and detailing requirements ensure that beams yield before columns. The ACI 318-19[12] code specifies that the moment capacity of columns should be higher than that of beams at their connections, thereby promoting ductile failure in beams first.

NZS 1170.5:2004[19] and associated codes explicitly incorporate capacity design principles to control the location of plastic hinges in structural members. These standards ensure that columns can support at least 1,2 times the sum of the demands placed on them by the beams, which is a clear application of the capacity design philosophy ensuring column strength exceeds that of beams. The Chilean code NCh433.Of1996[28] follows similar principles to NZS 1170.5:2004[19], emphasizing the prevention of column failure before beam yielding. It requires detailed calculations to ensure that columns have sufficient strength and stiffness to support the forces from beams during seismic events. The NTC-DS:2017[33] also adopt a strong column-weak beam strategy, requiring detailed design to ensure that beams yield before columns. The design philosophy aims to prevent building collapse by ensuring that the most critical structural elements (columns) have higher capacity than the demands. EN 1998-1:2004[37] emphasizes ensuring that plastic deformations occur in beams rather than in columns to prevent column failure, which can be catastrophic. This approach enforces the "strong column - weak beam" philosophy, where columns are designed to have more robust seismic resistance than beams. The code mandates that the capacity of columns must not be

exceeded by the demand from the beams they support, effectively shifting potential plastic hinges away from columns.

Concerning the detailing for local ductility, the seismic design codes either provide guidelines for designing and detailing RC buildings for different ductility classes or refer to complimentary design codes. These provisions, in general, consist of four requirements:

- Capacity design provisions to achieve a hierarchy of strength to avoid brittle failure modes
- Provisions of flexural reinforcement within potential plastic hinge locations for beams, columns and walls.
- Provisions for shear reinforcement potential plastic hinge locations beams, columns and walls.
- Provision of confining reinforcement at potential plastic hinge locations columns and walls.

In concrete members, the mechanisms of force transfer by shear or by flexure act in series, as both have to transfer the same force and ultimate strength is controlled by the weakest of the two mechanisms. The shear force transfer mechanism can be constrained to the elastic range through capacity design, namely, by dimensioning a concrete member in shear not for the force demand from the analysis but for the maximum shear force that may physically develop in it, as controlled by attainment of the force resistance in flexure.

Concerning concrete confinement, reinforcing steel is inherently ductile, and concrete is brittle unless its lateral expansion is well restrained by confinement. Flexure is the only force transfer mechanism allowing the advantage of tension reinforcement's fundamental ductility. It effectively enhances concrete and compression steel's ductility through lateral restraint. It induces principal stresses and strains in any inclined direction (especially for cyclic loading). It does not lend itself to effective inelastic action in reinforcement to control the extent of cracking and concrete confinement. Table 2-25 briefly compares the detailing for local ductility used across the codes for different structural types.

Table 2-25: Design and detailing provisions for different ductility classes in building codes

Ductile detailing criteria		ACI 3019-19 2019			NZS 1170.5 2004			NCh433 .Of1996			NTC-S 2017			EN 1998-1 2004		
		OMRF	IMRF	SMRF	NDS	SLD	DS	SWBS	MRF	DS	L	M	H	DCL	DCM	DCH
Capacity design	Flexural		●	●		●	●		●	●		●		●	●	
	Shear		●	●		●	●		●	●		●		●	●	
Flexural reinforcement			●	●		●	●		●	●	●			●	●	
Shear reinforcement			●	●		●	●		●	●	●			●	●	
Confinement reinforcement			●	●		●	●		●	●		●	●		●	●
Special rules for anchorage of reinforcement			●	●	●	●	●		●	●		●	●			●

It is not easy to compare different ductility classes of various codes directly. However, some conclusions can be drawn. All regulations prescribe the provision for highly ductile structures, capacity design, confinement reinforcement, and special anchorage and joint shear design rules. All five codes recognise the importance of the flexural overstrength of beam ends

and the ramifications for inelastic response control via capacity design. The five codes establish the capacity-design shear force of beams in about the same way.

2.1.7 RESEARCH AND DEVELOPMENT OF THE BEHAVIOUR FACTOR – LITERATURE REVIEW

The present chapter aims to provide the background of the definition and the evolution of the concept of the behaviour factor (q).

2.1.7.1 BACKGROUND

Design requirements for lateral loads, such as winds or earthquakes, differ fundamentally from those for loads. While design for wind loads is a primary requirement due to the frequency of the loading scenario, seismic design deals with events with a lower probability of occurrence. It may, therefore, be highly uneconomical to design structures to withstand earthquakes for the performance levels used for wind design. For example, building structures would typically be designed for lateral wind loads of 1% to 3% of their weight.

Earthquake loads may reach 30–40% of the structure's weight applied horizontally. If concepts of plastic design used for primary loads are employed for earthquake loads, cumbersome and expensive structures will ensue. Therefore, seismic design, by necessity, uses concepts of controlled damage and collapse prevention. Indeed, buildings are usually designed for only 15–20% of the elastic earthquake forces (V_e). This is illustrated in Figure 2-41, where the elastic and inelastic responses are depicted. The concept of equal energy reduces the design forces from (V_e) to ($V^*_{e,u}$), denoting elastic and design force levels. Therefore, the damage is inevitable in seismic response and design. It is the type, location and extent of damage that is the target of the design and detailing the process in earthquake engineering. The ratio between elastic base shear force (V_e)and horizontal shear force for a displacement (d_r) of a structure with all sections identical to the real ones but which would be made of a material with an infinite yield strength ($V^*_{e,u}$) shears is defined as 'force reduction factor (R) according to Elnashai et al.[36], as depicted in equation (2-64):

$$R = \frac{V^*_{e,u}}{V_e} \tag{2-64}$$

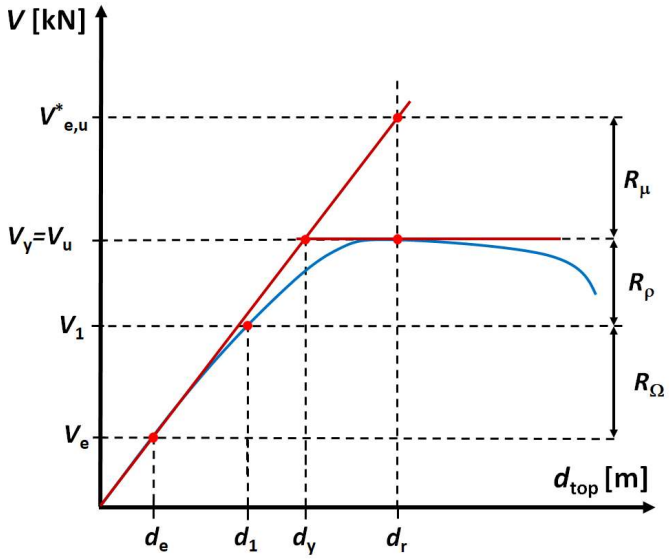


Figure 2-41: Value Base-shear versus roof displacement relationship. Overstrength (R_Ω), redundancy (R_ρ), and ductility (R_μ) reduction factors.

The force reduction factor is generally expressed as the product of different factors.

$$R = R_{\mu} \cdot R_{\rho} \cdot R_{\Omega} \tag{2-65}$$

R_{μ}, R_{ρ}, and R_{Ω} are ductility, redundancy, and overstrength. The meaning of the other parameters may be highlighted based on the pushover curve obtained by nonlinear static analysis.

The meaning of the other parameters is highlighted on the pushover curve obtained by nonlinear static analysis. Figure 2-41 shows the base shear force, V, versus roof displacement (d) relationship and its elastoplastic idealization [37]. The parameters in this figure, besides (V_e) and ($V^*_{e,u,}$)are defined as the first significant yield strength at the formation of the first hinge (V_1) and the idealized yield strength(V_y). The parameters are the ultimate displacement (d_r) and yield displacement (d_y). Concerning Figure 2-41 and equation (2-65), the components of force reduction factor (R) may be expressed in three distinct components: (R_{μ}), (R_{ρ}) and (R_{Ω}).

The ductility reduction force factor component (R_{μ}) is characterized by the ratio between horizontal shear force for a displacement (d_r) of a structure with all sections identical to the real ones but which would be made of a material with an infinite yield strength ($V^*_{e,u}$) and the horizontal shear force at yield (V_y), as depicted in expression (2-66), depends on both structural properties such as ductility, damping, and fundamental period and the characteristics of seismic ground motion.

$$R_{\mu} = \frac{V^*_{e,u}}{V_y} \tag{2-66}$$

The ratio between the horizontal shear force at yield (V_y) and the horizontal shear force at the formation of the first plastic hinge (V_1) characterizes the redundancy reduction factor (R_{ρ}) as depicted in equation (2-67):

$$R_{\rho} = \frac{V_y}{V_1} \tag{2-67}$$

Concrete structures should be designed and detailed to transfer the seismic forces to the foundation through a redundant framing system. The overstrength factor (R_{Ω}) is defined as the ratio between horizontal shear force at the formation of the first plastic hinge (V_1) and elastic base shear force (V_e), as depicted in expression (2-68):

$$R_{\Omega} = \frac{V_1}{V_e} \tag{2-68}$$

This parameter accounts for the overall structural overstrength, which is produced by the design assumptions that typically have a conservative design:

- The design strength of materials with partial safety factors.
- Code requirements for reinforcement details.
- Drift limitations.
- Design engineering practice.
- Design simplifications concerning the methods of analysis.

The force-based design concept prescribed by the modern design codes specifies that the structural system's design (sizing and detailing) of structural elements is developed under prescribed equivalent statically applied forces. In earlier generations of seismic design codes, the seismic loads applied to the structure were non-factored, and the verification at the member level was evaluated using linear elastic analysis. For seismic loading and following the assessment of the response of simple structures following severe earthquakes in California in the 1950s, it was demonstrated by Housner[38] that a sufficient inelastic deformation capacity by the structure through controlled "underdesign" of the elements and local detailing, led to these structures acquire an acceptable behaviour, during an extreme seismic event.

Considering the transient time-varying nature of the loading and the response, the type of the loading and inertia effects, and the extreme magnitude of the load intensity, a reduction factor was proposed to be used to reduce the design of the structure to levels lower than those required for complete elastic response, at the expense of increased inelastic global and local deformations of a controlled magnitude and distribution. This reduction is imposed by recognizing that an "underdesigned" structure with adequate ductility capacity survives the extreme loading event scenario with tolerable inelastic deformation acceptable for its occupants and equipment's safety. As an extension of this initial collapse limitation design approach implemented in the 1970s, additional intermediate performance requirements for occupancy and equipment have been adopted, leading to a complete performance-based design concept for the structures. In order to reduce the design load for elastic response, a factor greater than 1,0 is adopted and is denoted as the behaviour factor (q), as stated by Zeris[39].

The problem faced by standardisation committees worldwide is quantifying the behaviour factor. Recent research results regarding the assigned value for the behaviour factor have raised concerns regarding their scientific determination basis. Most seismic design codes use the behaviour factor to include the effects of plastification in structural systems when subjected to ground motions. Uang[40] stated that the most controversial part of seismic design provisions for building structures is developing the behaviour factors. The strength reduction factors, R (or behaviour factor), for various structural systems, have been determined based mainly on engineering judgment and accumulated experiences from past earthquakes rather than the theoretical background, according to Lee et.[41]

Several research works have attempted to include the energy-dissipating effects of inelastic yielding into the elastic response spectrum for seismic design. The observation that inelastic deformations in the structure may be utilised to absorb high levels of energy has allowed the seismic design to be significantly improved, as described by Borzi and Elnashai[42]. Such a design procedure significantly reduced the design seismic forces, allowing for more cost-effective buildings. For this reason, the behaviour factor has been introduced to seismic design.

Borzi et al.[42] have defined the behaviour factor as "the all-embracing" parameter that considers overstrength, energy absorption and dissipation and the structural capacity to redistribute actions from inelastic highly stressed regions to other less stressed locations in the structure". Most existing building design codes deal with this problem using the seismic force from spectra based on linear behaviour, using a behaviour factor that modifies the linear-elastic force system to an equivalent force that accounts for the nonlinear effects prescribed by Fathi et al.[43].

Experience shows that details with appropriate ductility and regularity levels are crucial in structural systems. Lam et al.[45] state that the behaviour factor values have been developed from Californian research and experience and broadly account for the structural system's characteristics, such as energy absorbing capacity, overstrength, and the degree of redundancy and performance in past seismic events. This reduction in the elastic demand placed on a structural system reduces the design strength by creating more affordability.

This concept's application has been proven to work effectively in many analytical and experimental studies during recent decades. However, for instance, Dimova et al.[46] concluded that the reduction of the design seismic loading due to underestimating the structural stiffness leads to a considerable decrease in the behaviour factor supply, dependent on the initial stiffness of the structure. Moreover, their experimental study concluded that the behaviour factor demand of 4,95 for ductility class H (High), preconized in EN 1998-1:2004[37], could be matched for the studied structure only by consideration of initial stiffness corresponding to the initiation of cracking and not by the stiffness corresponding to the initiation of yielding of the reinforcement.

2.1.7.2 PROPOSED FORMULATIONS OF THE BEHAVIOUR FACTOR

This subchapter intends to show some proposals' evolution to estimate the behaviour factor and their achievements, led by some research works in past decades.

The methods to evaluate the behaviour factor can be classified into two main categories based on the degree of freedom: strategies based on single-degree-of-freedom (SDOF) and multiple-degree-of-freedom (MDOF) systems. A range of properties, such as the natural period of vibration, the force-deformation relationship, and the level of viscous damping, may reasonably characterize the research using SDOF systems. Parameters like the SDOF system's vibration period and the ductility ratio significantly impacted the behaviour factor. Therefore, research works have proposed reduction factors in the interdependence between behaviour factor, ductility ratio and period (q-μ-T).

A comparison of the produced artificial inelastic spectra with the real ones showed that the reduction factor and reduction factor proposed by Vidic et al.[47] resulted from an accurate estimation of the inelastic spectra. In their work, it is concluded that the value of the response modification factor (R) depends on the natural period of the system, the prescribed ductility factor, the hysteretic behaviour, damping, and ground motion. Fairly accurate approximations of the inelastic spectra for strength and displacement can be derived from the elastic spectrum using the proposed values for (R) Vidic et al.[47]. The proposed behaviour factors have been incorporated in FEMA 273[9]. The above-mentioned behaviour factors resulted in a seismic design that assumed that structural damage occurs only due to maximum response deformation. Hence, the aspect of cumulative damage was ignored.

Uang[40] proposed in 1991 that the behaviour factor can be decomposed similarly to reduce ductility, overstrength and allowable stress. Uang[40] concluded that the response modification factor (R) is a function of the structural overstrength factor, structural ductility factor (μ), and equivalent viscous damping ratio and compares some definitions of the behaviour factor, as defined in the seismic provisions of various building codes. Whittaker[48] provided a technical basis for (R) values in seismic regulations and guidelines. The formulation splits (R) into factors related to reserve strength (R_s), ductility (R_m), and redundancy (RR), making no allowance for formulating the effects of plan and vertical irregularity on building response. They observed that the interdependence of the strength and ductility factors is likely

to be very strong. The evaluations made by Whittaker et al.[48] Studies performed by others show that the reserve strength of code-compliant buildings varies widely depending on the building type, building height, and seismic zone. Values for strength factors must address these variations, and the influence of higher-mode effects must be studied further. Also, it is stated that the relations between the ductility factor and the global displacement ductility are well established for SDOF systems but should be extended to multiple-degree-of-freedom systems, MDOF, using a modification factor.

Cuesta et al.[49] applied a reduction factor evaluated based on the pulse to the elastic spectra of ground motions to determine the corresponding inelastic spectrum. This procedure to generate the inelastic spectra was repeated using the reduction factors proposed by other researchers. Their research refers to inelastic response spectra that may be estimated by dividing elastic response spectra by *R*-factors derived from a simple pulse response. The pulse *R*-factor depends on the pulse shape, ductility (μ), load-deformation model and the system's period (T) relative to the pulse characteristic period (T_p). Inelastic spectra estimated using pulse *R*-factors are compared with those obtained using (R-μ-T) relationships for elastoplastic, bilinear, and stiffness-degrading systems. The estimations' accuracy is influenced by the characteristic period of the ground motion and soft soil deposits that lead to nearly harmonic surface waveforms.

The collapse limit state was assumed to be the one for the selected damage function. Using the behaviour factor, Chai et al.[50] formulated the duration-dependent inelastic seismic design spectrum. The results of Chai et al.[50] indicated an increased lateral strength demand could be expected for an increased ground motion duration. However, in the long period range, the increase in lateral strength due to the duration increase is relatively uniform. The duration-dependent inelastic seismic design spectrum is also significantly influenced by the strength deterioration parameter that characterizes the structure's plastic strain energy capacity as a function of the cyclic displacement ductility factor.

Manfredi[51] proposed an energy spectrum (E_h/M) using a behaviour factor, in which the hysteretic energy is calculated as a function of parameters like ductility, peak pseudo-velocity, and a seismic index I_D. The method evaluates the equivalent number of cycles correlated to the earthquake characteristics by a seismic index I_D. This procedure allows us to obtain peak values of the hysteretic and input energy that depend on the demanded ductility, seismic index I_D, and peak pseudo-velocity. The hysteretic energy spectra could be found approximately by introducing appropriate expressions of (R), a function of the cyclic ductility, reducing the elastic spectrum (S_a).

Many researchers have proposed behaviour factors that consider the SDOF system. Also, the majority of the researchers have correlated the required maximum storey ductility of the MDOF system to be equal to the ductility factor of the SDOF system by relating the shear at yield shear of an MDOF system with the yield strength of an SDOF system for the same period (T). Nassar et al.[52] comprehensively evaluated seismic demand parameters for bilinear and stiffness degrading on SDOF systems, a detailed study on three types of MDOF systems. This aspect of the study aims to evaluate the modification that must be applied to strength demand parameters derived from simplified SDOF models to account for multi-modal effects in real structures. In the SDOF study, the inelastic strength and cumulative damage demands are evaluated statistically for specified target ductility ratios. Strength demands are represented in inelastic strength demand spectra or spectra of strength reduction factors. Expressions that relate the strength reduction factor to the period and target ductility ratio are developed. The MDOF study found that the required strength for specified target ductility ratios depends strongly on

the type of failure mechanism that develops during severe earthquakes. Quantitative information is produced on relative strength requirements for three types of MDOF structures, showing the disadvantage of structures in which story mechanisms develop, particularly the great strength capacities needed to control inelastic deformations in structures with weak.

Mohammadi et al.[53] proposed a relation for this ratio, which was observed to be highly dependent on ductility. The research mentioned above evaluated the behaviour factor of MDOF structures based on reducing the force of SDOF systems. A statistical process is outlined in their study to estimate the seismic demands of roof and story for MDOF structures. Roof drift demand is related to the maximum story drift demand through a simplified process that reasonably evaluates the seismic demand. An attempt has been made to adjust and extend the response modification factor by introducing a modifying factor (R_T). This factor accounts for the inelastic response in the dynamic analysis of MDOF structures. The research involves MDOF shear and frame buildings with various dynamic characteristics. The study results can provide a practical and straightforward way to determine the nonlinear dynamic response of MDOF structures. An approximate approach is presented to calculate the maximum inelastic deformations in a structure with a given strength distribution. The inelastic response of an MDOF structure designed for a base shear coefficient equal to that estimated from SDOF systems with periods equal to the fundamental period of the structure for a given ductility ratio (μ) subjected to the same strong ground motion is considered. The nonlinear dynamic analysis of such a system may estimate the base shear capacity required for an MDOF system to limit the maximum inter-story ductility demand on the target value. However, the Mohamadi et al.[53]study shows that the base shear demands of MDOF systems are greater than those of the corresponding SDOF systems to limit the ductility demand in both systems to the same value of (μ).

Khashaee et al.[54] Significant issues in the damage assessment of buildings were observed in limited experimental results. It was observed that the behaviour factor, considering the MDOF system, is directly related to the limit states based on energy-based damage indices, which was yet to be proposed. His research concludes that the lower stories of uniform buildings may experience more damage, whereas the middle storeys of non-uniform buildings may undergo more damage. Damage indices may be used in a damage-based design approach to computing the earthquake forces and hysteretic energy demand rather than a response modification factor based on the structural systems and construction materials. A damage index can be used for the strength, ductility, stiffness, and energy demands, whereas a response modification factor accounts for the strength and ductility demands. However, previous research reports a correlation between maximum deformation and cumulative damage indices. Thus, the behaviour factors can be effectively used in the cumulative damage-based seismic design procedure. The proposed behaviour factor depended upon the limit state under consideration and basic structural characteristics, such as the number of storeys and the joint capacity design factor.

Any limit state can be explicitly defined by selecting the allowable value of a dimensionless deformation damage index, i.e., the maximum storey ductility. A correlation study between the maximum and the damage index developed by Park et al.[55] provided a useful tool for the damage-based interpretation of the limit states under consideration. In the same study, monotonic and cyclic test data were analysed to evaluate the statistics of the appropriate parameters of the proposed damage model. The lognormal distribution was suitable for the damage index of reinforced concrete components.

Dolšek et al.[56] proposed a reduction factor (R)as the elastic and inelastic spectral acceleration ratio. The behaviour factor was in conjunction with the equal displacement rule and the "N2 method" to determine the seismic response of the SDOF system. The analysis showed that the proposed procedure yielded comparable results with dynamic analysis. The reduction factor R due to energy dissipation capacity is defined as the acceleration demand ratio in terms of the elastic spectral acceleration for the period (T)to the acceleration capacity S_a. The reduction factors used in seismic codes, e.g., the behaviour factor (q) used in EN 1998-1:2004[37], take into account the reduction due to overstrength and are thus not equivalent to the reduction factor (R) defined as the ratio of the acceleration demand S_{ae} in terms of the elastic spectral acceleration for the period (T), to the acceleration capacity (S_a). The relationships between the reduction factor (R), the ductility (μ), and the period (T) depend on the structural system type [56]. Dolšek et al.[56] also, the "equal displacement rule" is the most straightforward relation, often used for ordinary structures with medium- and long-range periods [57].

Athanassiadou[58] used the EN 1998-1:2004[37] proposed behaviour factor for the seismic design of RC structures with setbacks for DCM and DCH ductility classes. The inelastic dynamic analysis conducted on the building models showed satisfactory seismic performance. Moreover, the ductility class was observed to have the most negligible impact on seismic performance, which was quite an important observation as the DCM class is more economical and yields comparable performance to that of the DCH class. The research concerns analysing multistorey reinforced concrete MRF buildings with irregular elevation. In this study, two sets of ten-storey two-dimensional plane frames with two and four large setbacks in the upper floors, respectively, as well as a third one, have been designed to the provisions of the EN 1998-1 for the high (DCH) and medium (DCM) ductility classes, and the same peak ground acceleration (PGA) and material characteristics. It was concluded that the effect of the ductility class on the cost of buildings is negligible. Simultaneously, all irregular frames' seismic performance appears equally satisfactory, not inferior to regular ones. DCM frames are stronger and less ductile than the corresponding DCH ones. The overstrength of the irregular frames is similar to that of the regular ones, while DCH frames dispose of higher overstrength than DCM ones. One of the main conclusions is that the irregular structures seem more robust due to the decreased values of the behaviour factors (since the behaviour factor is reduced by 20% for irregular structures) and less ductile than the corresponding regular ones.

Anagnwstopoulou[59] addresses the problem of seismic design of irregular RC buildings according to EN 1998-1:2004[37] and defines the complexity of the analysis model, the design analysis methods to use, and the design "penalty" through the specification of a lower behaviour factor (q). This approach's reliability is evaluated by investigating the seismic performance of typical forms of vertically irregular buildings with different levels of irregularity for different vertical and irregularity torsional flexibility levels. For the objective evaluation of the behaviour factor (q), a nonlinear analysis following an automated linear analysis and design procedure is employed to establish the inelastic demands. Three of these frames have different first ground story heights, while the fourth is regular in height but has recessed setbacks on the fifth and sixth floors, with the remaining geometric characteristics being the same for all frames. The research analysis results concluded that the estimated response reduction factors (q) are higher than those assumed for the design for all cases considered.

Magliulo[60] observed that EN 1998-1:2004[37] proposed behaviour factors that lead to conservatively estimating the seismic response for irregular structure elevation. His work

presents the results of a research study concerning the seismic response and design of RC frames with overstrength discontinuities in elevation. The discontinuities are obtained by assigning overstrength to the beams or the columns of a "regular frame" assumed as reference. According to EN 1998-1:2004[37] provisions, these analyses are carried out for nonlinear static analysis, the N2 method is applied, and in the case of nonlinear time-history analyses, seven real earthquakes, selected to fit on average the elastic design spectrum, are used as input. The seismic response of frames characterised by the assigned overstrength differs greatly from the "regular frame" one. Moreover, all the frames satisfy the Ultimate Limit State, verified by applying nonlinear static and dynamic analyses.

Žižmond et al.[62] proposed a formulation to calculate the risk-targeted behaviour factor. The risk-targeted behaviour factor is formulated as a product of the conventional reduction factor and two factors that account for the return period of design seismic action, the target collapse risk, and the difference between the shape of acceleration spectra from the demand and capacity domain. Consequently, the behaviour factor is expressed as a function of the target collapse risk, the parameter of seismic hazard, the uncertainty of the seismic response, the ability of the structure to deform in the collapse range and the overstrength factor. It should be highlighted that this proposal will be used in this research project as a probabilistic method to estimate the behaviour factor(q). Table 2-26 condenses the various research efforts and their implications on the behaviour factor's estimation and application in seismic design.

Table 2-26: Research on the basic value of the behaviour

Researcher(s)	Focus Area	Key Findings and Proposals
Vidic et al.	Inelastic spectra estimation	Accurate estimation methods for inelastic spectra are proposed, influencing FEMA 273 seismic design.
Uang	Decomposition of behaviour factor	Behaviour factor as a function of structural overstrength, ductility, and damping.
Whittaker et al.	Technical basis for (R)	(R) divided into components of reserve strength, ductility, and redundancy, highlights the strong interdependence of strength and ductility factors.
Cuesta et al.	Pulse-based reduction factors	Estimated inelastic spectra using pulse R-factors, highlighting the importance of ground motion characteristics.
Chai et al.	Duration-dependent design spectrum	The lateral strength demand with longer ground motion durations emphasized the strength deterioration parameter.
Manfredi	Energy spectrum (E_h/M)	An energy spectrum is developed based on behaviour factors, considering ductility and seismic index.
Nassar et al.	Seismic demand on SDOF and MDOF	A detailed evaluation of seismic demand parameters highlights the need to account for multi-modal effects in real structures.
Mohammadi et al.	Dynamic response of MDOF structures	It proposes a method to estimate seismic demands, emphasizing the importance of modifying factors to address MDOF systems' dynamic responses.
Khashaee et al.	Damage assessment in MDOF systems	Significant issues in damage assessment were observed, and damage indices were proposed for earthquake force computation.
Dolšek et al.	Seismic response of SDOF system	Proposed a reduction factor (R) based on energy dissipation and emphasized the "equal displacement rule" and "N2 method".
Athanassiadou	RC structures with setbacks	Analysed seismic performance of RC structures with different ductility classes and found minimal impact of ductility class on performance.
Anagnwstopoulou	Irregular RC buildings	It investigated the seismic performance of irregular RC buildings, proposing a lower behaviour factor for design due to complexity and irregularity.
Magliulo	Seismic response of RC frames	Highlighted that proposed behaviour factors might lead to conservative estimates, verified by nonlinear static and dynamic analyses.
Žižmond et al.	Risk-targeted behaviour factor	A risk-targeted behaviour factor is formulated by accounting for various risk and structural parameters for probabilistic estimation methods.

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2.1.9 SYMBOLOGY

The following tables of symbols and their descriptions are not exhaustive. It includes key terms used in seismic design, emphasizing those explicitly defined or frequently mentioned in the context provided. The selection is curated to balance comprehensiveness with relevance, noting that additional symbols and terms may be applicable but are not listed here due to their absence or indirect reference in the initial information. This approach ensures clarity and focus on the most pertinent symbols for understanding seismic design principles and calculations.

2.1.9.1 U.S. SEISMIC CODES SYMBOLOGY

Symbol description	*Symbol*
The deflection amplification coefficient adjusts computed lateral displacements to expected inelastic displacements during earthquakes	C_d
Vertical seismic load component	E_V
The lateral force applied at each building level during seismic event analysis	F_i
Specified compressive strength of concrete used in reinforcement calculations	f'_c
Specified yield strength of steel used in reinforcement calculations	f_y
The height of the story or the level below is used in stability calculations.	h_x
The important factor is adjusting the level of seismic safety based on the structure's occupancy or use	I_e
An exponent is used to distribute seismic forces along the height of the building	k
The clear span of a beam	l_n
The clear span of the column	l_u
Nominal moment capacities of a beam	M_{nb}
Nominal moment capacities of a column	M_{nc}
Moment capacity at plastic hinges in beam and column design	M_{pr}
The total vertical design load at and above the level is considered in stability calculations	P_x
Response Modification Coefficient, influencing the base shear calculation	R
Design spectral acceleration parameters used in seismic design for short periods	S_{DS}
Design spectral acceleration parameters used in seismic design for a period of 1 second.	S_{D1}
Fundamental Period of Vibration	T_L
Base Shear Force	V
Story shear is the shear force at a particular level of the building	V_x
The weight and height of the i-th level are used to calculate the distribution of seismic forces	$w_{i,\ hi}$
Shear due to gravity loads on beams	w_u
Ratio of story shear demand under design seismic forces to the story shear strength, used in determining story drift amplification due to (*P-Δ*) effects	β
Storey drift or displacement in various equations	Δ
The redundancy factor enhances strength in seismic force-resisting elements if the SFRS configuration does not meet certain conditions	ρ
Overstrength Coefficient, indicating the additional strength considered beyond the expected seismic demands	Ω_0
Factors considered include overstrength due to material properties, load path, and redundancy in shear wall design	Ω_v
Angle used to quantify the severity of P-Δ effects at each storey	θ

2.1.9.2 NEW ZEALAND SEISMIC CODES SYMBOLOGY

Symbol description	*Symbol*
Gross area of the section.	A_g
Area of steel reinforcement	A_s
Elastic site spectrum for horizontal loading in New Zealand	$C(T)$
Design action coefficient used in seismic design calculations.	$C_d(T)/S_p$
The horizontal design coefficient of the part varies based on the period of the part	$C_p(T_p)$
Spectral shape factors are used for different seismic hazard levels	$C_h(T)$
Ductility demand or factor	D
Modulus of elasticity of concrete	E_c
Specified compressive strength of concrete	f_c
Yield strength of steel reinforcement	f_y
Yield strength is used in shear reinforcement calculations	f_{yt}
Moment of inertia of the cracked section	I_{cr}
Nominal span length	L_n
Factored ultimate moment.	M^*_u
Secondary effect involving interaction between axial forces and lateral displacements	$P\text{-}\Delta$
Fundamental period of the structure	T_1
Corner period at the long-period end of the spectrum's peak	T_c
Design shear force in columns	V^*_{col}
A uniform load per unit length is used in the design for bending and shear in beams and slabs	w
Dynamic magnification factors important for considering higher mode effects in structural design	ω_{max}
Displacement at the ultimate limit state is used for walls	Δ_u
Inter-storey height is used in seismic calculations to assess building sway and displacement	Δh_i
Base shears are in orthogonal directions in the X direction	V_x
Base shears in orthogonal directions in the Y direction	V_y

2.1.9.3 CHILE SEISMIC CODES SYMBOLOGY

Symbol description	*Symbol*
The seismic zone determines the maximum effective soil acceleration.	A_0
Coefficient used in the vertical distribution of base shear force to each diaphragm level. Factor depending on the level height, at level *k*	A_k
Coefficients are used in the vertical distribution of base shear force to each diaphragm level. Factor of the effective seismic weight at level *k*	P_k
Amplification factor in the design spectrum equation.	α
The largest dimension of the structure at level k perpendicular to the direction of analysis is used to calculate accidental torsional moments.	b_k
The seismic coefficient used in calculating the total seismic base shear is dependent on various factors, including A_0 and the response modification coefficient *R*	C
Maximum values of the seismic coefficient based on the response modification coefficient RR.	C_{max}
Lateral design displacement at the roof, considered equal to the ordinate of the elastic displacement spectrum (S_{de}) for a given damping, multiplied by a factor for the design of reinforced concrete structures.	δ_u
Dual systems containing a combination of shear wall and moment-resisting frame systems.	DS
Static force at each diaphragm level is used to calculate torsional moments.	F_k
Factor reducing required seismic design forces for buildings with reinforced concrete walls, dependent on the fraction of the total shear resisted by these walls.	f_q
The total height of the structure above grade is used in the distribution of base shear force.	H
Occupancy importance factor influencing the seismic design loads.	I
Coefficient dependent on soil type, used in calculating the seismic coefficient CC.	n
The effective seismic weight of the building above the base is used to calculate the base shear.	P
Total seismic base shear in each direction, a function of the seismic coefficient CC, importance factor II, and the effective seismic weight PP.	Q_0
Response modification coefficient varies based on the structural system used, which is critical in determining the design base shear	R
Soil type parameter in the design spectrum equation.	S
Characteristic site period	T'
Characteristic period of the mode with the highest translational mass participation	T^*
Height of levels *k* and (*k*-1) above grade, used in calculating force distribution within the structure.	Z_k, Z_{k-1}

2.1.9.4 MEXICO CITY SEISMIC CODES SYMBOLOGY

Symbol description	*Symbol*
Acceleration coefficient for the design spectra	a
Ground acceleration coefficient	a_0
Reduction factor for supplementary damping effects	β
Coefficient at the plateau of the seismic response spectrum	β_c
Coefficient used in static analysis for determining shear forces	c
The effective depth of a structural element (beams/columns)	d
Specified compressive strength of concrete	f'_c
Yield strength of steel used in reinforcement	f_y
Yield strength of the hoop reinforcement	f_{yh}
Interstorey drift for a specific floor	γ_i
Maximum allowable interstorey drift	γ_{max}
Height of a building or structural element	h
Total wall height	H
Critical height for bending moment in walls	H_{cr}
Height of the mass above the base in static analysis	h_i
Parameter used in the formulation of reduction factors	k
Span length of beams or horizontal dimension of an element	l
Length of a wall or structural member	L
Positive bending moment at the end of a structural member	M^+
Negative bending moment at the end of a structural member	M^-
The external moment applied at a joint in capacity design	M_e
Moment generated by the gravitational loads at a joint	M_g
Coefficient for calculation in reduction factors	p
Minimum longitudinal reinforcement ratio	p_{min}
Maximum longitudinal reinforcement ratio	p_{max}
Seismic behaviour factor	Q
Modified seismic behaviour factor accounting for ductility and over-strength	Q'
Over-strength factor enhances actual strength beyond the minimum required for seismic forces	R
Response in terms of shear force, lateral displacement, or overturning moment	S
Period of vibration for a structure	T
Characteristic period of the spectral plateau	T_a
Ending period of the spectral plateau	T_b

2.1.9.5 EUROPEAN SEISMIC CODE SYMBOLOGY

Symbol description	*Symbol*
Length of the tension shift	a_l
Width of the confined compression zone	b_0
Width of the critical region of the column	b_{oc}
Effective depth of a beam or slab	d
Diameter of transverse (hoop) reinforcement	d_{bw}
Elastic displacement	d_e
Interstorey drift	$d_{r,i}$
Displacement under reduced seismic action	d_s
Static eccentricities along X and Y axes per storey	e_0
Critical height	h_{cr}
Torsional and lateral stiffnesses of the i-th storey	K
Factor reflecting the prevailing failure mode in wall systems	k_w
Radius of gyration of the floor mass	l_s
Resistance of beams	$M_{Rb,i}$
Resistance for beams of columns	$M_{Rc,i}$
End moments in beams for capacity design in seismic conditions	$M_{i,d}$
Total gravity load at and above the storey ii	$P_{tot,i}$
Behaviour factor	q
Behaviour factor basic value for systems regular in elevation	q_0
Torsional radii in X and Y directions	r_x, r_y
Soil factor for seismic design	S
Corner periods for the seismic response spectrum	T_C, T_D
Slenderness ratio, $Lmax/Lmin$Lmax/Lmin	λ
Normalized axial force	v_d
Interstorey drift sensitivity coefficient	θ_i

3 Comparative analysis of the impact of design and detailing provisions for RC moment resisting frames under the first- and second-generation of Eurocode 8

Helder Maranhão, Humberto Varum, José Melo, and António A. Correia, "Comparative analysis of the impact of design and detailing provisions for RC moment resisting frames under the first- and second-generation of Eurocode 8", Engineering Structures, vol. 306, p. 117809, May 2024, doi: 10.1016/j.engstruct.2024.117809

3.1 ABSTRACT

Eurocode 8 is undergoing a thorough revision process encompassing a significant number of changes. This work presents and discusses the novel ductility classes, drift limits for damage limitation, local ductility conditions, and corresponding detailing prescriptions. It is paramount to analyse and contrast these modifications with the current provisions of Eurocode 8.

To assess the impact of such revisions, an extensive examination was conducted on ten symmetrical moment-resisting frame (MRF) buildings. These buildings are regular in plan and elevation and are not torsionally flexible. The analysis was carried out following both the current and the second-generation Eurocode 8 provisions, with the results being compared to evaluate the effectiveness of the new design and detailing provisions.

The study outcomes revealed that the DCM, DC2, and DC3 structures demonstrated different trends in their detailing demands for local ductility and capacity design. Differences in reinforcement quantities were observed between DCM and DC2 and DC2 and DC3 for 5-storey and 10-storey buildings. The design approach focused on individual member detailing requirements to optimize design while maintaining the overall concrete quantities. These insights offer valuable economic considerations for various structural configurations under strong seismic actions.

3.2 INTRODUCTION

In recent decades, structural engineering has seen significant advancements in research and development of design practices, including theoretical, experimental, and numerical investigations by researchers working with practitioners' feedback. This has led to the development new construction materials and structural solutions and the evolution of engineering professionals' design methodologies.

Additionally, the study of the damage caused by recent seismic events to existing structures is critical in acquiring knowledge about their behaviour and identifying potential vulnerabilities, which, in turn, helps develop strategies and methodologies for strengthening existing structures and designing new ones.

This continuous development has led to the revision of the design codes, particularly Eurocode 8, and the need to validate the values and limits of different parameters, performance requirements and verification rules in the sections dedicated to reinforced concrete (RC) building design. Developing seismic design criteria based on performance requires a substantiated description of structures' probable response and behaviour under seismic excitations they may be subjected to during their lifespan, necessitating further advancements in estimating their expected behaviour.

Modern design procedures aim to produce a structural system with the required stiffness, strength, and deformation capacity to resist seismic events with acceptable performance. It thus requires an appropriately configured, sized, and detailed structural system to meet a building performance objective under earthquake loading.

Booth[1] states that the upcoming generation of Eurocodes aims to align national practices more closely, seeking to minimise variations in values and procedures that allow for national discretion, as outlined in each country's National Application Document. While this approach may adequately address the requirements for most Eurocodes, recent advancements in earthquake engineering call for a more comprehensive and tailored approach within Eurocode 8.

The structural design process involves two stages. The first stage comprises the analysis for design actions, which includes defining the interstorey drift limitation, determining structural member cross-sections' dimensions, and calculating reinforcement quantities for static and seismic actions to verify limit states. The analysis also considers second-order effects, global mechanism control checks (such as the capacity design procedure in flexure for members and beam-column joints), and capacity design in shear. The second stage involves detailing local ductility, including verifying the minimum and maximum longitudinal reinforcement ratio, hoop spacing, and anchoring provisions.

The current Eurocode 8, EN 1998-1:2004[2], and the second generation of Eurocode 8, prEN 1998-1-2:2023[3] and FprEN 1998-1-1:2024[4], together with EN 1992-1-1:2004[5], specify how structural analysis and design is to be done for new RC buildings, including the required stiffness, strength and ductility models. These design codes specify how to calculate member design strengths and detailing requirements to achieve a building's safe and economic performance during an earthquake. These specifications are defined in the Eurocodes under the design provisions.

To evaluate the effectiveness of the new design and detailing provisions in the second-generation Eurocode 8 compared to the existing version, a comparative design and analysis of ten symmetrical moment-resisting frame (MRF) buildings is conducted herein. These buildings are regular in plan and elevation, not torsionally flexible, and have varying height configurations and seismic demands for DCM, DC2, and DC3 ductility classes.

According to EN 1998-1:2004[2], the Ductility Class Medium (DCM) refers to a specific classification within the seismic design of structures where moderate ductility demands are expected for a given seismicity level. It represents an intermediate level of design requirements and is typically applied to structures located in regions with a moderate seismic hazard.

According to Plumier[6], within the framework of concrete building design as outlined in FprEN 1998-1-1:2024[4], DC2 is a classification that takes into consideration specific requirements: local overstrength capacity, local deformation capacity, and local energy dissipation capacity. Additionally, DC3 focuses also on the structure's ability to establish a global plastic mechanism at the Significant Damage (SD) limit state.

Additionally, Plumier[6] states that DC2 falls between the previous categories of DCL and DCM, aiming to achieve a more cost-effective design approach.

In the context of this study, the design calculations for DCM structures were performed following the guidelines outlined in EN 1998-1:2004[2]. On the other hand, the design

calculations for DC2 and DC3 structures were conducted following the provisions specified in prEN 1998-1-2:2023[3] and FprEN 1998-1-1:2024[4], respectively.

In the present work, the design method adopted is the force-based approach. The force-based approach includes the response spectrum method, i.e. a linear analysis incorporating the effects of overstrength and non-linear response through the behaviour factor (q). According to Elnashai[7], the behaviour factor may be defined as the minimum reduction coefficient corresponding to a specific level of ductility obtained from inelastic constant ductility spectra and elastic spectra at a given period (T).

3.3 OBJECT OF RESEARCH AND ANALYSIS

To conduct this study, a set of reinforced concrete moment-resistant frame (MRF) structures with different configurations was selected as the subject of analysis. These configurations were defined based on non-variable characteristics and parameters common to all buildings and specific variables for each building, as outlined in Table 3-1.

Table 3-1: Characteristics/parameters of the structures analysed

Variable	Non-variable
Buildings height Seismicity Ductility classes Inertial masses	Structural system (frames) Interstorey height Span length Materials Vertical loads

Ten structure models were built up, assumed symmetrical, and regular in plan and elevation as prescribed in EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. These structures, listed in Table 3-2, were assigned to three different ductility classes (DCM, DC2, and DC3) based on the number of storeys and the maximum response spectral acceleration at the constant acceleration range (S_α). In the present study,(S_α) is defined as being ($2{,}5 \cdot a_g \cdot S \cdot \eta$), as per EN 1998-1:2004[2].

Table 3-2: Adopted nomenclature for the structures

Number of storeys	S_α m/s²	Ductility class	Structure label
5	5,0	DCM	DCM.50.5
		DC2	DC2.50.5
		DC3	DC3.50.5
	7,5	DCM	DCM.75.5
		DC3	DC3.75.5
10	5,0	DCM	DCM.50.10
		DC2	DC2.50.10
		DC3	DC3.50.10
	7,5	DCM	DCM.75.10
		DC3	DC3.75.10

3.3.1 BUILDINGS GEOMETRY AND STRUCTURAL SYSTEM

The structures in this study consist of two building configurations: a five-storey building, shown in Figure 3-1(a) and a ten-storey building, shown in Figure 3-1 (b).

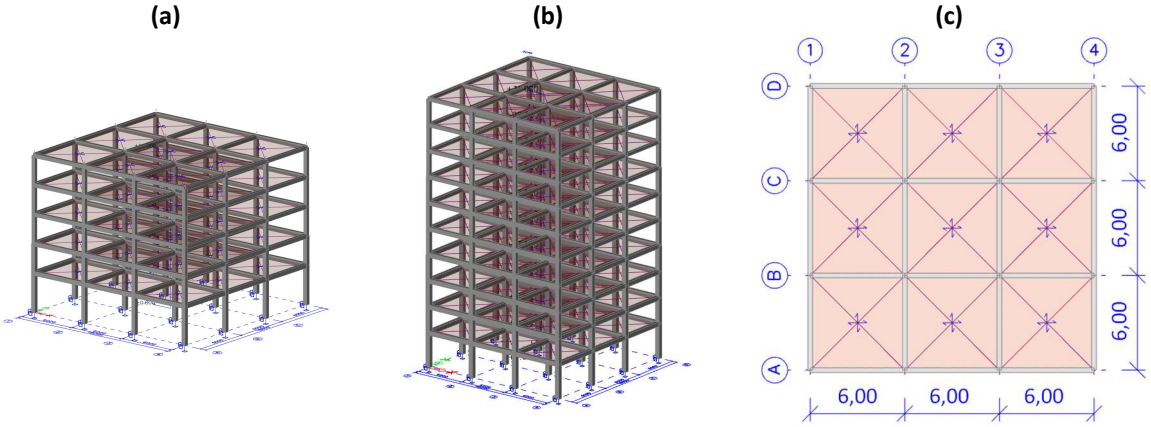


Figure 3-1: Frame building system configuration: (a) 5 storeys, (b) 10 storeys and (c) plan view

The structural system for the buildings consists of a series of frames with an equal spacing of 6,0 m, typical of office buildings, as illustrated in Figure 3-1 (c). The floor-to-floor height is 3,0 m for all storeys, except for the ground storey, which is 4,0 m. The solid slab thickness is 15 cm at each storey.

3.3.2 MATERIALS

The following materials are considered for all primary seismic members (beams and columns) of the structures under analysis:

- Concrete C30/37
- Reinforcement steel B500

The concrete properties are defined per EN 1992-1-1:2004[5] and EN 206-1:2007[8] standards, and its mechanical properties are shown in Table 3-3.

Table 3-3: Concrete mechanical properties

Mechanical properties of concrete C30/37		
Characteristic compressive cylinder strength of concrete	f_{ck} (MPa)	30
Mean value of concrete cylinder compressive strength	f_{cm} (MPa)	38
The design value of concrete compressive strength	f_{cd} (MPa)	20
Characteristic axial tensile strength of concrete	$f_{ctk,0,95}$ (MPa)	2,0
Mean value of axial tensile strength of concrete	f_{ctm} (MPa)	2,9
Design value of tensile strength	f_{ctd} (MPa)	1,3
Elasticity modulus	E_{cm} (GPa)	33
Ultimate compressive strain in the concrete	$\varepsilon_{cu,1}$ (‰)	3,5

The nominal concrete cover cnom is calculated by adding to the minimum cover (c_{min}) the allowance in design for deviation (Δc_{dev}) according to EN 1992-1-1:2004[5]. Assuming Exposure Class XC1, related to environmental conditions following EN 206-1:2007[8] as specified in EN 1992-1-1:2004[5], the assumed nominal cover (c_{nom}) is 30 mm.

The steel reinforcement utilised in this study is classified as B500, and its mechanical properties are characterised according to EN 10080:2005[9], as presented in Table 3-4.

Table 3-4: Reinforcement steel mechanical properties

Reinforcement steel B500		
Characteristic yield strength of reinforcement	f_{yk} (MPa)	500
Mean strength of reinforcement	f_{ym} (MPa)	500
Design yield strength of reinforcement	f_{yd} (MPa)	434,8
Elasticity modulus	E_s (GPa)	200
Yielding strain	ε_{sy} (‰)	2,5

3.4 DESIGN CRITERIA OPTIONS

A range of design options were considered for the analysis. The following subsections outline the options and criteria for describing and comparing earthquake-resistant structural performance and safety requirements.

3.4.1 DESIGN AND MODELLING OPTIONS

The structures are assumed to be regular in plan and elevation. So, no reduction of the behaviour factor (q) was adopted. The specifications in EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] were followed in the design of structures, although the following exceptions were considered:

- The response spectra adopted were based on EN 1998-1:2004[2] alone.
- The shear design of beams and columns followed the approach according to the current EN 1992-1-1:2004[5].

In the design of structural members, the following options were considered for simplicity:

- The minimum dimensions of the structural members concerning the design forces were obtained without adopting typical rules of common practice, such as assuming constant sections in columns along all their heights.
- A non-increasing rule of the columns' cross-section dimensions in height was imposed.
- No structural infills were considered in the analysis.

To evaluate the deformations and internal forces of the structures, a Finite Element (FE) model was meticulously constructed using Scia Engineer software [10]. The models were designed to meet all Eurocode 8 modelling requirements, and their fundamental characteristics are listed in Table 3-5.

Table 3-5: Modelling criteria

Modelling Aspects	Modelling Criteria
Earthquake demand direction	Two horizontal orthogonal directions, X and Y
Global torsional effects	Yes
Frame modelling	Three-dimensional (3D)
Slabs	As a rigid diaphragm and non-contributing to the flexural stiffness of the beams
Columns and beams modelling position	Central axis
Columns and beams type	Linear elastic bar elements (Bernoulli formulation)
Joints stiffness modelling	No
Cracked stiffness of the concrete	$E{\cdot}A/2; E{\cdot}I/2$; $G{\cdot}J/2$ As proposed in EN 1998-1:2004 and FprEN 1998-1-1:2024
Masses	(a) Masses of the slabs, other permanent loads and live loads with the quasi-permanent value were assumed uniformly distributed in the floors
	(b) masses of the frame elements (beams and columns) were considered automatically by the software (distributed along the axis of the elements)
Bending moments redistribution	No redistribution has been made.
Beam-to-column connection	concentric
Accidental eccentricities	Taken into account
Second-order effects P-Δ	Taken into account, calculated by the FE software
Infills	Stiffness and strengths were not considered. Their weight was included in the vertical self-weight and masses
Support at the foundation level	Restrained in all directions

To provide a visual representation of the finite element (FE) model used in the analysis of the 5 and 10-storey buildings, a schematic structural model is presented in Figure 3-2.

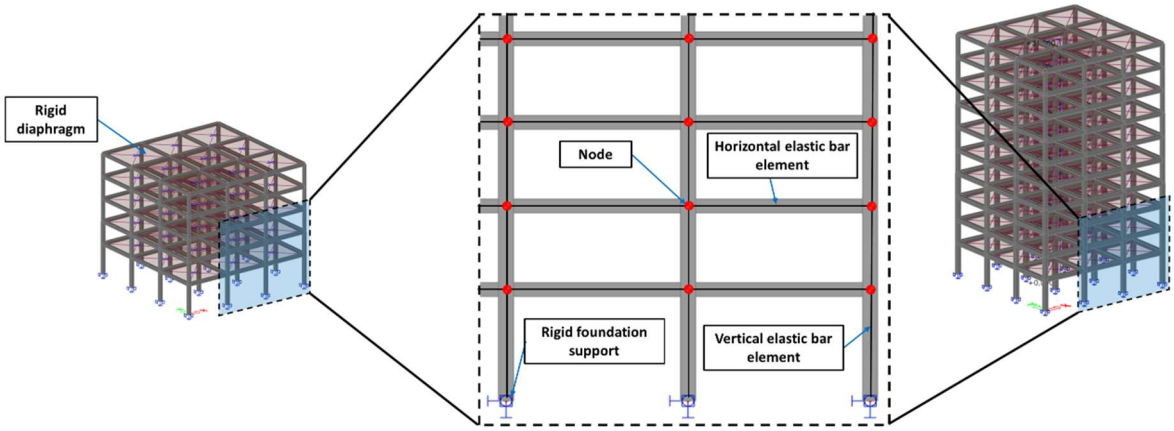


Figure 3-2: Frame building

3.4.2 SEISMIC ACTION

The capacity of structural systems to withstand seismic actions in the non-linear range often allows for their design to be based on seismic forces smaller than those required for a linear elastic response. To avoid the need for explicit inelastic structural analysis during the design phase, the structure's ability to dissipate energy through the ductile behaviour of its elements and other mechanisms is considered by performing an elastic analysis based on a reduced response spectrum concerning the elastic one. This reduction is achieved by introducing the behaviour factor (q), whose definition depends on the type of structure and its regularity in plan and elevation.

3.4.2.1 REGULARITY IN PLAN AND ELEVATION

The plan's regularity criteria were verified according to EN 1998-1:2004[2]. The slenderness of the building corresponds to λ=1,0 since the building is square in the horizontal plane and, therefore, (λ) is smaller than 4,0. The ratio between the torsional radius and the radius of gyration of the floor mass at each storey level in both horizontal directions (r_i/l_s) ranges between 1,18 and 1,25. Since this ratio surpasses 1,0, the corresponding criterion is satisfied. Additionally, the buildings possess double symmetry, ensuring that the centre of gravity aligns with the centre of rigidity. This results in an eccentricity (e_0) of zero in the orthogonal directions. Consequently, the specified criterion of ($e_0 \leq 0{,}3 \cdot r_i$)is also fulfilled. Since these three conditions are met, the buildings are categorised as regular in plan in both directions.

The buildings also meet the qualitative criteria for regular elevation, according to EN 1998-1:2004[2]. The prEN 1998-1-1:2023[3] refers to a quantitative criterion preconizing that "both the lateral stiffness and the mass of the individual storeys remain constant or decrease gradually by no more than 20% relative to the storey below, without abrupt changes, from the base to at least one storey below the top storey". Since interstorey stiffness and mass of the individual storeys decrease gradually by no more than 10%, the buildings are classified as regular in elevation.

To categorize a building as having "torsional flexibility", the verification involves comparing two uncoupled vibration frequencies: (ω_θ/ω_x) and (ω_θ/ω_y). Here, (ω_x) and (ω_y) denote the system's uncoupled angular natural frequencies in the x and y directions while ωθ representing the uncoupled circular natural frequency of torsional vibration. If the ratios (ω_θ/ω_x) and (ω_θ/ω_y) exceed 1,0, the response primarily exhibits translational characteristics, classifying the structure as non-torsionally flexible. Conversely, if (ω_θ/ω_x)and (ω_θ/ω_y) is below 1, the response is predominantly influenced by torsional behaviour. In the current study, the angular frequency ratios (ω_θ/ω_x)and (ω_θ/ω_y) exceed 1,0, thus categorizing non-torsionally flexible structures.

3.4.2.2 BEHAVIOUR FACTOR APPLIED IN THE ANALYSIS

To build up the design spectra for elastic analysis, and regarding the structural systems in question, the behaviour factor q and its components are defined per EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] for structures regular in plan and elevation.

The present work considers the analysis of the DCM ductility class, which is preconised in EN 1998-1:2004[2]. For the structures of interest for the current work, i.e., multistorey and multi-bay regular frame structures, the behaviour factor adopted is the basic value of the behaviour factor, defined by expression (3-1):

$$q_0 = 3{,}0 \cdot \frac{\alpha_u}{\alpha_1} \tag{3-1}$$

Where (α_1) is the value by which the horizontal seismic design action is multiplied first to reach the flexural resistance of any structure member. In contrast, all other design actions remain constant, and (α_u) is the value by which the horizontal seismic design action is multiplied to form plastic hinges in several sections sufficient to develop overall structural instability. In contrast, all other design actions remain constant. For multistorey, multi-bay frames, the ratio (α_u/α_1) takes the value of 1,3; therefore, q_0=3,9. The upper limit value of the behaviour factor (q)is expressed as depicted in expression (3-2):

$$q = q_0 \cdot k_w \geq 1{,}5 \tag{3-2}$$

Where (k_w) is the factor reflecting the prevailing failure mode in structural systems with walls, the parameter(k_w)takes the value of 1,0 for frame systems. Considering those above, the adopted behaviour factor (q) assumes a value of 3,9.

The new generation of Eurocode 8, prEN 1998-1-2:2023[3], introduces a behaviour factor composed of the product of the partial behaviour factor components q_S, q_R, and q_D. The parameter (q_R)is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures, (q_S) is the behaviour factor component accounting for overstrength due to all other sources, and (q_D) is the behaviour factor component accounting for the deformation and energy dissipation capacity.

Regarding the prEN 1998-1-2:2023[3], to build up the design spectra for elastic analysis, and regarding the regular structural systems under analysis herein, the behaviour factor and its components for multistorey, multi-bay moment resisting frames are presented in Table 3-6.

Table 3-6: Default values of the behaviour factors for MRF systems regular in plan and in elevation – prEN 1998-1-2[3]

Structure	q_R	q_S	q_D	$q=q_R \cdot q_S \cdot q_D$
DC2.50.5	1,3	1,5	1,3	2,5
DC3.50.5			2,0	3,9
DC3.75.5			2,0	3,9
DC2.50.10			1,3	2,5
DC3.50.10			2,0	3,9
DC3.75.10			2,0	3,9

3.4.2.3 RESPONSE SPECTRA

The seismic action is defined according to EN 1998-1:2004[2]. For structures of ordinary importance, the recommendation in EN 1998-1:2004[2] is for seismic action for (local) collapse prevention with a 10% exceedance probability in 50 years (i.e., a mean return period of 475 years). For the present study, the type 1 elastic and the design spectra for elastic analysis are drawn up according to EN 1998-1:2004[2] for ground type B and an importance class II with an importance factor (γ_I=1,0). Table 7 shows the values of the parameters describing the elastic response spectra (ξ=5%) for the structures under analysis.

Table 3-7 refers to the maximum response spectral acceleration at the constant acceleration range (S_α) and the different behaviour factor (q) values. For reliability differentiation, EN 1990:2002[11] establishes consequence classes (CC), which quantitatively define the consequences of failure or malfunction. A Consequence Class CC2 is assumed to have medium consequences for the loss of human life, with considerable impacts on economic, social, and environmental aspects. For the consequence class CC2, the design spectrum does not require amplification related to reliability differentiation.

Table 3-7: Properties of the horizontal response spectra

Structure	Ductility Class	S_α (m/s²)	a_g (m/s²)	q
DCM.50.5 DCM.50.10	DCM	5,00	1,67	3,9
DCM.75.5 DCM.75.10		7,50	2,50	
DC2.50.5 DC2.50.10	DC2	5,00	1,67	2,5
DC3.50.5 DC3.50.10	DC3	5,00	1,67	3,9
DC3.75.5 DC3.75.10		7,50	2,50	

3.4.3 STRUCTURAL SAFETY VERIFICATIONS TO LIMIT STATES

Deformation-related criteria are prescribed for all types of buildings in EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. However, in the case of tall (RC) moment-resistant frames, their flexibility often governs the design, and they are subjected to additional criteria. Eurocode 8 sets out safety verification criteria for the Ultimate Limit State (ULS) according to EN 1998-1:2004[2] and for the Significant Damage Limit State (SD) according to prEN 1998-1-2:2023[3]. These safety criteria include the control of second-order effects, interstorey drift (ISD) limitation, and control of the global plastic mechanism (resulting from capacity design in flexure). It should be noted that EN 1998-1:2004[2] considers only the serviceability condition as a Damage-Limitation Limit State (DL) for controlling ISD, whereas prEN 1998-1-2:2023[3] considers both DL and SD limit states.

3.4.3.1 CONTROL OF SECOND-ORDER EFFECTS

The second-order effects (P-Δ), which are associated with ULS effects, are specified through an interstorey drift sensitivity coefficient (θ) as shown in Equation (3-3) based on EN 1998-1:2004[2]:

$$\theta = \frac{P_{tot}}{V_{tot}} \cdot \frac{d_r}{h} \tag{3-3}$$

According to prEN 1998-1-2:2023[3], second-order effects (P-Δ) are associated with SD limit states and are defined by Equation (3-4):

$$\theta = \frac{P_{tot}}{q_R \cdot q_S \cdot V_{tot}} \cdot \frac{d_{r,SD}}{h_s} \tag{3-4}$$

Where (P_{tot}) and (V_{tot}) are the total cumulative gravity load and seismic shear, respectively, at the storey under consideration; (h_s) and h are the storey height.

EN 1998-1[2] and prEN 1998-1-2[3] both specify that when the interstorey drift sensitivity coefficient (θ) has a value between 0,1 and 0,2, an approximate adjustment for (*P-Δ*) effects can be made by increasing the seismic forces used in the analysis by a factor of 1/(1-θ). On the other hand, when (θ) it is between 0,2 and 0,3, a second-order analysis is necessary.

3.4.3.2 LIMITATION OF THE INTERSTOREY DRIFT ISD

In a current design with a force-based approach, the interstorey drifts are the product of elastic interstorey drift from analysis and the behaviour factor q. According to EN 1998-1:2004[2], the interstorey drift verification is included regarding the serviceability condition as damage-limitation (DL). The prEN 1998-1-2:2023[3] considers two limit states for the verification: the damage limitation (DL) and the significant damage limit state (SD).

Since the seismic analysis in a force-based approach is linear-elastic and based on the design response spectrum, the elastic spectrum with 5% damping divided by the behaviour factor q is used. The values of the displacements (d_s) to be used in the verification are calculated from the analysis (d_e) and multiplied by the behaviour factor (q), as shown in Equation (3-5):

$$d_s = q \cdot d_e \tag{3-5}$$

The design interstorey drift (d_r) is calculated as the difference of the average lateral displacements' ds induced by the reduced seismic action at the top and bottom of the storey under consideration as depicted in Equation (3-6):

$$d_{r,i,} = d_{s,i+1} - d_{s,i} \tag{3-6}$$

EN 1998-1:2004[2] defines the limit of interstorey drift ratio (IDR) according to Equation (3-7):

$$\frac{d_r}{h} = \frac{K}{\nu} \tag{3-7}$$

Where (K) is the coefficient that accounts for the sensitivity of ancillary elements to interstorey drift, h is the interstorey height, and (ν) is the reduction factor, which considers the lower return period of the seismic action associated with the damage limitation requirement. Table 3-8 presents the IDR limits according to EN 1998-1:2004[2] and assuming ν=0,5.

Table 3-8: Limits of IDR in % in buildings with masonry infills - EN 1998-1:2004[2]

Type	IDR limit (K/ν) at DL [%]
Buildings having non-structural elements of brittle materials attached to the structure	1,00
Buildings having ductile non-structural elements:	1,50
Buildings having non-structural elements fixed in a way so as not to interfere with structural deformations or without non-structural elements	2,00

The prEN 1998-1-2:2023[3] presents a similar formulation, where (λ_{NS}) is the coefficient accounting for the sensitivity of ancillary elements to interstorey drift, the subscript (LS) refers to limit state, and (h_s) has the same meaning as (h).

$$\frac{d_{r,LS}}{h_s} = \lambda_{NS} \tag{3-8}$$

The IDR limits as per prEN 1998-1-2:2023[3] are presented in Table 3-9.

Table 3-9: Limits of IDR in buildings with masonry infills – prEN 1998-1-2:2023[3]

Type	IDR limit (λ_{NS}) at DL [%]	IDR limit (λ_{NS}) at SD [%]
Buildings having ancillary elements of unreinforced masonry units of Group 4 attached to the structure	0,25	0,90
Buildings having ancillary elements of brittle materials attached to the structure, in particular unreinforced masonry with clay units of Groups 1, 2 or 3 with a thickness greater than 200mm and the normalised mean compressive strength $f_b \geq 3$MPa,	0,45	1,40
Buildings having ductile ancillary elements	0,75	2,00

3.4.3.3 GLOBAL MECHANISM CONTROL – CAPACITY DESIGN IN FLEXURE

The design rules for concrete moment-resisting frames according to EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] aim to prevent a soft-storey mechanism from developing by ensuring that flexural plastic hinges develop in the beams rather than in the columns. This results in the development of a beam-sway mechanism and prevents the top and bottom ends of all the columns in a storey from undergoing unrestrained flexural rotations without a notable increase of their bending moments beyond the corresponding moment resistance (M_{Rc}).

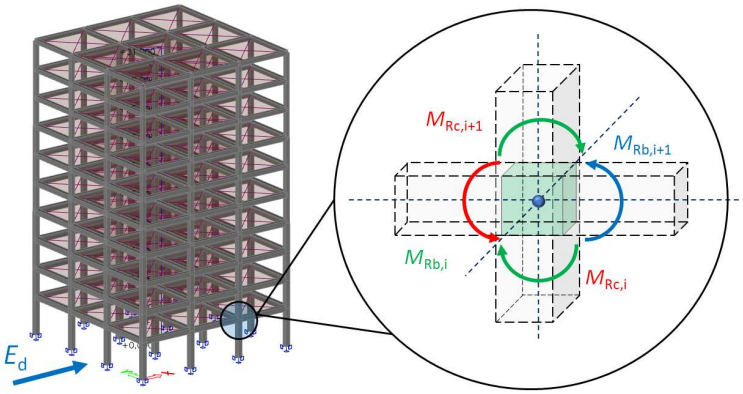


Figure 3-3: Vectorial system of the resistant moments for beams and columns at a node

To this end, within any vertical plane in which a soft-storey is to be prevented, the two columns framing into a beam-column joint from above and below are designed to be jointly stronger by 30% than the beams connected to the same joint from any side as depicted in expression (3-9):

$$\sum_{i=1}^{j} M_{Rc,i} \geq 1{,}3 \cdot \sum_{i=1}^{j} M_{Rb,i} \tag{3-9}$$

The sum of the design values of the flexural strength, in terms of moments of the columns framing the joint, is represented by ($\Sigma M_{Rc,i}$) while ($\Sigma M_{Rb,i}$) represents the sum of the design values of the moments of resistance of the beams framing the joint. Figure 3 illustrates the vectorial system of ($M_{Rc,i}$) and ($M_{Rb,i}$) at a node. The column's bending moment strength ($M_{Rc,i}$) is intricately linked to the magnitude of the axial force (N_{Ed}). By considering the seismic effects, the axial force (N_{Ed}) is calculated, considering the dynamic forces imposed on the column in the seismic design situation.

3.4.4 VERTICAL LOADS

The vertical loads specified in Table 3-10 are, per EN 1990[11] and EN 1991-1-1[12], for office buildings and are considered in all buildings under analysis.

Table 3-10: Vertical loads

Load type	Floor	Load	Value
Structure self-weight $G_{k,1}$	All floors	Primary seismic members (columns and beams): generated by the software	γ=25 kN/m^3
		Slabs: solid RC 15 cm thick	
Other permanent loads $G_{k,2}$	1st to (n-1)th	External walls and partitions with 3,0 kN/m (per unit of wall length). It is assumed a uniformly distributed load according to 6.3.1.2(8) of EN 1991-1-1:2002[12]	1,2 kN/m^2
		Finishings	1,3 kN/m^2
	n^{th}(roof)	Finishings	1,0 kN/m^2
Live loads Q_k	1st to (n-1)th	Building for use as civil dwellings, falling into usage category A, according to the classification in 6.3.1.2 of EN 1991-1-1:2002[12]	2,0 kN/m^2
	n^{th}(roof)	Roof category H – Roofs not accessible except for normal maintenance and repair, according to the classification defined in 6.3.4 of EN 1991-1-1:2002[12]	1,0 kN/m^2

3.4.5 LOAD COMBINATIONS

The structures used in the present analysis were designed for:

a) Ultimate Limit states (ULS) – Fundamental combinations

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j}" + "\gamma_{Q,1} \cdot Q_{k,1} \tag{3-10}$$

Where ($G_{k,j}$) is the partial factor for permanent actions, accounting for model uncertainties and dimensional variations for permanent action j; ($G_{k,j}$) is the characteristic value of permanent action j; ($\gamma_{Q,1}$) is the partial factor for variable actions.

b) Ultimate Limit States (ULS) – Seismic combinations

$$\sum_{j\geq 1} G_{k,j}" + "A_{Ed}" + "\sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{3-11}$$

The parameter ($G_{k,j}$) is the characteristic value of permanent action j, (A_{Ed}) is the design value of seismic action, and (Q_k) is the characteristic value of the variable action i, with ($\psi_{2,i}$) the combination factor for variable actions. In the present work, A_{Ed} is denominated as (E_d).

It should be noted that in the present work, wind, snow and thermal actions were not considered. The combinations of actions for the seismic design situation are defined in Table 3-11. The seismic combinations also consider the accidental eccentricities producing accidental torsional effects. According to Fardis et al.[13] three-dimensional structural models incorporate the interplay between translational and torsional vibrations. In the present work, horizontal ground motion does not induce a torsional response in buildings that exhibit full plan-wise symmetry along both axes. According to Fardis et al.[13], the seismic analysis fails to account for potential deviations in stiffness, mass, and strength distributions from their intended values for plan-wise symmetry. Additionally, certain torsional ground motion components remain

unaddressed in seismic assessments, leading to potential torsional responses even in fully symmetric structures.

To mitigate these uncertainties and ensure adequate torsional resistance and stiffness, as well as to mitigate the consequences of unexpected torsional responses, seismic codes such as EN 1998-1-1:2004[2] introduce the concept of accidental eccentricity and prEN 1998-1-2:2023[3] refers to it as minimum eccentricity in buildings. Accidental torsional effects may arise when masses are displaced from their intended positions by a distance equivalent to the accidental eccentricity, denoted as (e_{ai}). In EN 1998-1-1:2004[2] and prEN 1998-1-2:2023[3], this value is set at 5% of the floor dimension in the storey *i*, as depicted in expression (3-12):

$$e_{ai} = 0{,}05 \cdot L_i \tag{3-12}$$

Where (L_i) is the floor dimension perpendicular to the direction of the seismic action.

In the present analysis, the accidental eccentricity is addressed in only one direction since the buildings are plan-wise symmetric in the orthogonal directions.

Table 3-11: Load combinations

Load Combination	Nature	$\gamma_{G,1}$	$\gamma_{G,2}$	$\gamma_{Q,1}$	$\psi_{2,1}$	$E_{Ed,x}$	$E_{Ed,y}$	$A_{cce,i,x}$	$A_{cce,i,y}$
Combination 1	ULS – Fundamental	1,35	1,35	1,50	-	-	-	-	-
Combination 2	ULS – Seismic	1,00	1,00	1,00	0,30	0,30	1,00	0,30	1,00
Combination 3						-0,30	1,00	-0,30	1,00
Combination 4						0,30	-1,00	0,30	-1,00
Combination 5						-0,30	-1,00	-0,30	-1,00
Combination 6						1,00	0,30	1,00	0,30
Combination 7						1,00	-0,30	1,00	-0,30
Combination 8						-1,00	0,30	-1,00	0,30
Combination 9						-1,00	-0,30	-1,00	-0,30
Combination 10	ULS – Seismic	1,00	1,00	1,00	0,30	0,30	1,00	-0,30	-1,00
Combination 11						-0,30	1,00	0,30	-1,00
Combination 12						0,30	-1,00	-0,30	1,00
Combination 13						-0,30	-1,00	0,30	1,00
Combination 14						1,00	0,30	-1,00	-0,30
Combination 15						1,00	-0,30	-1,00	0,30
Combination 16						-1,00	0,30	1,00	-0,30
Combination 17						-1,00	-0,30	1,00	0,30

The parameter (E_{Edx}) represents the action effects due to the application of the seismic action along the chosen horizontal axis (*x*)of the structure, and (E_{Edy}) represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis (*y*) of the structure as defined in section 4.3.3.5.1(3) of the EN 1998-1:2004[2]. The parameters ($A_{cce,i,x}$) and ($A_{cce,i,y}$) represent the torsional moments resulting from mass eccentricity (e_{ai}) perpendicular to directions (*x*) and (*y*), respectively, at any storey *i*.

These moments exert their influence around the vertical axis of the building, as depicted in expression (3-13):

$$A_{cce,i,x} = F_{Ed,i,x} \cdot e_{ai,y} \tag{3-13}$$

and expression (3-14):

$$A_{cce,i,y} = F_{Ed,i,y} \cdot e_{ai,x} \tag{3-14}$$

Where ($F_{\mathrm{Edx,i}}$) and ($F_{\mathrm{Edy,i}}$) are the horizontal forces acting on the considered storey i in the directions (x) and (y), respectively. The parameters ($e_{\mathrm{ai,x}}$) and ($e_{\mathrm{ai,y}}$) are the accidental eccentricities in the directions x and y, respectively.

3.4.6 BEAM DESIGN AND DETAILING FOR LOCAL DUCTILITY

In the seismic design situation, the design values of bending moments and shear are obtained from the structural analysis as per EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. The bending and shear resistance is calculated following EN 1992-1-1[5] while taking into account the capacity design in shear and the detailing requirements for local ductility as prescribed in EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. Table 12 summarises the design and detailing requirements for the primary beams of ductility classes DCM, DC2, and DC3. The longitudinal reinforcement was calculated using the stress block model, where μ the reduced moment ω is the mechanical percentage of reinforcement. The transverse reinforcement in hoops was designed according to EN 1992-1-1:2004[5].

Table 3-12 shows the transverse reinforcement requirements, divided within the critical region (l_{cr}) and out of the critical region. The critical region of a beam (l_{cr}), in the context of Eurocode 8, refers to the region where the design forces and moments caused by seismic actions are most significant. According to Fardis[14], a "critical region" is defined at the end of a "primary" column or beam framing into a beam or vertical element, respectively, no matter whether the relative magnitude of the moment resistances of the cross-sections around the joint suggests that a plastic hinge at that end is likely to form.

Table 3-12: Rules for design detailing and dimensioning of primary beams

Beams		DCM	DC2	DC3
Critical region length		h_w		
Resistance in bending				
Calculation of longitudinal reinforcement area	A_s	Rectangular stress block; $\mu = \frac{M_{Ed}}{b \cdot d^2 \cdot f_{cd}}$; $\omega = 1 - \sqrt{1 - 2 \cdot \mu}$; $A_s = \omega \cdot b \cdot d \cdot \frac{f_{yd}}{f_{cd}}$		
Resistance in shear				
Capacity design	$V_{i,d}$	$V_{g+\psi_2 q} \pm \frac{\sum M_{Rb,i}}{l_{cl}}$	$V_{g+\psi_2 q} \pm 1{,}1 \frac{\sum M_{Rd,b,i}}{l_{cl}}$	
Maximum shear capacity	$V_{Rd,max}$	As in EN 1992-1-1:2004 – $V_{Rd,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot 0{,}6 \cdot \left[1 - \frac{f_{ck}}{250}\right] \cdot f_{cd}}{cot\theta + tan\theta}$		
Shear resistance inside and outside critical regions	$V_{Rd,s}$	As in EN 1992-1-1:2004 – $V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot cot\theta$		
Detailing for local ductility – Longitudinal bars				
Minimum longitudinal reinforcement ratio	ρ_{min}	$0{,}5 \cdot f_{ctm}/f_{yk}$		
Maximum longitudinal reinforcement ratio	ρ_{max}	$\rho' + \frac{0{,}0018}{\mu_\phi \cdot \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$	$\rho'_l + 0{,}8\%$	$\rho'_l + 0{,}6\%$
Maximum bar diameter crossing interior joints	d_{bL}	$\left[\frac{7{,}5 \cdot f_{ctm}}{\mathbf{1{,}0} \cdot f_{yd}} \cdot \frac{(1 + 0{,}8 \cdot \nu_d)}{1 + 0{,}75 \cdot \frac{\mathbf{2}}{\mathbf{3}} \cdot \frac{\rho'}{\rho_{max}}}\right] \cdot h_c$	$\left[\frac{7{,}5 \cdot f_{ctm}}{\mathbf{1{,}1} \cdot f_{yd}} \cdot \frac{(1 + 0{,}8 \cdot \nu_d)}{1 + 0{,}75 \cdot \frac{\mathbf{1}}{\mathbf{2}} \cdot \frac{\rho'_l}{\rho_{l,max}}}\right] \cdot h_c$	$\left[\frac{7{,}5 \cdot f_{ctm}}{\mathbf{1{,}2} \cdot f_{yd}} \cdot \frac{(1 + 0{,}8 \cdot \nu_d)}{1 + 0{,}75 \cdot \frac{\mathbf{2}}{\mathbf{3}} \cdot \frac{\rho'_l}{\rho_{l,max}}}\right] \cdot h_c$
Maximum bar diameter crossing exterior joints	d_{bL}	$\left[7{,}5 \cdot (1 + 0{,}8 \cdot \nu_d) \cdot \frac{f_{ctm}}{f_{yd}}\right] \cdot h_c$		
Detailing for local ductility – Transverse reinforcement				
Outside critical regions				
Maximum hoop spacing	s	$0{,}75 \cdot d$		
Minimum traversal reinforcement ratio	ρ_w	$0{,}08 \cdot \sqrt{\frac{f_{ck}}{f_{yk}}}$		
In critical regions				
Minimum hoop diameter	d_{bw}	6 mm		
Maximum hoop spacing	s	$\min(h_w/4;\ 24 \cdot d_{bw};\ 225;\ 8 \cdot d_{bL})$	$\min(h/4;\ 30 \cdot d_{bw};\ 12 \cdot d_{bL})$	$\min(h/4;\ 24 \cdot d_{bw};\ 8 \cdot d_{bL})$

3.4.7 COLUMN DESIGN AND DETAILING

According to EN 1998-1:2004[2] and prEN 1998-1-2:2023[3], the design values of bending moments and axial forces for seismic design are obtained from the structural analysis. Compliance with the capacity design requirements for columns when bending at beam/column joints is necessary to ensure the resisting capacity of the columns. Table 3-13 shows the design values of shear forces are determined using the capacity design rule, as prescribed in EN 1998-1:2004[2] and prEN 1998-1-2:2023[3], and the bending and shear resistance calculated per EN 1992-1-1:2004[5] as well, considering the detailing requirements and the value of the axial force from the analysis in the seismic design situation.

Table 3-13: Rules for design detailing and dimensioning of columns

Columns		DCM	DC2	DC3
Critical region length	l_{cr}	max(h_c; l_{cl}/5; 0,45m)		
Maximum admissible normalised axial force	ν_d	0,65	0,65	0,55
Resistance in bending				
Calculation of the longitudinal rebars	A_s	Truly biaxial flexure [N_{Rd}-$M_{Rd,y}$-$M_{Rd,z}$]		
Resistance in shear				
Capacity design	$V_{i,d}$	$V_{i,d} = 1{,}1 \cdot \frac{\sum M_{i,d}}{l_{cl}}$		
Maximum admissible shear capacity	$V_{Rd,max}$	As in EN 1992-1-1:2004 – $V_{Rd,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot 0{,}6 \cdot \left[1-\frac{f_{ck}}{250}\right] \cdot f_{cd}}{cot\theta + tan\theta}$		
Shear resistance inside and outside critical regions	$V_{Rd,s}$	As in EN 1992-1-1:2004 – $V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot cot\theta$		
Detailing for local ductility – Longitudinal bars				
Minimum longitudinal reinforcement ratio	ρ_{min}	1%		
Maximum longitudinal reinforcement ratio	ρ_{max}	4%		
Minimum reinforcement diameter	d_{bL}	8 mm	12mm	12mm
Minimum bars per side		3		
Maximum distance between restrained bars		200 mm		
Maximum distance of the unrestrained bar from the nearest restrained bar		150 mm		
Detailing for local ductility – Transverse bars				
Outside critical regions				
Minimum hoop bar diameter	d_{bw}	max (6mm; d_{bL}/4)		
Maximum hoop spacing	s	20d_{bL}; h_c, b_c; 400		
Within critical regions				
Minimum hoop bar diameter	d_{bw}	max (6, d_{bL}/4)		
Maximum hoop spacing	s	min (b_{oc}/2; 175; 8d_{bL})	min (b_{oc}/2; 200; 9·d_{bL})	min (b_{oc}/2; 175; 8d_{bL})
In critical region at the column base				
Minimum mechanical percentage of reinforcement	ω_{wd}	0,08	0,05	0,08
Confinement integrity verification	$\alpha \cdot \omega_{wd}$	30·μ_ϕ·ν_d·$\varepsilon_{sy,d}$·b_c/b_o-0,035	-	-

3.4.8 CHORD ROTATION

According to prEN 1998-1-2:2023[3], ensuring sufficient ductility and plastic rotation capacity in all critical regions of the beams is necessary. To achieve this, the product of the global displacement ductility factor, calculated as the product of the components of the behaviour factor (q_R) and q_D, and the chord rotation at yield (θ_y) of each member end should not exceed the chord rotation for the Significant Damage (SD) Limit State (θ_{SD}) as defined in expression (3-15):

$$q_D \cdot q_R \cdot \theta_y \leq \theta_{SD} \tag{3-15}$$

To provide a visual representation of the chord rotation in structures, Figure 3-4 depicts the vectorial diagram of the chord rotation at the left (θ_l) and right (θ_r) ends of beams and at the bottom (θ_b) and top (θ_t) ends of columns.

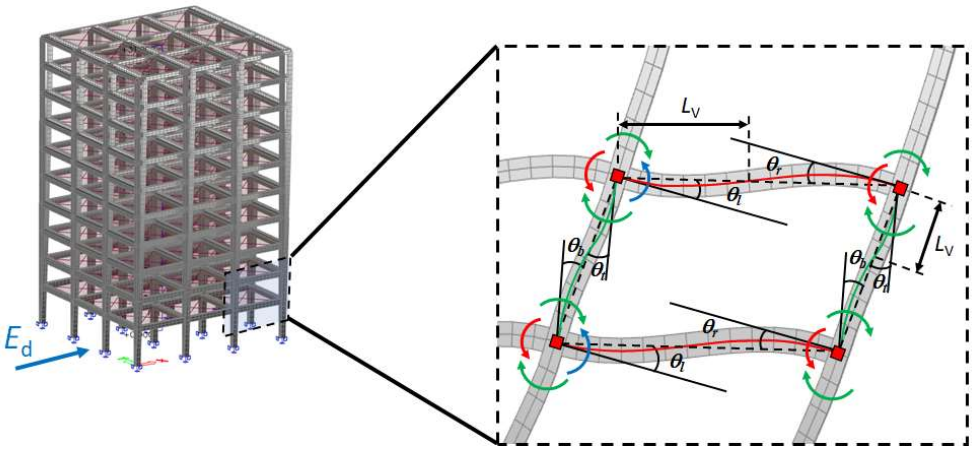


Figure 3-4: Vectorial representation of the chord rotation

The chord rotation of the shear span (L_V=M/V) at yield (θ_y) for rectangular beams and columns may be evaluated according to expression (3-16), as proposed in prEN 1998-1-1[4]:

$$\theta_y = \phi_y \cdot \frac{L_V + a_l}{3} + \frac{\phi_y \cdot d_{bL} \cdot f_y}{8 \cdot \sqrt{f_c}} + 0{,}0019 \cdot \left(1 + \frac{h}{1{,}6 \cdot L_V}\right) \tag{3-16}$$

According to the proposal by Romão et al.[15], the shear span can be defined as 50% of the length of the element. However, the current study calculated the beams' shear span (L_V) for each seismic combination.

The parameter (ϕ_y) is the yield curvature from a cross-sectional analysis of the end section. The tension shift of the bending moment diagram (a_l) is defined in the second generation of Eurocode 2 as being the same expression as in the current EN 1992-1-1:2004[5]. For columns, the shear span (L_V) adopted is 50% of the clear height, as referred by Fardis et al.[13]. The parameters (f_y) and (f_c) are the mean yield and compressive strength values of reinforcement steel and concrete, respectively.

For the verification of SD, the resistance of ductile mechanisms should be taken as given by expression (3-17), as defined in prEN 1998-1-1:2024[4]:

$$\theta_{SD} = \frac{1}{\gamma_{Rd}} \cdot \left(\theta_y + \alpha_{SD,\theta} \cdot \theta_u^{pl}\right) \tag{3-17}$$

The parameter ($\alpha_{SD,\theta}$) is the portion of the plastic part of the ultimate deformation $\theta_u^{pl} = \theta_u - \theta_y$ (computed with mean material properties) that corresponds to the attainment of SD, and γ_{Rd} is a partial factor on resistance at the SD limit state. In the current version of the prEN 1998-1-1:2024[4], the reference value for ($\alpha_{SD,\theta}$) is assumed to be 0,5.

3.5 RESULTS AND DISCUSSION

The following section discusses the results of the analysis of the parameters in section 3.4.

3.5.1 SEISMIC RESPONSE DEMANDS

Modal response spectrum analysis was performed independently for the ground excitation in two horizontal directions. The design spectra were used in both horizontal directions. The CQC rule for combining different modes was used according to EN 1998-1:2004[2]. The CQC rule accurately represents the combined response by explicitly accounting for mode interaction effects. The building's three fundamental modes of vibration are schematically represented in Figure 3-5.

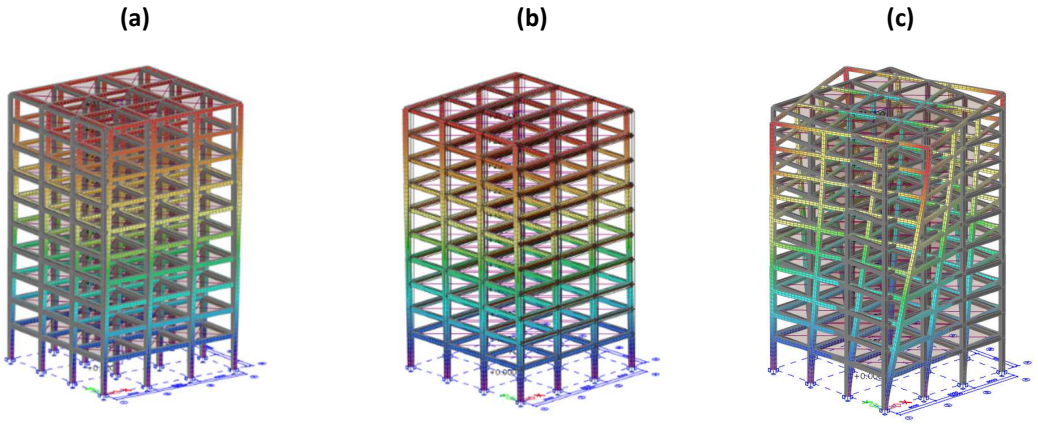


Figure 3-5: Example of modal shapes: (a) First vibration mode T_1, (b) Second vibration mode T_2 and (c) Third vibration mode T_3

The first vibration mode (T_1)is predominantly translational in the X direction (Figure 3-5(a)), and the second mode(T_2) is translational in the Y direction (Figure 3-5(b)). The third mode (T_3) is predominantly torsional (Figure 3-5 (c)). In the modal response spectrum analysis, the sum of the effective modal masses amounts to 98% of the total mass of the structure.

The shear force at the structure's base (V_b) was obtained by modal response spectrum analysis for X and Y directions. The roof displacement ($d_{e,roof}$) was also obtained by modal response spectrum analysis based on the design response spectrum, including torsional effects.

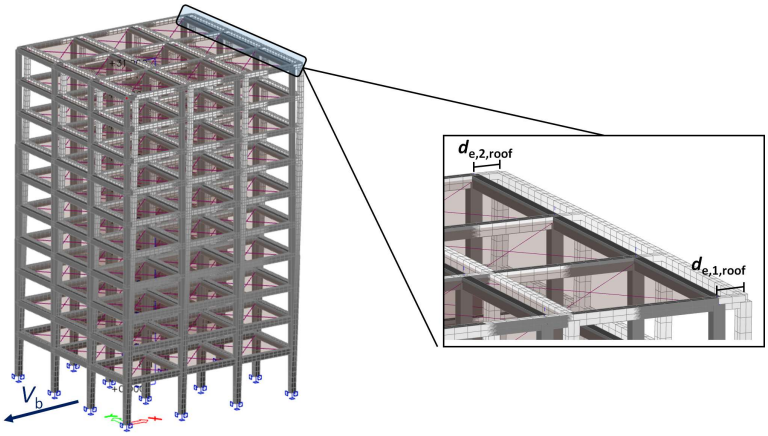


Figure 3-6: Scheme for base shear (V_b) and roof displacement ($d_{e,roof}$)

The roof displacement refers to the maximum displacement observed between the corners of the building. This is determined by comparing the two displacements: ($d_{e,1,roof}$) and ($d_{e,2,roof}$). The larger value between these two displacements is considered the roof displacement, as illustrated in Figure 3-6. The basic modal properties of the building are summarised in Table 3-14.

Table 3-14: Seismic response parameters

Structure	T_1 (s)	T_2 (s)	T_3 (s)	$S_d(T_1)$ (m/s²)	V_b (kN)	d_{eroof} (m)
DCM.50.5	1,49	1,49	1,30	0,43	532,37	0,033
DC2.50.5	1,43	1,43	1,28	0,69	866,51	0,051
DC3.50.5	1,49	1,49	1,30	0,43	532,37	0,033
DCM.75.5	1,03	1,03	0,91	0,93	1236,39	0,034
DC3.75.5	1,02	1,02	0,89	0,93	1250,01	0,033
DCM.50.10	2,14	2,14	1,86	0,33	862,50	0,055
DC2.50.10	2,10	2,10	1,85	0,44	1169,98	0,085
DC3.50.10	2,14	2,10	1,85	0,33	867,01	0,053
DCM.75.10	1,92	1,92	1,70	0,50	1382,10	0,065
DC3.75.10	1,92	1,92	1,70	0,50	1382,10	0,065

From Figure 3-7 and Figure 3-8, several conclusions can be drawn:

a) Buildings designed for DCM and DC3 ductility classes show similar base shear values (V_b) and roof displacement for different building heights and seismic actions.

b) Among the 5-storey buildings, DCM.50.5, DC2.50.5, and DC3.50.5 structures have a similar fundamental period (T_1). However, DC2.50.5 (q=2,5) has 35% higher roof displacement than DCM.50.5 and DC3.50.5 (q=3,9) due to its higher seismic action.

c) DC2. 50.5 (q=2,5) has a base shear value 38% higher than DCM.50.5 and DC3.50.5. (q=3,9).

d) DC2.75.5 and DC3.75.5 have similar base shear and roof displacement since they have the same stiffness and seismic action.

e) The 10-storey buildings, DCM.50.10, DC2.FRM.50.10, and DC3.50.10, have a similar fundamental period (T_1). However, due to higher seismic action, DC2.50.5 (q=2,5) experiences a 36% higher roof displacement than DCM.50.5 and DC3.50.5 (q=3,9)

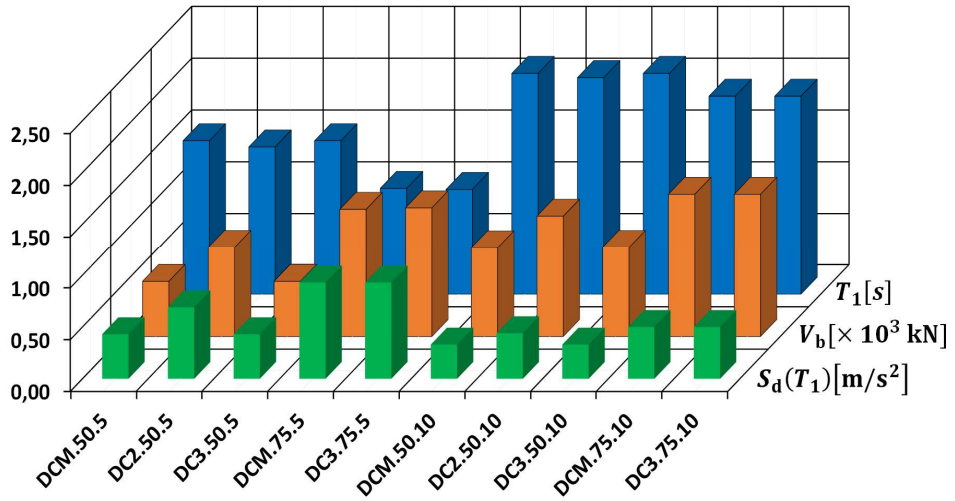


Figure 3-7: Design ground acceleration at fundamental period $S_d(T_1)$, Base shear (V_b) and fundamental period (T_1)

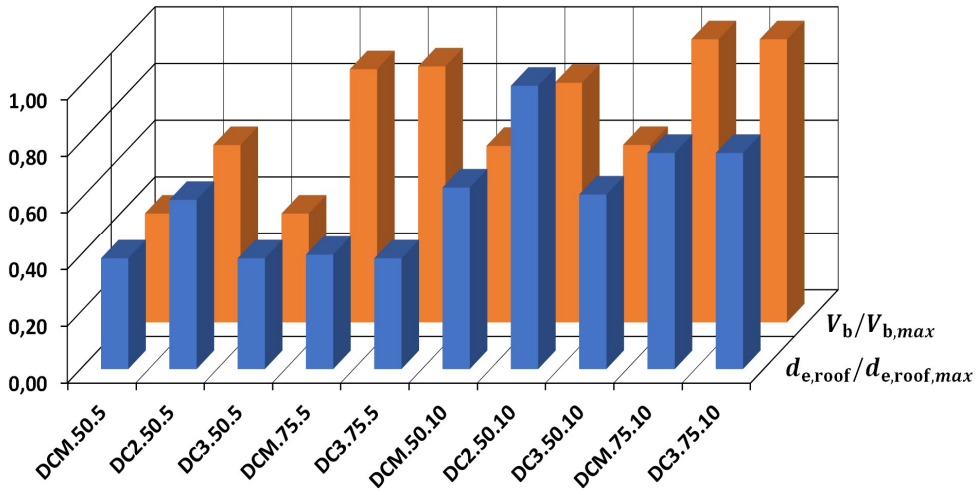


Figure 3-8: Normalised Base shear (V_b/ $V_{b,max}$) and roof displacement ($d_{e,roof}$/ $d_{e,roof,max}$)

DC2.50.10 (q=2.5) has a 26% higher base shear compared to DCM.50.5 and DC3.50.5 (q=3.9), which is less than the expected 35% increase. This difference is due to the influence of higher modes in mid-rise buildings. According to Maniatakis et al.[16], higher modes significantly affect inertial forces, storey accelerations, and shear forces in first-mode dominated RC frames, especially at higher storeys. However, their impact on displacements and interstorey drifts is less significant.

3.5.1.1 INTERSTOREY DRIFT RATIO

Figure 3-9(a) and (b) show the IDR for the 5 and 10-storey buildings, respectively, representing the *X* and *Y* directions. The interstorey drift is the same in both the *X* and *Y* directions, given the plan-wise symmetry nature of the structures.

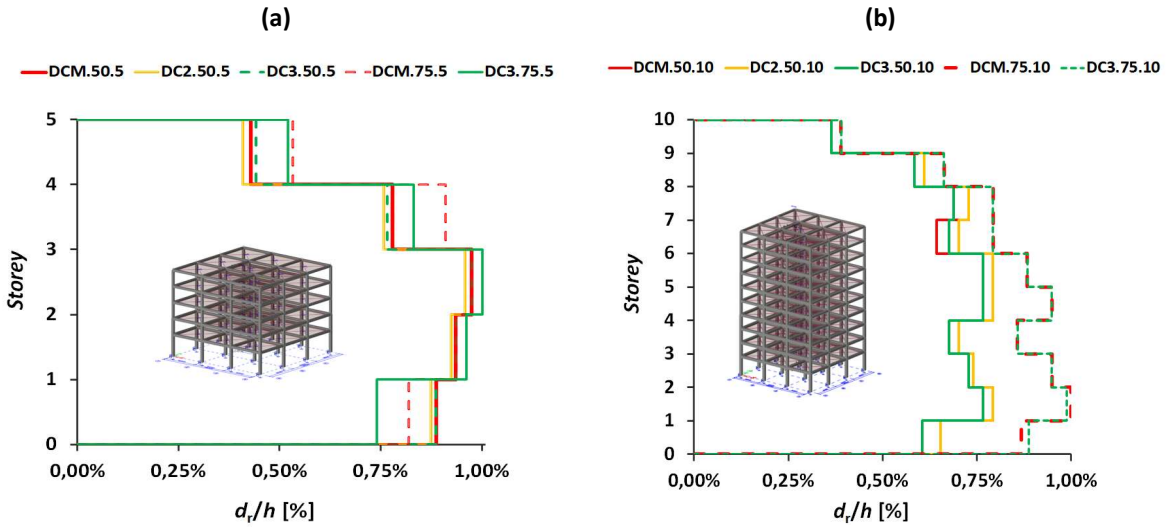


Figure 3-9: Interstorey drift ratio for *X* and *Y* directions: (a) 5-storey MRF buildings and (b) 10-storey MRF buildings

From the analysis of the IDR profiles presented in Figure 3-9 for the X and Y alignments, the following observations can be made:

a) For the 5-storey buildings, the maximum IDR values were observed on the 3rd floor in both directions, while for the 10-storey buildings, the highest IDR was recorded on the 2nd floor.

b) Among all the 5-storey numerical models, DC3.75.5 exhibited the highest IDR values along the X and Y directions, reaching a maximum value of 1,0% on the 3rd floor. DC2.50.5 had the lowest IDR of 0,43% on the 5th floor.

c) For all the 10-storey numerical models, DCM.75.10 and DC3.75.10 had the highest IDR values along both X and Y directions, with a maximum value of 1,0% on the 3rd floor. DC2.50.10 had the lowest IDR of 0,32% on the 5th floor.

d) The IDRs of the 5-storey buildings were relatively homogeneous along the height, with similar values at the 2nd and 3rd floors.

e) Due to non-rigid modelling on the interface between the linear elements and joints, the 10-storey model has significant heterogeneity in IDRs between storeys.

Table 3-9 presents the DL requirement for IDRs based on prEN 1998-1-2:2023[3], which is very restrictive compared to EN 1998-1:2004[2]. Consequently, the DL limit state preconised in prEN 1998-1-2:2023[3] may require thick elements for these seismic actions. To optimise the structure, the maximum IDR specified in EN 1998-1:2004[2] is adopted for the DL limit state.

3.5.1.2 ACCIDENTAL TORSIONAL EFFECTS

The torsional resistance of frames and the in-plane rotation of slabs are significantly affected by the relative stiffness of the horizontal to vertical resisting elements. To account for uncertainties in the location of masses and the spatial variation of the seismic motion, an accidental eccentricity, as mentioned in 3.5, should be considered for the calculated centre of mass at each floor *i*, which displaces it from its nominal location in each direction. In this study, since the buildings are regular in plan and elevation, the torsional effects are assumed to result from an accidental eccentricity (e_{ai}) of 5% of the floor dimension perpendicular to the direction of the seismic action, as prescribed by EN 1998-1:2004[2]and prEN 1998-1-2:2023[3]. Figure 10 illustrates the horizontal plan's torsional rotation angle (φ) for the regular building model and height.

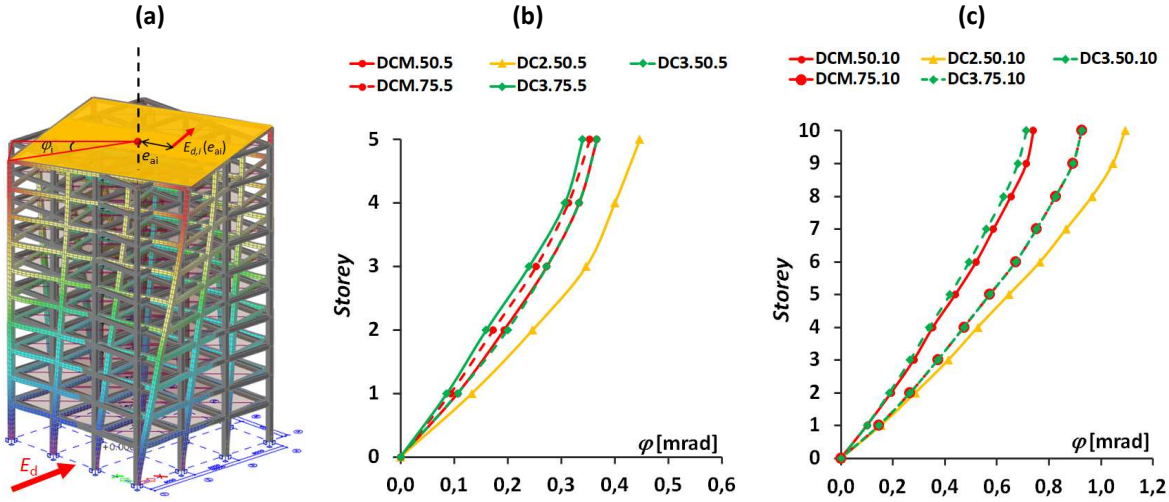


Figure 3-10: Torsional effects due to accidental eccentricities, e_{ai} (a) Illustration of the angle of torsion φ (b) 5-storey building angle of twist (c) 10-storey building angle of twist

Based on the analysis of the rotation angle profiles in Figure 10, the following observations can be made:

a) The profiles show that the structure's torsional rotation varies throughout the height of the building, with the highest value being at the top level. This finding is consistent with the conclusion reached by Özmen et al.[17].

b) Figure 10(a) and (b) also reveal the presence of three distinct groups of structures based on their rotation angles. The first group consists of DC2.50.5, DC2.50.10, and DC2.75.10, with relatively high rotation angles. The second group consists of DC3.50.5 and DC3.50.10, with moderate rotation angles. Finally, the third group includes DCM.50.5, DCM.50.10, DCM.75.5, DCM.75.10, and DC3.75.5, with the lowest rotation angles.

Overall, these findings suggest that the relative stiffness of the horizontal to vertical resisting element significantly impacts the torsional resistance of the frames and in-plane rotation of slabs. Additionally, considering uncertainties in the location of masses and the spatial variation of the seismic motion, the accidental eccentricity of 5% of the floor dimension perpendicular to the direction of the seismic action is applied in this study, as recommended by EN 1998-1:2004[2] and EN 1998-1-2:2023[3] for in plan and elevation regular buildings.

The analysis of the rotation angle profiles reveals that the torsional rotation of the structures varies significantly across the building's height, with the top level exhibiting the

highest rotation angle. Additionally, the study classifies the structures into three groups based on their rotation angles, which are closely linked to the relative stiffness of their horizontal and vertical resisting elements. The first group comprises DC2 structures with relatively high rotation angles, while the second group consists of DC3 structures with moderate rotation angles. The third group includes DCM structures with the lowest rotation angles. These findings imply that the DCM structures are the most torsionally rigid, while the DC2 structures are the least.

3.5.1.3 INTERSTOREY DRIFT SENSITIVITY COEFFICIENT Θ

Figure 3-11 shows the interstorey sensitivity coefficient (θ_i) at all storeys calculated according to Equation (3-3) for DCM structures and Equation (3-4) for DC2 and DC3 structures.

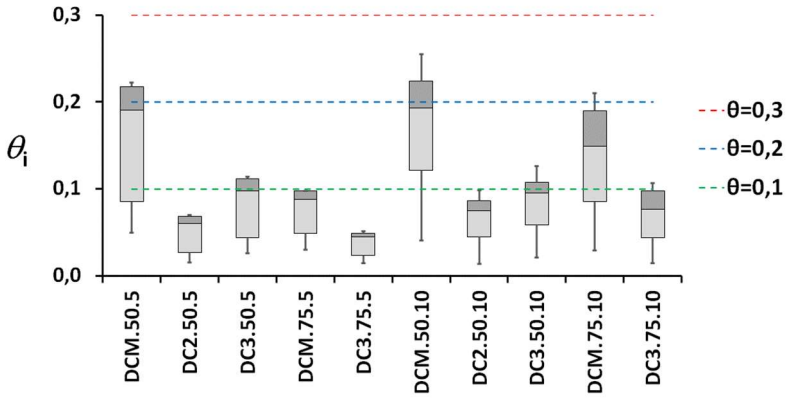


Figure 3-11: Interstorey sensitivity coefficient θ_i

Figure 3-11 presents several observations regarding the second-order effect demands:

a) The interstorey drift sensitivity coefficient (θ) values for DC3.50.5 and DC3.75.5 are 50% smaller than for DCM.50.5 and DCM.75.5, respectively, even though they have the same stiffness. This observation is consistent with the prEN 1998-1-2:2023[3], which divides the sensitivity coefficients by(q_S)and (q_R), as shown in Equation (3-4), for the ductility classes defined in that standard.

b) The same conclusion is observed for the 10-storey buildings.

c) A comparison between EN 1998-1:2004[2] and prEN1998-1-2:2023[3] shows that the latter relies on overstrength due to the redistribution and overstrength of all other sources.

d) The distance between quartiles in the distribution of θ shows that it is skewed towards the bottom 25%. This can be explained by the fact that the first floors are more rigid than the upper ones.

It is worth noting that although θ<0,1 for DC2.50.5, DC3.75.5, and DC2.50.10, all structures were designed considering the (P-Δ) effects.

3.5.2 BEAM DESIGN AND DETAILING FOR LOCAL DUCTILITY

A structure's ductility behaviour largely depends on the design and detailing of its beam cross-section and longitudinal and transverse reinforcement areas. Therefore, the design and detailing of beams play a crucial role in ensuring the desired level of ductility in a structure.

3.5.2.1 BENDING RESISTANCE OF BEAMS

The rectangular stress block model, as presented in Table 3-12, was used to calculate the resistant bending moment. This was done for all load combinations, with the acting bending moment (M_{Ed}) being obtained accordingly.

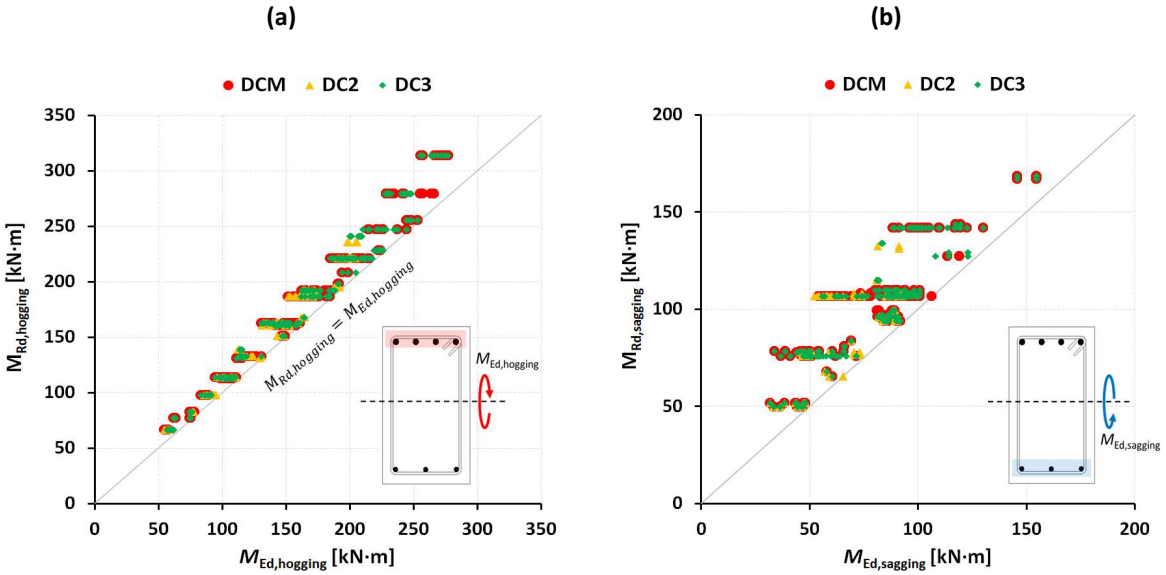


Figure 3-12: Comparison of resistant bending moment (M_{Rd}) and acting bending moment (M_{Ed}) for (a) Hogging Bending Moment and (b) Sagging Bending Moment

Based on Figure 3-12(a), the following conclusions can be drawn:

a) The hogging moments exhibit a nearly linear relationship close to the threshold line (M_{RD}=M_{Ed}), which indicates that the top reinforcement of the beam is well-optimised.

b) In some beams, the bending moment M_{Ed} is higher for DCM and DC3 structures. This can be attributed to the fact that DC2 structures have stiffer columns.

With regards to Figure 3-12(b), the following observations can be made:

a) Some beams' sagging moments do not show a clear linear pattern close to(M_{Rd}=M_{Ed}). This is because the bottom reinforcement was increased to respect the prescribed balance between the top and bottom reinforcement ratio (ρ'-ρ), resulting in higher M_{Rd} values.

b) Despite having similar fundamental periods, the acting bending moments MEd in some beams are higher for DCM and DC3 structures than for DC2 structures. This can be attributed to the fact that DC2 structures have stiffer central columns.

3.5.2.2 DETAILING OF BEAMS LONGITUDINAL REINFORCEMENT

Figure 3-13(a) and Figure 3-14(a) show the reinforcement ratio normalised to the maximum (ρ_i/ρ_{max}), and Figure 3-13(b) and Figure 3-14(b) show the reinforcement ratio normalised to the minimum reinforcement (ρi/ρmin) allowed in EN 1998-1:2004[2]and prEN 1998-1-2:2023[3] regarding the top and bottom reinforcement respectively.

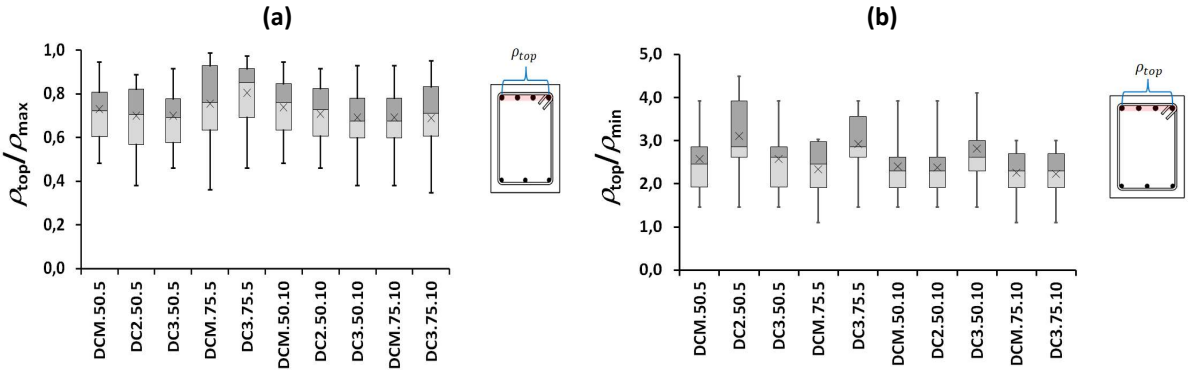


Figure 3-13: Normalised reinforcement ratio of beams: (a) Top normalised reinforcement ratio and (b) Top minimum normalised reinforcement ratio

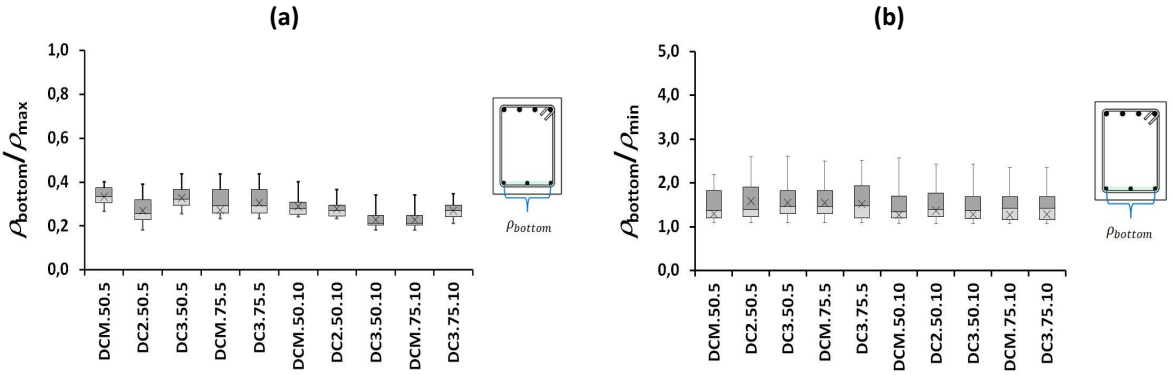


Figure 3-14: Normalised reinforcement ratio of (a) Bottom normalised reinforcement ratio and (b) Bottom minimum normalised reinforcement ratio

From the figures above, some observations can be made:

a) Regarding top reinforcement, both DCM and DC3 exhibit a closer approximation to the maximum reinforcement allowed compared to DC2, as seen in Figure 3-13 (a). Similarly, in Figure 3-13 (b), DCM and DC3 show a closer approximation to the minimum permitted reinforcement than DC2.

b) Figure 3-14 (a) indicates that bottom reinforcement ratios are far from the maximum allowed. However, some beams in DCM and DC3 have a higher approximation to the maximum permitted reinforcement ratio than DC2.

3.5.2.3 SHEAR RESISTANCE IN CRITICAL REGIONS

The design approach of using capacity design in shear ($V_{i,d}$) instead of relying on the shear force from the seismic design situation analysis ($V_{Ed,SEISM}$) not only prevents shear failure of the beams prior to flexural yielding but also ensures their resistance to seismic forces at all levels. This analysis is specifically carried out for the critical regions of the beams.

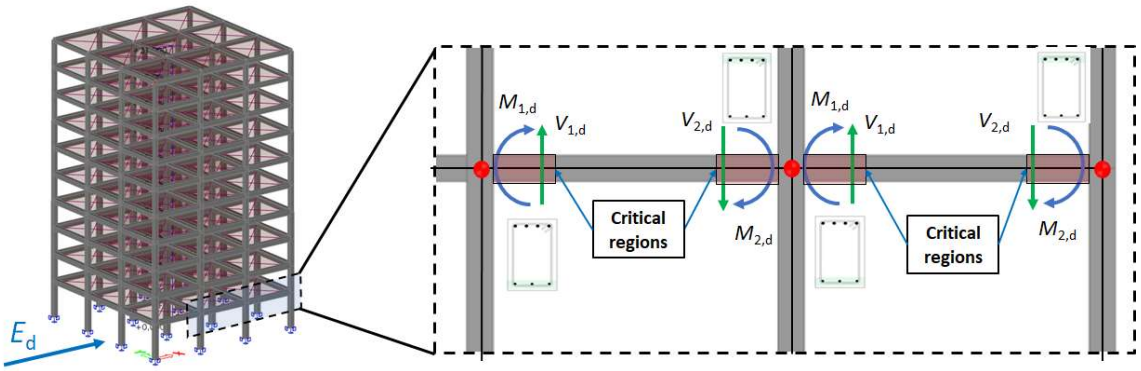


Figure 3-15: Illustration of the vectorial system of capacity design in shear for beams ($V_{i,d}$)

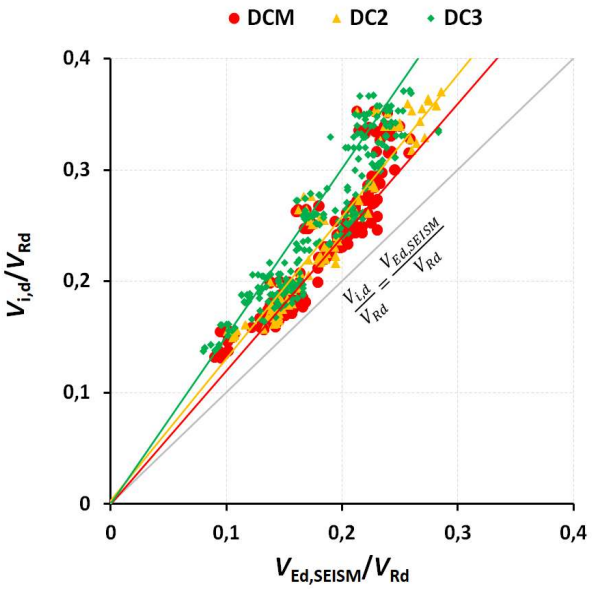


Figure 3-16: Normalised capacity design in shear ($V_{i,d}$) to shear force from the analysis for the seismic design situation ($V_{Ed,SEISM}$)

Analysis of Figure 3-16 reveals the following observations:

a) The capacity design in shear ($V_{i,d}$) surpasses the shear force from the seismic design situation analysis ($V_{Ed,SEISM}$).

b) The graph depicts a consistent, linear trend for all ductility classes, indicating that the capacity design in shear ($V_{i,d}$) is close to the shear force from the analysis for the seismic design situation ($V_{Ed,SEISM}$).

c) Interestingly, the slope of the trendline differs among the ductility classes. DCM structures exhibit a smaller trend slope, while DC3 has a higher slope.

3.5.2.4 DETAILING OF BEAMS TRANSVERSE REINFORCEMENT AT THE CRITICAL REGION

EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] specify that the maximum hoop spacing (s_{max}) is determined by a multiple of the beam longitudinal (d_{bL}) and transverse (d_{bw}) bar diameters and beam height (h_w), as shown in Figure 3-17. The values for this coefficient and the resulting maximum hoop spacing are specified in Table 12 for DCM, DC2, and DC3.

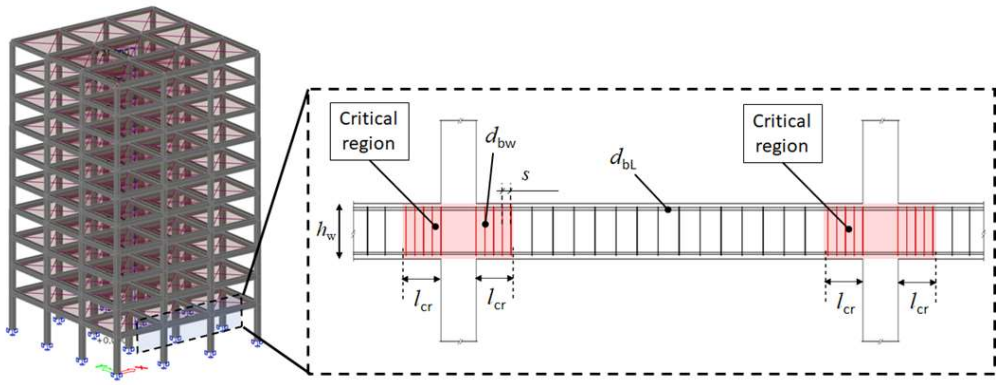


Figure 3-17: Transverse reinforcement in critical regions of beams

The following figures compare the criterium arguments by which the spacing (s) of the hoops shall not exceed, as defined in Table 3-12, the adopted hoop spacing ($s_{adopted}$).

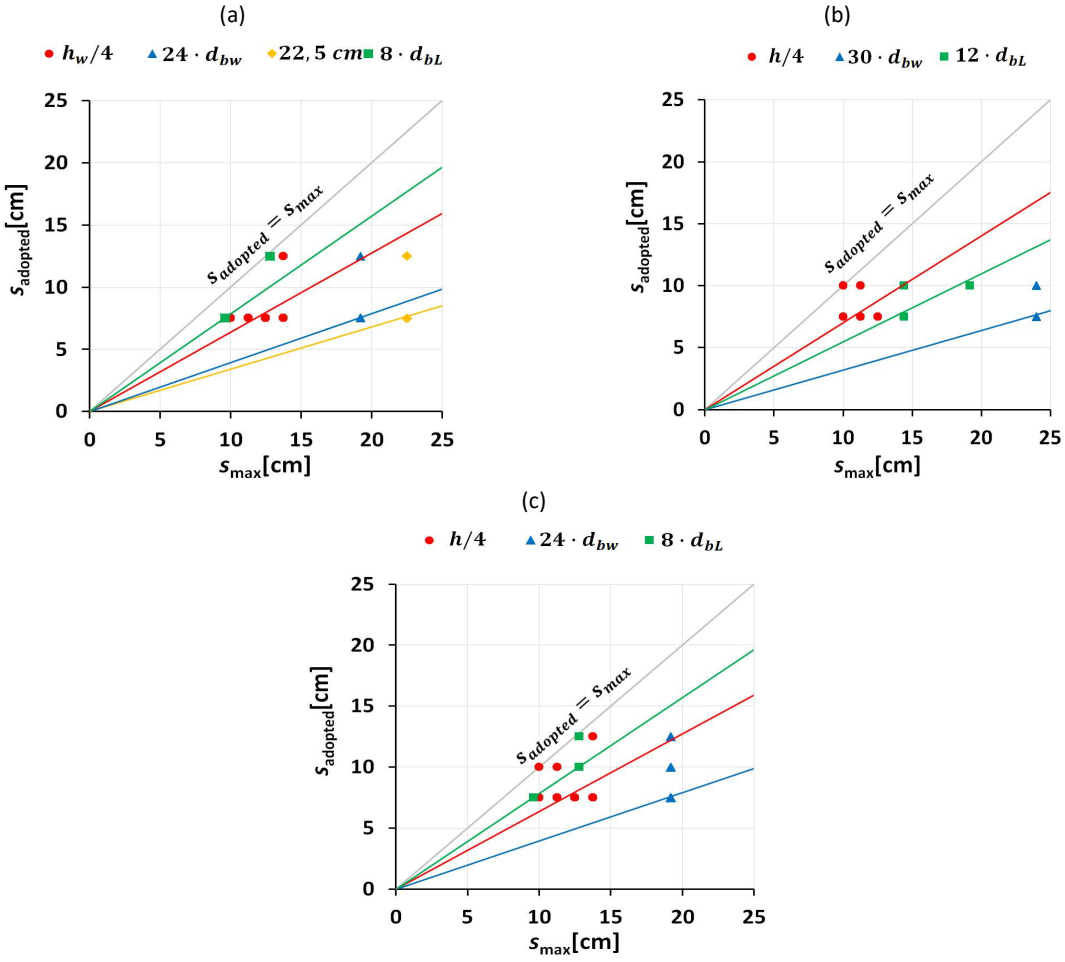


Figure 3-18: Hoop spacing (a) DCM (b) DC2, and (c) DC3

Figure 3-18 provides the following observations regarding the maximum hoop spacing (s_{max}) governing the shear resistance (V_{Rd}) of the beams for different structural systems:

a) For DCM structures, the hoop spacing is governed by the multiple 8 times the longitudinal bar diameter ($8 \cdot d_{bL}$), as shown in Figure 3-18(a).

b) For DC2 structures, the hoop spacing is governed by the multiple 1/4 times the beam height ($1/4 \cdot h$), Figure 3-18 (b).

c) Figure 3-18(c) shows that for DC3 structures, some beams are governed by the multiple ($8 \cdot d_{bL}$)spacing, while the multiple ($1/4 \cdot h$) governs others.

3.5.2.5 ANCHORAGE OF BEAM REBARS IN BEAM-COLUMN JOINTS

EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] prescribe a maximum rebar diameter limit ($d_{bL,max}$) for the beam rebar diameter (d_{bL}) as defined in Table 3-12. This accounts for the exceptionally high bond stresses that can occur along the length of a beam bar within an interior beam-column joint or for the anchorage of beam bars terminating in a joint, whether exterior or interior. Figure 3-19 illustrates such situations.

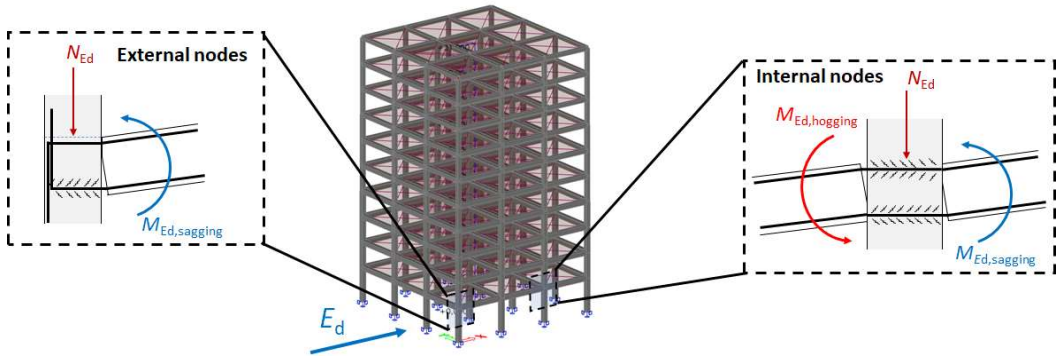


Figure 3-19: Illustration of bond stresses in beam-column joint

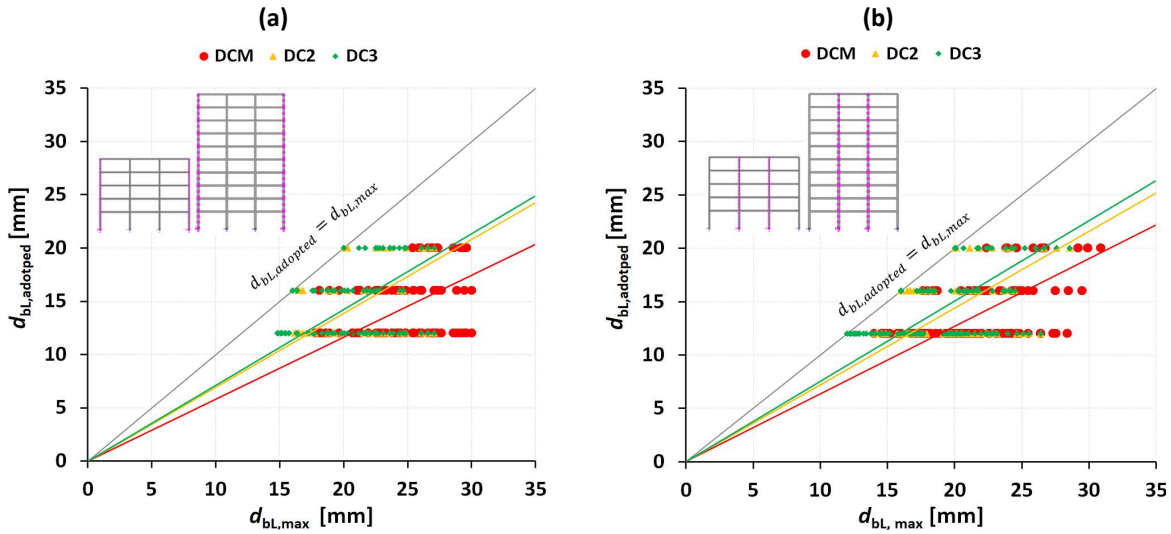


Figure 3-20: Beam rebar diameter d_{bL}: (a) Beams framing exterior columns and (b) Beams framing interior columns

One of the most critical issues for DC3 structures appears to be the internal nodes, where the beams typically require a significant amount of reinforcement that must be accommodated with the minimum possible number of bars.

To address this, prEN 1998-1-2:2023[3] provides a simplified method for calculating the rebar diameter limit ($d_{bL,max}$) on the rebar diameter in the beam that frames into the column. This limit is based on a portion of the column height, hc, and is intended to control the high bond stresses that occur along the length of a beam bar inside an interior beam-column joint or for the anchorage of beam bars terminating in a joint, either exterior or interior. The values for the parameters used in the present analysis are specified in Table 3-15.

Table 3-15: Limit to the beam rebar diameter d_{bLmax} prEN 19981-2:2023[3]

Joint position	Concrete grade range	Steel grade	Ductility class	$d_{bL,max}$
Interior beam-column joints	C30-C45	B500	DC2	5,0% h_c
			DC3	3,5% h_c
Exterior beam-column joints			DC2	5,5% h_c
			DC3	

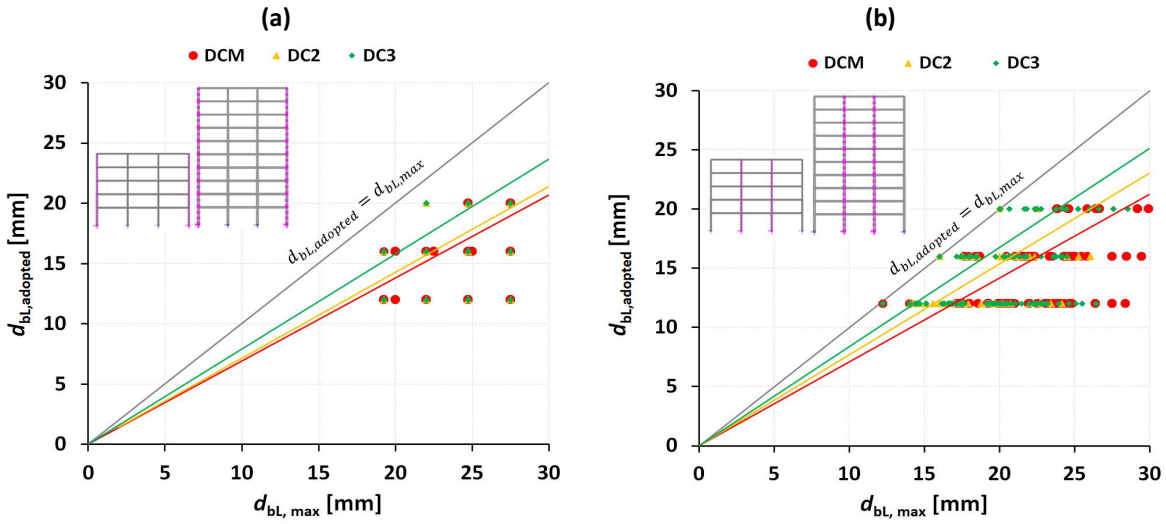


Figure 3-21: Beam rebar diameter d_{bL}: (a) Beams framing exterior columns and (b) Beams framing interior columns

Concerning Figure 3-21, while (a) and (b) depict varying values for some beams, they exhibit a similar trend.

3.5.2.6 CHORD ROTATION OF BEAMS

According to prEN 1998-1-2:2023[3], sufficient ductility and plastic rotation capacity should be provided in all critical regions of beams. Figure 22 represents the chord rotation verifications for seismic load acting in one direction.

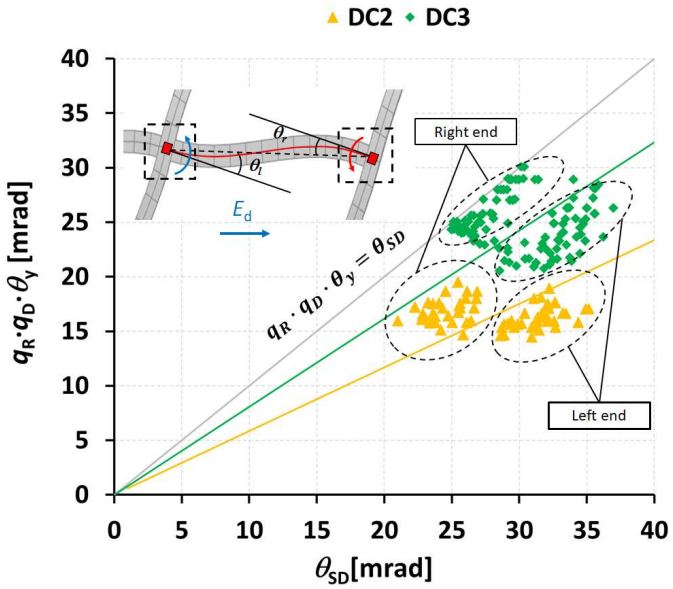


Figure 3-22: Verification of the chord rotation capacity of beams

Figure 3-22 provides some important insights:

a) On the right end of the beam, the value of θ_{SD} is closer to the threshold $q_R \cdot q_D \cdot \theta_y$ than on the left. This is because the top reinforcement ratio is higher than the bottom. Since θ_y is linearly dependent on the yield curvature from the section analysis of the end section ($\phi_y = \varepsilon_{sy}/(d\text{-}x)$), the higher the reinforcement quantity, the higher the value of ϕ_y. It should be noted that ϕ_y is the largest contributor to the value of θ_y, as shown in expression (16).

b) DC3 structures exhibit higher values of ($q_R \cdot q_D \cdot \theta_y$) and ($\theta_{SD}$) than DC2. Two reasons can explain this observation:

   i. DC3 structures have a higher reinforcement ratio compared to DC2 structures.

   ii. DC3 structures have a higher ductility than DC2 structures, provided the detailing rules have been followed. Therefore, the value of ($q_R \cdot q_D \cdot \theta_y$) is higher for DC3 than DC2.

3.5.3 COLUMN DESIGN AND DETAILING FOR LOCAL DUCTILITY

According to EN 1998-1:2004[2] and prEN 1998-1-2:2023[3], column design aims to achieve the desired global ductility and resistance by promoting beam-sway mechanisms and preventing soft-storey mechanisms, early shear damage, and section spalling.

3.5.3.1 NORMALISED AXIAL FORCE

To ensure a minimum flexural ductility of the column, EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] set upper limits on its axial force ratio (ν_d), as defined in Table 3-13.

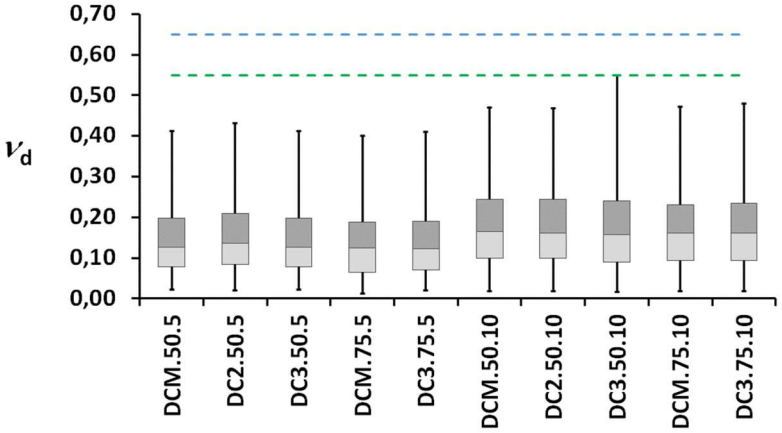


Figure 3-23: Normalised axial force (ν_d)

As shown in Figure 3-23, all structures have a normalised axial force lower than 0,55. The DC2.50.10 structure approaches the DC2 threshold due to seismic action and column cross-section area optimisation.

3.5.3.2 BENDING RESISTANCE OF COLUMNS

The column reinforcement design was performed by considering combined flexural actions (N_{Ed}, $M_{Ed,y}$, $M_{Ed,z}$) and using the capacity design method to ensure adequate resistance capacity as prescribed by EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. Figure 3-24 shows the relation between the beams (M_{Rb}) and columns (M_{Rc}) resistant bending moment for all ten structures in the analysis.

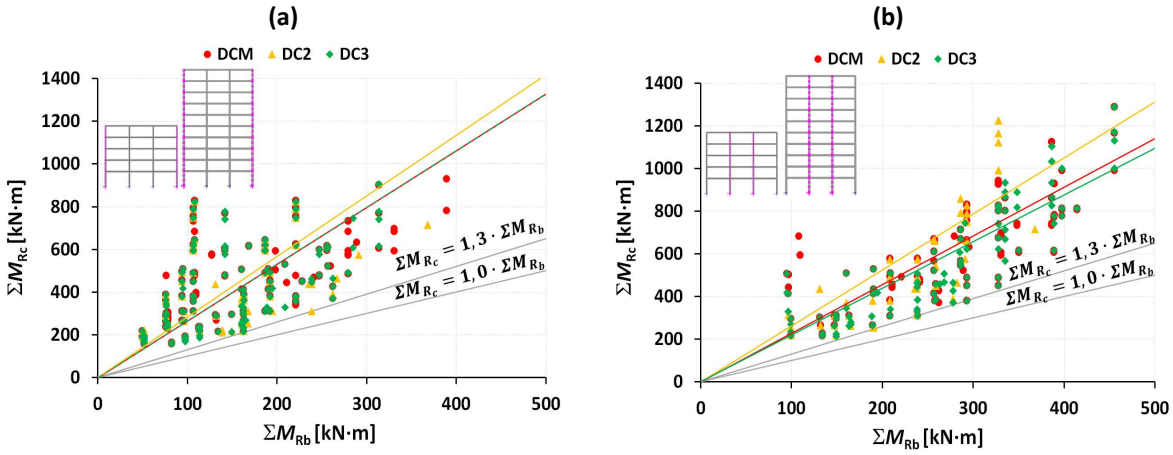


Figure 3-24: Capacity design in flexure, strong column/weak beam: (a) External columns and (b) Internal columns

From the figure above, the following observation can be made:

a) Figure 3-24(a) and (b) shows all structures fulfil the capacity rule for control of the global mechanism, $\Sigma M_{Rc} \geq 1{,}3 \cdot \Sigma M_{Rb}$.

b) Figure 3-24(a) shows that for structures DC2, the resistant bending moment ratio between the external columns and beams is, on average, M_{Rc}/M_{Rb}=2,8. For DCM structures, M_{Rc}/M_{Rb}=2,65 and for DC3 structures, M_{Rc}/M_{Rb}=2,71 on average.

c) Figure 3-24(b) reveals that for DC2 structures, the average ratio between the resistant bending moment of external columns and beams is M_{Rc}/M_{Rb}=2,62. For DCM, the ratio is M_{Rc}/M_{Rb}=2,27; for DC3 structures, the average ratio is M_{Rc}/M_{Rb}=2,18.

3.5.3.3 DETAILING OF COLUMNS LONGITUDINAL REINFORCEMENT

Figure 3-25 shows the normalised reinforcement ratio for each, and it was built up with the same procedures as in 3.5.2.2.

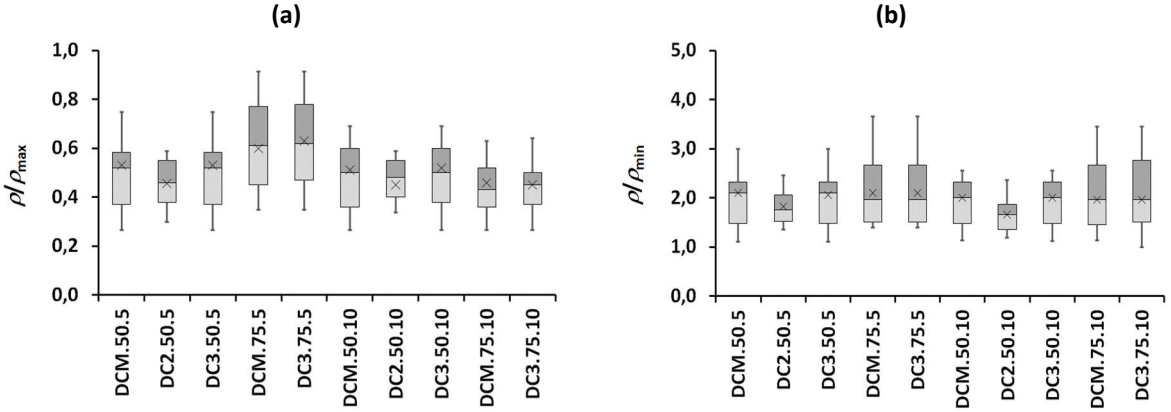


Figure 3-25: Normalised reinforcement ratio for columns: (a) Maximum reinforcement ratio and (b) Minimum reinforcement ratio

From Figure 3-25, it can be observed that:

a) In some cases, the reinforcement of columns has been selected to correspond to the minimum steel content required by EN 1998-1:2004[2] and prEN 1998-1-2:2023[3]. This is because the dimensions of columns have been determined based on damage limitations for DCM and significant damage limitations for DC2 and DC3, as shown in Figure 3-25(b). However, this approach has led to the overstrength of some columns, as seen in Figure 3-25(a). Overstrength occurs when the strength of a structure or a structural component exceeds the design strength, which can have economic and safety implications.

b) In other cases, the capacity in flexure dictated the increase of longitudinal reinforcement. This indicates that the capacity in flexure, or the ability of the column to resist bending moments, is a crucial factor in the design of reinforced concrete columns. Increasing longitudinal reinforcement can improve the capacity in flexure and ensure that the column meets the required design strength.

3.5.3.4 CAPACITY DESIGN IN SHEAR OF COLUMNS

The capacity design of columns in shear presumes that plastic hinges form at the member ends, and the equilibrium of moments suffices to establish the maximum possible shear force in the

column. Figure 3-26 depicts the vectorial components (shear and moments) acting on the structural columns involved in capacity design in shear.

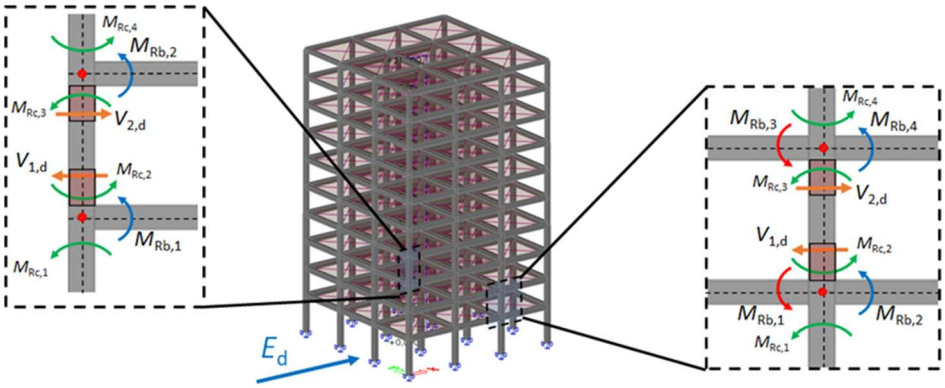


Figure 3-26: Illustration of the vectorial system of capacity design in shear ($V_{i,d}$) for columns

Figure 3-27 shows the relationship between capacity design in shear ($V_{i,d}$) instead of the shear force from the analysis for the seismic design situation ($V_{Ed,SEISM}$) and the shear resistance (V_{Rd}) for the internal and external columns.

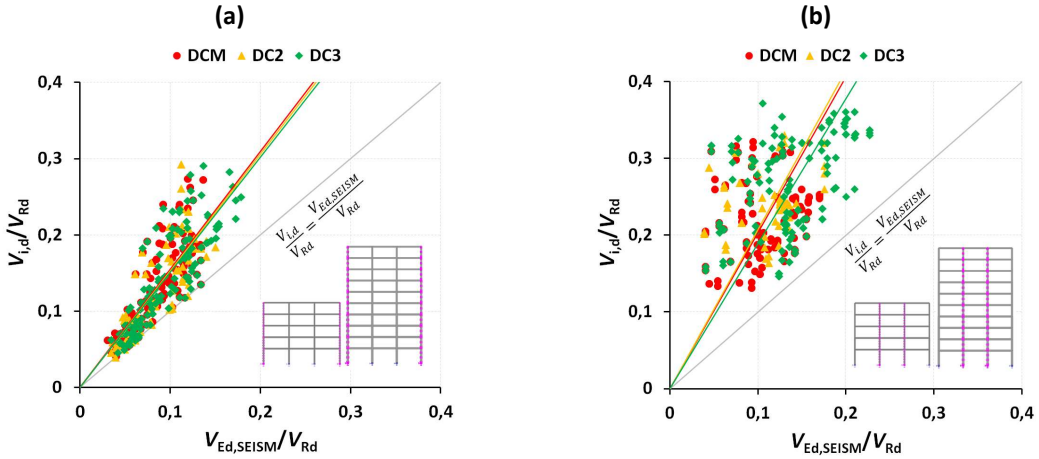


Figure 3-27: Normalised shear capacity design of columns (a) Exterior columns (b) Interior columns

Upon analysing Figure 3-27, several observations can be made:

a) The distribution of capacity design in shear ($V_{i,d}$) is greater than the distribution of shear force from the seismic design situation analysis ($V_{Ed,SEISM}$), indicating that the structure's capacity to resist shear is higher than the seismic design demand for both internal and external columns.

b) For external columns, Figure 3-27(a) shows a linear trend for all ductility classes, indicating that the capacity design in shear ($V_{i,d}$) is generally close to the shear force from the analysis for the seismic design situation ($V_{Ed,SEISM}$). However, for some columns, the values of ($V_{i,d}$) are substantially higher than ($V_{Ed,SEISM}$) due to the fact that (M_{Rb}) at the first storeys is higher.

c) For internal columns, Figure 3-27(b) exhibits a similar distribution pattern for all ductility classes.

d) The slope of the distribution trendline differs among the ductility classes, with DC3 structures exhibiting a smaller slope for the external columns in Figure 3-27 (a) and DC2 for interior columns in Figure 3-27 (b).

3.5.3.5 DETAIL OF COLUMNS TRANSVERSE REINFORCEMENT

According to the specifications in EN1998-1:2004[2] and prEN 1998-1-2:2023[3], Figure 3-28 illustrates the maximum hoop spacing within the critical region (s_{max}).

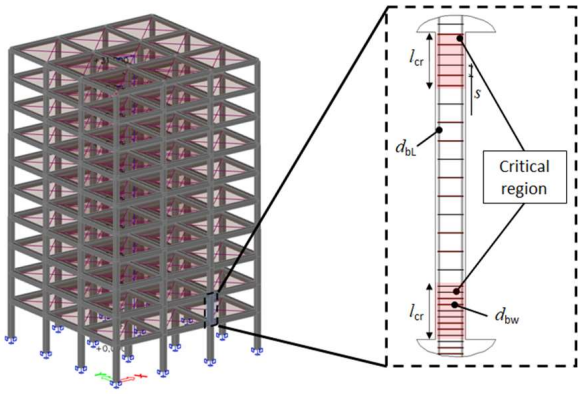


Figure 3-28: Illustration of the transverse reinforcement in critical regions of columns

The following figures compare the criterium arguments by which the spacing (s) of the hoops shall not exceed, as defined in Figure 3-13, m with the adopted hoop spacing ($s_{adopted}$).

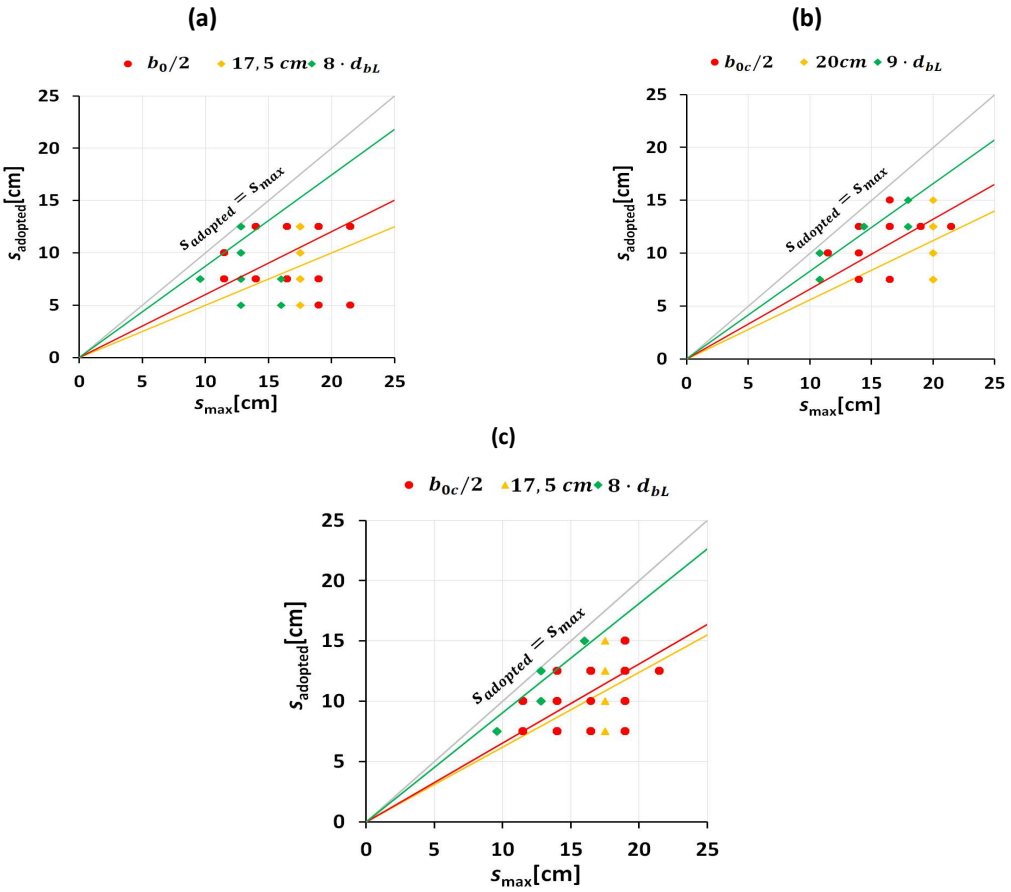


Figure 3-29: Hoop spacing (a) DCM, (b) DC2, and (c) DC3

Figure 3-29 provides the following observations regarding the maximum hoop spacing (smax) governing the shear resistance (V_{Rd}) of the beams for different structural systems:

a) For DCM structures, the hoop spacing is governed by the multiple 8 times the longitudinal bar diameter ($8 \cdot d_{bL}$) and half of the smallest dimension of the concrete core ($b_{0c}/2$), as shown in Figure 3-29(a).

b) For DC2 structures, the hoop spacing is governed by the multiple 9 times the longitudinal bar diameter ($9 \cdot d_{bL}$) and the half of the smallest dimension of the concrete core ($b_{0c}/2$), Figure 3-29(b).

c) In the case of DC3 structures, the maximum hoop spacing is determined by multiplying the longitudinal bar diameter (d_{bL}) by 8, as depicted in Figure 3-29(c).

3.5.3.6 CONFINING REINFORCEMENT IN CRITICAL REGIONS OF PRIMARY COLUMNS

The EN 1998-1 takes into account the necessity of verifying the confinement capacity ($\alpha \cdot \omega_{wd}$) against the confinement demand ($30 \cdot \mu_\phi \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot b_c/b_0$-0,035). However, the second generation of Eurocode 8, prEN 1998-1-2[3], does not include this verification. Instead, it focuses on achieving the required overall ductility of the structure by considering the chord rotation verification in potential regions for plastic hinge formation, which are defined for each type of building member. This subchapter conducts a comparative analysis between the DCM, DC2, and DC3 ductility classes for analysis.

So, the value (μ_ϕ) cannot be achieved in a column's flexural plastic hinge like a beam. Instead, the extreme concrete fibres can reach their ultimate strain, ε_{cu2}=0,0035, and spall. The plastic hinge relies on the enhanced ultimate strain of the confined concrete core inside the hoops to provide the required value of (μ_ϕ) through confinement. The concept of effective confinement area is based on recommendations by Mander et al.[18].

Figure 3-30 shows the relation between confinement capacity ($\alpha \cdot \omega_{wd}$) against the confinement demand at the base of the columns ($30 \cdot \mu_\phi \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot b_c/b_0$-0,035), as depicted in expression (3-18):

$$\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_\phi \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} - 0{,}035 \qquad (3\text{-}18)$$

Where(ω_{wd}) is the mechanical volumetric ratio of confining hoops within the critical regions, (b_0) is the minimum dimension of the concrete core, and (α) is a confinement effectiveness factor, depending on concrete core dimensions, confinement spacing and the arrangement of hoops and ties.

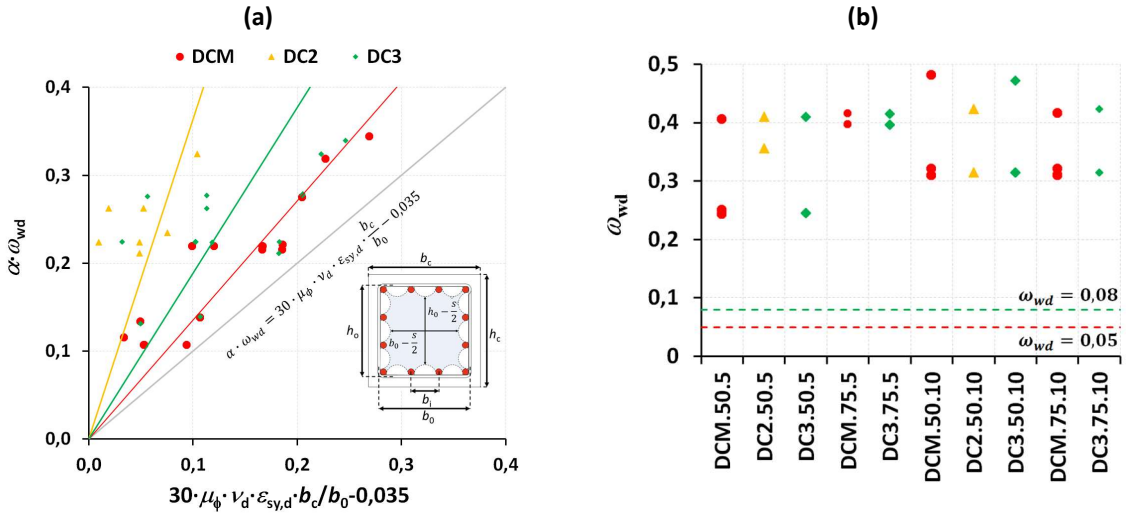


Figure 3-30: Column confinement: (a) Confinement capacity versus demand and (b) Mechanical reinforcement ratio

From Figure 3-30, some conclusions can be drawn about the need for confinement reinforcement at the base of the columns:

a) The confinement reinforcement is determined using the same hoops to resist shear forces. In certain situations, the maximum allowable spacing between the hoops (s_{max}) governs the mechanical reinforcement ratio (ω_{wd}).

b) Figure 30(a) displays distinct confinement demands for different types of structures. DCM structures exhibit a lower confinement capacity, while DC2 structures demonstrate the highest. This can be attributed to the fact that higher design axial forces result in increased confinement requirements. It is important to note that the quantity of hoops does not solely dictate the confinement demand.

3.5.3.7 CHORD ROTATION OF COLUMNS

As previously mentioned, in prEN 1998-1-2:2023[3], it is important to ensure adequate ductility and plastic rotation capacity in critical areas of beams. Figure 3-31 illustrates the evaluations of chord rotation for seismic loads applied in a single direction.

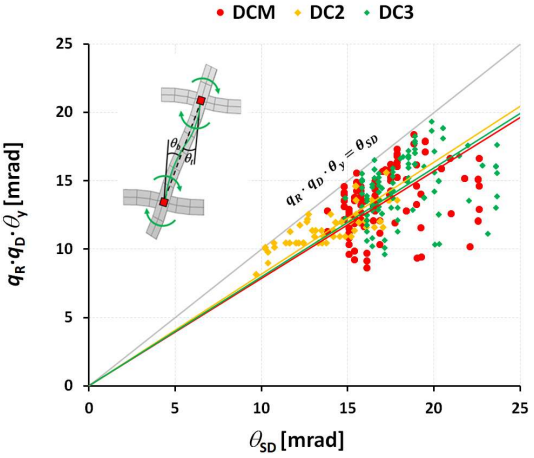


Figure 3-31: Verification of the chord rotation capacity of column

Based on the information presented in Figure 3-31, certain observations can be made regarding the chord rotation:

a) Chord rotation refers to the rotational deformation experienced by columns, leading to tension or compression stresses on the longitudinal reinforcement within a concrete column when subjected to applied loads. The extent of chord rotation is influenced by multiple factors, such as the magnitude and distribution of the applied loads, as well as the geometric properties of the column. This can be observed in Figure 3-32, which showcases DCM and DC3 structures.

b) While chord rotation and concrete confinement are distinct considerations in the design of concrete columns, they are interconnected and mutually influence each other. Proper concrete confinement can help mitigate excessive chord rotation by enhancing the column's overall ductility and strength. Additionally, analysing Figure 3-30(a) and Figure 3-31 designing for chord rotation can influence the requirements for concrete confinement for DCM, DC2 and DC3 structures, as the two aspects are interconnected regarding column performance.

3.5.4 STRUCTURES INVENTORY

A structure inventory, also known as a bill of quantities (BOQ), is a detailed document that provides a comprehensive list of all the materials and quantities for each structure, as represented in Table 3-16.

Table 3-16: Bill of quantities

Structure	Concrete		Reinforcement steel									
			Beam					Column				
	Beams m^3	Columns m^3	Total long. ton	Density Long. kg/m^3	Total Transv. ton	Density Transv. kg/m^3	Total Density kg/m^3	Total Long. ton	Density Long. kg/m^3	Total transv. ton	Density Transv. kg/m^3	Total Density kg/m^3
DCM.50.5	90,7	33,00	7,5	82,4	5,8	63,7	146,1	5,0	152,7	2,7	80,2	232,9
DC2.50.5	90,7	35,4	8,7	95,4	6,4	70,8	166,1	5,4	153,2	2,7	77,2	230,4
DC3.50.5	90,7	33,00	7,5	82,9	5,8	63,9	146,8	5,2	157,6	2,7	80,2	237,8
DCM.75.5	125,6	62,1	8,9	71,4	7,9	63,5	134,9	7,7	124,5	5,0	81,0	205,6
DC3.75.5	125,6	62,1	8,9	71,2	7,9	63,2	134,3	7,6	122,9	5,1	82,8	205,7
DCM.50.10	235,1	100,2	17,1	72,7	15,2	64,5	137,2	12,8	127,7	9,3	92,5	220,3
DC2.50.10	235,1	105,9	19,2	81,8	16,6	70,5	152,3	13,7	129,0	10,7	101,4	230,4
DC3.50.10	235,1	100,2	16,9	72,2	15,6	65,8	138,0	13,2	131,7	9,6	96,0	227,7
DCM.75.10	281,2	105,9	19,5	69,4	17,4	61,8	131,2	12,5	117,7	9,6	90,5	208,2
DC3.75.10	281,2	105,9	19,5	69,3	17,4	61,8	131,0	12,4	117,0	9,6	90,5	207,5

Table 3-17 shows the differences in quantities of concrete volume, longitudinal reinforcement (Long.) and transverse reinforcement (Transv.).

Table 3-17: Quantities differences between structures

Structures	S_α m/s²	Number of storeys	Concrete volume difference		Long. Reinforcement weight difference		Transv. Reinforcement weight difference	
			Beams	Columns	Beams	Columns	Beams	Columns
DCM to DC2	5,00	5	0,0% =	6,7% ↗	13,6% ↗↗	7,0% ↗	9,9% ↗	3,0% ↗
DC2 to DC3			0,0% =	-7,2% ↘	-15,1% ↘↘	-4,2% ↘	-10,7% ↘↘	-3,1% ↘
DCM to DC3			0,0% =	0,0% =	0,6% ↗	3,1% ↗	0,3% ↗	0,0% →
DCM to DC3	7,50		0,0% =	0, 0% =	-0,4% ↘	-1,3% ↘	-0,5% ↘	2,1% ↗
DCM to DC2	5,00	10	0,0% =	5,8% ↗	11,2% ↗↗	6,4% ↗	8,5% ↗	13,7% ↗↗
DC2 to DC3			0,0% =	-5, 4% ↘	-13,3% ↘↘	-3,6% ↘	-7,1% ↘	-11,6% ↘↘
DCM to DC3			0,0% =	0, 0% =	-0,6% ↘	3,0% ↗	1,94% ↗	3,6% ↗
DCM to DC3	7,50		0,0% =	0,0% =	0,3% ↗	0,6% ↗	0,0% =	0,0% =

Based on the information provided in Table 3-17, specific conclusions can be drawn for both 5-storey and 10-storey buildings subjected to a seismic action S_α=5,0m/s^2.

For the 5-storey buildings:

a) The concrete volume remains unchanged compared to DCM and DC2 (5 storeys). However, there is an increase in longitudinal reinforcement weight, with beams experiencing a 6,70% increase and columns a 13,6% increase. Similarly, the transverse reinforcement weight increases by 7,0% for beams and 9,9% for columns.

b) When transitioning from DC2 to DC3, the concrete volume remains the same. However, there is a decrease in the longitudinal reinforcement weight, with beams experiencing a 7,18% decrease and columns a 15,0% decrease. Likewise, the transverse reinforcement weight decreases by 4,2% for beams and 10,7% for columns.

c) In the case of DCM to DC3, there is an increase in the longitudinal reinforcement weight by 0,6% for beams and 3,1% for columns. The transverse reinforcement weight increases by 0,3% for beams, while columns experience no change.

Comparing DCM to DC3 for the 5-storey buildings under the seismic action S_α=5,0m/s^2, it can be observed that the concrete volume remains the same. However, there is a decrease in longitudinal reinforcement weight, with beams experiencing a 0,4% decrease and columns a 1,3% decrease. Conversely, the transverse reinforcement weight decreases by 0,5% for beams but increases by 2,1% for columns.

For the 10-storey buildings:

a) The concrete volume remains the same in the transition from DCM to DC2. The longitudinal reinforcement weight increases by 5,8% for beams and 11,2% for columns. Similarly, the transverse reinforcement weight increases by 6,4% for beams and 8,5% for columns.

b) When comparing DC2 to DC3, the concrete volume remains unchanged. However, there is a decrease in longitudinal reinforcement weight, with beams experiencing a 5,4% decrease and columns a 13,3% decrease. The transverse reinforcement weight decreases by 3,6% for beams and 7,1% for columns.

c) In the case of DCM to DC3, the concrete volume remains the same. The longitudinal reinforcement weight decreases by 0,7% for beams but increases by 3,0% for columns. The transverse reinforcement weight increases by 1,9% for beams and 3,6% for columns.

It was anticipated that DC2 would offer greater cost efficiency compared to DC3, as mentioned by Plumier[6]. However, the present study's findings reveal that DC2 exhibits higher concrete consumption for columns and an overall greater utilization of reinforcement steel than DC3. This can be attributed to the earlier drafts of prEN 1998-1-2-N 1150[19] from 2022, where the control mechanism to prevent the collapse of soft stories was based on the criterion that the virtual work of external forces ($V \cdot \delta$) multiplied by the behaviour factor, should be smaller than the potential virtual work generated by the internal resisting bending moments ($M_{\mathrm{Rdc,i}}$) of the columns multiplied by the plastic chord rotation capacity ($\theta_{\mathrm{u}}^{\mathrm{pl}}$), with($V$) representing external forces and (δ) representing the displacement imposed in a kinematically acceptable yield mechanism, as discussed by Maglio et al.[20]. Comparing Figure 3-32 to Figure 3-24, this condition leads to a less demanding resistant capacity of the columns ($M_{\mathrm{Rc,i}}$) concerning the resistant capacity of the beams ($M_{\mathrm{Rb,i}}$).

Notably, the latest version of prEN 1998-1-2:2023[3] no longer includes this condition. Instead, DC2 and DC3 adhere to the strong column/weak beam rule described in section 3.4.3.3. Since DC2 has a higher seismic action, the material consumption is higher despite the differences in detailing for local ductility between DC2 and DC3.

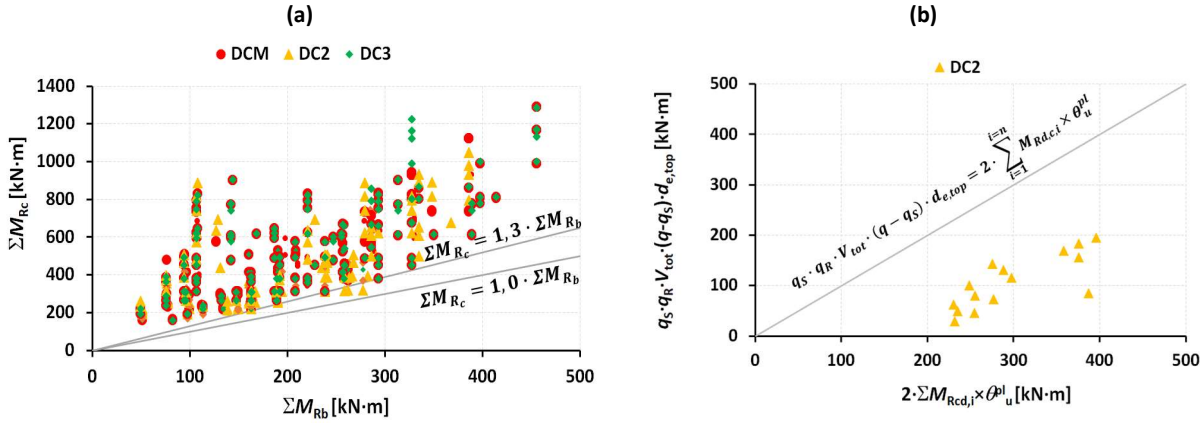


Figure 3-32: Capacity design in flexure: (a) Beams and columns(b) Virtual work

It is worth mentioning that Sextos et al.[21] concluded for EN 1998-1:2004[2] that selecting the ductility class (DCM/DCH) does not substantially impact the overall quantity of reinforcement needed or the total cost of a regular dual structural system in low-to-moderate seismicity regions. This is because most of the cross-sectional dimensions and the corresponding reinforcement are determined by the minimum requirements prescribed in the building code.

3.6 SUMMARY AND CONCLUSIONS

This study aimed to compare the design provisions and requirements for different buildings, classes of ductility and seismic action intensity levels according to the first and second generations of Eurocode 8. The seismic response, stability, and detailing of local ductility were analysed for the case studies. The major findings and suggestions for improvement are summarised here forth:

- Buildings designed for DCM and DC3 present similar behaviour, but DC2 structures experience higher roof displacements and base shear for the same seismic action intensity level. Higher modes influence the increase in base shear for DC2.50.10 in mid-rise buildings, affecting inertial forces and shear forces at higher storeys but with less impact on displacements and interstorey drifts. These findings are relevant for the seismic design of reinforced concrete structures, especially mid-rise buildings.
- The interstorey drift sensitivity coefficient analysis indicates that buildings with the same stiffness have smaller sensitivity coefficients for DC3.50.5 and DC3.75.5 compared to DCM.50.5 and DCM.75.5, per prEN 1998-1-2:2023[3]. This holds true for both 5-storey and 10-storey buildings. The comparison between EN 1998-1:2004[2] and prEN 1998-1-2:2023[3] demonstrates the latter's reliance on overstrength due to the redistribution and overstrength of all other sources. The distribution of (θ) is skewed towards the bottom 25%, which can be attributed to the greater rigidity of the first floors compared to the upper ones. It is worth noting that all structures were designed considering (P-Δ) effects, even though some have θ<0,1.
- Based on the analysis of the hogging and sagging moments of the beams, it can be concluded that the top reinforcement of the beam is adequately optimised, as exhibited by the nearly linear relationship as evidenced in Figure 3-12(a) close to the threshold line (M_{Rd}=M_{Ed}). However, in some beams, the bending moment M_{Ed} is higher for DCM and DC3 structures, which can be attributed to the fact that DC2 structures have stiffer columns.
- In terms of the sagging moments, it was observed that, in some beams, there is no clear linear pattern, as demonstrated in Figure 3-12(b) close to (M_{Rd}=M_{Ed}), as the bottom reinforcement was increased to meet the prescribed balance between the top and bottom reinforcement ratio (ρ'-ρ), resulting in higher (M_{Rd}) values. Additionally, the acting bending moments M_{Ed} are higher for DCM and DC3 structures than for DC2 structures despite having similar fundamental periods, which can be attributed to the stiffer central columns of DC2 structures.
- Regarding the beam capacity design in shear, a consistent linear trend, where ($V_{i,d}$) is higher than ($V_{Ed,SEISM}$), is verified across all ductility classes, indicating that the capacity design in shear results in action effects close to the shear force from the analysis for the seismic design situation. It is worth noting that the trendline slope, as shown in Figure 3-16, differs among the ductility classes, with DCM structures exhibiting a smaller trend slope and DC3 structures showing a higher slope.
- Regarding the transverse reinforcement of beam detailing, as depicted in Figure 3-18, the analysis of the hoop spacing for different ductility classes shows that DCM structures are governed by the spacing limit of 8 longitudinal bar diameters. In contrast, DC2 structures

are governed by the limit of 1/4 of the beam height. However, DC3 structures have a mix of beams governed by the (d_{bL})and the ($1/4 \cdot h$) spacing limits.

- Regarding the chord rotation, Figure 3-22 demonstrates that the top reinforcement ratio of a beam affects the yield curvature (θ_y) value and, consequently, the SD limit state chord rotation capacity (θ_{SD}) value. Beams with higher top reinforcement ratios have higher values of (ϕ_y) and (θ_{SD}). Moreover, DC3 structures have higher reinforcement ratios and ductility than DC2 structures, resulting in higher values of both ($q_R \cdot q_D \cdot \theta_y$) and ($\theta_{SD}$).
- Concerning the capacity design in flexure for the global mechanism control, Figure 3-24 shows that DC2 structures have the highest average value of ($\Sigma M_{Rc}/\Sigma M_{Rb}$) ratio of external columns to beams resisting bending moment, followed by DC3 and DCM structures.
- Concerning the longitudinal reinforcement ratio (Figure 3-25), columns have been reinforced with the minimum steel content required by EN 1998-1:2004[2], prEN 1998-1-2:2023[3] and EN 1992-1-1:2004[5], leading to significant overstrength in some cases. Such overstrength from detailing requirements can have economic and safety implications. However, in other cases, the flexural demand has dictated the increase of longitudinal reinforcement, governing the design of reinforced concrete columns.
- This study found that the shear action effects distribution resulting from capacity design is generally close to the seismic demand from analysis for internal and external columns, indicating that the structure's capacity to resist shear demand is adequate. However, for some columns, shear values from capacity design are substantially higher than the shear force from the seismic analysis, possibly due to higher (M_{Rb}) values at the lower storeys. The distribution pattern of capacity design in shear is similar for all ductility classes for internal columns, as shown in Figure 3-27(a). However, there is a variation in slope among the ductility classes for both internal and external columns, as depicted in Figure 3-27(b). DC3 structures exhibit a smaller slope for the external columns, while DC2 structures exhibit a smaller slope for the interior columns.
- Despite chord rotation capacity and concrete confinement being distinct provisions/requirements in concrete column design, they are interconnected and mutually affect each other. Adequate concrete confinement can help mitigate excessive chord rotation by enhancing the column's ductility and strength. Moreover, analysing Figure 3-30(a) and Figure 3-31 reveals that designing for chord rotation capacity can impact the requirements for concrete confinement in DCM, DC2, and DC3 structures. These two aspects are closely interconnected in terms of column performance.
- Adopting DC2/DC3 instead of DCM significantly impacts the total reinforcement needed. This is because most cross-sectional dimensions and corresponding amounts of reinforcement are determined by the minimum requirements set forth by Eurocode 8.

In summary, the comparison between DCM and DC2 and DC2 and DC3 shows variations in the quantities required for longitudinal and transverse reinforcement for both 5-storey and 10-storey buildings. The design was performed for each member, considering specific requirements instead of relying on a standardized cross-sectional detailing for similar members. This approach aimed to optimize and enhance the design. The required concrete volume amount remains similar in all cases. These findings provide valuable insights into the economic

implications of design considerations for different structural configurations under seismic actions.

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3.8 SYMBOLOGY

3.8.1 UPPER-CASE LATIN SYMBOLS

Symbol description	*Upper-case Latin symbols*		
	DCM	DC2	DC3
Cross-sectional area of reinforcement.	A_s		
Cross-sectional area of shear reinforcement	A_{sw}		
Horizontal force acting on the considered storey i	$F_{Ed,i}$		
Floor-dimension perpendicular to the direction of the seismic action	L_i		
Beam design value of the applied internal bending moment	M_{Ed}		
End moment of a beam or column for the calculation of its capacity design shear	$M_{i,d}$		
Resistant moment about the Y axis	$M_{Rd,y}$		
Resistant moment about the Z axis	$M_{Rd,z}$		
Design moment of resistance of a beam at end *i*	$M_{Rb,i}$	$M_{Rd,b,i}$	
Design the axial resistance of the section	N_{Rd}		
Design shear force for quasi-permanent load combinations	$V_{g+\psi_2 \cdot q}$		

3.8.2 LOWER CASE LATIN SYMBOLS

Symbol description	*Lower case Latin symbols*		
	DCM	DC2	DC3
Overall width of a beam cross-section	b		
Width of the web on T, I or L beams	b_w		
Cross-sectional dimension of column	b_c		
Width of confined core in a column	b_0	-	-
Smallest dimension of the concrete core (to the centreline of the hoops)	-	b_{0c}	
Effective depth of a cross-section	d		
Accidental eccentricity	e_{ai}		
Design value of concrete compressive strength	f_{cd}		
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}		
Mean value of axial tensile strength of concrete	f_{ctm}		
Design yield strength of reinforcement	f_{yd}		
Design yield strength of the shear reinforcement	f_{ywd}		
Characteristic yield strength of reinforcement	f_{yk}		
Cross-sectional depth of column in the direction of interest	h_c		
cross-sectional depth of a beam	h_w	-	-
cross-section depth of a beam or column	-	h	
Beam or column clear length	l_{cl}		
Spacing of the stirrups	s		
Inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration	z		

3.8.3 GREEK SYMBOLS

Symbol description	Greek symbols		
	DCM	DC2	DC3
Coefficient taking account of the state of the stress in the compression chord	α_{cw}		
Confinement effectiveness factor	α	–	–
Design value of steel strain at yield	$\varepsilon_{sy,d}$		
Reduced moment	μ		
Curvature ductility factor	μ_ϕ		
Normalised axial load	ν_d		
Longitudinal reinforcement ratio in the compression zone	ρ'	ρ'_l	
Maximum longitudinal reinforcement ratio	ρ_{max}	$\rho_{l,max}$	
Angle between the concrete compression strut and the beam axis perpendicular to the shear force	θ		
Mechanical ratio of longitudinal reinforcement	ω		
Mechanical volumetric ratio of confining reinforcement	ω_{wd}	–	–

Symbol description	Greek symbols		
	DCM	DC2	DC3
Coefficient taking account of the state of the stress in the compression chord	α_{cw}		
Confinement effectiveness factor	α	–	–
Design value of steel strain at yield	$\varepsilon_{sy,d}$		
Reduced moment	μ		
Curvature ductility factor	μ_ϕ		
Normalised axial load	ν_d		
Longitudinal reinforcement ratio in the compression zone	ρ'	ρ'_l	
Maximum longitudinal reinforcement ratio	ρ_{max}	$\rho_{l,max}$	
Angle between the concrete compression strut and the beam axis perpendicular to the shear force	θ		
Mechanical ratio of longitudinal reinforcement	ω		
Mechanical volumetric ratio of confining reinforcement	ω_{wd}	–	–

4 RC wall building structures: comparison of first and second generation Eurocode 8 seismic design and detailing strategies

Helder Maranhão, Humberto Varum, José Melo, António A. Correia, "RC wall building structures: Comparison of first and second generation Eurocode 8 seismic design and detailing strategies", Engineering Structures, Volume 315, 2024, 118463, ISSN 0141-0296, https://doi.org/10.1016/j.engstruct.2024.118463.

4.1 ABSTRACT

This study provides a thorough comparative analysis of structural design strategies across different ductility classes and seismic action levels, as outlined in Eurocode 8 provisions. The analysis is carried out for 5- and 10-storey building examples. The research highlights key findings regarding base shear demand, storey displacements, sensitivity coefficients, and other crucial structural performance parameters. Those findings indicate that while buildings designed for DCM and DC3 ductility classes generally exhibit similar base shear values and roof displacements, exceptions arise, particularly in the lower-storey buildings where DC2 demonstrates significantly lower roof displacements. The influence of strut inclination on design bending moments is also noted, underscoring its significance in structural behaviour. Moreover, the study delves into interstorey drift ratio (IDR) demands from analysis, revealing consistent displacement patterns across various structures with notable implications for deflection and stability. Additionally, insights into chord rotation control, sensitivity coefficients, and reinforcement quantities shed light on their behaviour under diverse seismic conditions, aiding in optimising structural design strategies.

4.2 INTRODUCTION

In recent decades, structural engineering has seen significant advancements in research and development of design practices, including theoretical, experimental, and numerical investigations by researchers working with practitioners' feedback. Those advancements led to the development of new construction materials and structural solutions and the evolution of engineering professionals' design methodologies, as referred to by Maranhão et al.[1].

The second-generation Eurocode 8 introduces a displacement-based design for new structures alongside the traditional force-based design using reduced equivalent forces. The second-generation Eurocode 8 drafts, FprEN 1998-1-1:2024[2] and prEN 1998-1-2:2023[3], provide resistance models which are required to apply the displacement-based design approach to reinforced concrete wall structures in the form of chord rotation verification for members, as stated by Franchin and Noto[4].

In this second-generation Eurocode 8, the structure of its parts was revised, concentrating in Part 1-1, FprEN 1998-1-1:2024[2], the general aspects independent of the type of structures and materials, and in Part 1-2, prEN 1998-1-2:2023[3], the specific rules for new buildings.

The second generation of Eurocode 8 introduces new classes of ductility to deal with identified practical design issues in the first generation. According to the first generation, EN 1998-1:2004[5], three ductility classes were defined, namely the high (DCH) and medium (DCM), which are recommended for areas of high or medium seismicity, and the low ductility class (DCL) for low seismicity regions. These ductility classes allow the designer, within strictly prescribed limits, to balance the provision of strength against ductility. Thus, concrete structures with high ductility class (DCH) requirements or the more straightforward medium

ductility class (DCM) can be designed. However, DCH concrete structures' rules are sometimes reported as challenging, often appearing hard to satisfy in practice. In contrast, the DCM rules are sometimes complex (for example, for capacity design in shear of beams and columns).

In any case, there are two general reasons why providing high ductility is usually of no advantage, according to Booth[6]. One reason is that tall or flexible buildings, governed by deflection rather than strength, have no advantage from high ductility design. Another reason is that the high ductility option for any building type is becoming less attractive with the increasing emphasis on serviceability performance. The best option would thus likely be to eliminate the DCM/DCH distinction. On the other hand, a review of DCM provisions is also needed for concrete structures. Therefore, FprEN 1998-1-1:2024[2] overcame this issue by introducing new ductility classes known as DC1, DC2, and DC3, where DC1 and DC3 can be related to the previous DCL and DCM, regarding EN 1998-1:2004[5], thus letting DCH out of the new generation of Eurocode 8.

Another crucial novelty of the second-generation Eurocode 8 is the decomposition of the behaviour factor in its different components. The influence of the behaviour factor on the shape of the design response spectrum, intrinsically on the spectral acceleration, and therefore the design force, is sensitive to the numerical value of the behaviour factor. The prEN 1998-1-2:2023[3] introduces a behaviour factor composed of the product of the behaviour factor components (q_R), (q_S), and (q_D) used in the force-based design. Additional differences in the second generation of Eurocode 8 also arise when calculating the envelope of bending moment and shear actions for RC wall structures.

The structural design process involves two stages. The first stage comprises the analysis for design actions, which includes defining the interstorey drift limitation, calculating the member cross-sections' dimensions, and reinforcement quantities for static and seismic actions to verify the limit states' requirements. The analysis also considers second-order effects, global mechanism control checks (such as the capacity design procedure in flexure for members and beam-column joints), and capacity design in shear. The second stage involves detailing local ductility, including verifying the minimum and maximum longitudinal reinforcement ratio, hoop spacing, and anchoring provisions, as noted in Maranhão et al.[1].

The present study aims to compare and analyse the structural behaviour of wall structures for ductility classes DCM, as preconised in EN 1998-1:2004[5], DC2 and DC3, as established in prEN 1998-1-2:2023[3]. It also aims to analyse the influence of the displacement-based design under the form of chord rotation checks, the influence of the behaviour factor components on the interstorey drift, and (P-Δ) effects, as introduced by FprEN 1998-1-1:2024[2] and prEN 1998-1-2:2023[3]. This study also intends to compare the influence of the envelope of bending moment and shear action calculation models preconised in EN 1998-1:2004[5] for DCM structures and in prEN 1998-1-2:2023[3] for DC2 and DC3 structures. Lastly, the present research aims to compare and observe the influence and differences of provisions for local ductility between the current EN 1998-1:2004[5] and the second-generation prEN 1998-1-2:2023[3].

The current study builds upon the work developed by Maranhão et al.[1]. It involves the comparison of a set of structures designed to be regular in both plan and elevation and non-torsionally flexible, considering various classes of ductility, concerning both the current EN 1998-1:2004[5] and second-generation prEN 1998-1-2:2023[3] and FprEN 1998-1-1:2024[2]. Therefore, the design calculations for DCM wall structures followed the guidelines outlined in

EN 1998-1:2004[5]. On the other hand, the design calculations for DC2 and DC3 structures were conducted following the provisions specified in prEN 1998-1-2:2023[3] and FprEN 1998-1-1:2024[2]. It should be noted that the structures were calculated rigorously, with a high level of optimisation, to identify the differences between the different ductility classes.

4.3 OBJECT OF RESEARCH AND ANALYSIS

To conduct this study, a set of reinforced concrete wall system structures with different configurations were selected as the subject of analysis. These configurations were defined based on non-variable characteristics and parameters common to all buildings and specific variables for each building, as outlined in Table 4-1.

Table 4-1: Characteristics/parameters of the structures analysed

Variable	Non-variable
Buildings height Seismicity Ductility classes Inertial masses	Structural system (wall system) Inter-storey height Span length Materials Vertical loads

Ten structures were built up, assumed to be symmetrical, isotropic, and regular in plan and elevation as prescribed in EN 1998-1:2004[5], prEN 1998-1-2:2023[3]. These structures, listed in Table 4-2, were assigned to three different ductility classes (DCM, DC2, and DC3) based on the number of storeys and the maximum response spectral acceleration at the constant acceleration range (S_α). In the present study, (S_α) is defined as being ($2{,}5 \cdot a_g \cdot S \cdot \eta$), as per EN 1998-1:2004[5].

Table 4-2: Adopted nomenclature for the structures

Number of storeys	S_α m/s²	Ductility class	Structure label
5	5,0	DCM	DCM.W.50.5
		DC2	DC2.W.50.5
		DC3	DC3.W.50.5
	7,5	DCM	DCM.W.75.5
		DC3	DC3.W.75.5
10	5,0	DCM	DCM.W.50.10
		DC2	DC2.W.50.10
		DC3	DC3.W.50.10
	7,5	DCM	DCM.W.75.10
		DC3	DC3.W.75.10

4.3.1 BUILDINGS GEOMETRY AND STRUCTURAL SYSTEM

The structures in this study consist of two building geometrical configurations: a five-storey building, shown in Figure 4-1(a) and a ten-storey building, shown in Figure 4-1(b).

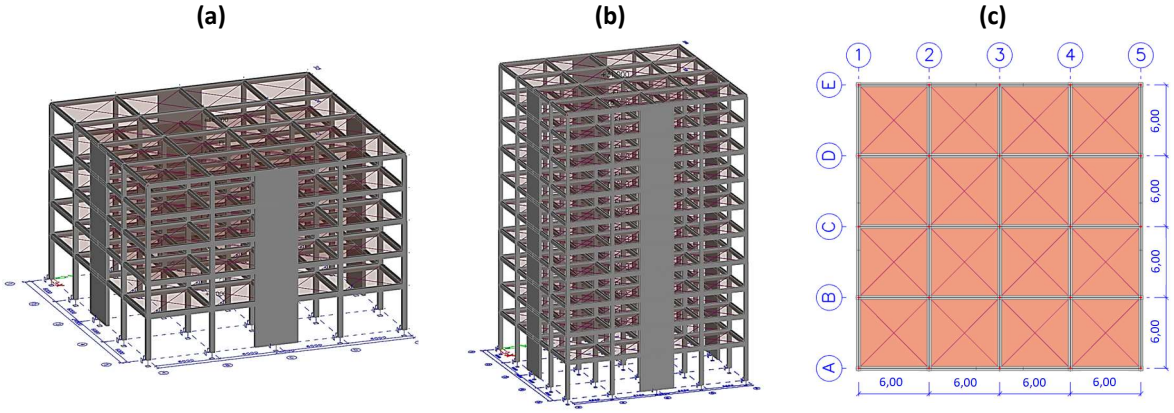


Figure 4-1: Wall system configuration: (a) 5 storeys, (b) 10 storeys and (c) plan view

The structural system for the buildings consists of a series of frames with an equal spacing of 6,0 m between axes, typical of office buildings, as illustrated in Figure 4-1(c), with exterior walls positioned at the axis centres in an orthogonal arrangement on the facades. The floor-to-floor height is 3,0 m for all storeys, except for the ground storey, which is 4,0 m. The solid slab thickness is 15,0 cm at each storey. Table 4-3 presents the main geometrical characteristics.

Table 4-3: Geometry of the wall structures.

Structure	Number of storeys	Total height (m)	Number of bays	Inter-storey height (m)	Wall width b_w (m)	Wall length l_w (m)
DCM.W.50.5	5	16,0	4	4,0 for the ground storey 3,0 for the others	0,30	4,00
DC2.W.50.5					0,30	4,00
DC3.W.50.5					0,30	4,00
DCM.W.75.5					0,30	4,00
DC3.W.75.5					0,30	4,00
DCM.W.50.10	10	31,0			0,30	4,50
DC2.W.50.10					0,30	4,50
DC3.W.50.10					0,30	4,50
DCM.W.75.10					0,30	4,50
DC3.W.75.10					0,30	4,50

For the analysis, it is assumed that the walls have the same geometry along the height for each group of 5-storey and 10-storey buildings. This assumption simplifies the analysis by considering consistent wall dimensions and characteristics within each group of buildings.

4.3.2 MATERIALS

The following materials are considered for all primary seismic members (beams, columns, and walls) of the structures under analysis:

- Concrete C30/37
- Reinforcement steel B500

The concrete properties are defined per EN 1992-1-1:2004[7] and EN 206-1:2007[8] standards, and its mechanical properties are shown in Table 4-4.

Table 4-4: Concrete mechanical properties

Mechanical properties of concrete C30/37		
Characteristic compressive cylinder strength of concrete	f_{ck} (MPa)	30
Mean value of concrete cylinder compressive strength	f_{cm} (MPa)	38
The design value of concrete compressive strength	f_{cd} (MPa)	20
Characteristic axial tensile strength of concrete	$f_{ctk,0,95}$ (MPa)	2,0
Mean value of axial tensile strength of concrete	f_{ctm} (MPa)	2,9
Design value of tensile strength	f_{ctd} (MPa)	1,3
Elasticity modulus	E_{cm} (GPa)	33
Ultimate compressive strain in the concrete	$\varepsilon_{cu,1}$ (‰)	3,5

The nominal concrete cover (cnom) is calculated by adding to the minimum cover (cmin) the allowance in design for deviation (Δc_{dev}) according to EN 1992-1-1:2004[7]. Assuming Exposure Class XC1, related to environmental conditions following EN 206-1:2007[8] as specified in EN 1992-1-1:2004[7], the assumed nominal cover (c_{nom}) is 30 mm. The steel reinforcement adopted in this study is classified as B500, and its mechanical properties are characterised according to EN 10080:2005[9], as presented in Table 4-5.

Table 4-5: Reinforcement steel mechanical properties

Reinforcement steel B500		
Characteristic yield strength of reinforcement	f_{yk} (MPa)	500
Mean strength of reinforcement	f_{ym} (MPa)	500
Design yield strength of reinforcement	f_{yd} (MPa)	434,8
Elasticity modulus	E_s (GPa)	200
Yielding strain	ε_{sy} (‰)	2,5

4.4 DESIGN CRITERIA OPTIONS

The following subsections outline the options and criteria for describing and comparing earthquake-resistant structural performance and safety requirements.

4.4.1 DESIGN AND MODELLING OPTIONS

The structures are assumed to be regular in plan and elevation. Therefore, no reference behaviour factor (q) reduction was required.

The provisions in EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] for the design of structures were followed, although the following exceptions were considered:

- The response spectra adopted were based on EN 1998-1:2004[5] alone.
- Concerning the shear design of beams, columns, and walls, the approach of the current EN 1992-1-1:2004[7] was followed.

In the design of structural members, the following options were considered for simplicity:

- The minimum dimensions of the structural members concerning the design forces were obtained without adopting typical rules of common practice, such as assuming constant sections of columns and walls along all their heights. In particular, no geometrical reduction of the wall cross-sections was done along the height.
- No structural infills were considered in the analysis.

Ten finite element (FE) models were created using Scia Engineer version 22.0[10] to assess structural deformations and internal forces. The finite element software used in the present work considers non-linearities such as concrete cracking, buckling and P-□ effects. Therefore, axial loads causing additional moments due to deflections that induce geometric non-linearity were considered. The non-linear behaviour resulting from column interaction effects, namely from axial-flexural interaction, was considered since the columns are subjected to combined axial and lateral loads.

These models adhere to all Eurocode 8 modelling criteria, and their key specifications can be found in Table 4-6.

Table 4-6: Modelling criteria

Modelling Aspects	Modelling Criteria
Earthquake demand direction	Two horizontal orthogonal directions, X and Y
Global torsional effects	Yes
Frame modelling	Three-dimensional (3D)
Slabs	As a rigid diaphragm and non-contributing to the flexural stiffness of the beams
Columns, beams and walls modelling position	Central axis
Columns, beams and walls type	Linear elastic bar elements (Bernoulli formulation)
Joints stiffness modelling	No
Cracked stiffness of the concrete	$E \cdot A/2$; $E \cdot I/2$; $G \cdot J/2$ as proposed in EN 1998-1:2004[5] and FprEN 1998-1-1:2024[2]
Masses	(a) Masses of the slabs, other permanent loads and live loads with the quasi-permanent value were assumed uniformly distributed in the floors
	(b) Masses of the frame elements (beams, columns and walls) were considered automatically by the software (distributed along the axis of the elements)
Bending moments redistribution	No redistribution has been made
Beam to column/wall connection	Concentric
Accidental eccentricities	Taken into account
Second-order effects P-Δ	Taken into account, calculated by the software
Infills	Stiffness and strengths were not considered. Their weight was included in the vertical self-weight and masses
Support at the foundation level	Restrained in all directions

To provide a visual representation of the finite element (FE) model used in the analysis of the 5 and 10-storey buildings, a schematic structural model is presented in Figure 4-2.

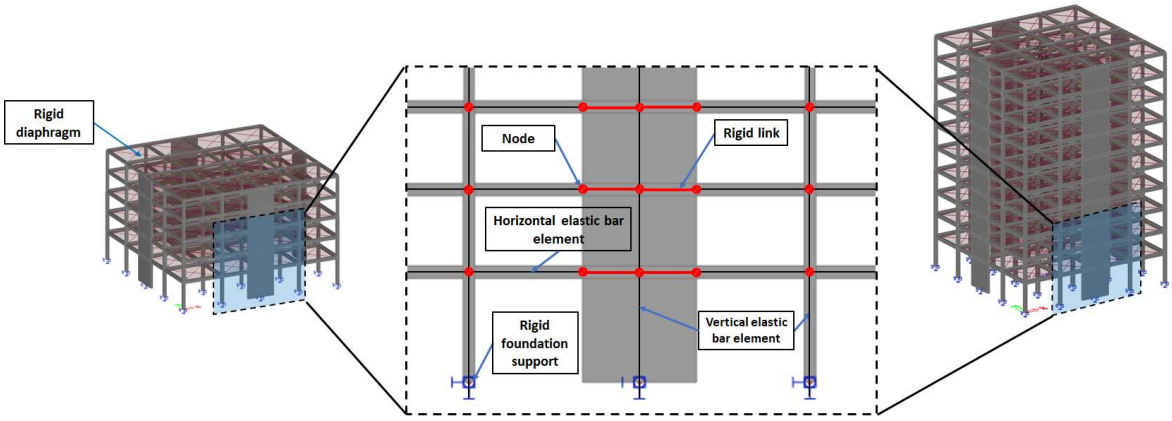


Figure 4-2: Schematic representations of the FEM model for 5 and 10-storey buildings

4.4.2 SEISMIC ACTION

The capacity of structural systems to withstand seismic actions in the non-linear range often allows for their design to resist seismic forces smaller than those required for a linear elastic response. To avoid the need for explicit inelastic structural analysis during the design phase, the structure's ability to dissipate energy through the ductile behaviour of its elements and other mechanisms is considered by performing an elastic analysis based on a reduced response spectrum concerning the elastic one. This reduction is achieved by introducing the behaviour factor (q), whose definition depends on the type of structure and its regularity in plan and elevation.

4.4.2.1 REGULARITY IN PLAN AND ELEVATION

The plan regularity criteria were assessed per EN 1998-1:2004[5]. The building exhibits a slenderness (λ) of 1,0, as it is square in the horizontal plane, making it smaller than 4,0. The ratio between the torsional radius and the radius of gyration of the floor mass at each storey level in both horizontal directions (r_i/l_s) falls within the range of 1,71 to 1,73. Given that this ratio exceeds 1,0, the criterion is met. Additionally, the buildings display double symmetry, ensuring alignment of the centre of gravity with the centre of rigidity. This results in an eccentricity (e_0) of zero in both orthogonal directions. Consequently, the specified criterion of ($e_0 \leq 0{,}3 \cdot r_i$) is also satisfied. With all these conditions fulfilled, the buildings are classified as having plan regularity in both directions.

Likewise, the buildings meet the qualitative criteria for regular elevation according to EN 1998-1:2004[5]. The prEN 1998-1-1:2023[3] refers to a quantitative criterion preconising that “both the lateral stiffness and the mass of the individual storeys remain constant or decrease gradually by no more than 20% relative to the storey below, without abrupt changes, from the base to at least one storey below the top storey”. Since interstorey stiffness and the individual storeys' mass decrease gradually by no more than 8%, the buildings are classified as regular in elevation.

To categorise a building as having torsional flexibility, the verification involves the comparison of two uncoupled vibration frequencies: (ω_θ/ω_x) and (ω_θ/ω_y). In this context, (ω_x) and (ω_y) represent the system's uncoupled angular natural frequencies in the x and y directions. The parameter (ω_θ) represents the uncoupled circular natural frequency of torsional vibration. If the ratios (ω_θ/ω_x) and (ω_θ/ω_y) exceed 1,0, the response primarily exhibits translational

characteristics, classifying the structure as non-torsionally flexible. Conversely, torsional behaviour predominantly influences the response if (ω_θ/ω_x) and (ω_θ/ω_y) are below 1,0. In the present scenario, where(ω_θ/ω_x) and (ω_θ/ω_y)are greater than 1,0, the analysed structures are classified as non-torsional flexible.

4.4.2.2 BEHAVIOUR FACTOR APPLIED IN THE ANALYSIS

To build up the design spectra for elastic analysis, and regarding the structural systems in question, the behaviour factor q and its components are defined per EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] for structures regular in plan and elevation.

The present work considers the analysis of the DCM ductility class, which is preconised in EN 1998-1:2004[5]. For the structures of interest for the current work, i.e., uncoupled wall systems, the behaviour factor adopted is the basic value of the behaviour factor, defined by q_0=3,0. The upper limit value of the behaviour factor (q) is expressed as depicted in expression (4-1):

$$q = q_0 \cdot k_w \geq 1{,}5 \tag{4-1}$$

Where (k_w) reflects the prevailing failure mode in structural systems with walls. The parameter (k_w) takes the value of 1,0 for the uncoupled wall system. The adopted behaviour factor (q) assumes a value of 3,0 in light of those above.

The new generation of Eurocode 8, prEN 1998-1-2:2023[3], introduces a behaviour factor composed of the product of three partial behaviour factor components: q_S, q_R, and q_D. Here, q_R represents the behaviour factor component responsible for accounting for overstrength due to the redistribution of seismic action effects in redundant structures. At the same time, (q_S) addresses overstrength from all other sources, and (q_D) deals with deformation and energy dissipation capacity. To build up the design spectra for elastic analysis, and regarding the structural systems in question, the behaviour factor and its components used for the present research are drawn for regular structures. Table 4-7 shows the default behaviour factor for wall systems regular in plan and in elevation according to prEN 1998-1-2:2023[3]

Table 4-7: Default values of the behaviour factors wall systems regular in plan and in elevation according to prEN 1998-1-2:2023[3]

Structure	q_R	q_D	$q=q_R \cdot q_S \cdot q_D$
DC2.50.5	1,0	1,3	2,0
DC3.50.5		2,0	3,0
DC3.75.5		2,0	3,0
DC2.50.10		1,3	2,0
DC3.50.10		2,0	3,0
DC3.75.10		2,0	3,0

4.4.2.3 RESPONSE SPECTRA

The seismic action is defined according to EN 1998-1:2004[5]. For structures of ordinary importance, the recommendation in EN 1998-1:2004[2] is for seismic action for (local) collapse prevention with a 10% exceedance probability in 50 years (mean return period: 475 years). For the present study, the type 1 elastic and the design spectra for elastic analysis are drawn up according to EN 1998-1:2004[2] for ground type B and an importance class II with an importance

factor γ_I= 1,0. Table 4-8 shows the values of the parameters describing the elastic response spectra (ξ=5%) for the structures under analysis.

Table 4-8 refers to the maximum response spectral acceleration at the constant acceleration range (S_α) and the different behaviour factor (q) values. A Consequence Class CC2 is assumed to have medium consequences for the loss of human life, with considerable impacts on economic, social, and environmental aspects. For the consequence class CC2, the design spectrum does not require amplification related to reliability differentiation.

Table 4-8: Properties of the horizontal response spectra

Structure	Ductility Class	S_α (m/s²)	a_g (m/s²)	q
DCM.W.50.5 DCM.W.50.10	DCM	5,00	1,67	3,0
DCM.W.75.5 DCM.W.75.10		7,50	2,50	
DC2.W.50.5 DC2.W.50.10	DC2	5,00	1,67	2,0
DC3.W.50.5 DC3.W.50.10	DC3	5,00	1,67	3,0
DC3.W.75.5 DC3.W.75.10		7,50	2,50	

4.4.3 STRUCTURAL SAFETY VERIFICATIONS TO LIMIT STATES

Deformation-related criteria are prescribed for all types of buildings in EN 1998-1:2004[5] and prEN 1998-1-2:2023[3]. Eurocode 8 sets out safety verification criteria for Ultimate Limit States (ULS) according to EN 1998-1:2004[5] and for Significant Damage Limit State (SD) according to prEN 1998-1-2:2023[3]. These safety criteria include the control of second-order effects, limitation of interstorey drift (ISD), and control of the global mechanism, also known as capacity design in flexure. It should be noted that EN 1998-1:2004[5] considers only the serviceability condition as a Damage-Limitation Limit State (DL) for controlling ISD, whereas prEN 1998-1-2:2023[3] considers both DL and SD limit states.

4.4.3.1 CONTROL OF SECOND-ORDER EFFECTS

The second-order effects (P-Δ), which are associated with ULS effects, are specified through an inter-storey drift sensitivity coefficient (θ) as shown in Equation (4-2) based on EN 1998-1:2004[5]:

$$\theta = \frac{P_{tot}}{V_{tot}} \cdot \frac{d_r}{h} \tag{4-2}$$

According to prEN 1998-1-2:2023[3], second-order effects (P-Δ) are associated with SD limit states and are defined by Equation (4-3):

$$\theta = \frac{P_{tot}}{q_R \cdot q_S \cdot V_{tot}} \cdot \frac{d_{r,SD}}{h_s} \tag{4-3}$$

where P_{tot} and V_{tot} are the total cumulative gravity load and seismic shear, respectively, at the storey under consideration; hs is the storey height. The parameter (q_R) is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in

redundant structures. The (q_S) is the behaviour factor component accounting for overstrength due to all other sources.

EN 1998-1:2004[5]and prEN 1998-1-2:2023[3] both specify that when the inter-storey drift sensitivity coefficient (θ) has a value between 0,1 and 0,2, an approximate adjustment for (P-Δ) effects can be made by increasing the seismic forces used in the analysis by a factor of 1/(1-θ). On the other hand, when (θ) it is between 0,2 and 0,3, a second-order analysis is necessary.

4.4.3.2 LIMITATION OF THE INTERSTOREY DRIFT ISD

The interstorey drifts (δ_r) are the product of elastic inter-storey drift (d_e) from analysis and the behaviour factor q. According to EN 1998-1:2004[2], the interstorey drift is included regarding the serviceability condition as damage-limitation (DL). The prEN 1998-1-2:2023[4] considers two limit states for the verification: the damage limitation (DL) and the significant damage limit state (SD).

Since the seismic design action analysis is linear elastic based on the design response spectrum, the elastic spectrum with 5% damping divided by the behaviour factor (q) is used. The values of the displacements (d_s) to be used for ds are calculated from the analysis (d_e) and multiplied by the behaviour factor (q), as shown in Equation (4-4):

$$d_s = q \cdot d_e \tag{4-4}$$

The design interstorey drift (d_r) is calculated as the difference of the average lateral induced by the reduced seismic action displacements (d_s) at the top and bottom of the storey under consideration as depicted in Equation (4-5):

$$d_{r,i,} = d_{s,i+1} - d_{s,i} \tag{4-5}$$

EN 1998-1:2004[5] defines the limit of interstorey drift ratio (IDR) according to Equation (4-6):

$$\frac{d_r}{h} = \frac{K}{\nu} \tag{4-6}$$

where (K) is the coefficient that accounts for the sensitivity of ancillary elements to interstorey drift, (h) is the interstorey height, and (ν) the reduction factor considers the lower return period of the seismic action associated with the damage limitation requirement. The prEN 1998-1-2:2023[3] presents a similar formulation, where (λ_{NS}) is the coefficient accounting for the sensitivity of ancillary elements to interstorey drift, the subscript LS refers to limit state and (h_s) has the same meaning as (h).

$$\frac{d_{r,LS}}{h_s} = \lambda_{NS} \tag{4-7}$$

4.4.4 VERTICAL LOADS

The vertical loads specified in Table 4-9 are according to EN 1990:2002[11] and EN 1991-1-1:2002[12] for office buildings and are considered in all the buildings under analysis.

Table 4-9: Vertical loads

Load type	Floor	Load	Value
Structure self-weight $G_{k,1}$	All floors	Primary seismic members (columns, beams and walls): generated by the software	γ=25 kN/m^3
		Slabs: solid RC 15 cm thick	
Other permanent loads $G_{k,2}$	1st to (n-1)th	Externa non-structural walls and partitions with 3,0 kN/m (per unit of wall length). It is assumed a uniformly distributed load according to EN 1991-1-1[12] 6.3.1.2(8)	1,2 kN/m^2
		Finishings	1,3 kN/m^2
	nth (roof)	Finishings	1,0 kN/m^2
Live loads Q_k	1st to (n-1)th	Building for use as civil dwellings, falling into usage category B, according to the classification in EN 1991-1-1[12][6.3.1.2]	2,0 kN/m^2
	nth (roof)	Roof category H – Roofs not accessible except for normal maintenance and repair, according to the classification EN 1991-1-1[12], 6.3.4	1,0 kN/m^2

4.4.5 DESIGN ACTION EFFECTS

This subsection explains the action effects of bending and shear as specified in EN 1998-1:2004[5] and prEN 1998-1-2:2023[3]. A comprehensive explanation of how these effects influence structural behaviour and design considerations will be provided.

4.4.5.1 BENDING MOMENT EFFECTS

According to Elgazhouli[10], for the best seismic performance, the walls must act as vertical cantilevers and only be allowed to develop a single plastic hinge at the base.

Moelhe[14] states that in cases where a design aims for a wall to possess a sole critical segment for both moment and axial force, the allocation of flexural strength should be chosen to encourage substantial yielding exclusively at that specific critical point. One strategy involves devising the designated critical height to possess flexural strength that closely aligns with the necessary strength while incorporating a degree of flexural overstrength at other points.

The non-linear behaviour of an uncoupled reinforced concrete wall is governed by a single plastic hinge at its base. This section must be designed flexibly for the bending moment resulting from the seismic design situation analysis. However, unlike the frames, the bending moment diagram does not change signs between successive floor levels, which creates uncertainties in moment distribution along the wall. In order to avoid yielding above the base hinge, EN 1998-1:2004[2] prescribes that the design bending moment diagram is based on an envelope derived from the bending moment diagram obtained from analysis vertically displaced, as known as tension shift (a_l), as shown in Figure 4-3.

According to EN 1998-1:2004[5], the envelope may be assumed linear if the structure does not exhibit significant mass discontinuities, stiffness, or resistance over its height. The tension shift (a_l) should be consistent with the strut inclination (θ) taken in the ultimate limit states (ULS) verification for shear, as defined in EN 1992-1-1:2004[7].

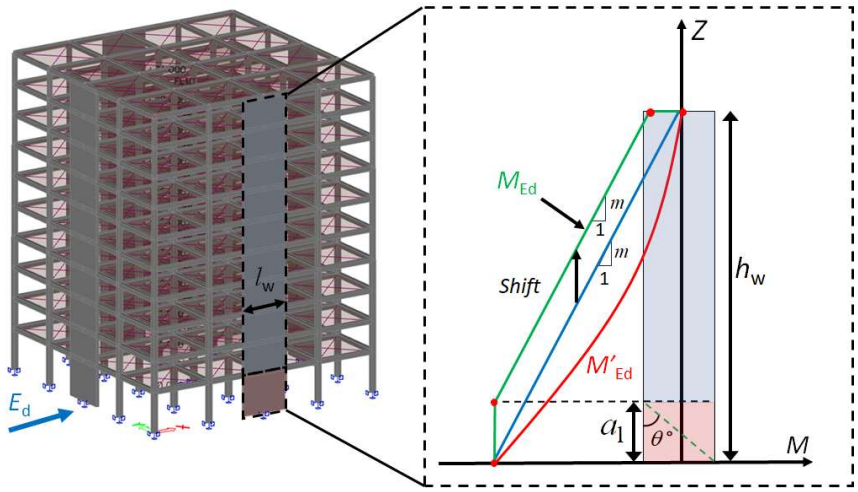


Figure 4-3: Design envelope for bending moments for wall systems EN 1998-1:2004[5]

On the other hand, prEN 1998-1-2:2023[3] adopts a different approach. The flexural resistance of the wall at its base should not be lower than the design moment ($M'_{Edw,base}$) from analysis. Along the height of the wall, the design bending moment (M_{Edw}) should be given by a vertically displaced (tension shift) envelope of the bending moment diagram obtained from the analysis, with the value at the base section replaced by the flexural overstrength moment, $M_0=\gamma_{Rd}\cdot M_{Rdw,base}$, with $\gamma_{Rd}=1,2$, as illustrated in Figure 4-4. This conservativism is still under discussion in preparation for the final version of the second-generation Eurocode 8.

Within the maximum between the critical height of the wall (h_{cr}) and the tension shift (a_l), the design moment should equal the resisting moment of the (MRdw,base.) The envelope may be assumed linear if the structure does not exhibit significant mass, stiffness, or resistance discontinuities over its height. The tension shift (a_l) depends on the axial force level at the base and should be consistent with the strut inclination (θ) taken in the SD limit state verification for shear.

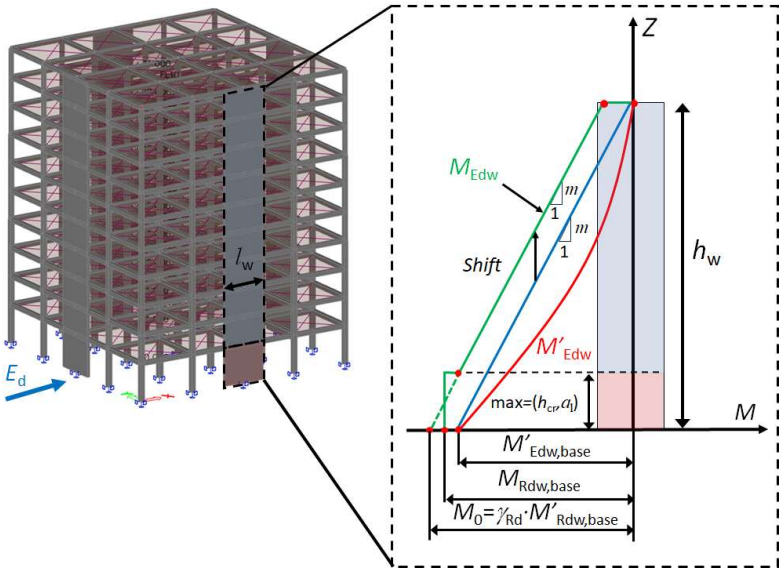


Figure 4-4: Design envelope for bending moments for wall systems prEN 1998-1-2:2023[3]

The parameter ($M_{Rdw,base}$) is the flexural resistance of the wall at the base section within the maximum between the critical height (h_{cr}), and the tension shift (a_l) is dependent on the level of axial force at the base.

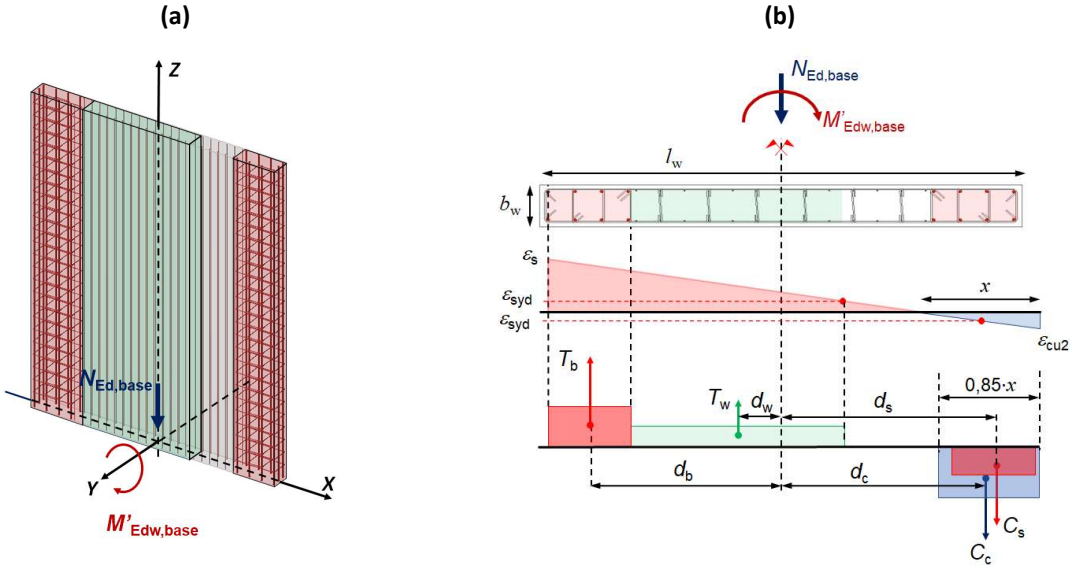


Figure 4-5: Calculation model of the resistant bending moment $M_{Rdw,base}$ (a) Wall vectorial system (b) Stress block model

According to prEN 1998-1-2:2023[4], the flexural resistance at the base of the wall ($M_{Rdw,base}$) is calculated, including the vertical web reinforcement, as illustrated in Figure 4-5.

The mathematical model to calculate the ($M_{Rdw,base}$) in the present work is adapted from the model presented by Lestuzzi et al.[15]. The longitudinal reinforcement of the plastic hinge must be determined by combined flexure design. Once the values of the internal forces have been determined, the most stressed section must be considered. The calculation uses an iterative reinforced concrete procedure based on Bernoulli's hypothesis (conservation of plane sections) and the equilibrium conditions. The usual hypotheses concerning concrete and reinforcing steel behaviour are also used. The design value of the specific concrete failure deformation defines the deformation plane. The steel's specific elastic elongation (or shortening) design value delimits the zones where the reinforcement strains. The rebar tensile forces in the boundary (T_b) and the web (T_w) are determined with the design value (f_{yd}).

$$\begin{cases} T_b + T_w - C_c - C_s - N_{Ed,base} = 0 \\ M_{Rdw,base} = T_b \cdot d_b + T_w \cdot d_b + C_c \cdot d_c + C_s \cdot d_c \end{cases} \quad (4\text{-}8)$$

The parameter (C_s) is the compression force of steel in the wall boundary element. The reinforcement area concerning the reinforcement subjected to plastification (i.e., $\varepsilon_s \geq \varepsilon_{sy,d}$) should be note$_d$. The stresses of the compressed concrete (C_c) are defined as an equivalent block of constant stresses to consider the real distribution of the compressive stresses in the concrete. The length of the equivalent stress block is limited to 85% of that of the compressed zone. The balance of vertical forces must be satisfied by iteratively determining the value of the neutral axis position (x). It should be noted that the quantity of the end reinforcement does not influence the position of the neutral axis if it is fully compressed because the antisymmetric contributions compensate each other in this case. The resistant bending moment of resistance of the section

concerning its axis can then be determined by taking into account the corresponding lever arms (d_i) as defined in expression (4-8) and depicted in Figure 4-5(b).

4.4.5.2 SHEAR EFFECTS

In EN 1998-1:2004[5], the concept of ductile walls is utilized to create a plastic hinge only at the base while maintaining elasticity along the rest of their height. While insights are provided by the moment resistance value at the base of the wall, known as ($M_{Rdw,base}$), and isolated equilibrium, they fall short in determining the maximum seismic shears that can occur at various wall levels. This deficiency arises from the dynamic nature of forces applied to the wall by the floors, which experience fluctuations during seismic events.

To address this challenge, a fundamental assumption is introduced: if ($M_{Rdw,base}$) exceeds the elastic analysis-based bending moment at the base for the design seismic action M'Ed, then the seismic shears at any point on the wall surpass those calculated from the same elastic analysis in proportion to the ratio ($M_{Rdw,base}/M'_{Edw}$). Consequently, a logical strategy is adopted to adjust the shear force derived from the design seismic action analysis (V'_{Ed}) using a capacity-design amplification factor (ε). This factor is directly linked to the ratio ($M_{Rdw,base}/M'_{Ed}$) and scales the shear force accordingly, as appointed by Fardis et al.[16].

The EN 1998-1:2004[5] stipulates that, for DCM structures, the potential increase in shear forces after yielding at the base of a primary seismic wall must be accounted for. The design shear forces (V_{Ed}), as depicted in expression (4-9), are determined to be 50% higher than the shear forces obtained from the analysis (V'_{Ed}). Consequently, a magnification factor (ε) of 1,5 is assumed for this specific class of ductility.

$$V_{Ed}(z) = 1{,}5 \cdot V'_{Ed}(z) \tag{4-9}$$

The prEN 1998-1-2:2023[3] considers that in ductile walls when the force-based approach is used, the design shear force (V_{Edw}) at level (z) should be derived from expression (4-10).

$$V_{Edw}(z) = \varepsilon(z) \cdot V'_{Edw,1}(z) \leq q \cdot V'_{Edw} \tag{4-10}$$

where $V'_{Edw}(z)$ is the shear force at level z calculated as the combination of shear force in all modes from the analysis, $V'_{Edw,1}(z)$ is the effect due to the mode with the largest participating mass in the direction of V_{Edw}. The shear magnification factor ε preconizes in prEN 1998-1-2:2023[4] follows the approach Eibl[17] and Keintzel[18] as depicted in Equation (4-11):

$$\varepsilon(z) = \sqrt{\left(\frac{\gamma_{Rd} \cdot M_{Rdw,base}}{M'_{Edw,base}}\right)^2 + m(z) \cdot \left(\frac{q \cdot S_e(T_C)}{S_e(T_1)}\right)^2} \tag{4-11}$$

where $m(z)$ is a factor varying along the height of the wall (h_w), as defined in prEN 1998-1-2:2023[3] depicted in Equation (4-12):

$$m(z) = \begin{cases} 0{,}10 \; for \; 0 \leq z \leq 0{,}33 \cdot h_w \\ 0{,}05 \; for \; 0{,}33 \cdot h_w \leq z \leq 0{,}67 \cdot h_w \\ 0{,}25 \; for \; 0{,}67 \cdot h_w \leq z \leq h_w \end{cases} \tag{4-12}$$

4.4.6 LOADS COMBINATIONS

The structure used for the present analysis was designed for Ultimate Limit states (ULS) – Fundamental and seismic combinations according to EN 1990[11].

The combinations of actions for the seismic design situation are defined in Table 4-10. The seismic combinations consider accidental eccentricities.

Table 4-10: Load combinations

Load Combination	Nature	$\gamma_{G,1}$	$\gamma_{G,2}$	$\gamma_{Q,1}$	$\psi_{2,1}$	$E_{Ed,x}$	$E_{Ed,y}$	$A_{cce,i,x}$	$A_{cce,i,y}$
Combination 1	ULS – Fundamental	1,35	1,35	1,50	-	-	-	-	-
Combination 2	ULS – Seismic	1,00	1,00	1,00	0,30	0,30	1,00	0,30	1,00
Combination 3						-0,30	1,00	-0,30	1,00
Combination 4						0,30	-1,00	0,30	-1,00
Combination 5						-0,30	-1,00	-0,30	-1,00
Combination 6						1,00	0,30	1,00	0,30
Combination 7						1,00	-0,30	1,00	-0,30
Combination 8						-1,00	0,30	-1,00	0,30
Combination 9						-1,00	-0,30	-1,00	-0,30
Combination 10	ULS – Seismic	1,00	1,00	1,00	0,30	0,30	1,00	-0,30	-1,00
Combination 11						-0,30	1,00	0,30	-1,00
Combination 12						0,30	-1,00	-0,30	1,00
Combination 13						-0,30	-1,00	0,30	1,00
Combination 14						1,00	0,30	-1,00	-0,30
Combination 15						1,00	-0,30	-1,00	0,30
Combination 16						-1,00	0,30	1,00	-0,30
Combination 17						-1,00	-0,30	1,00	0,30

The parameter ($E_{Ed,x}$) represents the action effects due to the application of the seismic action along the chosen horizontal axis X of the structure and ($E_{Ed,y}$) represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis Y of the structure as defined in EN 1998-1:2004[5]. The parameters ($A_{cce,i,x}$) and ($A_{cce,i,y}$) are the seismic and accidental torsional effects produced by ($E_{Ed,x}$)and ($E_{Ed,y}$), respectively.

4.4.7 WALL DESIGN AND DETAILING

According to EN 1998-1:2004[5] and prEN 1998-1-2:2023[3], the design values of bending moments and axial forces for seismic design are obtained from the structural analysis. Compliance with the capacity design requirements for walls is necessary to ensure the resistant capacity. The design values of shear forces are determined using the capacity design rule, as prescribed in EN 1998-1:2004[5] and prEN 1998-1-2:2023[3]. The design and detailing of the wall comprehend different regions and components. The wall is divided by the elements within and above the critical regions. According to Fardis et al.[19] usually, energy dissipation takes place by alternate positive and negative bending in flexural plastic hinges at member ends. EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] call the zones where energy is dissipated “critical regions”. The wall components, Figure 4-6, are defined as the boundary elements and web, which play complementary roles.

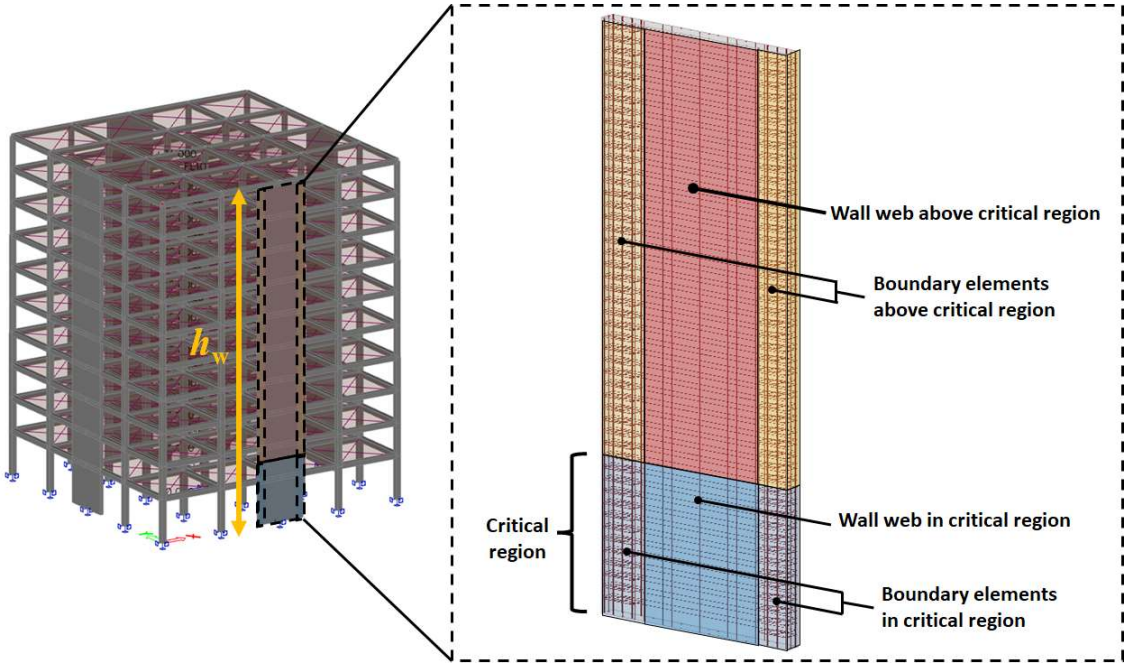


Figure 4-6: Wall components

The critical region height (h_{cr}) is defined in the same way as for EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] for DCM, DC2 and DC3 as defined in expression (4-13):

$$h_{cr} = \begin{cases} \geq max\left(l_w, \frac{h_w}{6}\right) \\ \leq min(2 \cdot l_w, h_s)\ if\ wall\ \leq 6\ storeys \\ \leq min(2 \cdot l_w, 2 \cdot h_s)\ if\ wall\ > 6\ storeys \end{cases} \quad (4\text{-}13)$$

where (h_w) is the total height of the wall, hs is the clear storey height, and the base is defined as the foundation level or the top of basement storeys with rigid diaphragms and perimeter walls.

4.4.7.1 BENDING SHEAR DESIGN

The bending and shear resistance is calculated by considering the detailing requirements and the value of the axial force from the analysis in the seismic design situation.

Table 4-11: Longitudinal and vertical reinforcement design

Design parameters		DCM	DC2	DC3
Maximum admissible normalised axial force	ν_d	0,40	0,40	0,35
Resistance in bending				
Calculation of the longitudinal rebars	A_s	As defined in 4.4.5.1		
Resistance in shear				
Capacity design	V_{Ed}	$V_{Ed} = \varepsilon \cdot V'_{Ed}$ where ε=1,5	As defined in 4.4.5.2	
Maximum admissible shear capacity	$V_{Rd,max}$	As in EN 1992-1-1:2004[9]- $V_{Rd,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot 0{,}6 \cdot \left[1-\frac{f_{ck}}{250}\right] \cdot f_{cd}}{cot\theta + tan\theta}$		
Shear resistance inside and outside critical regions	$V_{Rd,s}$	As in EN 1992-1-1:2004[9] - $V_{Rd,s} = \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot cot\theta$		

4.4.7.2 DETAILING OF WALL ELEMENTS

According to Wight et al.[20], boundary elements are segments of a concrete wall located at the ends or edges of the wall. These elements are designed to concentrate and distribute the seismic forces and deformations, preventing premature failure and enhancing the overall structural performance during earthquakes. Figure 7 represents the details of the boundary elements. It should be noted that according to EN 1998-1:2004[5] and prEN 1998-1-2:2023[3], hoops used as transverse reinforcement in walls and closed stirrups should be tied with 135° bents.

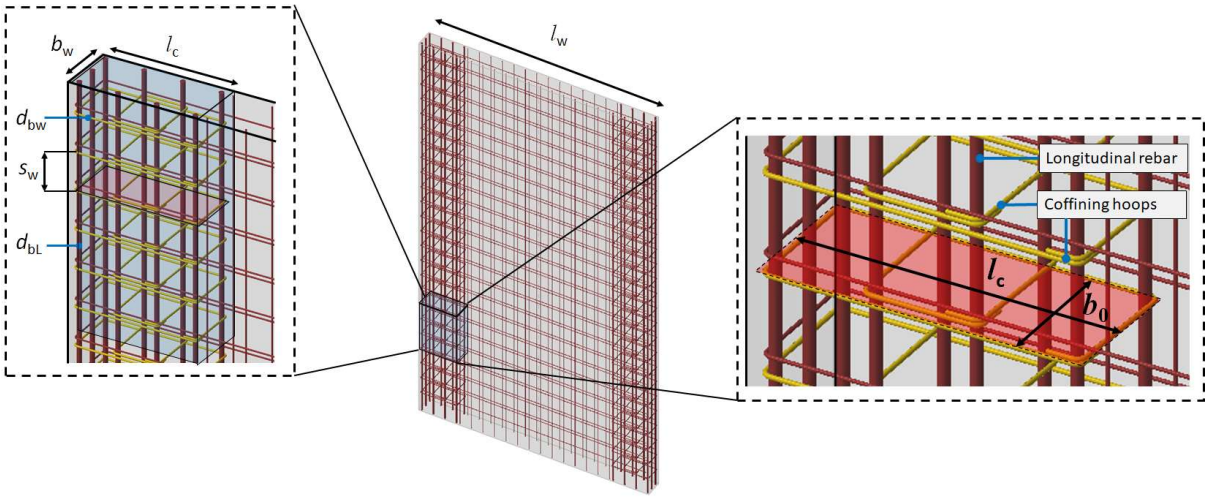


Figure 4-7: Wall boundary elements

Table 4-12 provides a comprehensive summary of the detailing provisions for the boundary elements.

Table 4-12: Detailing boundary elements

Detailing rules		DCM	DC2	DC3
In critical height region:				
Length from wall edge	l_c	0,15·l_w; 1,5·b_w part of the section where ε_c>0,0035		
Thickness	b_{wc}	≥ (0,2m; $h_{s,cl}$/15) if l_c≤max(2·b_w; 0,2·l_w)[(a)]		
		≥ (0,2m; h_{scl}/10) if l_c>max(2·b_w; 0,2·l_w)[(a)]		
Vertical reinforcement within the critical region				
Minimum longitudinal reinforcement ratio	ρ_{min}	0,5%	1,0%	
Maximum longitudinal reinforcement ratio	ρ_{max}	4,0%		
Maximum distance between restrained bars		200mm	In the boundary elements of walls, a hoop or cross-tie should engage every other longitudinal bar.	
Minimum longitudinal rebar diameter	d_{bL}	-	12	
Vertical reinforcement on the rest of the wall (outside the critical region)				
In parts where the longitudinal reinforcement ratio ε_c>0,002	ρ_{min}	0,5%		
In parts where the longitudinal reinforcement ratio ε_c<0,002	ρ_{min}	$max\left(\frac{0,1 \cdot N_{Ed} \cdot b_w \cdot l_c}{f_{yd}}; 0,002 \cdot b_w \cdot l_c\right)^{(b)}$		
Maximum longitudinal reinforcement ratio	ρ_{max}	4%		
Confining hoops within critical regions				
Minimum hoop rebar diameter	d_{bw}	6mm	max(6mm, d_{bL}/4)	
Maximum hoop spacing	s_w	b_0/2; 175mm; 8·d_{bL}	b_0/2; 200 mm; 9·$d_{bL,min}$	b_0/2; 175mm; 8·$d_{bL,min}$
Minimum mechanical percentage of reinforcement	ω_{wd}	0,08	0,05	0,08
Confinement integrity verification	$\alpha\cdot\omega_{wd}$	30·μ_ϕ·(ν_d+ω_v)·$\varepsilon_{sy,d}$·b_c/b_0-0,035	-	
Confining hoops over the rest of the wall (outside the critical region)				
In parts where the longitudinal reinforcement ratio ρ>2%	d_{bw}	max(6mm, d_{bL}/4)[(b)]		
	s	20·d_{bL}; b_w; 400mm[(b)]		

(a) According to EN 1998-1:2004 $h_{s,cl}$ is denominated as h_s
(b) According to EN 1992-1-1:2004[7]

The detailing of the web reinforcement in EN 1998-1:2004[2] and EN 1998-1-2:2023[4] does not differentiate between detailing within the critical region and detailing above it.

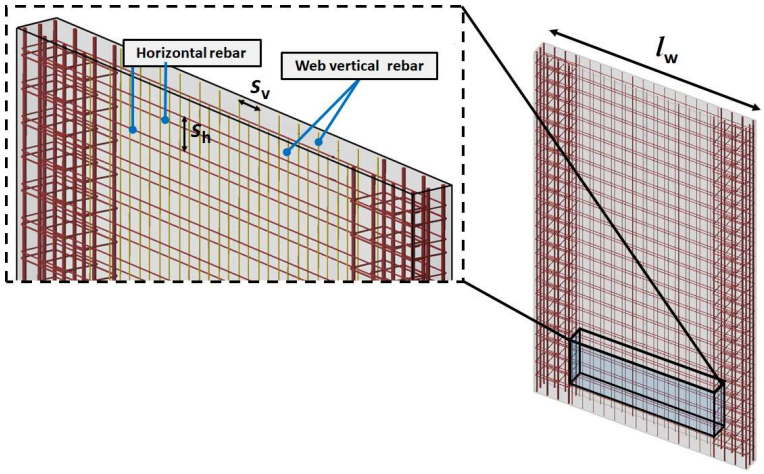


Figure 4-8: Wall boundary elements

The horizontal reinforcement in the web is also positioned within the boundary elements of the wall. These horizontal bars play a crucial role in absorbing shear forces. It is important to emphasize that within the critical region of the wall, the horizontal reinforcement that contributes to shear strength must maintain continuity. There should be no bars overlapping beyond the boundary zones, and their arrangement must exhibit uniform distribution.

Table 4-13: Detailing web elements

Detailing rules		**DCM**	**DC2**	**DC3**
Minimum web thickness	b_{w0}	max(0,15m; h_s/20)	max(0,15m; $h_{s,cl}$/20; l_w/40)	
Vertical bars				
Minimum reinforcement ratio	$\rho_{v,min}$	0,2%[(a)]		
Maximum reinforcement ratio	$\rho_{v,max}$	4,0%[(a)]		
In critical region				
Maximum spacing	s_v	min(3·b_{wo}; 400mm) [(a)]	300mm	250mm
Above the critical region				
Maximum spacing	s_v	min(3·b_{wo}; 400mm)[(a)]		
Horizontal bars[(b)]				
Minimum reinforcement ratio	$\rho_{h,min}$	max(0,1%; 0,25·ρ_v)[(a)]		
In the critical region				
Maximum spacing	s_h	400mm	300mm	250mm
Above the critical region				
Maximum spacing	s_h	400 mm[(a)]		

(a) According to EN 1992-1-1:2004[7]
(b) Hooks anchor the horizontal bars or bends in the confined core of the boundary elements.

In the critical region of the wall, the total vertical reinforcement ratio should not be smaller than 0,25%, and the horizontal reinforcement ratio at each wall face should not be smaller than 0,125%.

4.4.8 CHORD ROTATION

According to prEN 1998-1-2:2023[3], ensuring sufficient ductility and plastic rotation capacity in all critical regions of the wall is necessary. To achieve this, the product of the global displacement ductility factor, calculated as the product of the components of the behaviour factor (q_R) and (q_D), and the chord rotation at yield (θ_y) of each member end should not exceed the chord rotation for the Significant Damage (SD) Limit State (θ_{SD}) as defined in expression (4-14):

$$q_D \cdot q_R \cdot \theta_y \leq \theta_{SD} \tag{4-14}$$

To provide a visual representation of the chord rotation in structures, Figure 4-9 depicts the vectorial diagram of the chord rotation (θ_w) at the base of the wall since this structural element is defined as a cantilever wall.

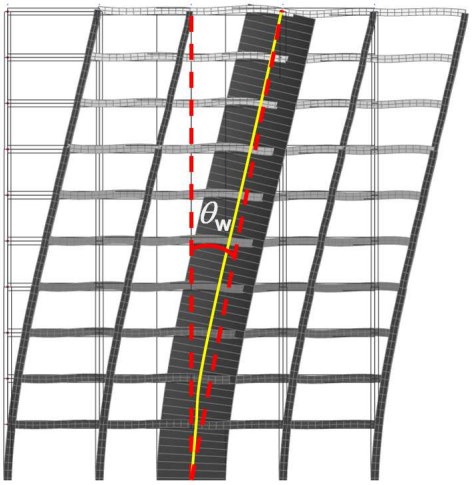


Figure 4-9: Vectorial representation of the chord rotation

The chord rotation of the shear span (L_V=M/V) at yield (θ_y) for walls of any cross-sectional shape may be evaluated according to expression (4-15) as proposed in FprEN 1998-1-1:2024[2]:

$$\theta_y = \phi_y \cdot \frac{L_V + a_l}{3} + \frac{\phi_y \cdot d_{bL} \cdot f_y}{8 \cdot \sqrt{f_c}} + 0{,}0011 \cdot \left(1 + \frac{h}{3{,}0 \cdot L_V}\right) \tag{4-15}$$

The parameter ϕ_y is the yield curvature from section analysis of the end section. The tension shift of the bending moment diagram (a_l) is defined in the second generation of Eurocode 2, prEN 1992-1-1:2021[18], as being the same expression as the current EN 1992-1-1:2004[5]. For walls, the shear span (L_V) adopted is the total clear height since there are no points of contra-flexion. The parameters (f_y) and (f_c) are the mean yield and compressive strength values of reinforcement steel and concrete, respectively. In the case of walls, the section height (h) is the wall length (l_w). For the verification of SD, the resistance of ductile mechanisms should be taken as given by expression (4-16) as defined in FprEN 1998-1-1:2024[2]:

$$\theta_{SD} = \frac{1}{\gamma_{Rd}} \cdot \left(\theta_y + \alpha_{SD,\theta} \cdot \theta_u^{pl}\right) \tag{4-16}$$

The parameter $\alpha_{SD,\theta}$ is the portion of the plastic part of the ultimate deformation $\theta_u^{pl}=\theta_u-\theta_y$ (computed with mean material properties) that corresponds to the attainment of SD, and γ_{Rd} is a partial factor on resistance at the SD limit state. In the current version of the FprEN 1998-1-1:2024[3], the reference value for ($\alpha_{SD,\theta}$) is assumed to be 0,5.

4.5 RESULTS AND DISCUSSION

The following section presents and discusses the results obtained from analysing the various parameters in section 4.4.

4.5.1 SEISMIC RESPONSE DEMANDS

Modal response spectrum analysis was performed independently for the ground excitation in two horizontal directions. The design spectra were used in both horizontal directions. The CQC rule for combining different modes was used according to EN 1998-1:2004[5]. The CQC rule accurately represents the combined response by considering linear and quadratic terms in mode combinations and explicitly accounts for mode interaction effects. The shear force at the structure's base (V_b) was obtained by modal response spectrum analysis for X and Y directions. The roof displacement ($d_{e,roof}$) was obtained by modal response spectrum analysis based on the design response spectrum, including torsional effects.

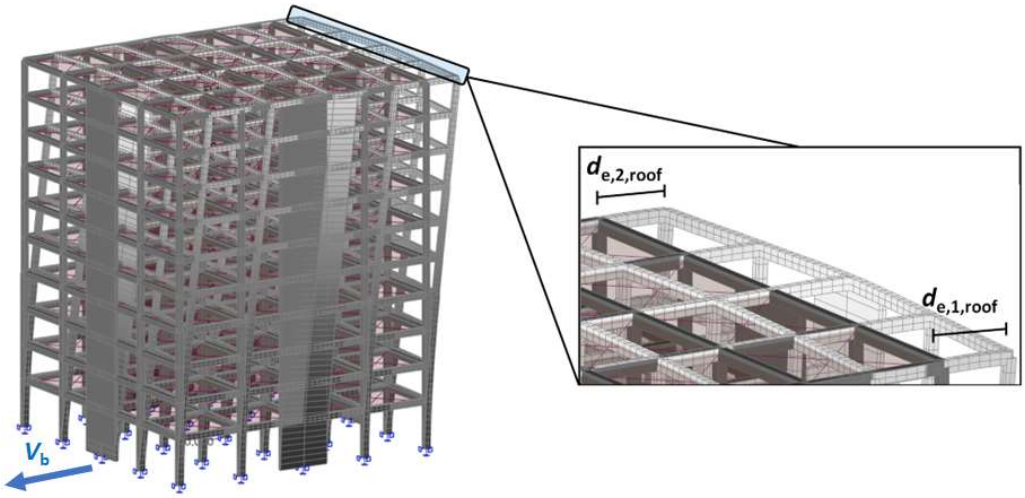


Figure 4-10: Scheme for base shear (V_b) and roof displacement ($d_{e,roof}$)

The roof displacement refers to the maximum displacement observed between the corners of the building. This is determined by comparing the two displacements: $d_{e,1,roof}$ and $d_{e,2,roof}$. The larger value between these two displacements is the roof displacement, as illustrated in Figure 4-10.

The basic modal properties of the building are summarised in Table 4-14.

Table 4-14: Seismic response parameters

Structure	T_1 (s)	T_2 (s)	T_3 (s)	$S_d(T_1)$ (m/s²)	V_b (kN)	$d_{s,roof}$ (m)
DCM.W.50.5	1,16	1,16	0,70	0,72	1582,16	0,081
DC2.W.50.5	1,11	1,11	0,90	1,13	2478,78	0,078
DC3.W.50.5	1,16	1.16	0,70	0,72	1582,16	0,081
DCM.W.75.5	1,16	1,16	0,70	1,08	2373,24	0,122
DC3.W.75.5	1,16	1,16	0,70	1,08	2373,24	0,122
DCM.W.50.10	1,95	1,95	0,70	0,43	1993,36	0,167
DC2.W.50.10	1,70	1,70	1,24	0,74	3633,88	0,153
DC3.W.50.10	1,90	1,90	1,34	0,44	1993,36	0,167
DCM.W.75.10	1,70	1,70	1,24	0,74	3628,89	0,228
DC3.W.75.10	1,67	1,67	1,23	0,75	3733,99	0,225

From Figure 4-11 and Figure 4-12, we can draw several conclusions:

a) Buildings designed for DCM and DC3 ductility classes show similar base shear values (V_b) and roof displacements for different building heights and seismic action levels.

b) Among the 5-storey buildings, DCM.W.50.5, DC2.W.50.5, and DC3.W.50.5 structures have a similar fundamental period (T_1). However, DCM.W.50.5 and DC3.W.50.5 (q=3,0) have a 3,67% higher roof displacement (d_s) than DC2.W.50.5 (q=2,0). This may be attributed to the fact that DC2 is subject to higher seismic actions, resulting in elements with cross-sections designed to withstand these forces being larger than those designed for DC3 and DCM.

c) DC2.W.50.5 (q=2,0) has a base shear value 36% higher than DCM.W.50.5 and DC3.W.50.5 (q=3,0), which is not far from the expected ratio of q-factor values.

d) The structure with the highest Vb value is DC3.W.75.10, and the structures with the lowest (V_b) values are DCM.W.50.5 and DC3.50.5.

e) The structures with the highest ($d_{s,roof}$) values are DCM.W.50.10 and DC3.W.75.10 (both 0,081 m), and the structure with the lowest (ds,roof) values is DC2.W.50.5.

f) DC2.W.50.5 has the lowest fundamental period values, indicating stiffer and thus more resistant to deformations than the other structures.

g) DCM.W.50.10 and DC3.W.50.10 have the highest fundamental period values, suggesting they are more flexible and thus more prone to higher deformations under applied loads.

h) Among the 10-storey buildings, DCM.W.50.10 and DC3.W.50.10 (q=3,0) have an 8,4% higher roof displacement (ds) than DC2.W.50.10 (q=2,0). As referred to in item b), This may be attributed to the fact that DC2 is subject to higher seismic actions, resulting in elements with cross-sections designed to withstand these forces being larger than those designed for DC3 and DCM.

i) The structure DC2.W.50.10 (q=2,0) has a base shear value 45% higher than DCM.W.50.10 and DC3.W.50.10 (q=3,0), which is expected since the fundamental period (T_1) of DC2.W.50.10 is 13% lower than DCM.W.50.10 and DC3.W.50.10.

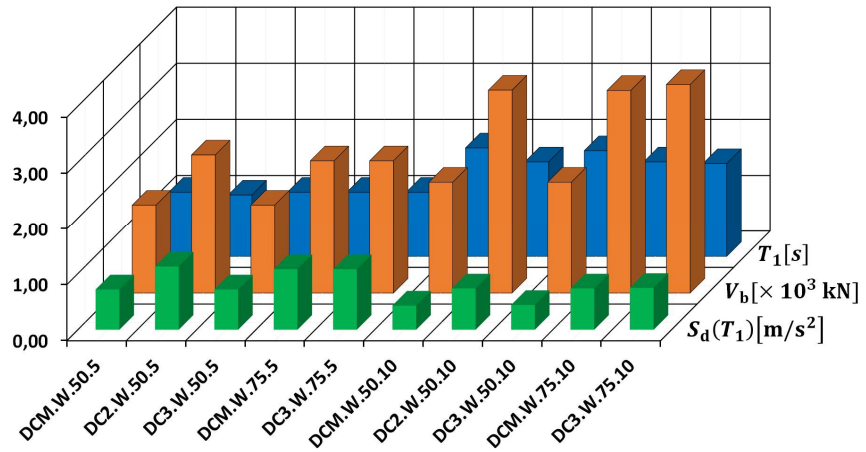


Figure 4-11: Design ground acceleration at fundamental period $S_d(T_1)$, Base shear (V_b) and fundamental period (T_1)

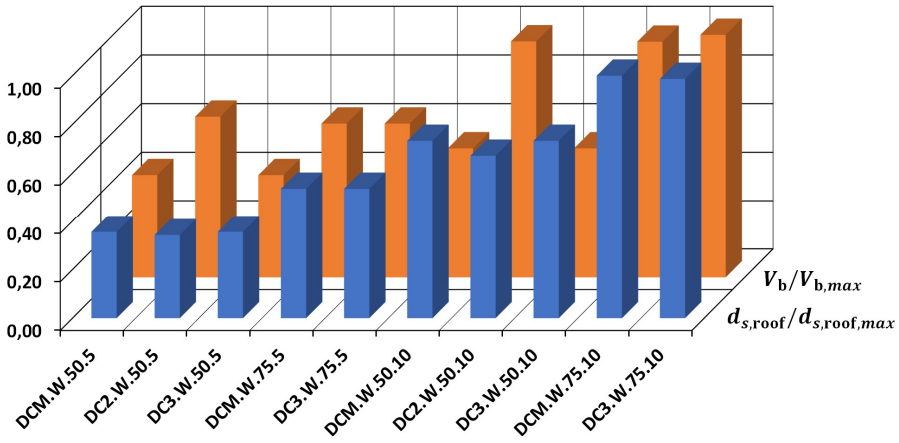


Figure 4-12: Normalised base shear ($V_b/V_{b,max}$) and roof displacement ($d_{s,roof}/d_{s,roof,max}$)

4.5.1.1 INTERSTOREY DRIFT RATIO

Figure 4-13(a) and (b) show the interstorey drift ratio (IDR) ratio for the 5 and 10-storey buildings, respectively, representing both the X and Y directions. The interstorey drift is the same in both the X and Y directions, given the symmetric and isotropic nature of the structures.

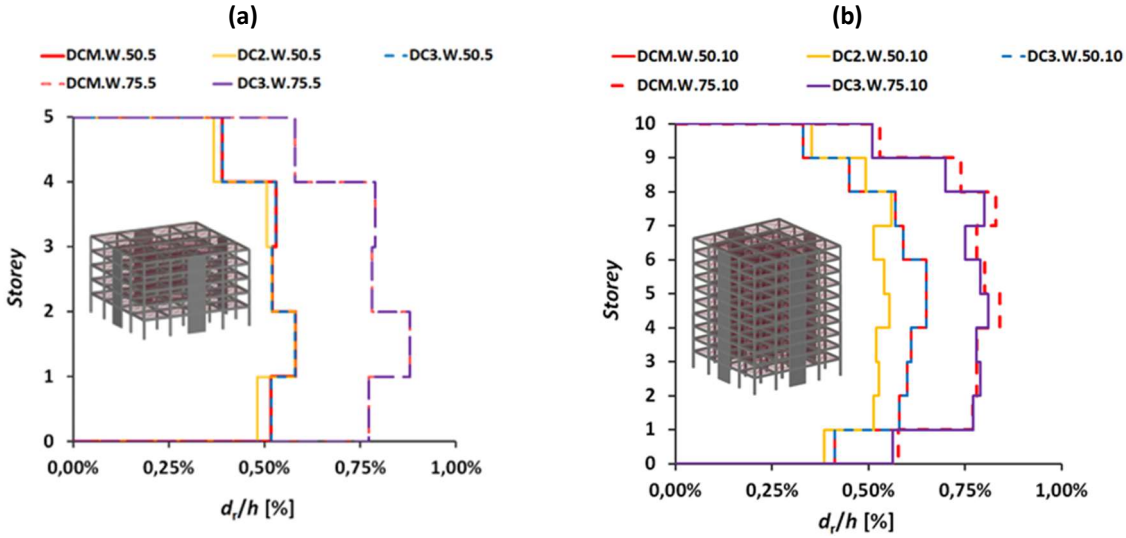


Figure 4-13: Interstorey drift ratio for X and Y directions: (a) 5-storey wall buildings and (b) 10-storey wall system buildings

From the analysis of the IDR profiles presented in Figure 4-13 for the X and Y alignments, the following observations can be made:

a) The absolute IDR differences between DCM.W.50.5 and DC3.W.50.5 (Figure 4-13 (a))are generally small. Most of the absolute differences are around 0,02% to 0,04%, indicating that the IDR for these structures are the same in practice. The largest absolute difference is 0,04% (at storey 1), and the smallest is 0,02% (for most storeys). It is observed that there are no differences between DCM.W.75.5 and DC3.W.75.5.

b) Based on this analysis, DCM.W.50.5, DC3.W.50.5, DCM.W.75.5, and DC3M.W.75.5 seem to have more consistent displacement patterns across different storeys. The largest absolute IDR difference of 7% occurs at first storey among DCM.W.50.5, DC3.W.50.5, and DC2.W.50.5, regarding the structures with the same (S_α).

c) In Figure 4-13(b), it can be observed that there are no IDR differences between DCM.W.50.10 and DC3.W.50.10. The absolute differences between DCM.W.75.10 and DC3.W.75.10 are around 3,8% at storey 6, which is consistently small. The largest absolute difference of 17% occurs at storey 6 among DCM.W.50.10, DC3.W.50.10, and DC2.50.10 in what concerns the structures with the same (S_α).

d) Based on the provided data of Figure 4-13 (b), the pair DCM.W.75.10 and DC3.W.75.10 exhibit different IDR patterns compared to the pair DCM.W.50.10 and DC3.W.50.10, which are consistently slight across all storeys.

4.5.1.2 ACCIDENTAL TORSIONAL EFFECTS

The strength of frames and slab rotations are affected by how stiff horizontal and vertical elements are relative to each other. To handle uncertainties in mass placement and seismic forces, an accidental eccentricity should be considered for the calculated centre of mass at each floor *i*, which displaces it from its nominal location in each direction. For regular buildings, the torsional effects are assumed to result from an accidental eccentricity (e_{ai}) of 5% of the floor dimension perpendicular to the direction of the seismic action, as prescribed by EN 1998-1:2004[5]and prEN 1998-1-2:2023[3]. Figure 4-14 illustrates the horizontal plan's torsional rotation angle (φ) for the regular building model and height. This aligns with the established as prescribed by EN 1998-1:2004[5]and prEN 1998-1-2:2023[3]. Figure 4-14 shows the buildings' resulting rotational angle (φ).

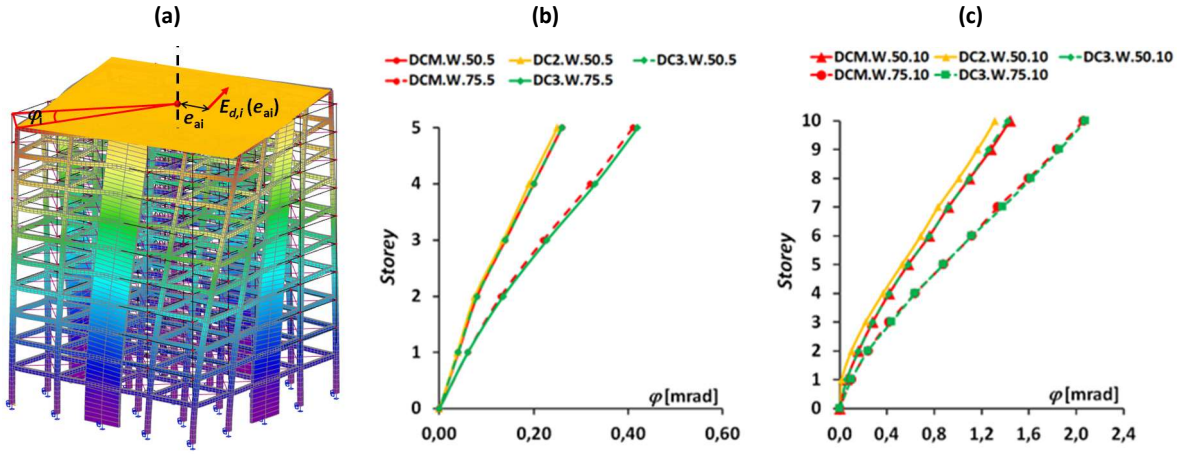


Figure 4-14: Torsional effects due to accidental eccentricities, e_{ai} (a) Illustration of the angle of torsion (φ) (b) 5-storey building angle of twist (c) 10-storey building angle of twist (φ)

Based on the analysis of the rotation angle profiles in Figure 4-14, the following observations can be made:

a) The profiles show that the structure's torsional deformation increases along the height of the building, as expected, with the highest value being at the top level since this has a smaller stiffness, consistent with the adopted design strategy for the present analysis. This finding is consistent with the conclusion reached by Özmen et al.[22].

b) Concerning the 5-storey building for DC3.W.75.5, the highest torsional angle (φ) is approximately 0,42 milliradians. The building with the lowest torsional angle is DC2.W.50.5 with 0,25 milliradians. These findings suggest that DC3.W.75.5 exhibits the highest average torsional angle across all storeys, while DC2.W.50.5 shows the lowest average.

c) The 10-storey building for DC3.W.75.10 displays a torsional angle of approximately 2,08 milliradians, suggesting a moderate torsional effect on the structural elements. DC2.W.50.10 exhibits the lowest torsional angle of approximately 1,31 milliradians, indicating a significant torsional impact. This highlights the importance of considering torsional effects, particularly in highrise structures.

These findings suggest that the relative stiffness of the horizontal to vertical resisting element significantly impacts the frames' torsional resistance and in-plane slabs' rotation, as Khatiwada et al.[23] pointed out. It should be noted that the distribution of lateral loads to the frame and the shear wall depends not only on their relative stiffness but also on the total number of storeys in a building, according to Khan et al.[24].

4.5.1.3 INTERSTOREY DRIFT SENSITIVITY COEFFICIENT θ

Figure 4-15 shows the interstorey sensitivity coefficient (θ_i) at all storeys calculated according to Equation (4-2) for DCM structures and Equation (4-3) for DC2 and DC3 structures.

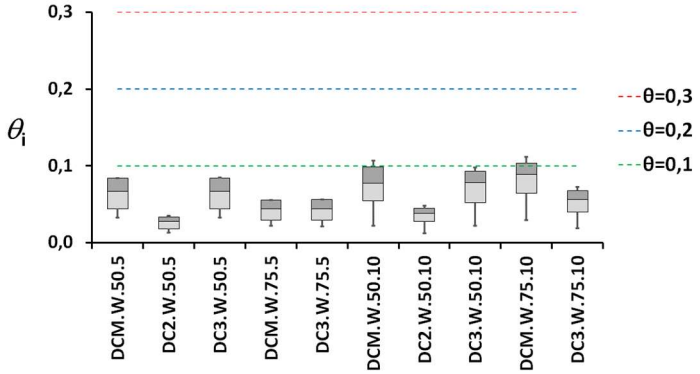


Figure 4-15: Interstorey sensitivity coefficient θ_i

a) In the context of the 5-storey buildings, DC3.W.50.5 boasts the highest sensitivity coefficient at 0,085, implying a significant influence within this category.

b) This means that, for the 5-storey buildings, the sensitivity coefficient is less than 0,1, which indicates that the change in internal forces due to lateral deflection is relatively small compared to the initial forces. In such cases, the (P-Δ) effect is negligible and does not significantly affect the structural analysis or design.

c) Among the 10-storey buildings, DCM.W.75.10 exhibits the highest sensitivity coefficient at 0,112, implying a substantial influence of second-order effects on structural behaviour. On the other hand, DC2.W.50.10 has the lowest maximum sensitivity coefficient at 0,048.

d) This means that, for some of the 10-storey buildings, the sensitivity coefficient is higher than 0,1, which indicates that the change in internal forces due to lateral deflection is relatively moderate compared to the initial forces for DCM.W.50.10 and DCM.W.75.10. In such cases, the (P-Δ) effects need to be considered.

It can be firmly concluded that walls are pivotal in controlling second-order effects observed in structures. Their substantial impact on structural behaviour, particularly deflection and stability, cannot be understated. Walls significantly influence the redistribution of forces and moments within a structure, affecting its overall response and performance.

4.5.2 WALL DESIGN AND DETAILING FOR LOCAL DUCTILITY

The design and detailing of the longitudinal and transverse reinforcement within the wall cross-section significantly influence the ductility behaviour of a structure. Consequently, meticulous design and precise wall detailing are essential for achieving the desired ductility levels in a structure.

4.5.2.1 NORMALIZED AXIAL FORCE

To ensure a minimum flexural ductility of the base of the wall, EN 1998-1:2004[5] and prEN 1998-1-2:2003[3] set upper limits on its axial force ratio (ν_d), as defined in Table 4-11.

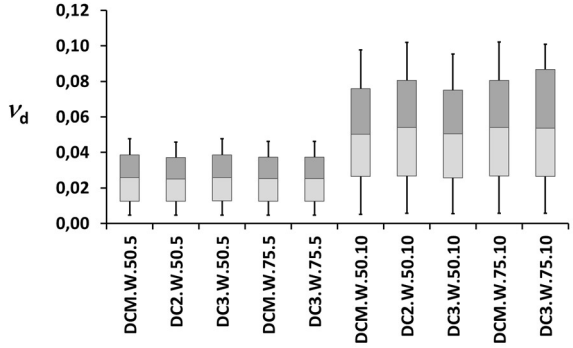


Figure 4-16: Normalized axial force ν_d

Based on the analysis of Figure 4-16, the following observations can be made:

a) Upon comparison, for the structures DCM.W.50.5, DC2.W.50.5, and DC3.W.50.5, the maximum values of a parameter, the normalized axial force, range from 0,046 to 0,048. The calculated averages for these scenarios fall within the range of approximately 0,025 to 0,0262, with consistent minimum values at 0,005.

b) Conversely, for the structures DCM.W.75.5 DC3.W.75.5, the normalized axial force extends to 0,048 and 0,046, respectively. The computed averages for these scenarios hover around 0,0254 and 0,0252, while the minimum values remain at 0,005.

c) Upon comparison of the 10-storey buildings, it is evident that the numerical values within each scenario are closely situated. Particularly, within the buildings DCM.W.50.10, DC2.W.50.10, and DC3.W.50.10, the maximum values range from 0,095 to 0,102, showcasing the peak performance concerning the parameter under scrutiny.

d) It should be noted that these values closely align with one another due to the central placement of the walls along the principal axis and the regularity of the structures in terms of height and plan, as can be seen in Figure 4-1.

4.5.2.2 WALL BENDING ACTION EFFECTS

The subchapter is dedicated to analysing the bending moment diagram. The main objective is to compare the parameter of the area of the bending moment diagram from the analysis. ($A_{M_{Edw}}$), with the area of the design bending moment diagram ($A_{M_{Edw}}$). On a preliminary note, it is important to point out the differences in the notation standards in the design. EN 1998-1:2004[5] designates the wall design bending moment as (M_{Ed}), whereas prEN 1998-1-2:2023[3] uses (M_{Edw}). The wall bending moment will be consistently defined as (MEdw) to maintain clarity in the current analysis and avoid ambiguities.

According to EN 1998-1:2004[5], the design of the bending moment diagram depends on the tension shift (a_l). The analysis is performed for the walls with ductility class DCM below the tension shift (a_l), as illustrated in Figure 4-17(a) and (b). In the present analysis, the shift is considered for two situations: one with a strut inclination of θ=22° and the other with a strut inclination of θ=45°.

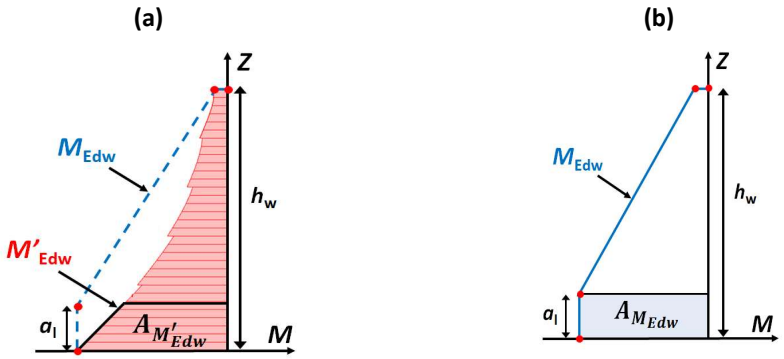


Figure 4-17: Scheme of the different areas to analyse below al according to EN 1998-1:2004[5](a) bending moment diagram from the analysis area ($A_{M_{Edw}}$) (b) Design bending moment diagram area ($A_{M_{Edw}}$)

According to EN 1998-1-2:2023[3], the bending moment diagram shift depends on the maximum between the critical height (h_{cr}) and the tension shift (a_l) for the walls with ductility class DC2 and DC3 as illustrated in Figure 4-18(a) and (b).

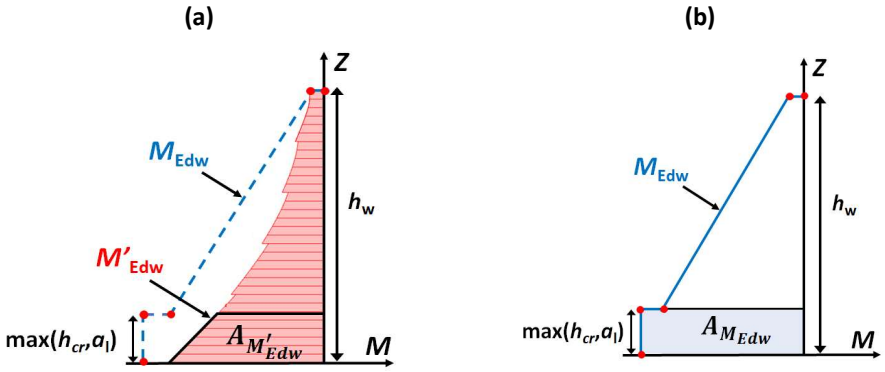


Figure 4-18: Scheme of the different areas to analyse below max(h_{cr}, a_l) according to prEN 1998-1-2:2023[4](a) bending moment diagram from the analysis area ($A_{M_{Edw}}$)(b) Design bending moment diagram area ($A_{M_{Edw}}$)

It should be noted that the (h_{cr}) does not depend on the angle of the strut. It is also considered a situation for al corresponding to a strut of θ=22° and another for a struct of θ=45°. Table 4-15 shows the critical height (h_{cr}) values and the tension shift (al) for a strut of θ=22° and other for a struct of θ=45°.

Table 4-15: Critical height (h_{cr}) and tension shift (a_l) for θ=22° and θ=45°

Structure	h_w m	h_s m	l_w m	h_{cr} m	a_l for θ=22° m	a_l for θ=45° m
DCM.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DC2.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DC3.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DCM.W.75.5	16	4,0	4,0	4,0	8,24	3,33
DC3.W.75.5	16	4,0	4,0	4,0	8,24	3,33
DCM.W.50.10	31	4,0	4,5	5,2	8,98	3,63
DC2.W.50.10	31	4,0	4,5	5,2	9,22	3,73
DC3.W.50.10	31	4,0	4,5	5,2	9,22	3,73
DCM.W.75.10	31	4,0	4,5	5,2	8,98	3,63
DC3.W.75.10	31	4,0	4,5	5,2	9,22	3,73

The following figures make the comparison for ($A'_{M_{Edw}}$), and for ($A_{M_{Edw}}$), for a struct of θ=22°as illustrated in Figure 4-19(a) and θ=45°as depicted in Figure 4-19(b). It should be noted that for DC2 and DC3, in some cases, hcr is higher than al for θ=45°, as shown in Table 4-15.

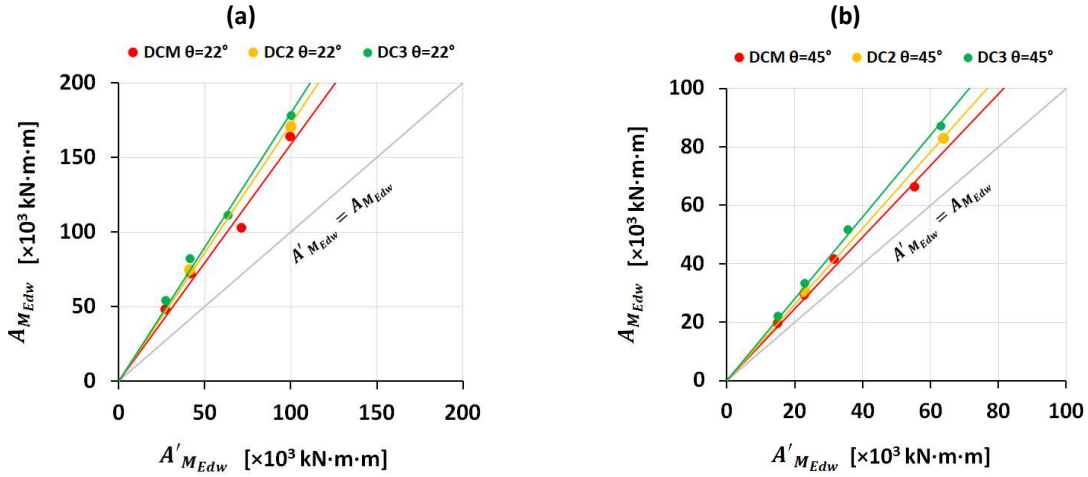


Figure 4-19: Wall bending moment area (a) $A'_{M_{Edw}}$ *vs* $A_{M_{Edw}}$ for θ=22° (b) $A'_{M_{Edw}}$ *vs* $A_{M_{Edw}}$ for θ=45°

Examining Figure 4-19 leads to the following observations:

a) As anticipated, the DC2 and DC3 structures demonstrate higher ($A_{M_{Edw}}$) values compared to the DCM structures. This disparity is attributed to the distinct calculation models for design bending moment (M_{Edw}), as specified in EN 1998-1:2004[2] and prEN 1998-1-2:2023[4], showcasing the significant impact of tension shift on (M'_{Edw}) variations among these structural configurations.

b) Concerning Figure 4-19(a), the distances of each data point from the line ($A'_{M_{Edw}} = A_{M_{Edw}}$) revealing variations ranging from approximately 20,34% to 77,77%. DCM.W.50.5 and DCM.W.75.5, with smaller distances in the range of about 20,34% to 30,04%, demonstrated a remarkably close alignment between ($A'_{M_{Edw}}$), and ($A_{M_{Edw}}$), as it would be expected due to the calculation model for the design bending moment M_{Edw} preconizes by EN 1998-1:2004[5].

c) Regarding structures of Figure 19(b), the distances of each data point from the line ($A'_{M_{Edw}} = A_{M_{Edw}}$) DCM.W.75.10 demonstrated a relatively smaller distance of about 19,39%, indicating a closer alignment between ($A'_{M_{Edw}}$), and ($A_{M_{Edw}}$), suggesting an efficient design for this particular structure. Conversely, structures like DC3.W.50.5 and DC3.W.75.5 exhibited larger distances, approximately 43,11% to 44,38%, signifying a notable divergence.

Above the tension shift (a_l), the parameter ($A'_{M_{Edw}}$), is defined as illustrated in Figure 20(a) and ($A'_{M_{Edw}}$), as depicted in Figure 20(b) for the structures with ductility class DCM as defined in EN 1998-1[5]:2004.

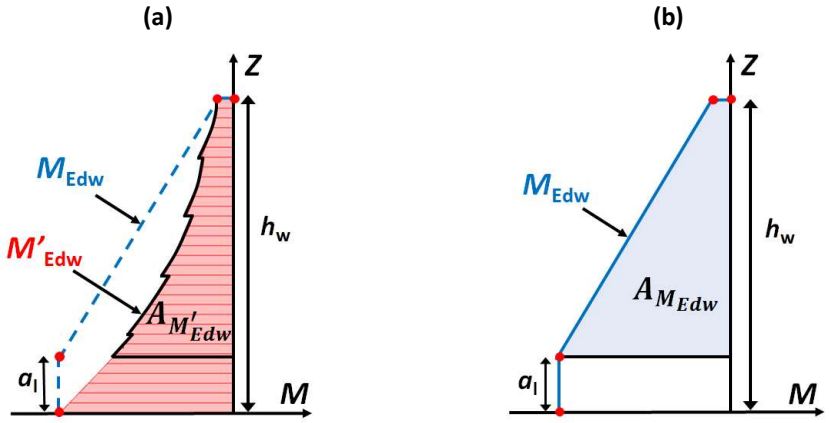


Figure 4-20: Scheme of the different areas to analyse above al according to EN 1998-1:2004[5](a) bending moment diagram from the analysis area ($A'_{M_{Edw}}$)(b) Design bending moment diagram area ($A_{M_{Edw}}$)

For structures with ductility class DC2 and DC3 below the maximum of hcr and al is defined for ($A'_{M_{Edw}}$), as illustrated in Figure 4-21(a) and for ($A_{M_{Edw}}$), as depicted in Figure 4-21(b). It is also considered a situation for al corresponding to a strut of θ=22° and another for a struct of θ=45°.

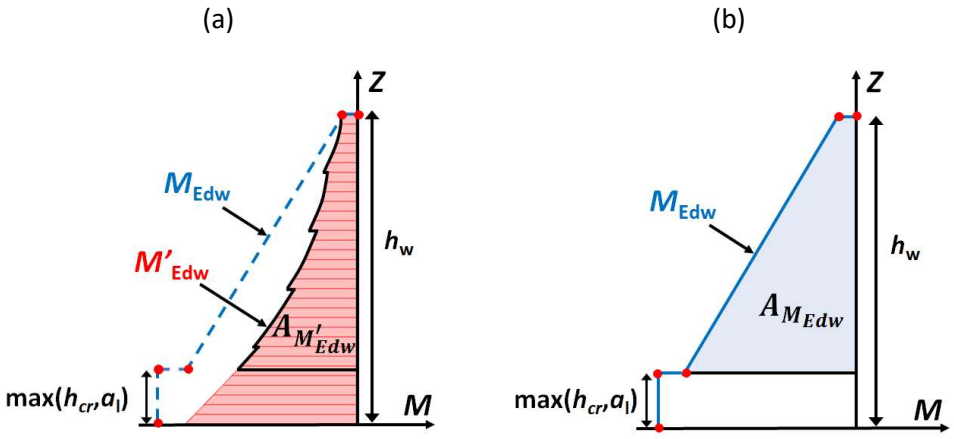


Figure 4-21: Scheme of the different areas to analyse above max(h_{cr},a_l) according to EN 1998-1-2:2023[3](a) bending moment diagram from the analysis area ($A'_{M_{Edw}}$)(b) Design bending moment diagram area ($A_{M_{Edw}}$)

The following figure makes the comparison between ($A'_{M_{Edw}}$), and ($A_{M_{Edw}}$) above (a_l) for DCM and the maximum between (h_{cr}) and (a_l) for DC2 and DC3, considering a struct of θ=22°as depicted in Figure 4-22(a) and θ=45°as depicted in Figure 4-22(b).

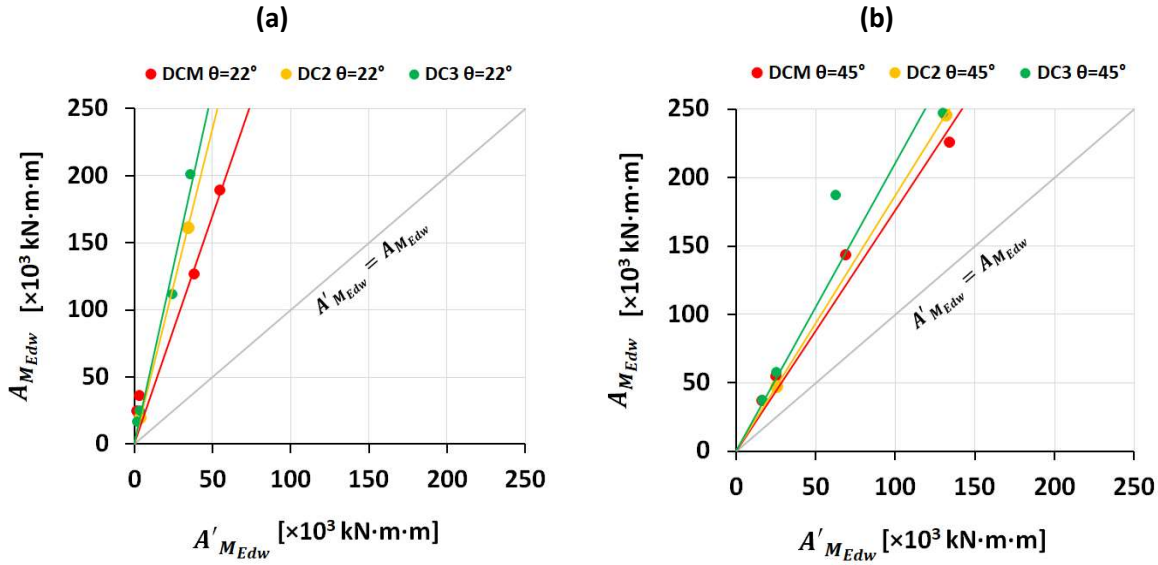


Figure 4-22: Wall bending moment area (a) $A'_{M_{Edw}}$ *vs* $A_{M_{Edw}}$ for θ=22° (b) $A'_{M_{Edw}}$ *vs* $A_{M_{Edw}}$ for θ=45°

From the data presented in Figure 4-22, it can be inferred the following:

a) Regarding Figure 4-22(a) for ($A'_{M_{Edw}}$) the maximum value is 55,0×10^3 kN·m·m, observed in the case DCM.W.75.10, while the minimum is 2,4×10^3 kN·m·m, seen in multiple cases such as DCM.W.50.5 and DC3.W.50.5. The average value for ($A'_{M_{Edw}}$)across all cases is approximately 22,1×10^3 kN·m·m. On the other hand, for ($A_{M_{Edw}}$) the maximum value is 201,0×10^3 kN·m·m in the case DC3.W.75.10, whereas the minimum value is 16,2×10^3 kN·m·m, observed in DC3.W.50.5. The average ($A_{M_{Edw}}$) value across all cases is approximately 83,0×10^3 kN·m·m.

b) Concerning the Figure 4-22(b) when considering the maximum values, ($A_{M_{Edw}}$)reaches its highest point of 246,9×10^3 kN·m in the case DC3.W.75.10, while ($A'_{M_{Edw}}$) peaks at 134,7×10^3 kN·m·m in DCM.W.75.10. Conversely, the minimum values for ($A_{M_{Edw}}$) (17,9×10^3 kN·m·m) and ($A'_{M_{Edw}}$) (16,9×10^3 kN·m·m) are both found in the case DCM.W.50.5. Lastly, in terms of averages, the average ($A_{M_{Edw}}$) value across all cases is approximately 96,7×10^3 kN·m·m, while the average ($A'_{M_{Edw}}$) value is about 55,1×10^3 kN·m·m.

c) According to the data presented in Figure 4-22(a) and (b), when evaluating a strut with an angle of inclination θ=22°, the design bending moment appears significantly higher than the case with θ=45°. Specifically, the average design bending moment in Figure 4-22(a) is substantially greater than that in Figure 4-22(b) by approximately 40,3%.

4.5.3 BENDING RESISTANCE

For the bending resistance calculation, the models described in Table 4-11 were used to calculate the resistant bending moment (M_{Rdw}).

Figure 4-23 illustrates the design bending moment (M_{Edw}) against the resistant moment (M_{Rdw}) that concerns the zone below the shift (a_l) for DCM and below the maximum between the critical height (h_{cr}) and the shift (a_l) for DC2 and DC3. Figure 4-23(a) concerns the walls designed considering a strut of θ=22°, and Figure 4-23(b) refers to struct θ=45°.

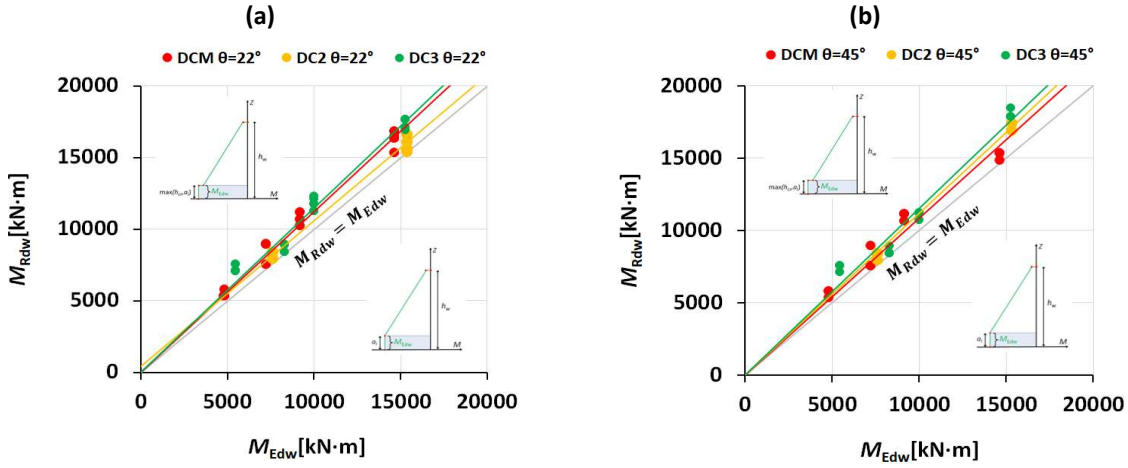


Figure 4-23: Wall bending moment below tension shift (a_l) (a) M_{Edw} *vs* M_{Rdw} for θ=22° (b) M_{Edw} *vs* M_{Rdw} for θ=45°

Based on Figure 4-23(a) and (b), some observations can be drawn:

a) The moments exhibit a nearly linear relationship close to the threshold line M_{Rdw}=M_{Edw}, indicating that the walls' reinforcement is well-optimised.

b) In some walls, the bending moment (M_{Ed}) is higher for DC3 and DCM structures. This can be attributed to the fact that the model used for calculating the design bending moment according to prEN 1998-1-2:2023[4] is higher than the one preconized for EN 1998-1:2004[2].

c) The binomial relation of (M_{Rdw}-M_{Edw}) for Figure 4-23(b) is similar to Figure 4-23(a), which means that the bending resistance in the critical zone in practical terms is the same. The main difference is that this reinforcement is more prolonged for the θ=22°, which means more reinforcement consumption.

Figure 4-24 illustrates the design bending moment (M_{Edw})against the resistant moment (M_{Rdw}) for the area above the shift (a_l) for DCM and above the maximum between the critical height (h_{cr}) and the shift (a_l) for DC2 and DC3. Figure 4-24(a) concerns the walls designed considering a strut inclination of θ=22°, and Figure 4-24(b) refers to struct inclination θ=45°.

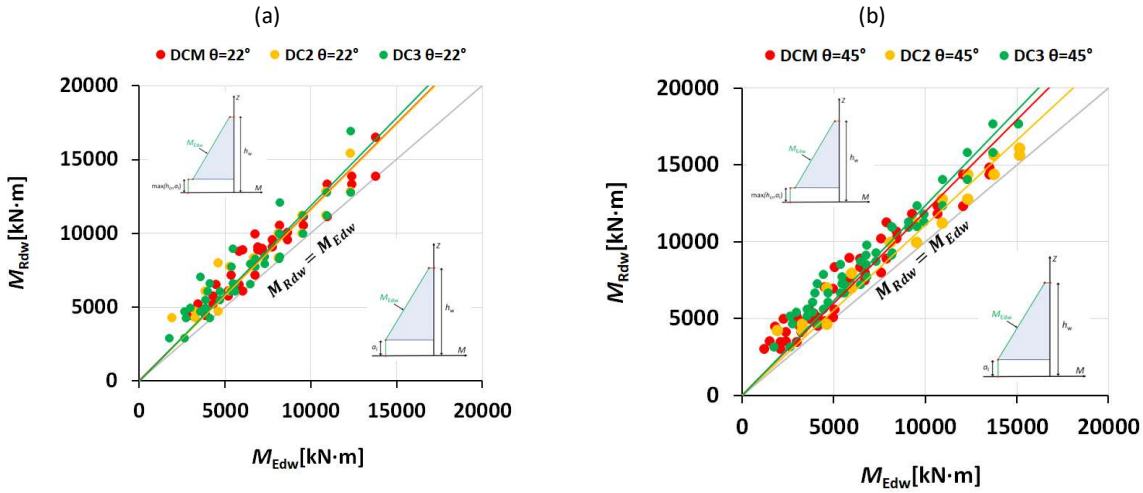


Figure 4-24: Wall bending moment above tension shift (a_l) (a) M_{Edw} *vs* M_{Rdw} for θ=22° (b) M_{Edw} *vs* M_{Rdw} for θ=45°

From Figure 4-24, some observations can be drawn:

a) The moments display a nearly linear relationship near the threshold line ($M_{Rdw} = M_{Edw}$), indicating that the wall reinforcement is well-optimized.

b) In some walls, the bending moment (M_{Ed}) is higher for DC3 than for DCM structures. This can be attributed to using a model for calculating the design bending moment as per prEN 1998-1-2:2023[3], which is higher than the one recommended by EN 1998-1:2004[5]. However, it is worth noting that the trend lines for both models are quite close.

c) The relationship between (M_{Rdw}) and (M_{Edw}) for Figure 4-24(a) is similar to Figure 4-24(b). There is evident greater dispersion for θ=45° as depicted in Figure 4-24 (b). This may be explained by the smaller tension shift, resulting in a more diversified quantity of reinforcement in the boundary elements.

4.5.3.1 DETAILING OF WALLS LONGITUDINAL REINFORCEMENT IN THE BOUNDARY ELEMENTS

Figure 4-25(a) shows the reinforcement ratio normalised to the maximum (ρ/ρ_{max}), and Figure 4-25(b) shows the reinforcement ratio normalised to the minimum reinforcement (ρ/ρ_{min}) allowed in EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] for the critical zone reinforcement, respectively.

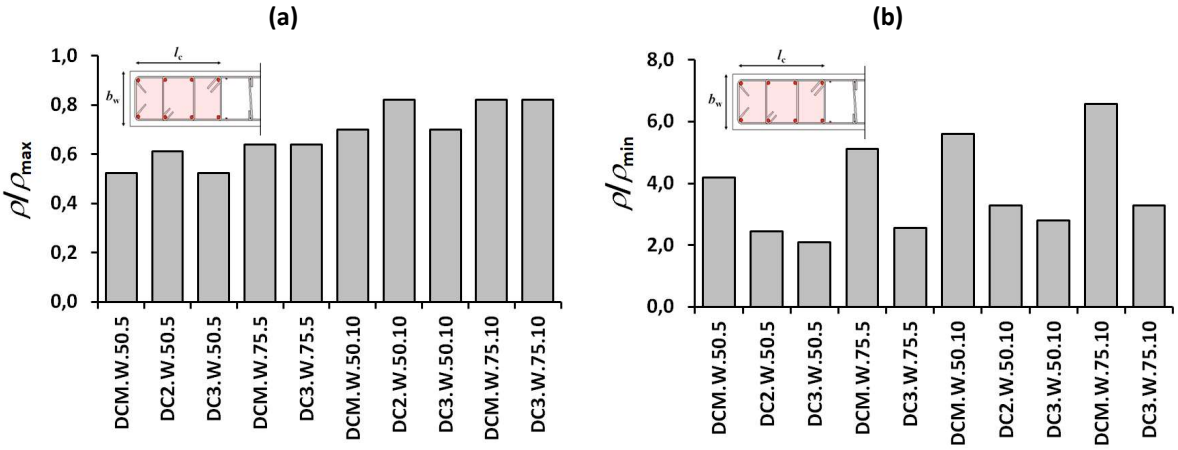


Figure 4-25: Normalized reinforcement ratio of boundary elements: (a) Top normalized reinforcement ratio and (b) Top minimum normalized reinforcement ratio

From the figures above, some observations can be made:

a) Regarding top reinforcement, both DCM and DC3 exhibit a closer approximation to the maximum reinforcement allowed compared to DC2, as seen in Figure 4-25(a). Similarly, in Figure 4-25(b), DCM and DC3 show a closer approximation to the minimum permitted reinforcement than DC2.

b) Figure 25(b) indicates that bottom reinforcement ratios are far from the maximum allowed. However, some wall boundary elements in DCM and DC3 have a higher approximation to the maximum permitted reinforcement ratio than DC2.

c) While one might expect DC3 to always have a higher reinforcement ratio due to its likely higher ductility class, the bar plot does not show a simple linear increase from DCM to DC2 to DC3. In some cases, DC2 has a higher ratio than DC3, which indicates that the design requirements for these classes are nuanced and not solely based on increasing ductility demands. In the present work, the specific behaviours of structures under seismic loading result in different reinforcement strategies optimised for each class.

d) It should be noticed that the length of the confined boundary element(l_c) is not the same for all structures. The geometrical rules are the same for DCM, DC2 and DC3, but DCM EN 1998-1:2004[5] demands verification of its structural integrity where ε_{cu2}>0,0035.

4.5.3.2 DETAILING OF CONFINING HOOPS IN THE CRITICAL REGION

EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] specify that the maximum confining hoop spacing ($s_{w,max}$) is determined by a coefficient multiplied by the wall boundary elements longitudinal rebar diameter (d_{bL}) a and the width (b_0) of the confined zone boundary element as shown in Figure 4-7. The values for this coefficient and the resulting maximum hoop spacing are specified in Table 12 for DCM, DC2, and DC3. The following figures compare these multiples with the adopted hoop spacing ($s_{w,adopted}$).

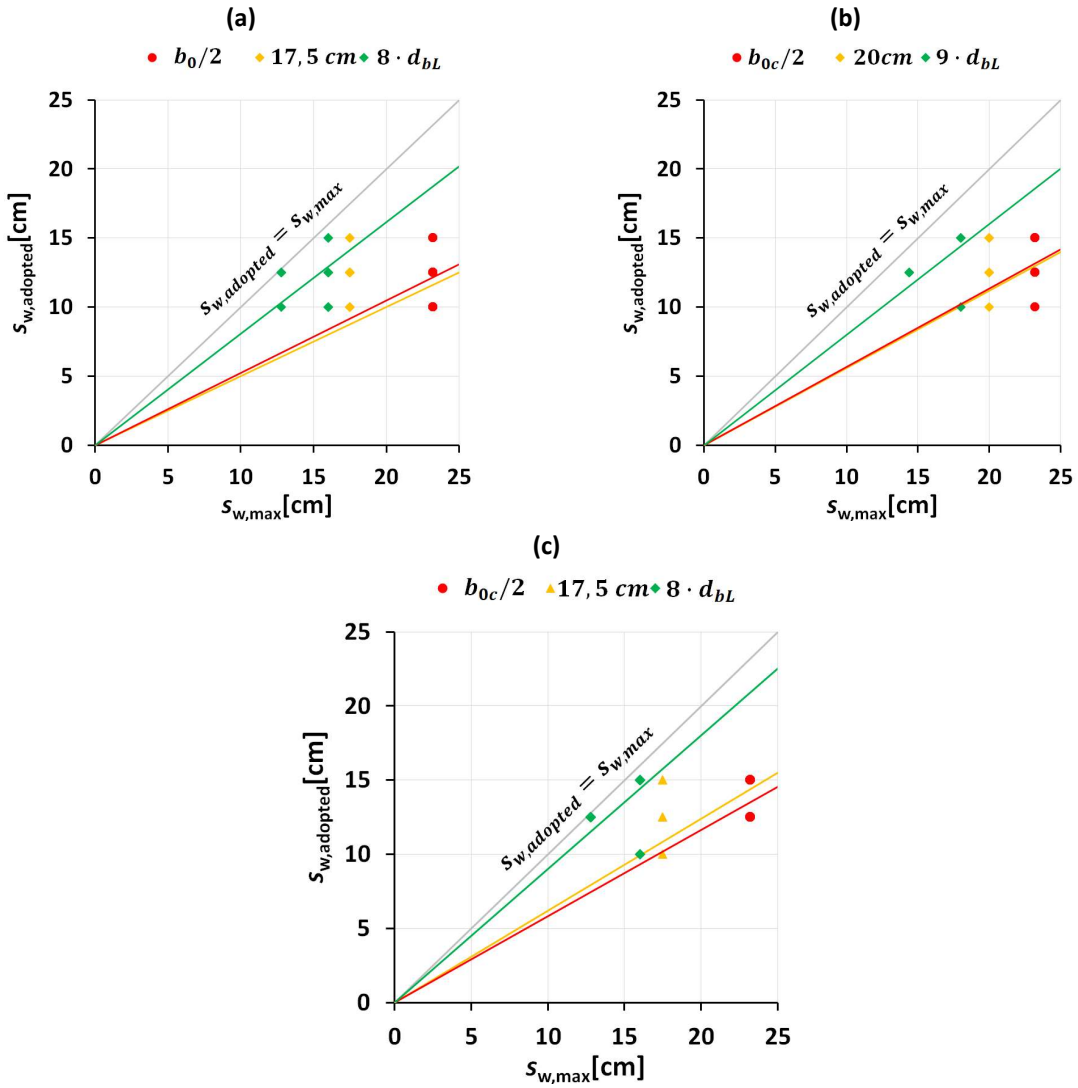


Figure 4-26: Hoop spacing against the multiples: (a) DCM (b) DC2, and (c) DC3

Figure 4-26 provides the following observations regarding the maximum boundary elements confining hoops spacing ($s_{w,max}$) of the walls for different structural systems:

a) For all the structures, Figure 4-26 demonstrates that the diameter of the longitudinal reinforcement of the boundary elements governs the spacing of the confining hoops.

b) The width of the confined zone boundary element has less impact on the spacing of the coffining hoops.

c) In the transition from (b_0/2) to (8·d_{bL}), DCM decreased by approximately 33%, DC2 increased by about 14%, and DC3 decreased by 25%. The comparison to (9·d_{bL}) saw DCM experiencing a reduction of around 47%, DC2 decreasing by 20%, and DC3 decreasing by 25%. These percentage changes demonstrate the variations in values for these specific prefixes between different conditions.

4.5.4 SHEAR RESISTANCE IN WALLS – CAPACITY DESIGN

The capacity design of columns in shear is presumed that plastic hinges form at the member ends, and the equilibrium of moments suffices to establish the maximum possible shear force in the wall. The EN 1998-1:2004[5] standard employs the notation (V'$_{Ed}$) for the shear diagram resulting from analysis and (V_{Ed}) for the design shear diagram. On the other hand, the EN 1998-1-2:2023[3] standard adopts (V'$_{Edw}$) for the shear diagram from analysis and (V_{Edw}) for the design shear diagram. To eliminate any potential confusion in the analysis, the shear diagram derived from the analysis is referred to as (V'$_{Edw}$), while the design shear diagram is designated as (V_{Edw}).

Figure 4-27 shows the relationship between the design in shear (V_{Edw}) of the shear force from the analysis for the seismic design situation (V'$_{Edw}$) and the shear resistance (V_{Rd}), for the for the walls in analysis.

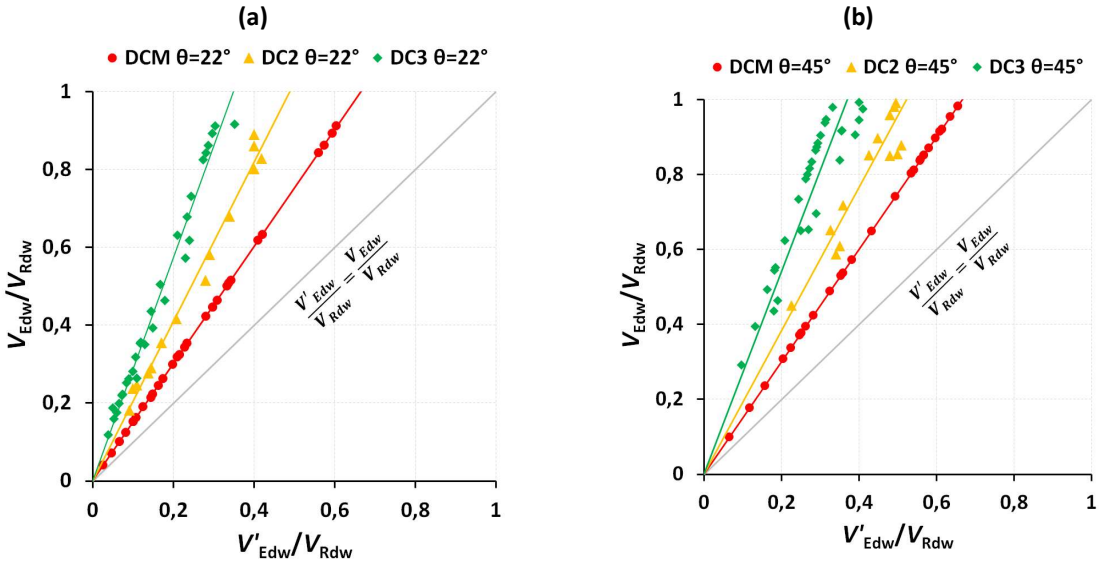


Figure 4-27: Normalized shear capacity design of walls (a)Strut inclination of θ=22° (b) Strut inclination of θ=45°

Upon analysing Figure 4-27, several observations can be made:

a) The relation between (V'$_{Edw}$/V_{Rdw}) and (V_{Edw}/V_{Rdw})for DCM structures follows a straight line since the magnification factor(ε) is 1,5, as expected.

b) For the structures with ductility classes DC2 and DC3, the relation between (V'$_{Edw}$/V_{Rdw}) and (V_{Edw}/V_{Rdw}) does not follow a straight line since these are influenced by the magnification factor ε(z) defined by the expression (4-11) according to EN 1998-1-2:2023[3].

c) The DC3 structures exhibit a steeper slope than the DC2 structures, primarily due to the difference in the behaviour factor. Specifically, the behaviour factor is q=2,0 for DC2 and q=3,0 for DC3.

d) It is also observed in Figure 4-27 (a) and (b) that the strut inclination has little influence on the relation between (V'_{Edw}/V_{Rdw}) and (V_{Edw}/V_{Rdw}).

e) It is also observed from Figure 4-27(a) and (b) that the inclination of the strut has little influence on the relationship between (V'_{Edw}/V_{Rdw}) and (V_{Edw}/V_{Rdw}).

4.5.5 DETAILING OF WALLS TRANSVERSE REINFORCEMENT

Horizontal reinforcement in the web anchored at the edges of the wall. These horizontal bars provide the wall shear resistance.

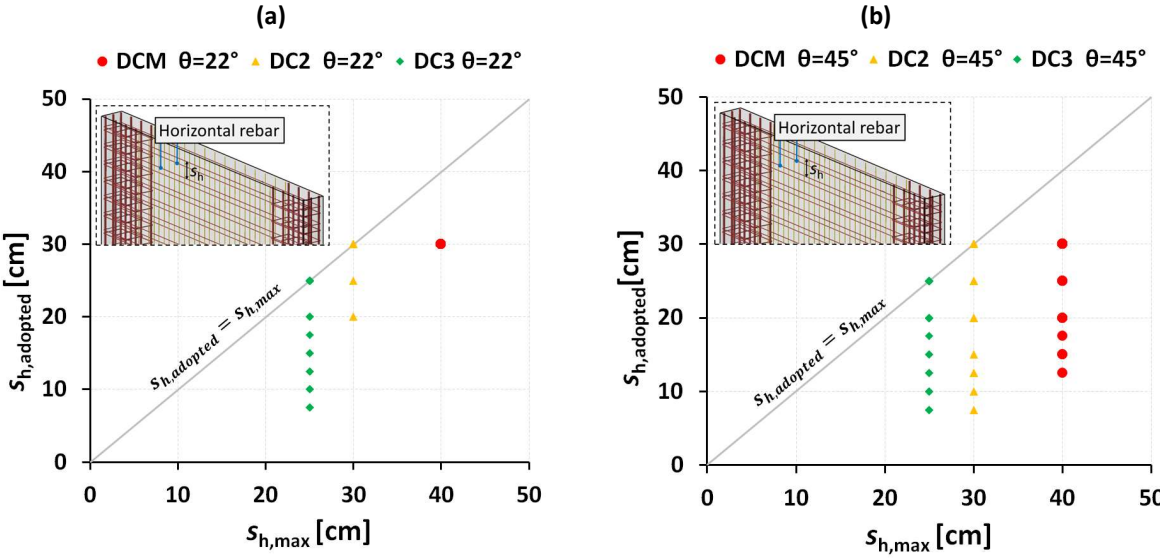


Figure 4-28: Horizontal reinforcement (a) Strut inclination of θ=22° (b) Strut inclination of θ=45°

Figure 4-28 provides the following observations regarding the horizontal bar spacing (s_h):

a) There is a noticeable distribution pattern where, generally, DCM points are closer to the line of maximum allowed spacing, while DC2 and DC3 points are further below the line. This could imply that designs conforming ductility classes preconised in prEN 1998-1-2:2023[3] (DC2, DC3) tend to use more closely spaced reinforcement than required by the maximum limits

b) For the DCM structures calculated considering strut inclinations of θ=22° and θ=45°, the maximum horizontal rebar spacing ($s_{h,max}$) preconised in EN 1998-1:2004[5] and EN 1992-1-1:2004[7] did not govern the value of the shear resistance (V_{Rdw}) as shown in Figure 28(a) and (b).

c) Regarding the structures DCM calculated with a strut of inclination of θ=22° the maximum due to the exigences shear resistance (V_{Rdw}), it was possible to use horizontal rebars spacing ($s_{h,adopted}$) close to the maximum spacing allowed per prEN 1998-1-2:2023[3] and EN 1992-1-1:2004[7].

d) For the DC2 and DC3 structures calculated considering strut inclinations of θ=22° and θ=45°, the maximum horizontal rebar spacing ($s_{h,max}$) preconised in EN 1998-1-2:2023[3] and EN 1992-1-1:2004[7] did not govern the value of the shear resistance (V_{Rdw}) as shown in Figure 28 (a) and (b).

4.5.6 CHORD ROTATION

Figure 4 29 represents the chord rotation verifications for seismic load acting in one direction.

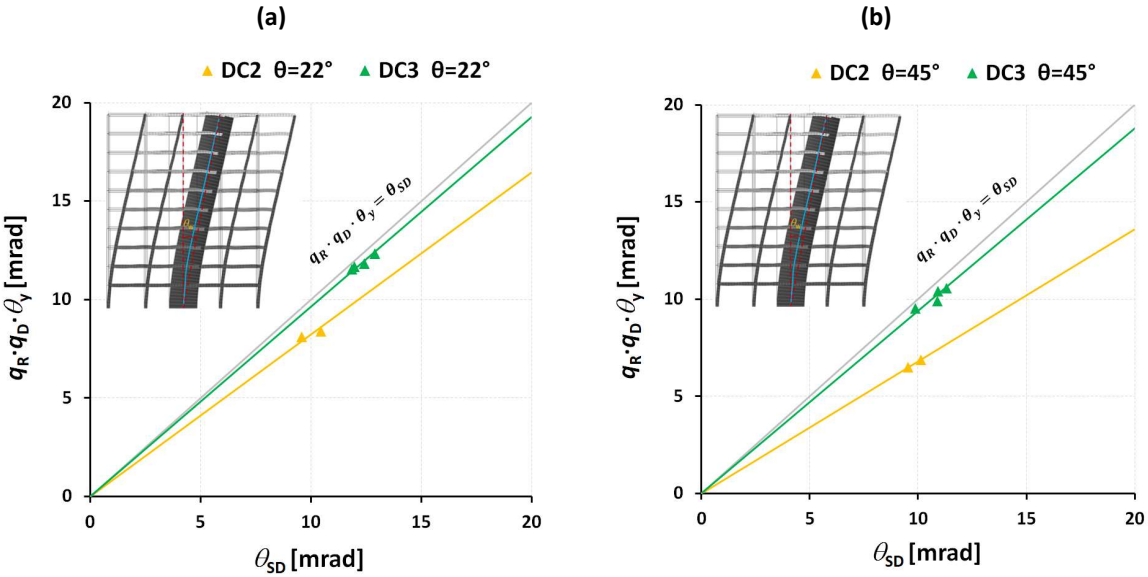


Figure 4-29: Verification of the chord rotation capacity (a) Strut inclination of θ=22° (b) Strut inclination of θ=45°

From Figure 4-29, the following observations can be made regarding the chord rotation:

a) The analysis of chord rotation control demonstrates that, among the structures calculated with different strut inclinations, those with a strut inclination of θ=22° exert greater influence on the control of the chord rotation than those with a strut inclination of θ=45°. This discrepancy arises mainly from the influence of the tension shift (a_l) in chord rotation at yield (θ_y), as depicted in expression(4-15).

b) While the plot does not show a dramatic difference between DC2 and DC3 in their respective curves, it does suggest some distinction in the performance levels that can be expected based on the selected ductility class. It can be seen that due to prEN 1998-1-2:2023[3] provision for local ductility, DC3 is pore ductile than DC2, highlighting the differences between the two ductility classes

c) The underlying design philosophy of the prEN 1998-1-2:2023[3] aims for a performance-based approach, where structures are designed for survival and functionality post-earthquake, the SD limit state. This approach would naturally result in higher deformation capabilities for higher ductility DC3 than for DC2.

4.5.7 STRUCTURES INVENTORY

A structure inventory, also known as a bill of quantities (BOQ), is a detailed document that provides a comprehensive list of all the materials and quantities for each structure, as represented in Table 4-16.

Table 4-16: Bill of quantities

Structure	Concrete	Reinforcement steel									
		Strut inclination θ=22°					Strut inclination θ=45°				
	m^3	Total Long. ton	Density Long. kg/m³	Total Transv. ton	Density Transv. kg/m³	Total Density kg/m³	Total Long. ton	Density Long. kg/m³	Total Transv. ton	Density Transv. kg/m³	Total Density kg/m³
DCM.W.50.5	19,20	1,9	97,5	0,66	34,2	131,7	1,8	94,2	0,73	38,3	132,5
DCM.W.75.5	19,20	2,1	109,8	0,8	41,7	151,4	2,0	103,3	1,1	57,8	161,1
DCM.W.50.10	41,85	4,3	101,8	1,4	34,0	135,8	3,9	93,1	1,6	37,6	130,7
DCM.W.75.10	41,85	5,1	121,1	1,4	34,2	155,3	4,7	112,7	1,9	44,9	157,5
DC2.W.50.5	19,20	1,95	96,9	0,9	49,3	146,2	1,86	101,6	1,1	54,7	156,3
DC2.W.50.10	41,85	4,6	109,7	1,5	35,4	145,1	4,8	114,1	1,6	37,8	151,9
DC3.W.50.5	19,20	1,8	94,8	0,9	46,9	141,7	1,9	97,9	1,1	57,1	155,0
DC3.W.75.5	19,20	2,1	109,8	1,1	57,6	167,3	2,0	103,3	1,2	62,0	165,2
DC3.W.50.10	41,85	3,72	88,9	2,1	50,2	139,1	3,63	86,7	2,3	56,3	143,0
DC3.W.75.10	41,85	4,8	113,8	2,1	50,3	164,1	4,7	113,1	2,9	58,9	172,0

Table 4-17 shows the differences in quantities of concrete volume, the longitudinal reinforcement (Long.) and the transverse reinforcement (Transv.).

Table 4-17: Quantities differences between structures

Structures	S_α m/s^2	Number of storeys	Long. Reinforcement weight difference [%]		Transv. Reinforcement weight difference [%]	
			θ=22°	θ=45°	θ=22°	θ=45°
DCM to DC2	5,00	5	-4,2%↘	-2,7%↘	-44,1%↘↘	-28,9%↘↘
DC2 to DC3			6,7%↗	-1,2%↘	-15,5%↘↘	4,9%↗
DCM to DC3			2,7%↗	2,7%↗	-66,5%↘↘	-22,5%↘↘
DCM to DC3	7,50		0,0%→	0,0%→	0,0%→	0,0→%
DCM to DC2	5,00	10	-7,8%↘	-22,5%↘↘	-11,1%↘↘	-0,6%↘
DC2 to DC3			21,0%↗↗	22,0%↗↗	-42,8%↘↘	-37,8%↘↘
DCM to DC3			14,8%↗↗	4,4%↗	-58,8%↘↘	-49,6%↘↘
DCM to DC3	7,50		6,0%↗	-0,4%↘	-47,0%↘↘	-31,2%↘↘

Figure 4-30 depicts the variations in the quantities of longitudinal and transverse reinforcement for walls designed with strut inclinations of θ=22° and θ=45°.

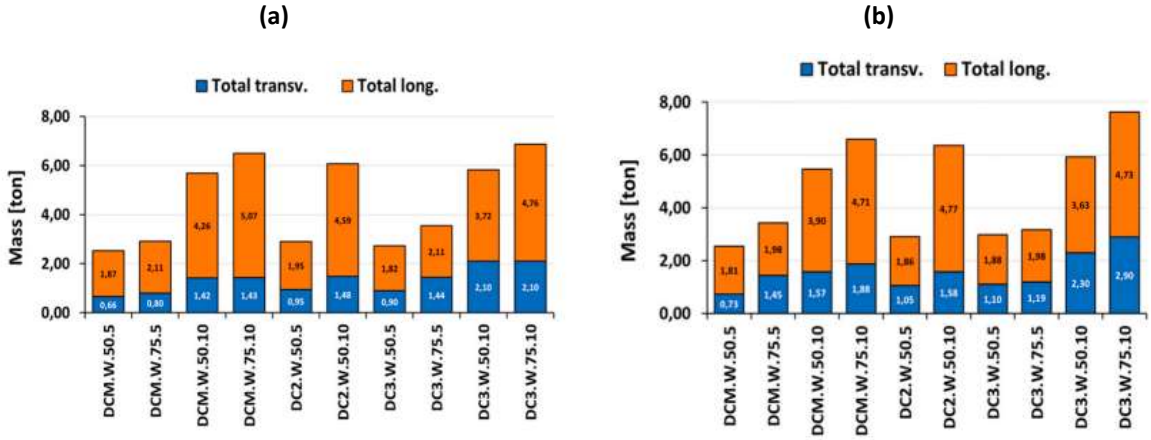


Figure 4-30: Longitudinal (Long.) and transverse (transv.) reinforcement (a) Strut inclination of θ=22° (b) Strut inclination of θ=45°

Based on the information provided in Table 4-17 and Figure 4-30, the following observations may be made:

a) When comparing DCM to DC2 and DCM to DC3 for structures with 5 storeys and an S_α of 5,00 m/s², there is a decrease in both longitudinal and transverse reinforcement weight when transitioning from DC2 to DC3.

b) For structures with 5 storeys and S_α of 5,00 m/s², DCM to DC2 experiences a decrease in longitudinal reinforcement weight when transitioning to DC3, but transverse reinforcement weight increases.

c) For structures with 7,50 m/s² acceleration, there is no change in reinforcement weight when transitioning from DCM to DC3.

d) When comparing structures with 10 storeys and an S_α of 5,00 m/s², transitioning from DCM to DC2 to DC2 to DC3 significantly increases longitudinal and transverse reinforcement weight.

e) For structures with 10 storeys and an S_α of 5,00 m/s², transitioning from DCM to DC3 results in a substantial decrease in longitudinal reinforcement weight, but transverse reinforcement weight remains relatively unchanged.

f) The transverse reinforcement quantity is higher for a strut inclination θ=45° than for θ=22°. Regarding the ($V_{Rd,s}$) equation, the demand for shear transverse reinforcement for the θ=45° strut is more pronounced, and the quantity of longitudinal reinforcement is greater for a strut inclination of θ=22° than for θ=45°. Walls with a θ=22° strut exhibit a higher tension shift, leading to increased consumption of longitudinal reinforcement.

4.6 SUMMARY AND CONCLUSIONS

This study aimed to compare the design for different buildings, classes of ductility and seismic action intensity levels according to the first and second generations of Eurocode 8 provisions for seismic response, stability and detailing for local ductility. The significant findings and suggestions are summarised here forth:

Buildings designed to DCM and DC3 ductility classes generally exhibit similar base shear values and roof displacements across various building heights and seismic action levels. Notably, 5-storey buildings such as DCM.W.50.5, DC2.W.50.5, and DC3.W.50.5 share comparable fundamental period values. Nevertheless, the roof displacement for DCM.W.50.5 and DC3.W.50.5 is 3.67% higher than DC2.W.50.5, possibly due to DC2 facing more severe seismic challenges, necessitating larger cross-sections in its structural design. DC2.W.50.5 shows a 36% higher base shear value than its DCM and DC3 counterparts, aligning closely with the differences expected from their respective q-factor values.

The sturdiest structure among those evaluated is DC3.W.75.10, which also demonstrates the highest base shear value, whereas the structures DCM.W.50.5 and DC3.50.5 have the lowest. Conversely, DC2.W.50.5 has the lowest fundamental period, indicating higher stiffness and resistance to deformation and the lowest roof displacement. For 10-storey buildings, DCM.W.50.10 and DC3.W.50.10 experience 8.4% greater roof displacement compared to DC2.W.50.10, reflecting a similar trend in design and response to seismic actions. Additionally, DC2.W.50.10 exhibits a 45% higher base shear value than its DCM and DC3 counterparts, which correlates with its significantly lower fundamental period, making it stiffer and less prone to deformations.

The interstorey drift ratio (IDR) differences between DCM.W.50.5 and DC3.W.50.5 are minimal, with most discrepancies ranging only between 0.02% to 0.04%, effectively indicating no practical difference in IDR values across various storeys. The largest observed difference is 0,04% at the first storey, while the smallest is 0,02% at most other storeys. Notably, the IDR values for the higher-storey structures DCM.W.75.5 and DC3.W.75.5 show no differences, suggesting highly consistent displacement patterns within these designs.

Further analysis of the displacement patterns, especially in taller structures, reveals a consistent trend in minimal IDR differences among DCM.W.50.10, DC3.W.50.10, and DC2.W.50.10, despite some variance at specific storeys. For instance, at storey 6, there is a significant absolute difference of 17% in IDR among these structures, indicating a variation in how they react to seismic forces at this level. Meanwhile, the discrepancies between DCM.W.75.10 and DC3.W.75.10 remain modest at around 3,8%, except for the noted 17% variance at the same storey, underscoring different structural responses under similar seismic conditions.

The analysis of torsional deformation in buildings reveals that it increases with the height of the structure, achieving the highest values at the top levels where stiffness is reduced, in line with the design strategy adopted for this study and confirming conclusions by Özmen et al.[22] For instance, in a 5-storey building, DC3.W.75.5 records the highest torsional angle at approximately 0,42 milliradians. In comparison, DC2.W.50.5 presents the lowest at 0,25 milliradians, indicating a higher average torsional response in DC3.W.75.5 across all levels compared to DC2.W.50.5. Similarly, the 10-storey DC3.W.75.10 experiences a torsional angle of around 2,08 milliradians, demonstrating a moderate torsional effect, whereas DC2.W.50.10 has a significantly lower torsional impact with an angle of 1,31 milliradians. These results

underscore the importance of accounting for torsional effects in building design, particularly in taller structures, to enhance structural integrity and safety. Furthermore, the research suggests that the relative stiffness of horizontal to vertical resisting elements and the distribution of lateral loads significantly influence torsional resistance and in-plane rotation of slabs, echoing findings by Khatiwada[23] and Khan et al.[24], who emphasize the critical role of building height and stiffness distribution in managing torsional responses.

According to the data presented in Figure 19(a) and (b), when evaluating a strut with an angle of inclination θ=22°, the design bending moment appears significantly higher compared to the case with θ=45° the average percentage difference for $A_{M_{Edw}}$ is approximately 59,44% across the provided structures when comparing values from Figure 4-19(a) and (b).

The observed differences in design bending moments, especially the higher moments for struts inclined at 22° compared to those with 45° struts, and among various ductility classes (DC2, DC3, DCM) as shown in Figure 4-19 (a) and (b), can be explained by the geometric effects of strut inclination, differences in envelope bending moment models outlined in EN 1998-1:2004[24] and prEN 1998-1-2:2023[3], and specific seismic design considerations. A lower strut inclination (θ=22°) increases bending moments due to a more prolonged lever arm effect, whilst variations in structural calculations arise from adherence to or deviations from EN 1998-1:2004[24] and prEN 1998-1-2:2023[3] specifications. This highlights the interplay between structural geometry and the envelope of bending moment models.

The variations in design bending moments observed in Figure 22 (a) and (b) across different building configurations are influenced by several factors, including the building's height, ductility class, and the strut inclination of the walls. Taller buildings and those designed to have higher ductility (DCM and DC3) typically show higher bending moments due to increased seismic demands. Specifically, struts inclined at a shallower angle of 22° generate significantly higher bending moments than those at a strut angle of 45°, with a roughly 40,3% higher average bending moment observed, attributable to the lever arm effect, which amplifies force impact over a longer distance. Additionally, differences in the EN 1998-1:2004[5] and prEN 1998-1-2:2023[3] envelope bending moment models further contribute to the discrepancies in bending moment values, reflecting the interplay between structural design seismic loading requirements.

Regarding the bending resistance below the tension shift (a_l), the analysis indicates effective top wall reinforcement optimisation with moments closely following a linear correlation near the threshold line (M_{Rdw}=M_{Edw}). However, in some instances, particularly for DC3 and DCM structures, higher bending moments (M_{Edw}) are observed, potentially attributed to the utilisation of a model with elevated design bending moments in compliance with prEN 1998-1-2:2023[3] compared to EN 1998-1:2004[3]. Despite this, the bending resistance in critical zones remains similar, with the primary distinction being the prolongation of reinforcement for θ=22°, leading to increased reinforcement consumption.

The bending resistance above the tension shift (a_l) analysis indicates that the moments exhibit a nearly linear relationship near the threshold line (M_{Rdw}=M_{Edw}), reflecting well-optimised wall reinforcement. Some walls show higher bending moments (M_{Edw}) for DC3 compared to DCM structures, attributed to the use of a calculation model with higher design bending moments according to prEN 1998-1-2:2023[3], as opposed to EN 1998-1:2004[5]; however, both models exhibit similar trends. Additionally, the relationship between MRdw and MEdw is comparable in Figure 4-24(a) and Figure 4-24(b), with greater dispersion at θ=45° in Figure 4-24(b),

possibly due to smaller tension shifts, leading to varied reinforcement quantities in the boundary elements.

Regarding the shear resistance in the analysis, it is evident that for DCM structures, the relationship between (V'_{Edw}/V_{Rdw}) and (V_{Edw}/V_{Rdw}) forms a straight line, aligning with the expected magnification factor of 1,5. However, in structures with ductility classes DC2 and DC3, this relationship does not follow a straight line because it is influenced by the magnification factor $\varepsilon(z)$ according to EN 1998-1-2:2020[3]. DC3 structures exhibit a steeper slope than DC2 structures, mainly due to differences in the behaviour factor (q=2,0 for DC2 and q=3,0 for DC3). The inclination of the strut has little impact on the relationship between (V'_{Edw}/V_{Rdw}) and (V_{Edw}/V_{Rdw}), but it significantly affects the value of (V_{Rdw}).

Concerning the chord rotation, the structures with a strut inclination of 22° have a more significant impact on chord rotation control than those with a 45° strut inclination, primarily due to the influence of tension shift (a_l) in chord rotation at yield (θ_y). Particularly in DC2 and DC3 structures, especially those with a 22° strut inclination, horizontal rebar spacing ($s_{h,adopted}$) is determined by the need to control chord rotation at the base of the walls, highlighting its critical role in these cases. The reason walls designed with a 22° strut inclination exhibit more significant chord rotation control challenges than those with a 45° inclination is primarily how tension shifts within the structure affect its stability and integrity during yield. A lower inclination angle intensifies the tension shift, increasing the chord rotation, particularly at yield points. This phenomenon necessitates a strategic approach to the placement and spacing of horizontal rebar ($s_{h,adopted}$), especially in buildings designed for higher ductility classes like DCM and DC3. In these cases, the closer spacing of horizontal rebar is a direct response to managing increased chord rotation.

In conclusion, analysing different structural configurations and seismic scenarios highlights several key findings. The comparison of buildings in DCM and DC3 ductility classes reveals similar base shear values and roof displacements, with some exceptions in 5-storey buildings. Notably, DC2.50.5 exhibits significantly higher roof displacement due to increased seismic activity compared to DCM.W.50.5 and DC3.W.50.5. Strut inclination influences design bending moments, emphasising the importance of this parameter.

The interstorey drift ratio (IDR) analysis demonstrates consistent displacement patterns between various structures, and the role of walls in second-order structural effects cannot be understated, affecting deflection and stability. Additionally, the study delves into chord rotation control, sensitivity coefficients, and reinforcement weight, providing insights into their behaviour under different seismic conditions, ductility classes, and storey heights. These findings contribute to our understanding of structural performance and the impact of design parameters.

The data provided showed a nuanced interplay between structural requirements and material economy when comparing DCM, DC2, and DC3. DCM configurations generally necessitate a minor reinforcement, potentially making them the most economical option for longitudinal and transverse steel, particularly in 5-storey buildings. As we progress to higher ductility classes, DC2 and DC3, the trend shifts, with DC2 occasionally requiring more longitudinal reinforcement but less transverse reinforcement than DCM, reflecting a moderate cost increase but enhanced seismic performance. DC3, while often the least economical due to the highest reinforcement demands, offers the greatest ductility, indicating a design prioritization of resilience and energy dissipation capacity over the material economy,

particularly evident in taller, 10-storey structures. This progression underscores a deliberate trade-off within the Eurocode framework between initial material costs and the expected performance of structures in seismic events, with each step up in ductility class aiming to provide more significant safety margins and post-earthquake functionality.

As a final note, for future research, more structures shall be analysed with different heights, number and span lengths, regularities and torsional flexibilities.

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4.8 SYMBOLOGY

4.8.1 UPPER CASE LATIN SYMBOLS

Symbol description	*Upper case Latin symbols*		
	DCM	DC2	DC3
Cross sectional area of reinforcement	A_s		
Cross sectional area of shear reinforcement	A_{sw}		
Wall design value of the applied internal bending moment	M_{Ed}		
Design value of the applied axial force (tension or compression)	N_{Ed}		

4.8.2 LOWER CASE LATIN SYMBOLS

Symbol description	*Lower case Latin symbols*		
	DCM	DC2	DC3
Wall thickness	b_c,b_w	b_w	
Wall web thickness	b_{w0}		
Width of confined core in the boundary element of a wall (to the centreline of hoops)	b_0		
Effective depth of a cross-section	d		
Design value of concrete compressive strength	f_{cd}		
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}		
Mean value of axial tensile strength of concrete	f_{ctm}		
Design yield strength of reinforcement	f_{yd}		
Design yield strength of the shear reinforcement	f_{ywd}		
Characteristic yield strength of reinforcement	f_{yk}		
Storey height (also named interstorey height)	-	h_s	
Clear storey height	h_s	$h_{s,cl}$	
Cross-sectional depth of column in the direction of interest	h_c		
cross-sectional depth of a beam	h_w	-	-
cross-section depth of a beam or column	-	h	
Wall length	l_w		
Spacing of the stirrups	s_w		
Inner lever arm, for a member with constant depth, corresponding to the bending moment in the element under consideration	z		

4.8.3 GREEK SYMBOLS

Symbol description	*Greek symbols*		
	DCM	**DC2**	**DC3**
Coefficient taking account of the state of the stress in the compression chord	α_{cw}		
Confinement effectiveness factor	α	–	–
Shear magnification factor for walls	ε		
Design value of steel strain at yield	$\varepsilon_{sy,d}$		
Curvature ductility factor	μ_ϕ		
Normalised axial load	ν_d		
Longitudinal reinforcement ratio in the compression zone	ρ'	ρ'_l	
Maximum longitudinal reinforcement ratio	ρ_{max}	$\rho_{l,max}$	
Angle between the concrete compression strut and the beam axis perpendicular to the shear force	θ		
Mechanical ratio of vertical web reinforcement	ω_v		
Mechanical volumetric ratio of confining reinforcement	ω_{wd}	–	–

5 Exploring behaviour factors and chord rotation in RC MRF Structures: Nonlinear analysis based on second-generation of Eurocode 8

Helder Maranhão, Humberto Varum, José Melo (submitted in press) Exploring behaviour factors and chord Rotation in RC MRF Structures: Nonlinear analysis based on second-generation of Eurocode 8

5.1 ABSTRACT

Eurocode 8 is undergoing revisions, including introducing new ductility classes and displacement-based design methods alongside traditional force-based ones. Additionally, new detailing provisions for local ductility are being considered. A notable change in the second generation of Eurocode 8 is that it no longer requires direct verification of concrete core confinement at the base of columns when detailing for local ductility. This study examines these changes concerning moment-resisting frames that are regular in plan and elevation and non-torsionally flexible. The behaviour factor (q) components q_R, q_D and q_S, and chord rotation (displacement-based) are analysed using nonlinear analysis and compared with the current and second-generation Eurocode 8. A mathematical model based on nonlinear analysis models has been developed to understand the implications of forfeiting the direct verification of concrete core confinement at the base of columns. Nonlinear analysis reveals higher behaviour factor values than those prescribed by the code, highlighting the safety characteristics of the second generation of Eurocode 8 and the implications of three-dimensional modelling. Regarding the displacement-based design, results show that chord rotation models preconised in the second generation of Eurocode 8 demonstrate good approximation to the ones obtained by nonlinear. The strong correlation observed in the developed mathematical model for concrete core confinement indicates that displacement-based design alone sufficiently ensures the safety of core confinement at the base of columns.

5.2 INTRODUCTION

EN 1998-1:2004[1] represents the state-of-the-art seismic design in Europe, although some sections need to be updated regarding the latest research developments and practitioners' feedback. A review of some fundamental concepts is required; for example, the concept of ductility class allows state-of-the-art buildings yet provides clear and practical guidance for traditional buildings. Currently, three ductility classes are defined, namely the high (DCH) and medium (DCM), which are recommended for areas of high or medium seismicity and low (DCL) for low seismicity regions, according to Booth et al.[2].

The second generation of Eurocode 8 introduces several improvements, aiming to be state-of-the-art in seismic structural engineering. In this new code generation, the structure of the parts was revised, concentrating in FprEN 1998-1-1:2024[2] the general aspects independent of the type of structures and materials and in prEN 1998-1-2:2023[3] included the specific rules for buildings.

Regarding the ductility classes, one factor is that tall or flexible structures primarily influenced by deflection rather than strength do not benefit from high ductility levels. Additionally, the appeal of high ductility options is diminishing due to a growing focus on serviceability performance. It seems most prudent to remove the distinction between DCM and DCH. The FprEN 1998-1-1:2024[3] attempts to overcome this issue by introducing new

ductility classes known as DC1, DC2, and DC3, where DC1 and DC3 can be related to the previous DCL and DCM regarding EN 1998-1[2], letting this way, DCH out of the new generation of Eurocode 8.

The EN 1998-1:2004[1] allows the definition of the design seismic base shear force from elastic analysis, reduced by a constant factor to include the effects of the ductile, nonlinear behaviour of the structural system, which is intrinsically connected to the ductility classes (DC). This reduction factor is the behaviour factor (q) that accounts for the structure's capacity to dissipate hysteretic energy by ductile behaviour, overstrength, and redundancy of the structural system. The quantification of different parameters associated with buildings' structural behaviour, such as the behaviour factor q, associated with the reference earthquake intensity and the definition of relevant design criteria, are still open issues among the structural researchers and engineering practitioners' community that need further studies.

The prEN 1998-1-2:2023[4] defines a behaviour factor composed of the product of the partial behaviour factors (components) q_R, q_S, and q_D, where (q_R) is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures (q_S) is the behaviour factor component accounting for overstrength due to all other sources, and (q_D) is the behaviour factor component accounting for the deformation and energy dissipation capacity.

Another important novelty is that the prEN 1998-1-2:2023[4] introduces a displacement-based design alongside the traditional force-based one with reduced equivalent forces. The FprEN 1998-1-1:2024[2] and prEN 1998-1-2:2023[3] provide resistance models needed to apply the displacement-based design approach to reinforced concrete elements (beams and columns) in the form of chord rotation verification, or control, for members, as stated by Franchin et al.[5].

The prEN 1998-1-2:2023[3] preconises that the required overall ductility of the structure to be achieved, the potential regions for plastic hinge formation, to be defined for each type of building member, should be provided with a sufficient ductility and plastic rotation capacity in all critical regions of primary seismic members. For that, it is necessary to perform the chord rotation control. The chord rotation refers to the rotational flexibility of the structural elements that resist lateral loads, such as beams or columns. This flexibility allows the structure to deform and distribute loads more effectively, improving its overall performance and safety.

The present work aims to analyse the behaviour factor q (and its components) as proposed in the prEN 1998-1-2:2023[4] of a set of reinforced concrete Moment-Resisting Frame structures (MRF) through nonlinear analysis and compare with the ones obtained by nonlinear analysis, using the models proposed by Plumier[6].

A notable novelty in the second generation of Eurocode 8 is that it no longer requires direct verification of concrete core confinement at the base of columns when detailing for local ductility. In this research work, a mathematical model has been developed to understand the implications of forfeiting the direct verification of concrete core confinement at the base of columns. The present work attempts to establish a link between the effectiveness of the core confinement ($\alpha \cdot \omega_{wd}$) as preconised in EN 1998-1:2004[1] at the base of the columns by developing a mathematical model-based nonlinear analysis in order to validate that the displacement-based design alone sufficiently ensures the safety of core confinement at the base of columns

The current study involves a set of reinforced concrete (RC) structures designed to be regular in both plan and elevation, non-torsionally flexible, based on the work developed by Maranhão et al.[7], furthermore, its findings are limited to this set.

5.3 OBJECT OF RESEARCH AND ANALYSIS

To conduct this study, ten reinforced concrete moment-resistant frame (MRF) structures were selected from the analysis performed by Maranhão et al.[7]. All buildings were designed according to the force-based design concept (FB). It should be noted that all buildings are designed in a three-dimensional environment (3D). A 3D model allows a better idea of how the forces are transferred to the structure's elements and foundations. These structures were designed in strict adherence to the standards outlined in EN 1998-1:2004[1], FprEN 1998-1-1:2024[3], prEN 1998-1-2:2023[4] and EN 1992-1-1:2004[8]. The structures included in this study consist of five 5-storey and another set of five 10-storey buildings, as illustrated in Figure 5-1, designed for DCM, DC2 and DC3 ductility classes. All structures are regular in plan and elevation and non-torsionally flexible.

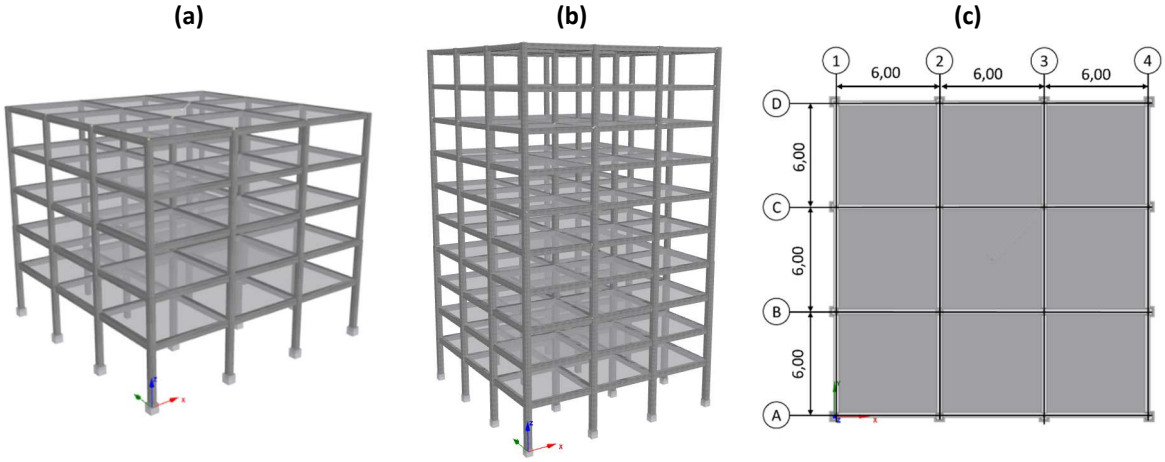


Figure 5-1: Buildings geometry model [4], [7]: (a) 5-storey building; (b) 10-storey building; (c) Plan view

The structural system for the buildings consists of a series of frames with an equal spacing of 6,0 m, typical of office buildings. The floor-to-floor height is 3,0 m for all storeys, except for the ground storey, which is 4,0 m. The solid slab thickness is 15,0 cm at each storey. The building seismic responses are summarised in Table 5-1.

Table 5-1: Seismic response parameters (Adapted from Maranhão et al.[7])

Structure label	Ductility class	Number of storeys	S_α m/s²	Ductility class	q	T_1 s	$S_d(T_1)$ m/s²	V_b kN	$d_{e,roof}$ m
DC2.50.5	DC2	5	5,0	DC2	2,5	1,43	0,69	866,51	0,051
DCM.50.5	DCM			DCM	3,9	1,49	0,43	532,37	0,033
DC3.50.5	DC3			DC3	3,9	1,49	0,43	532,37	0,033
DCM.75.5	DCM		7,5	DCM	3,9	1,03	0,93	1236,39	0,034
DC3.75.5	DC3			DC3	3,9	1,02	0,93	1250,01	0,033
DC2.50.10	DC2	10	5,0	DC2	2,5	2,10	0,44	1169,98	0,085
DCM.50.10	DCM			DCM	3,9	2,14	0,33	862,50	0,055
DC3.50.10	DC3			DC3	3,9	2,14	0,33	867,01	0,053
DCM.75.10	DCM		7,5	DCM	3,9	1,92	0,50	1382,10	0,065
DC3.75.10	DC3			DC3	3,9	1,92	0,50	1382,10	0,065

The building's primary seismic members (beams and columns) were designed with concrete C30/37 and reinforcement steel B500, as mentioned in Maranhão et al.[7].

5.4 MODELLING AND ANALYSIS

According to Antoniou et al.[9] in the Nonlinear Static Procedure, the structural behaviour is no longer linear, and the elasticity rules do not apply any more. Stresses are not proportional to strains, forces are not proportional to displacement, and bending moments are not proportional to curvatures. A lateral force profile approximating the earthquake loading is applied with several types of force distributions: triangular, uniform, modal or even adaptive distributions that change from step to step.

5.4.1 PUSHOVER PROCEDURE

The nonlinear analysis performed in the present work is known as adaptive pushover analysis recurring to finite element software SeismoStruct from Seismosoft[10] in a three-dimensional environment. According to Seismosoft[10], loads are applied to the structure in this type of analysis, like in conventional pushover analysis.

In the context of a pushover analysis following the EN 1998-3:2004[11] standard and utilising a uniform load pattern, the objective is to assess the structure's seismic performance. The uniform load pattern represents lateral forces applied uniformly across the structure's height. In the analysis, particular attention is given to the mode of vibration that mobilises the most mass. This involves identifying the vibration mode with the highest effective modal mass value, emphasising its influence on the structural response to seismic action. By focusing on the most significant mode, the pushover analysis aims to capture the critical aspects of the structure's behaviour under seismic loading.

The nonlinear models were developed using the nonlinear analysis software SeismoStruct[10]. The analysis through the distributed fibre formulation (distributed inelasticity) approach considers the 3 integration sections with 150 fibres in each integration section. The integration sections are located along the columns and beams in the distributed fibre formulation, as depicted in Figure 5-2.

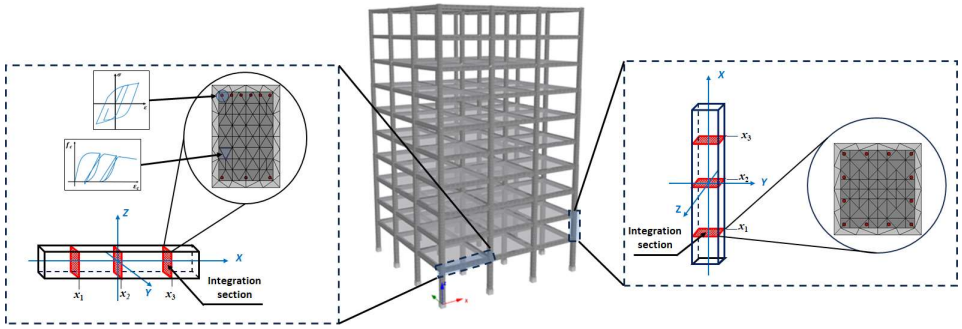


Figure 5-2: Schematic illustration of 3 integration sections and meshing for 150 fibres

The distributed inelasticity in elements using the force-based approach was implemented according to the formulation of Neuenhofer et al.[12]. In a force-based element, equilibrium is strictly satisfied, and no restraints are placed on developing inelastic deformations throughout the member. The incorporation of three integration sections per element was determined based on the findings made by Maranhão et al.[13]. According to Di Domenico et al.[35] is expected that the use of only three integration points introduces a limited error in element response when adopting the force-based formulation with Gauss–Lobatto integration scheme.

Using 3 integration sections instead of a higher number in seismic structural analysis in the context of the SeismoStruct[10], has several key advantages. These are derived from the handling of convergence norms, iterative strategies, and stiffness updates as described as follows:

a) Improved Convergence: with fewer integration sections, the program may find it easier to achieve convergence in challenging analysis steps. This is because convergence difficulties often arise from the use of a large number of integration sections. Reducing the number to 3 can mitigate these issues, facilitating smoother progression through analysis steps without the need for excessive adjustments to convergence norms. According to SeismoStruct[10] convergence difficulties in force-based elements are often caused by the employment of a large number of integration sections together with element discretisation (typically in elements where the reinforcement details change). In such cases, users should decrease the number of integration sections to 3.

b) Iterative Strategy: The iterative approach employed by SeismoStruct[10] can benefit from a reduced number of integration sections as it can lower the complexity of the equilibrium process within each load increment. This simplification can lead to more stable and consistent iterations, especially when dealing with high geometric nonlinearity or material inelasticity.

c) Number of Stiffness Updates: with 3 integration sections, the recalculation and updating of the tangent stiffness matrix may become more efficient. This can lead to an optimal balance between computation time and stability. Since the ideal number of stiffness updates is between 50% and 75% of the maximum iterations, fewer integration sections can streamline this process, reducing the need for frequent stiffness updates and thus enhancing computational efficiency.

d) Reduced Computational Load: Computational Efficiency: Fewer integration sections directly translate to reduced computational load. This means faster analysis times and less computational resources required, which is particularly beneficial for complex models or when running multiple simulations.

e) Simplified Modelling and element Discretization: In elements such as beams where reinforcement details change, using 3 integration sections simplifies the element discretization process. This can prevent convergence issues that arise from intricate discretization schemes, making the modelling process more straightforward and less error prone.

Overall, using 3 integration sections can lead to a more efficient, stable, and reliable analysis process in SeismoStruct, especially when dealing with complex structures or those subjected to significant nonlinearity. This approach optimizes the balance between computational demand and analysis accuracy, facilitating better overall performance of the seismic structural analysis.

According to SeismoStruct[10], the solution algorithm employs the modified Newton-Raphson method. The iterative procedure follows the conventional schemes applied in nonlinear analysis, whereby the internal forces corresponding to a displacement increment are computed, and convergence is checked. Regarding the convergence criteria, SeismoStruct[10] uses two distinct convergence check scheme criteria, the displacement-rotation and force-moment based. The solution convergence is checked at the end of every single iteration, as referred to by Maranhão et al.[13].

5.4.2 MATERIAL CONSTITUTIVE MODELS

The concrete constitutive relationship proposed by Mander et al.[14] is a uniaxial nonlinear constant confinement model, as shown in Figure 5-3(a). The confinement effects provided by the transverse reinforcement are incorporated, whereby constant confining pressure is assumed throughout the stress-strain range. The concrete compressive strength considered in the analysis is the mean characteristic cylinder compressive strength, which, in the case of a concrete C30/37, the value of f_{cm}=38 MPa, and its constitutive law, used in the nonlinear FE models under analysis, is depicted in Figure 5-3(b).

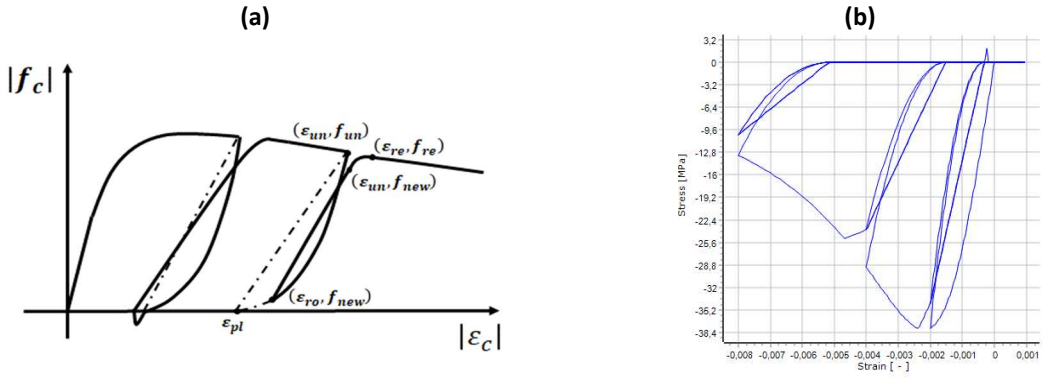


Figure 5-3: Concrete stress-strain curve: (a) Proposed by Mander et al. [14] (b) Actual curve used in the FE modelling [10]

The constitutive law for steel, utilised in the present modelling, was initially programmed by Menegotto et al.[15] Moreover, coupled with the isotropic hardening rules proposed by Taucer et al.[16] furthermore, it is represented in Figure 5-4(a). Figure 5-4(b) shows the steel stress-strain curve for nonlinear FE modelling. The yield strength of the reinforcement considered in the analysis is the mean yield strength (f_{ym}), which, for B500 grade reinforcement steel, is f_{ym}=555,56 MPa.

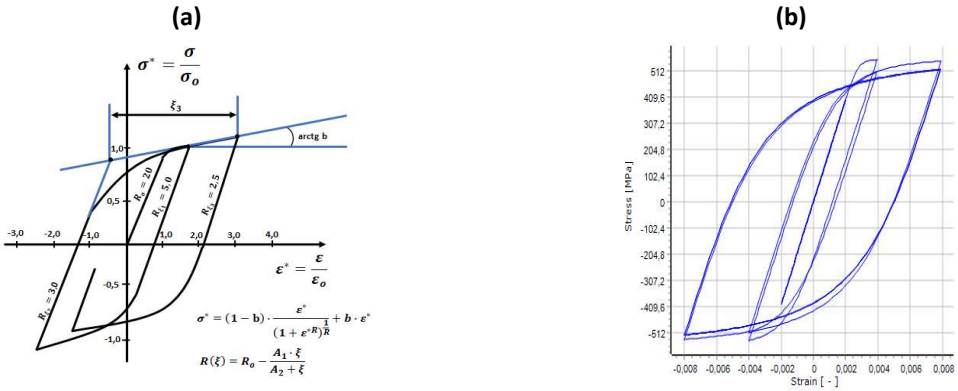


Figure 5-4: Steel stress-strain curve: (a) Proposed by Menegotto et al.[15]; (b) Actual curve used in finite element modelling [7]

5.5 BEHAVIOUR FACTOR ANALYSIS

The various structural behaviour parameters, such as the behaviour factor (q), are associated with the reference earthquake intensity and the definition of relevant design criteria. According to Elnashai et al.[17], the behaviour factor (q) may be defined as the ratio of the peak force ($V_{el,max}$) that would have developed if the SDOF system were linear-elastic to the yield force of the system (V_y), as depicted in Figure 5-5.

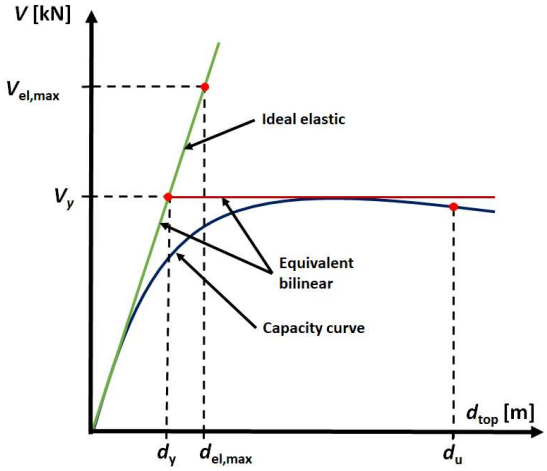


Figure 5-5: Capacity curve and idealised elastic-perfectly plastic force-displacement relationship (adapted from Elnashai et al.[18])

The behaviour factor (q) generally may be computed as depicted in Equation (5-1), and it expresses the energy absorption and dissipation capacity of structural systems, as stated by Elnashai et al.[17].

$$q = \frac{V_{el,max}}{V_y} \tag{5-1}$$

Zeri[18] defines the behaviour factor for ductility of the equivalent SDOF system as $q_\mu = V_{ed}/V_{ud}$. This relates the elastic spectral base shear demand (V_{ed}) to the equivalent SDOF bilinear yield strength (V_{ud}) obtained by assuming an elastoplastic equal area approximation of the inelastic base shear–roof displacement response under a given lateral load profile up to the ultimate deformation. One can draw a parallel with Elnashai et al.[17], where (V_{ed}) corresponds to ($V_{el,max}$) and (V_{ud}) corresponds to (V_y).

According to Fardis et al.[19], to account for the nonlinear response of a structure associated with the material, the structural system, and the design procedures, the behaviour factor (q) is defined as the ratio between the peak force that would have developed if the single-degree-of-freedom system (SDOF) were linear-elastic, to the yield force of the system. According to EN 1998-1:2004[1], the behaviour factor (q) is an approximation of the ratio of the seismic forces that the structure would experience if its response were completely elastic with 5% viscous damping to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The behaviour factor influences the shape of the design response spectrum, intrinsically on the spectral acceleration and, consequently, the design force.

As referred before, FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[4] introduce a behaviour factor composed by the product of the partial behaviour factors (components) q_R, q_S, and q_D, as depicted in Equation (5-2).

$$q = q_R \cdot q_S \cdot q_D \tag{5-2}$$

The component (q_R) is the behaviour factor component that accounts for overstrength due to the redistribution of seismic action effects in redundant structures, (q_S) is the behaviour factor

component accounting for overstrength due to all other sources, and (q_D) is the behaviour factor component accounting for the deformation and energy dissipation capacity.

The behaviour factor q (and its components) for the different RC buildings are analysed using nonlinear analysis through the adaptive static pushover method. The evaluation of the behaviour factor using the adaptive static pushover method involves the estimation of the capacity curve of the structure under a prescribed lateral load profile and crucial modelling assumptions of the strength and stiffness dependence of the member characteristics, including second-order effects at the global and local element levels. It is worth noting to refer that Repapis et al.[20] have adopted static pushover procedures to evaluate the behaviour factor of existing RC plane irregular frames, typical of structural designs between the 1960s and 1990s in Greece, as stated by Zeris[18].

5.5.1 ESTIMATION OF THE BEHAVIOUR FACTOR COMPONENTS

Estimating the behaviour factor and its components is performed, recurring to the mathematical model adapted from Plumier[6] and FprEN 1998-1-1:2024[3]. Plumier[6] proposes that the behaviour factor components q_R, q_S, and q_D are studied and analysed, recurring to the building capacity curve analysis at its key points (V_e,d_e), (V_1,d_1), (V_u,d_y) and (V_u,d_r) as shown in Figure 5-6.

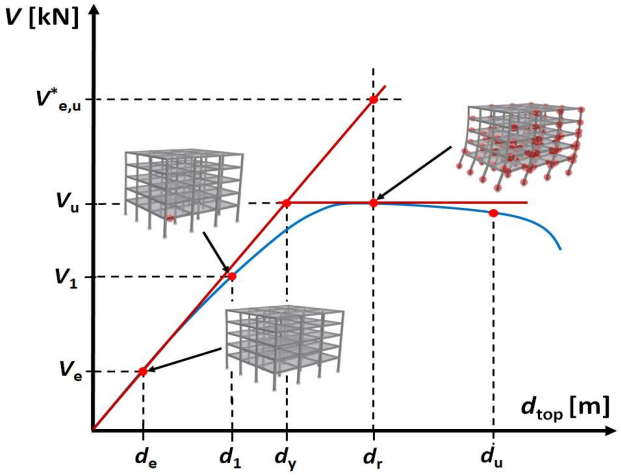


Figure 5-6: Capacity curve (adapted from Plumier [21])

The parameter (V_e) represents the horizontal shear force from the elastic analysis using a design spectrum reduced by the behaviour factor (q), and (d_e) is the displacement correspondent to the base shear force (V_e) measured at the roof top of the building as depicted in Figure 5-7(a). The parameter (V_1) is the horizontal base shear force at which the first plastic hinge is formed, and (d_1) is the displacement at the formation of the first plastic hinge, as shown in Figure 5-7(b).

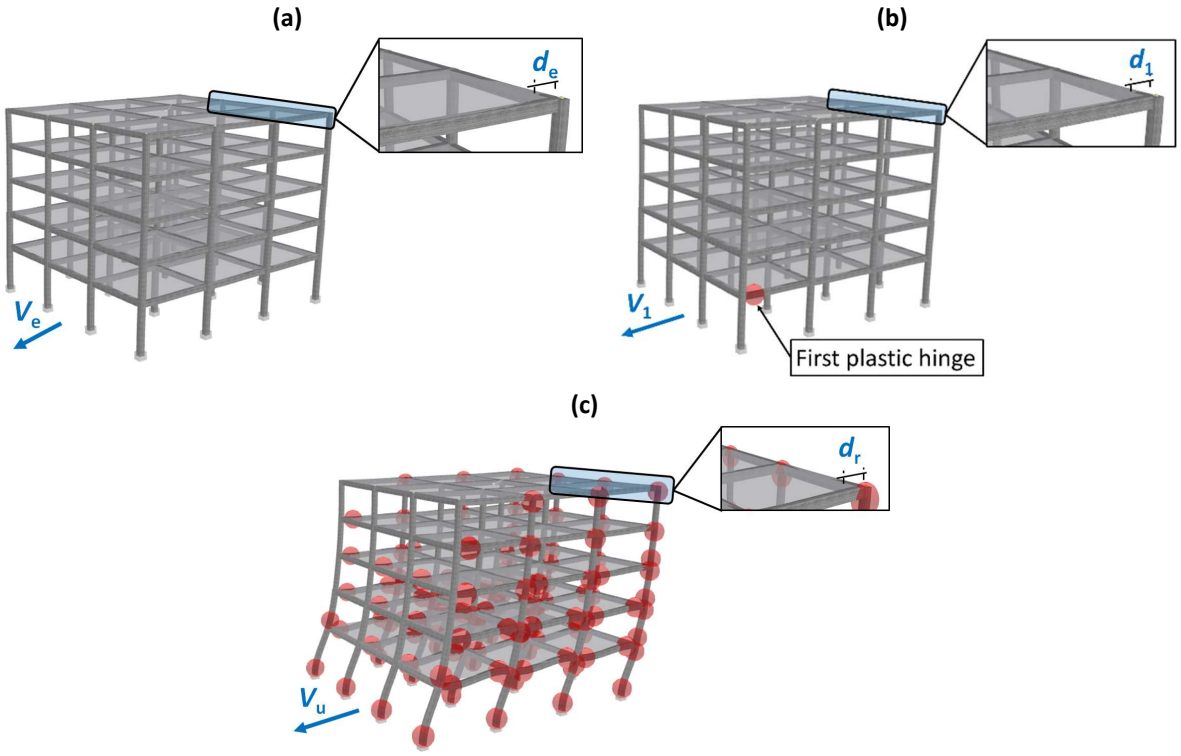


Figure 5-7: Structure damage states: (a) Elastic state (b) Attainment of the first plastic hinge; (c) Plastic mechanism

The parameter (V_u) is the horizontal shear force corresponding to the formation of a global plastic mechanism, and (d_r) is the correspondent displacement, as defined in Figure 7(c).

Finally, ($V^*_{e,u}$) is the hypothetical horizontal shear force for a displacement (d_r) of a structure with all sections identical to the real ones but which would be made of a material with an infinite yield strength. The displacement du displacement is defined as the ultimate limit state attained locally somewhere in the structure due to excess plastic rotation inducing a resistance drop by buckling, among other phenomena.

The present study introduces a methodology proposed by Plumier[6]. It is important to note that this analysis finds its roots in the case studies developed by Varum et al.[21]. The methodology consists of building a bilinear model of the force-deformation relationship at the member level for each building, used to represent ductile behaviour. Two damage levels should be defined in terms of the deformation thresholds of the member, which are structurally relevant points along the force-deformation relationship. A displacement at yield (d_y) corresponding to the attainment of effective yield strength is linked to the elastic stiffness ($K_e=V_e/d_e$), and the displacement at the formation of the global plastic mechanism (d_r) corresponds to the attainment of an effective ultimate strength (V_u). To determine the displacement (d_r), at least one of the two following criteria were considered:

1) The global plastic mechanism is observed as depicted in Figure 5-8(a)
2) If the global plastic mechanism is not observed, the ultimate capacity (V_u,d_r) is defined as a 20% reduction in strength relative to the maximum resistance, as described in Figure 5-8(b)

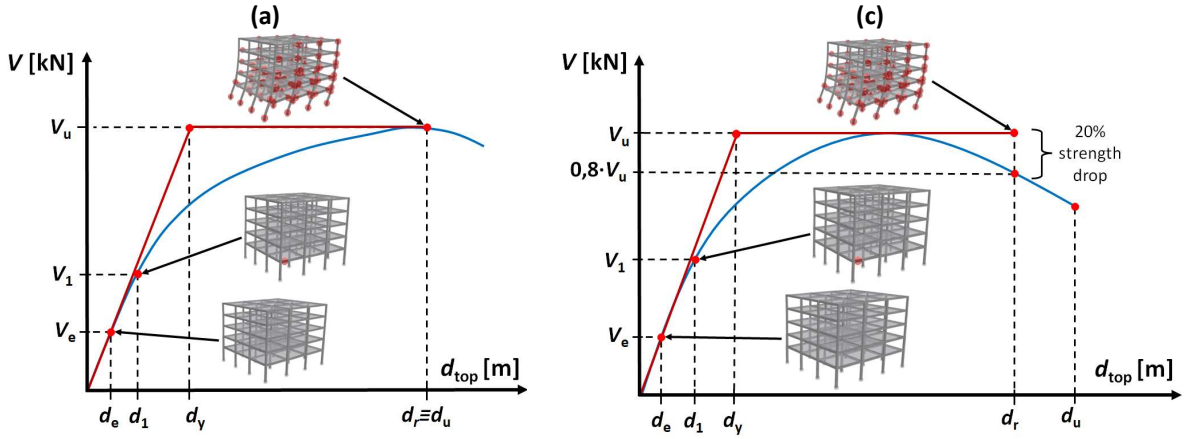


Figure 5-8: General definition of bilinear load-deformation relationship: (a) global plastic mechanism is observed; (b) strength drop of 20% relative to the maximum resistance.

5.5.1.1 BEHAVIOUR FACTOR COMPONENT q_R

The component (q_R) is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures.

According to Beyer et al.[22], a (q_R) value exceeding unity is attainable solely in cases of structural redundancy. However, it is essential to note that (q_R) does not quantify redundancy or plastic deformation capacity. Instead, it indicates the extent to which the internal force distribution, as determined by linear elastic analysis, deviates from the distribution under close-to-ultimate conditions, considering all sources of nonlinearities.

The present analysis considers the proposition to determine the behaviour factor component (q_R), adapted from Plumier[6]. This factor relates the single-degree of freedom (SDOF) bilinear effective ultimate strength (V_u) to the base shear (V_1) at the formation of the first plastic hinge in the members, as depicted in Figure 5-9.

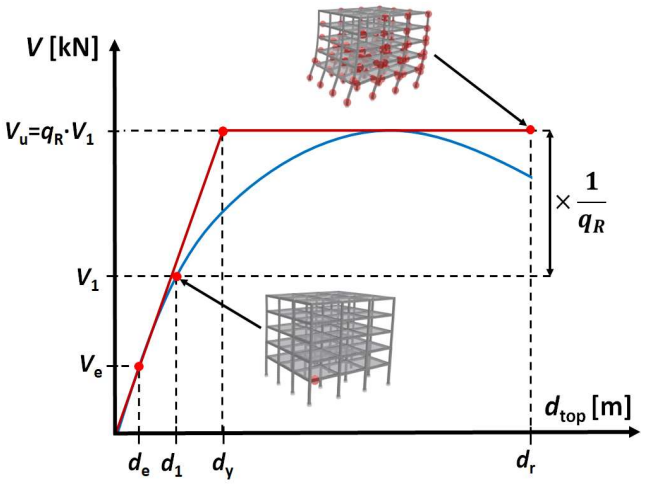


Figure 5-9: Definition of the behaviour factor component q_R

The behaviour factor component (q_R) is defined as the ratio between the ultimate strength (V_u) and base shear at the formation of the first plastic hinge in the members (V_1), as illustrated in Figure 5-9 and depicted in expression (5-3).

$$q_R = \frac{V_u}{V_1} \tag{5-3}$$

As per prEN 1998-1-2:2023[4], the behaviour factor (q_R) takes a value of 1,3 for both DC2 and DC3 ductility classes for MRF RC buildings.

5.5.1.2 BEHAVIOUR FACTOR COMPONENT q_S

The behaviour factor component (q_S) accounts for overstrength due to all other sources. Beyer et al.[22] define that the component (q_S) accounts to a large extent for the material overstrength and excess resistance due to the overdesign of sections. For MRF-reinforced concrete buildings, the component (q_S) results from sources such as material safety factors, the effects that increase the structural strength, such as the capacity design in bending (or mechanism control) and the capacity design in shear, and the contribution of slabs. In a 3D structural framework, the torsional resistance of elements positioned orthogonally becomes a crucial factor contributing significantly to the overall overstrength of the system. In the present analysis, the (q_S) component is defined as the ratio between the base shear force at the formation of the first plastic hinge (V_1) and the base shear force calculated in the elastic analysis (V_e), as presented in Figure 5-10.

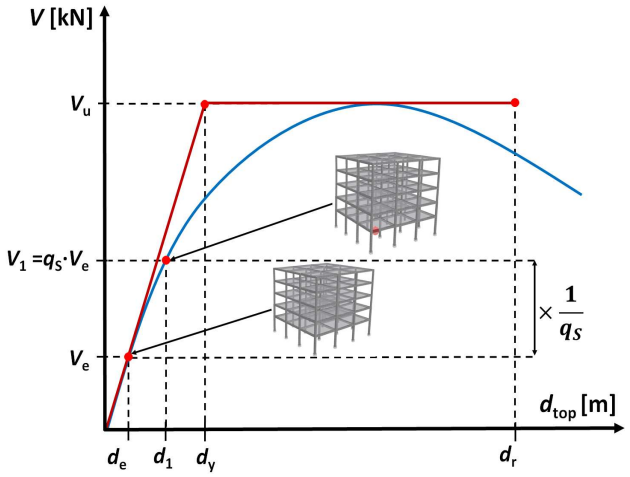


Figure 5-10: Definition of the behaviour factor component q_S

Therefore, the behaviour component (q_S) may be written as shown in Equation (5-4):

$$q_S = \frac{V_1}{V_e} \tag{5-4}$$

As for prEN 1998-1-2:2023[4], the behaviour factor (q_S) takes a value of 1,48 for DC2 and 1,50 for DC3.

5.5.1.3 BEHAVIOUR FACTOR COMPONENT q_D

The behaviour factor component (q_D)is the behaviour factor component accounting for the deformation capacity and energy dissipation capacity. Beyer et al.[22] states that this capacity depends on many building characteristics, including the deformation capacity of column beams, slabs, and torsional effects.

In the present analysis, the (q_D) component is defined as the ratio between the displacement corresponding to the formation of a global plastic mechanism (d_r) and displacement at yield (d_y), as presented in Figure 5-11.

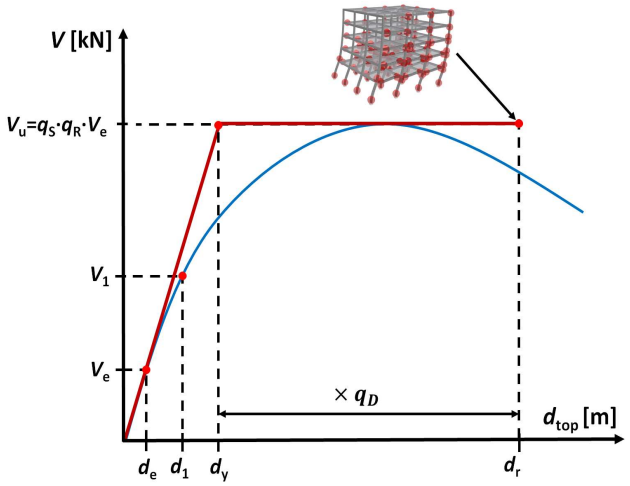


Figure 5-11: Definition of the behaviour factor component q_D

The behaviour factor component (q_D) is defined according to expression (5-5):

$$q_D = \frac{d_r}{d_y} \tag{5-5}$$

The displacement at yield (d_y) is defined according to Equation (5-6):

$$d_y = \frac{V_u \cdot d_e}{V_e} \tag{5-6}$$

As per prEN 1998-1-2:2023[4], the behaviour factor (q_D) takes a value of 1,3 for DC2 and 2,0 for DC3.

5.6 CHORD ROTATION ANALYSIS

According to prEN 1998-1-2:2023[4], ensuring sufficient ductility and plastic rotation capacity in all critical structural elements (beams and columns) is necessary. To achieve this, the product of the global displacement ductility factor, calculated as the product of the components of the behaviour factor (q_R) and (q_D), and the chord rotation at yield (θ_y) of each member end should not exceed the chord rotation corresponding to the attainment of the Significant Damage (SD) limit state (θ_{SD}) as defined in expression (5-7):

$$q_D \cdot q_R \cdot \theta_y \leq \theta_{SD} \tag{5-7}$$

To provide a visual representation of the chord rotation in structures, Figure 5-12 depicts the vectorial diagram of the chord rotation at the left end (θ_l) and the right end (θ_r) of beams and at the bottom end (θ_b) and the top end (θ_t) of columns.

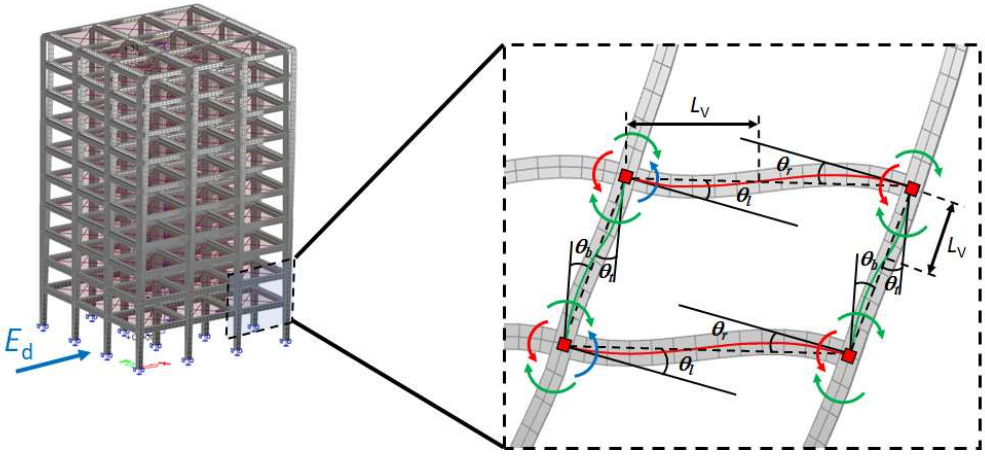


Figure 5-12: Vectorial representation of the chord rotation (adapted from Maranhão et al.[4])

The chord rotation of the shear span (L_V=M/V) at yield (θ_y) for rectangular beams and columns is composed of three parcels for beams and columns as defined in Equation (5-8):

$$\theta_y = \theta_{y,M} + \theta_{y,slip} + \theta_{y,V} \tag{5-8}$$

Where ($\theta_{y,M}$)is the contribution of the longitudinal reinforcement, ($\theta_{y,slip}$)is the contribution of the reinforcement bond and ($\theta_{y,V}$) is the contribution of the traversal reinforcement.

The Equation (5-8) may be written as per FprEN 1998-1-1:2024[2], for rectangular beams and columns may be evaluated according to expression (5-9).

$$\theta_y = \phi_y \cdot \frac{L_V + a_l}{3} + \frac{\phi_y \cdot d_{bL} \cdot f_y}{8 \cdot \sqrt{f_c}} + 0{,}0011 \cdot \left(1 + \frac{h}{3{,}0 \cdot L_V}\right) \tag{5-9}$$

The parameter (ϕ_y) is the curvature at yield from a cross-sectional analysis at the end section. The tension shift of the bending moment diagram (a_l) is defined in the second generation of EN 1992-1-1:2021[22] as being the same expression as in the current EN 1992-1-1:2004[6]. For columns, the shear span (L_V) adopted is 50% of the clear height, as referred by Fardis et al.[24]. The parameters (f_y) and (f_c) are the mean yield and compressive strength values of reinforcement steel and concrete, respectively.

To calculate the chord rotation corresponding to the attainment of the SD limit state, one should consider the expression(5-10), as defined in FprEN 1998-1-1:2024[3].

$$\theta_{SD} = \frac{1}{\gamma_{Rd}} \cdot \left(\theta_y + \alpha_{SD,\theta} \cdot \theta_u^{pl}\right) \quad (5\text{-}10)$$

The parameter $\alpha_{SD,\theta}$ is the portion of the plastic part of the ultimate rotation $\theta_u^{pl} = \theta_u - \theta_y$ (computed with mean material properties) that corresponds to the attainment of the SD limit state, and (γ_{Rd}) is a partial factor on resistance at the SD limit state. In the current version of the FprEN 1998-1-1:2024[3]. According to FprEN 1998-1-1:2024[3], plastic chord rotation capacity (θ_u^{pl}) is defined according to Equation (5-11).

$$\theta_u^{pl} = \kappa_{conform} \cdot \kappa_{axial} \cdot \kappa_{reinf} \cdot \kappa_{concrete} \cdot \kappa_{shearspan} \cdot \kappa_{confinement} \cdot \theta_{u0}^{pl} \quad (5\text{-}11)$$

Where θ_{u0}^{pl} is the basic value of a member's plastic chord rotation capacity, taking the value 0,039 rad if the member is a beam or a column with a section consisting of rectangular parts. The ($\kappa_{conform}$) factor is the correction factor linked to whether the structures have ductility class DC1, DC2 or DC3; (κ_{axial}) correction factor for an axial force different than zero; (κ_{reinf}) is the correction factor for asymmetrical reinforcement; κconcrete is the correction factor for concrete strength different than 25 MPa; ($\kappa_{shearspan}$) is the correction factor for a shear span-to-depth different than 2,5; and ($\kappa_{confinement}$) is the correction factor taking into account the confinement of concrete due to transverse bars.

The ultimate chord rotation is obtained according to expression (5-12) preconised by FprEN 1998-1-1:2024[3]:

$$\theta_u = \theta_y + \theta_u^{pl} \quad (5\text{-}12)$$

In the present analysis, the primary focus is on evaluating the essential elements required for conducting chord rotation safety verification. This involves assessing the chord rotation that corresponds to the attainment of the SD limit state obtained calculated according to (θ_{SD}), the chord rotation at yield (θ_y), the ultimate chord rotation (θ_u) at each member end, and the plastic chord rotation capacity (θ_u^{pl}).

5.6.1 ESTIMATION OF THE CHORD ROTATION PARAMETERS

The assessment involves a comparison of the chord rotation as follows:

a) The chord rotation that corresponds to the attainment of SD limit state obtained by nonlinear analysis ($\theta_{SD,NLA}$) against the chord rotation that corresponds to the attainment of the SD limit state obtained calculated (θ_{SD}) according to FprEN 1998-1-1:2024[3].

b) The chord rotation at yield obtained by nonlinear analysis ($\theta_{y,NLA}$) against the chord rotation at yield (θ_y) calculated according to FprEN 1998-1-1:2024[2].

c) The ultimate chord rotation obtained by nonlinear analysis ($\theta_{u,NLA}$) against the ultimate chord rotation (θ_u), calculated according to FprEN 1998-1-1:2024[3].

d) The plastic chord rotation capacity obtained by nonlinear analysis ($\theta_{u,NLA}^{pl}$) against the plastic chord rotation capacity calculated according to FprEN 1998-1-1:2024[3], (θu^{pl}).

The plastic chord rotation capacity (θ_u^{pl}) at each member end, as defined by expression (15), is compared with the one obtained by nonlinear analysis.

The values θ_{SD}, θ_y, θu and θ_u^{pl} are derived from the work of Maranhão et al.[7]. The chord rotation components $\theta_{SD,NLA}$, $\theta_{y,NLA}$, $\theta_{u,NLA}$ and $\theta_{u,NLA}^{pl}$ are estimated by nonlinear analysis according to the criteria depicted in Figure 13 adapted from FprEN 1998-1-1:2024[2]. The chord rotation at yield, $\theta_{y,NLA}$, corresponding to the attainment of an effective yield rotational strength (M^*_y) linked to the elastic stiffness (Ke) by $K_e = M^*_y / \theta_{y,NLA}$. The ultimate chord rotation of a member ($\theta_{u,NLA}$), corresponding to the attainment of a rotational effective ultimate strength (M^*_u), is estimated considering one of two criteria:

1) The ultimate chord rotation at the member end corresponds to a 20% drop in strength concerning the maximum strength of the primary seismic member, as depicted in Figure 5-13(a)
2) If criteria 1) is not met, it is considered the maximum strength at the end of the pushover, as illustrated in Figure 5-13(b)

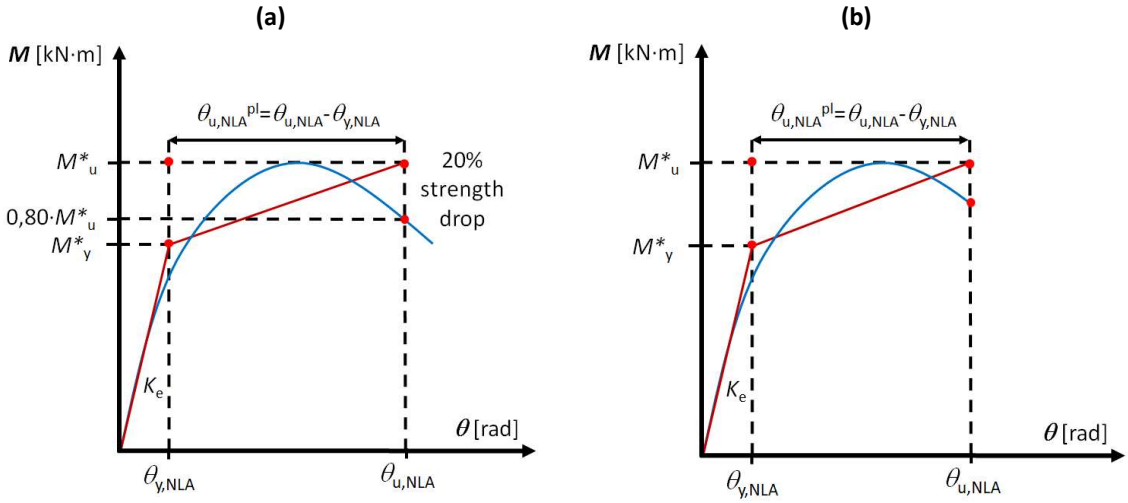


Figure 5-13: Definition of the chord rotation at yield $\theta_{y,NLA}$ and ultimate chord rotation $\theta_{u,NLA}$

The plastic chord rotation capacity obtained by nonlinear analysis ($\theta_{u,NLA}^{pl}$) is calculated as the difference between ultimate chord rotation ($\theta_{u,NLA}$) and chord rotation at yield, ($\theta_{y,NLA}$) as being $\theta_{u,NLA}^{pl} = \theta_{u,NLA} - \theta_{y,NLA}$, and illustrated in Figure 5-13.

To estimate the elements chord rotation that corresponds to the attainment of SD limit state ($\theta_{SD,NLA}$), the Capacity Spectrum Method, also known as the N2 method, developed by Fajfar[24] and preconised in the EN 1998-1:2004[1] and FprEN 1998-1-1:2024[3], is used in the analysis. The objective is to find the target displacement correspondent attained at significant damage limit state (d_{SD}).

The resulting capacity curve of the multi-degree-of-freedom (MDOF) structure should plot the total base shear (V_b) versus the control displacement (d_{top}), which is the node located at the roof buildings. The pushover analysis progresses until a displacement (d_u), where an ultimate local deformation occurs when the global plastic mechanism is attained. Using transformation relations proposed by Fajfar[25], EN 1998-1:2004[1] and FprEN 1998-1-1:2024[3], the curve (V_b-d_{top}) should then be converted into a force–deformation relationship of an equivalent SDOF system (F^*-d^*).

The resulting capacity curve of the MDOF, as shown in Figure 5-14(a) structure, should depict the (V_b) against the (d_{top}). The capacity curve (in Figure 5-14(a)) transformation to the equivalent SDOF model (in Figure 5-14 (b)) is built up by reducing (V_b) and (d_{top}) into the equivalent force (F^*) and the equivalent displacement (d^*_{top}) using the transformation factor (Γ) as defined by expression (5-13):

$$\Gamma = \frac{m^*}{\sum_{i=1}^{n} m_i \cdot \Phi_i^2} \tag{5-13}$$

where (m^*) is the equivalent SDOF system's mass, (m_i) is each storey's inertial mass, and (Φ_i) are the normalised displacements.

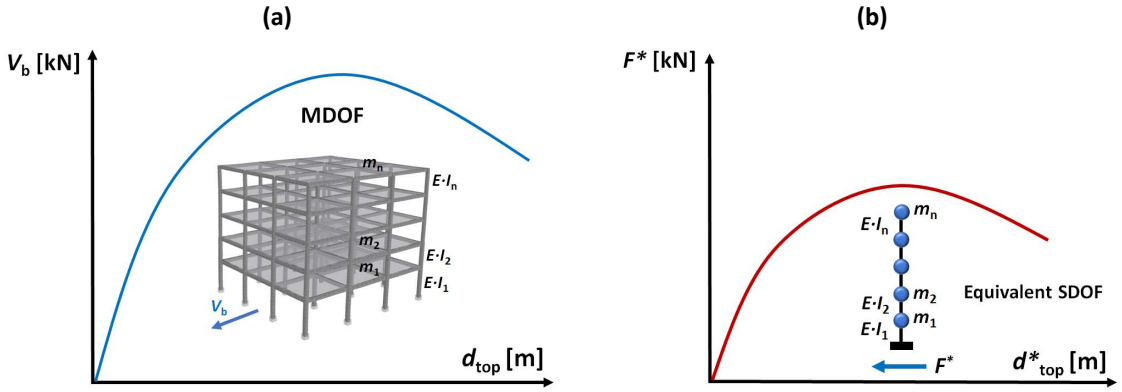


Figure 5-14: Capacity curve reduction to an equivalent SDOF: (a) Capacity curve MDOF; (b) Capacity curve SDOF

FprEN 1998-1-1:2024[3] proposes an idealised elastic-perfectly plastic force-displacement relationship is considered, where the (F^*-d^*) relation may always be idealised as bilinear if the capacity curve is non-decreasing, as depicted in Figure 5-15(a). FprEN 1998-1-1:2024[3] states that if the capacity curve exhibits a peak and a post-peak negative stiffness branch due to geometric nonlinearities member strength degradation, the bilinear idealisation may be adopted up to the peak as shown in Figure 5-15(b).

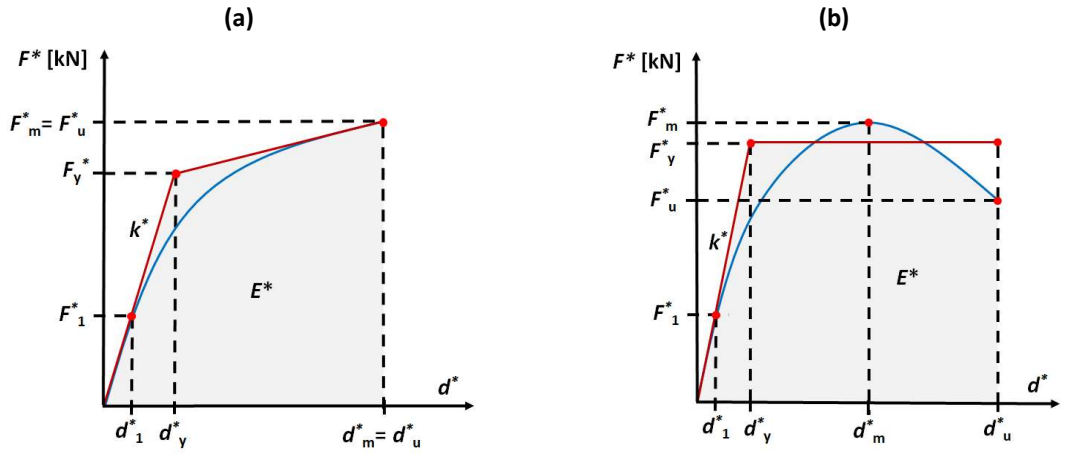


Figure 5-15 Determination of the bilinear idealization of the force-deformation relationship in the case of (a) non-decreasing and (b) decreasing capacity curve (adapted from FprEN 1998-1-1:2024[3])

The coordinates (d^*_1, F^*_1) correspond to the first plastic hinge attainment in the structure. This point defines the slope of the elastic region, characterised by an equivalent stiffness (k^*) as depicted in Equation (5-14):

$$k^* = \frac{F_1^*}{d_1^*} \tag{5-14}$$

The plastification or yield point (d^*_y, F^*_y) is computed such that the dissipated energy (E^*) up to the displacement at the attainment of the global plastic mechanism (d_u). The displacement at yield (d^*_y) is obtained as a function of the actual deformation energy (E^*) up to the formation of the plastic mechanism, following a system of Equations (5-15) preconised in FprEN 1998-1-1:2024[3]. Finally, the coordinates (d^*_m,F^*_m) correspond to the maximum force, as depicted in Figure 5-15.

$$d_y^* = \begin{cases} \dfrac{2 \cdot E^* - F_m^* \cdot d_m^*}{k^* \cdot d_m^* - F_m^*} & \text{for a bilinear idealisation with hardening} \\ d_u^* - \sqrt{{d_u^*}^2 - \dfrac{2 \cdot E^*}{k^*}} & \text{for a bilinear idealisation without hardening} \end{cases} \tag{5-15}$$

Determining the target displacement at significant damage limit state (SD) requires converting the bilinear SDOF force-displacement curve into a bilinear capacity spectrum in the acceleration-displacement response spectrum (ADRS). The displacement (d^*_{SD}) can be calculated by transforming the bilinear (F^*-d^*) in [(F^*/m^*)-d^*] plot. The calculation involves identifying the intersection of the extended first linear segment, as illustrated in Figure 5-16(a), or where the bilinear curve intersects the demand spectrum, as depicted in Figure 5-16(b). This intersection provides the displacement at the significant damage limit state (d^*_{SD}).

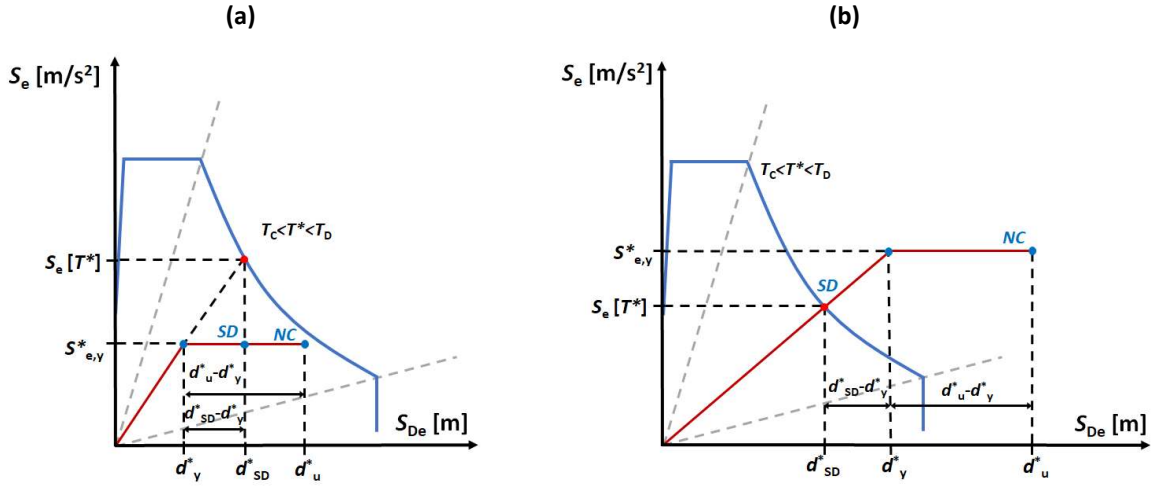


Figure 5-16: Determination of the target displacement for the equivalent SDOF system: (a) Structure with an inelastic response (b) Structure with elastic response (adapted from Franchin[25])

Alternatively, simply using expression (5-16):

$$d_{SD}^* = S_e \cdot \left(\frac{T^*}{2 \cdot \pi}\right)^2 \tag{5-16}$$

Where (S_e) is the acceleration of the elastic response spectrum at the period (T^*). The period (T^*) of the idealised equivalent SDOF system is determined by expression (5-17):

$$T^* = 2 \cdot \pi \sqrt{\frac{m^*}{k^*}} \tag{5-17}$$

To transform SDOF displacement demand to the top displacement of the MDOF model, expression (5-18) may be used:

$$d_{SD} = \Gamma \cdot d^*_{SD} \tag{5-18}$$

Regarding Urbański[27] and Fajfar[25], two situations may occur: (i) If $d^*_{SD} \geq d^*_y$, the structural response is considered inelastic as depicted in Figure 5-16(a); (ii) If $d^*_{SD} < d^*_y$, the structure responds elastically, as shown in Figure 5-16(b). The chord rotation ($\theta_{SD,NLA}$) is obtained from the correspondent structural displacement (d_{SD}) of the bilinear corresponding to the attainment of the SD limit state.

5.6.2 RELATION BETWEEN CHORD ROTATION AND EFFECTIVE CONFINEMENT AT COLUMNS BASE

EN 1998-1:2004[1] demands the assessment of the concrete core confinement in the critical region of a column, where a plastic hinge may form by design, namely at the base columns. The prEN 1998-1-2:2023[4] does not assess the impact of core confinement reinforcement. Instead, it considers that the confinement effectiveness is intrinsic to verifying the chord rotation.

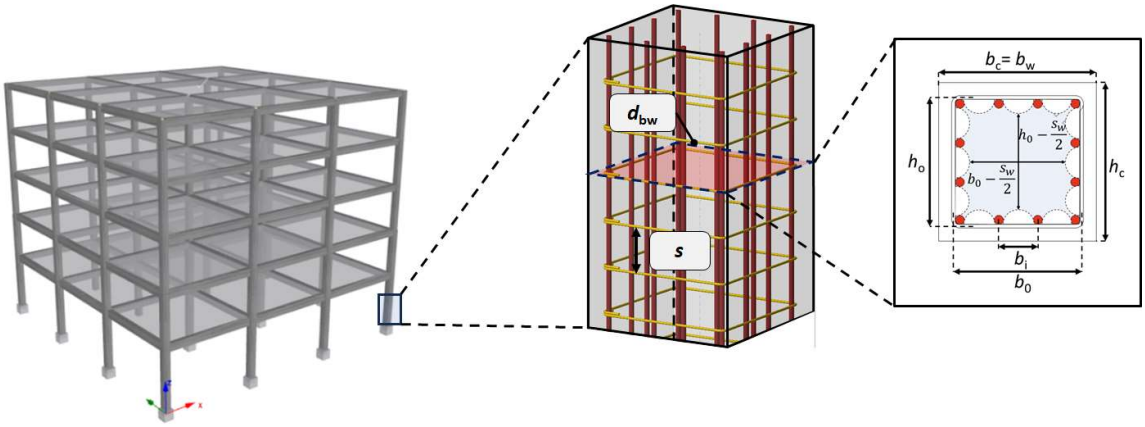


Figure 5-17: Geometrical properties and confinement at the base of a column

To assess the impact of concrete core confinement, EN 1998-1:2004[1] compares the effective concrete core confinement ($\alpha \cdot \omega_{wd}$) with the product of the curvature ductility factor (μ_ϕ), the design value of steel strain at yield ($\varepsilon_{sy,d}$), the axial force ratio (ν_d) and the ratio between cross-sectional dimension of column (b_c) and width of confined core in a column (to centreline of hoops) (b_0), as defined in Equation (5-19) and depicted in Figure 5-17:

$$\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_\phi \cdot \varepsilon_{sy,d} \cdot \nu_d \cdot \frac{b_c}{b_0} - 0{,}035 \tag{5-19}$$

The parameter (α) is the confinement effectiveness factor, and (ω_{wd}) is the mechanical volumetric ratio of confining reinforcement.

Considering expressions (5-11) and (5-19), it is possible to establish a relationship between the plastic chord rotation capacity (θ_u^{pl}) and the mechanical volumetric ratio of the transverse reinforcement (ω_{wd}).

The plastic chord rotation capacity, as described in expression (5-11), is defined according to the Equation that may be rewritten as depicted in Equation (5-20):

$$\theta_u^{pl} = \kappa_{conform} \cdot \kappa_{axial} \cdot \kappa_{reinf} \cdot \kappa_{concrete} \cdot \kappa_{shearspan} \cdot 24^{\left(\alpha \cdot \frac{A_{sw}}{b_w \cdot s_w} \cdot \frac{f_{yw}}{f_c}\right)} \cdot \theta_{u0}^{pl} \tag{5-20}$$

Assuming that the mechanical volumetric ratio of the confining reinforcement (ω_{wd}) is defined according to Equation (5-21):

$$\omega_w = \frac{A_{sw} \cdot (2 \cdot b_0 + 2 \cdot h_0)}{h_0 \cdot b_0^2 \cdot s_w} \cdot \frac{f_{yw}}{f_c} \tag{5-21}$$

Since it is considered squared section columns and for simplification, let's assume that $h_0 \equiv b_0$, as depicted in Figure 5-18.

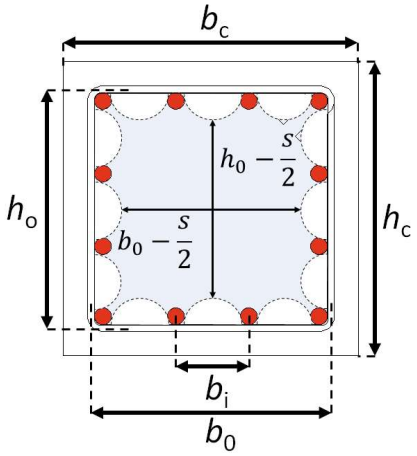


Figure 5-18: Columns confinement

Therefore, the Equation (5-21) is rewritten as in Equation (5-22):

$$\omega_{wd} = \frac{A_{sw} \cdot 4 \cdot b_0}{b_0^2 \cdot s} \cdot \frac{f_{yw}}{f_c} = \frac{A_{sw} \cdot 4}{[b_w - (2 \cdot c + d_{bw})] \cdot s_w} \cdot \frac{f_{yw}}{f_c} \tag{5-22}$$

Where (b_c) is the cross-sectional dimension of the column, c is the concrete cover, (d_{bw}) is the hoop diameter, s is the spacing between hoops, (f_{yw}) is the mean value of yield strength of transverse reinforcement, and (f_c) is the mean of concrete compressive strength.

Regarding Equations (5-20) and (5-22), the effective concrete core confinement as a function of plastic chord rotation capacity [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$] may be written as depicted in Equation (5-23):

$$\alpha \cdot \omega_{wd} = 1{,}26 \cdot b_w \cdot \frac{\ln\left(\frac{\theta_u^{pl}}{\kappa_{conform} \cdot \kappa_{axial} \cdot \kappa_{reinf} \cdot \kappa_{concrete} \cdot \kappa_{shearspan} \cdot \theta_{u0}^{pl}}\right)}{(b_w - 2 \cdot c - d_{bw})} \tag{5-23}$$

It should be noted that (α) for squared cross-section columns is calculated in the same way as for EN 1998-1:2004[1], prEN 1998-1-2:2023[3] and prEN 1992-1-1:2021[23].

Regarding the demonstration above, the objective is to compare the effective core confinement illustrated in expression (5-24) with the effective concrete core confinement depicted in Equation (5-23):

$$a \cdot \omega_{wd} = \left(1 - \sum_{i=1}^{n} \frac{b_i^2}{6 \cdot b_0 \cdot h_0}\right) \cdot \left[\left(1 - \frac{s}{2 \cdot b_0}\right) \cdot \left(1 - \frac{s}{2 \cdot h_0}\right)\right] \cdot \frac{Volume\ of\ the\ confining\ hoops}{Volume\ of\ the\ concrete\ core} \cdot \frac{f_y}{f_c} \quad (5\text{-}24)$$

To ensure consistency in the analysis and to conduct a proper comparison, the design value of yield strength of transverse reinforcement (f_{ywd}) and the design value of concrete compressive strength (f_{cd}) are replaced by their mean values in expression (5-24). In order to perform the comparison, equation (5-23) is calculated by assuming the absolute logarithmic ratio of the nonlinear analysis plastic chord rotation capacity obtained by nonlinear analysis ($\theta_{u,NLA}^{pl}$) and the plastic chord rotation capacity (θu^{pl}) calculated according to FprEN 1998-1-1:2024[3], as depicted in (5-25):

$$\alpha \cdot \omega_{wd} = 1{,}26 \cdot b_w \cdot \frac{\left| ln\left(\frac{\theta_{u,NLA}^{pl}}{\theta_u^{pl}}\right)\right|}{(b_w - 2 \cdot c - d_{bw})} \quad (5\text{-}25)$$

The parameter ($\theta_{u,NLA}^{pl}$) is obtained by nonlinear analysis as explained in 5.6.1, and (θ_u^{pl}) is calculated according to expression (5-11).

5.7 RESULTS AND DISCUSSION

The resulting capacity curves, obtained from adaptive pushover analysis, are shown in Figure 5-19. It is important to refer that both epistemic and aleatory uncertainties influence numerical, theoretical, and computational models of reinforced concrete (RC) structures. Epistemic uncertainties arise from gaps in knowledge and can be reduced, whereas aleatory uncertainties stem from inherent randomness. Studies such as Gino et al.[36] highlight the importance of accounting for both uncertainties.

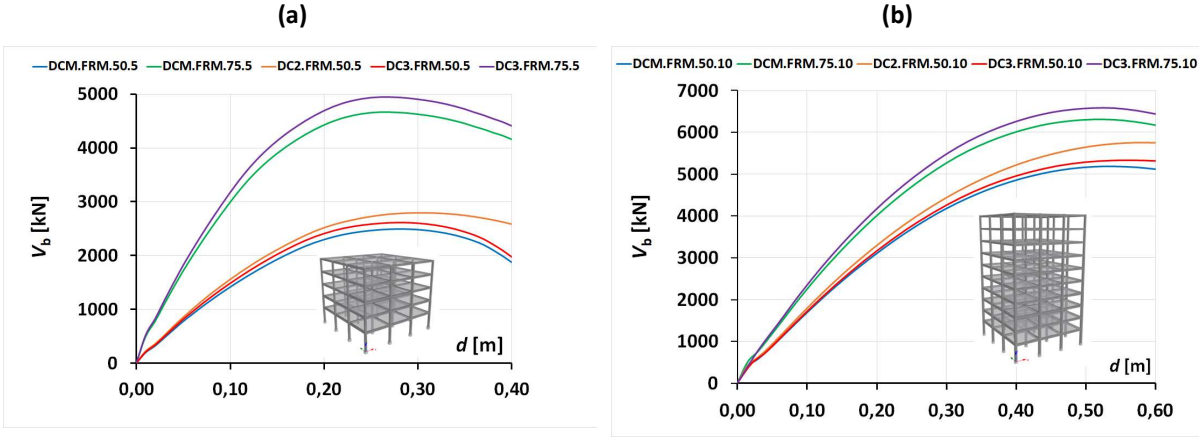


Figure 5-19: Capacity curves(a) 5-storey buildings(b) 10-storey buildings

Figure 5-20 illustrates the ductility ratio (μ=d_r/d_y), where (d_r) is the maximum displacement reached during the analysis, and (d_y) represents the yield displacement. Under identical design response spectra (S_α), the DCM and DC3 frames exhibit more ductile behaviour than DC2.

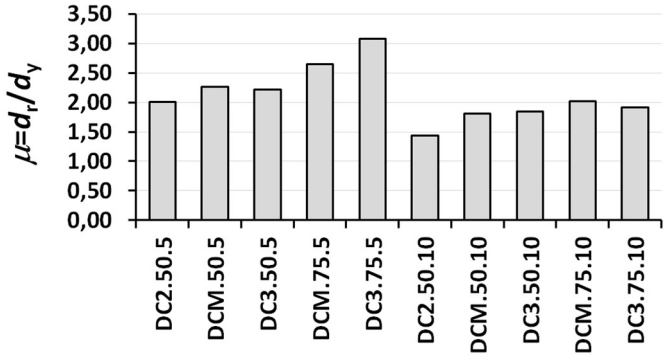


Figure 5-20: Ductility ratio (μ) for the different structures

The following observations can be made from Figure 5-19 and Figure 5-20.

a) The structures DC3.75.5, DC3.50.5, and DCM.75.5 exhibit a more ductile behaviour, indicating they can undergo more significant deformations than their displacements at yield.

b) There is variability in ductility ratios (μ) even within structures of the same category (e.g., DCM.50.5 and DCM.75.5). This suggests that factors such as design parameters or loading conditions may influence the observed ductility.

c) In the case of DC2 structures, specifically DC2.50.10, there is a notable reduction in the ductility ratio when transitioning from DC2.50.5 to DC2.50.10. This observed change could be attributed to the design strategies implemented during the structural design process.

5.7.1 BEHAVIOUR FACTOR ANALYSIS

The parameters extracted from the pushover curves from Figure 5-19 are presented in Table 5-2, allowing the behaviour factor components q_R, q_S, and q_D to be evaluated.

Table 5-2: Nonlinear analysis parameters

Structure label	V_e kN	d_e m	V_1 kN	d_1 m	V_y kN	d_y m	V_u kN	d_r m
DC2.50.5	866,51	0,051	1675,91	0,110	2789,82	0,164	2789,82	0,330
DCM.50.5	532,37	0,033	1414,86	0,140	2494,11	0,155	2494,11	0,350
DC3.50.5	532,37	0,033	1486,93	0,100	2619,09	0,162	2619,09	0,360
DCM.75.5	1236,39	0,034	2993,94	0,100	4949,01	0,136	4949,01	0,360
DC3.75.5	1250,01	0,033	3173,58	0,100	4668,88	0,123	4668,88	0,380
DC2.50.10	1169,98	0,085	3426,40	0,210	5757,81	0,418	5757,81	0,600
DCM.50.10	862,50	0,055	2848,84	0,180	5188,79	0,331	5188,79	0,600
DC3.50.10	867,01	0,053	3166,05	0,200	5338,56	0,326	5338,56	0,600
DCM.75.10	1382,10	0,065	3927,02	0,195	6305,99	0,297	6305,99	0,600
DC3.75.10	1382,10	0,065	4321,87	0,210	6574,47	0,309	6574,47	0,590

The values of the behaviour factor components are presented in Figure 5-21. It should be noted that EN 1998-1:2004[1] does not prescribe the behaviour factor components. However, the DCM structures' behaviour factor components were also calculated to compare the current [1] and second-generation [3] versions of Eurocode 8.

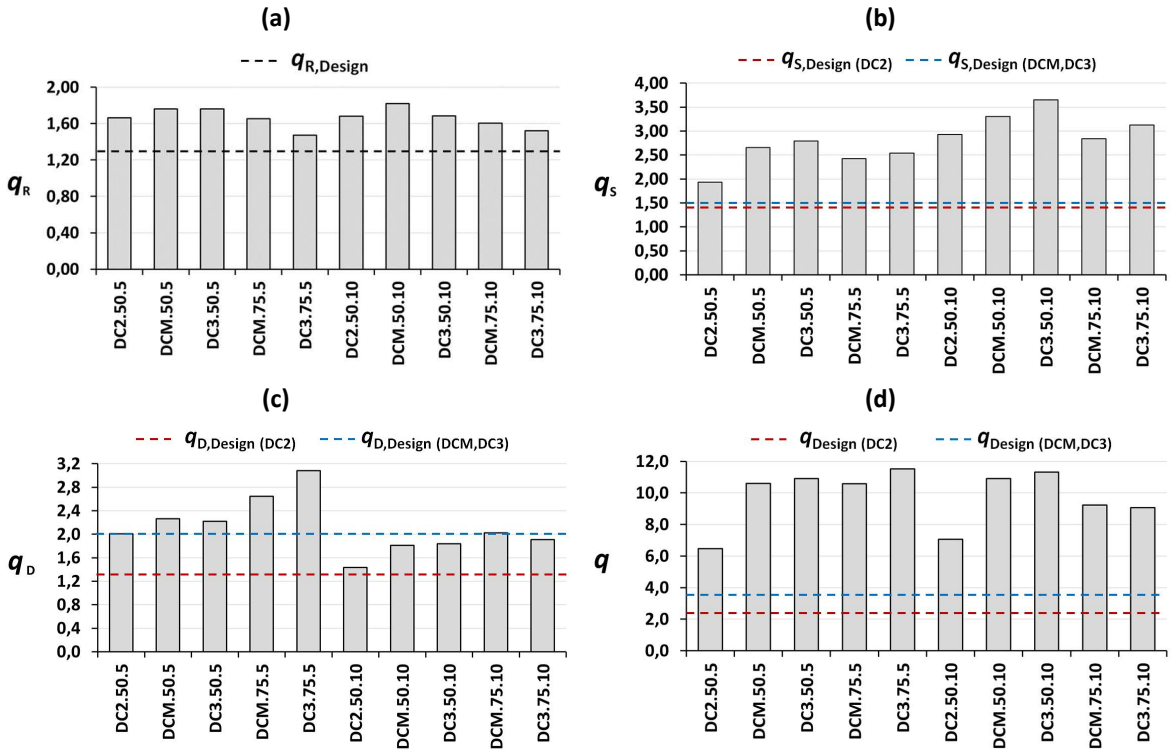


Figure 5-21: Behaviour factor and its components: (a) q_R, (b) q_S, (c) q_D and (d) q

Regarding Figure 5-21, the following observations can be drawn:

a) Upon analysis of Figure 5-21(a), it becomes evident that the behaviour factor component accounting for the overstrength (q_R) exhibits specific ranges. For DC2 structures, the observed range is 1,66 to 1,68, while for DCM structures, it extends from 1,61 to 1,82. For DC3 structures, the range is from 1,47 to 1,76. Notably, all these values surpass the recommended value (q_R=1,3) outlined in prEN 1998-1-2:2023[4], indicating a consistent overperformance in the structural behaviour across the various structures. The acquired results align with findings from other research studies, such as Zhu et al.[28], that reported overstrength ratios for RC-frame structures ranging between 1,2 and 1,7. Similarly, Sharifi et al.[29] observed values falling within the span of 1,2 to 2,2 for ordinary and special RC frames, and Ferraioli[30] observed values between 1,10 and 1,51.

b) In Figure 5-21(b), the behaviour factor component (q_S), defined as the factor representing overstrength from all other sources, takes on values for DC2 structures ranging from 1,93 to 2,93, and for DCM structures, it spans from 2,42 to 3,30. For DC3 structures, it ranges from 2,54 to 3,65. The prEN 1998-1-2:2023[4] recommends q_S as 1,48 for DC2 and 1,5 for DC3 for structures regular in plan and elevation and non-torsionally flexible. Consequently, it can be inferred that prEN 1998-1-2:2023[4] provides a conservative estimate for the overstrength from all other sources (structural redundancy). The number of stories appears to exert the most significant influence on the qS factor, as it increases with the growing number of stories. The analysis is carried out in a three-dimensional environment, considering overstrength from multiple sources. This includes factors such as the torsional resistance of the beams and the redistribution of stresses on the slabs. Incorporating these aspects ensures the formation of multiple load paths, providing a safeguard: if one path is compromised, the load can be effectively redistributed through alternative routes. The capacity design in shear the control of the global mechanism also contributes to redundancy by allowing controlled energy dissipation. These considerations contribute to an augmentation of the values of (q_S).

c) The behaviour factor component (q_D) is a behaviour factor component accounting for the deformation and energy dissipation capacities. Figure 21(c) shows that for DC2 structures, the value of (q_D) averages 1,61. In the case of DCM and DC3 structures, the average values are 2,19 and 2,26, respectively. As can be observed, the behaviour factor component (q_D), on average, is quite close to the one prescribed for DC3. It can also be observed that (q_D) decreases with the number of stories and the seismic action.

d) Figure 5-21(d) shows the behaviour factor (q) calculated according to expression (5-2). In all cases examined here, the q-factor values determined from the nonlinear static analysis were higher than the code-prescribed values. For DC2 structures, the observed range is 5,6 to 7,2, while for DCM structures, it extends from 9,22 to 10,33. Similarly, for DC3 structures, the range is from 9,10 to 10,71. A similar trend was observed also by Louzai et al.[31] and Ferraioli[30]. The results indicate that the structure's height also influences the elevated value of the behaviour factor (q). Figure 5-21(c) shows that the (q_S) factor is the primary contributor to the higher values compared to the behaviour factor (q). Specifically, the (q_S) component accounts for 46% of the behaviour factor (q).

e) The (q) values presented in the plots are potential outcomes derived from non-linear analysis under various conditions. These values represent the system's potential performance. Further studies must be conducted to validate and refine these design values.

5.7.2 ESTIMATION OF THE BILINEAR PARAMETERS

The participation factors converted the previous capacity curves into equivalent SDOF force-deformation relations (Γ). Such capacity spectra were then bilinearised according to the indications of FprEN 1998-1-1:2024[3], as described in subsection 5.6.1. The parameters of the bilinearised capacity spectra are reported in Table 5-3.

Table 5-3: Elastic properties, effective elastic period, and target displacements

Structure label	$S_{e,SD}$ m/s²	T_1 s	Γ	T^* s	d^*_y m	d^*_{SD} m
DC2.50.5	1,68	1,43	1,32	1,49	0,137	0,095
DCM.50.5	1,60	1,49	1,31	1,56	0,134	0,099
DC3.50.5	1,64	1,49	1,32	1,52	0,130	0,097
DCM.75.5	3,40	1,03	1,35	1,10	0,121	0,106
DC3.75.5	3,50	1,02	1,35	1,07	0,134	0,102
DC2.50.10	1,25	2,10	1,41	2,01	0,235	0,128
DCM.50.10	1,20	2,14	1,41	2,09	0,234	0,132
DC3.50.10	1,23	2,14	1,41	2,04	0,226	0,130
DCM.75.10	1,99	1,92	1,38	1,89	0,212	0,179
DC3.75.10	2,01	1,92	1,38	1,87	0,222	0,177

The target displacements for the limit state SD were obtained with the N2 method, as described in subsection 5.5.1.

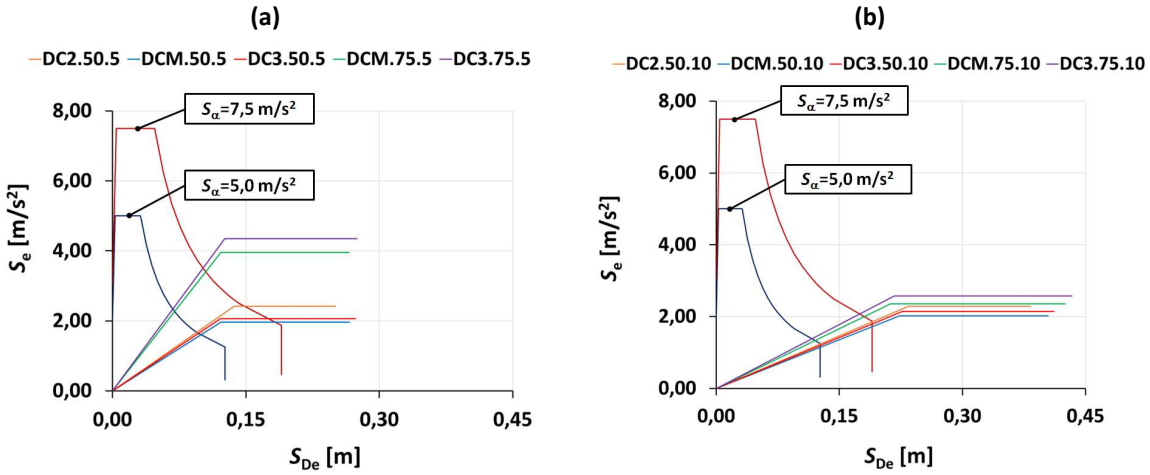


Figure 5-22: Determination of the bilinear idealization of the capacity curve: (a) 5-storey buildings; (b) 10-storey buildings

Regarding Table 5-3 and Figure 5-22, the following conclusions can be drawn:

a) It can be seen that (d^*_{SD}) is lower than (d^*_y) for all structures, meaning that idealised elastoplastic structures remain in the elastic range. This observation may be interpreted as the structure experiencing forces or deformations below the point of yielding, indicating that the structure is not being pushed beyond its intended capacity under the given spectrum. This is generally considered favourable, as it implies that the structure has a certain margin of safety against the forces associated with the analysed seismic conditions.

b) The structure aligns with the principles of capacity design. The capacity design is an approach that intentionally allows structural components (columns and beams) to yield in a controlled manner while other elements remain elastic.

c) In capacity design, the structures are designed to detail specific parts of a structure that undergo plastic deformations (yield) during seismic events. This ensures that the structure behaves in a ductile and predictable manner, dissipating seismic energy through controlled yielding. The observation that ($d^*_{SD}<d^*_y$) suggests that, in the context of a structure designed using capacity design features and based on the analysis using the capacity demand spectrum, the current demand on the structure is within the elastic range or controlled yielding range, and it has not yet reached the point of uncontrolled yielding or failure. This aligns with the objectives of capacity design preconised by EN 1998-1:2004[1] and the prEN 1998-1-2:2023[4], where yielding is intentionally concentrated in specific locations to enhance the seismic performance of the overall structure. In the context of seismic design and, more specifically, the capacity design (control of the global mechanism), the relationship between the capacity demand (d^*_{SD}) and the yield demand (d^*_y) is such that ($d^*_{SD}<d^*_y$) is a reflection of intentional and controlled yielding of the structural elements of a structure.

d) Capacity design ensures that key structural elements such as beams and columns possess adequate strength and ductility to withstand seismic forces. This approach typically dictates that beams fail first under designed seismic conditions, preserving the columns. The observed behaviour under seismic loads, including which parts fail first and how structural integrity is maintained, is influenced by this design principle. Nonlinear analysis confirms this approach by demonstrating that beams are the first to develop plastic hinges before columns. This corroborates the "strong column-weak beam" deterministic design strategy, as demonstrated in Figure 5-23, specifically with examples like DC2.50.5 and DC2.50.10. The illustration helps explain that the first hinges form beyond the linear elastic base shear (V_e) used in the design, surpassing the deterministic values prescribed in the current and second generation of Eurocode 8. This may significantly influence the observed results in Figure 5-22 and corroborate with the findings of item c).



Figure 5-23: First hinges appearance (a) DC2.50.5 (b) DC3.50.5

e) The intentional yielding in capacity design reduces the overall demand on the structure during a seismic event. This is reflected in the relationship ($d^*_{SD}<d^*_y$) where (d^*_{SD}) represents the demand based on the capacity design, and (d^*_y) represents the yield demand.

f) Proper core confinement of the bottom of columns and the control of the chord rotation can reduce the demand on structural elements during a seismic event. The reduced demand, reflected in (d^*_{SD}), signifies that the structure is designed to undergo controlled yielding in localised regions without exceeding the intended capacity.

5.7.3 ESTIMATION OF THE CHORD ROTATION PARAMETERS OF COLUMNS AND BEAMS

This subsection compares and discusses the values obtained as defined in subsection 5.1. The following analysis is divided by chord rotation components.

It is observed that there is a dispersion between the pairs of values (θ_{SD}; $\theta_{SD,NLA}$), (θ_y;$\theta_{y,NLA}$), (θ_u;$\theta_{u,NLA}$), (θ_u^{pl};$\theta_{u,NLA}^{pl}$). The dispersion of points for chord rotation components is calculated according to the given equations (5-9), (5-10), (5-11) and (5-12)from FprEN 1998-1-1:2024[3] and chord rotation components from pushover analysis may arise due to several reasons:

a) Simplifications and assumptions: The equations (5-9), (5-10), (5-11) and (5-12) are simplified analytical expressions derived from several numerical studies and laboratory testing such as the works performed by Panagiotakos et al.[32] and Biskinis et al.[33], or simplifications in the structural model or material behaviour. On the other hand, pushover analysis may consider more complex and realistic aspects of the structure, leading to differences in results.

b) Modelling accuracy: Pushover analysis involves more detailed and accurate modelling of the structure, considering factors like material nonlinearity, geometric nonlinearities, and boundary conditions. The analytical equation may overlook certain complexities, leading to disparities in results.

c) Numerical Methods: Pushover analysis often employs numerical methods to solve the nonlinear equations governing the structural response. The analytical equations may not consider all the complexities these numerical methods capture.

d) Dynamic effects: Pushover analysis can also consider dynamic effects, while the analytical equation might be static or semi-static. Dynamic effects, such as mass participation and damping, can influence the results.

5.7.3.1 CHORD ROTATION ANALYSIS AT THE ATTAINMENT OF SD LIMIT STATE

Figure 5-24 compares the chord rotation that corresponds to the attainment of the SD limit state obtained by nonlinear analysis ($\theta_{SD,NLA}$) against the chord rotation that corresponds to the attainment of the SD limit state (θ_{SD}) calculated according to FprEN 1998-1-1:2024[3] regarding the bottom and top ends of the columns, and left and right ends of beams, for the structures under analysis.

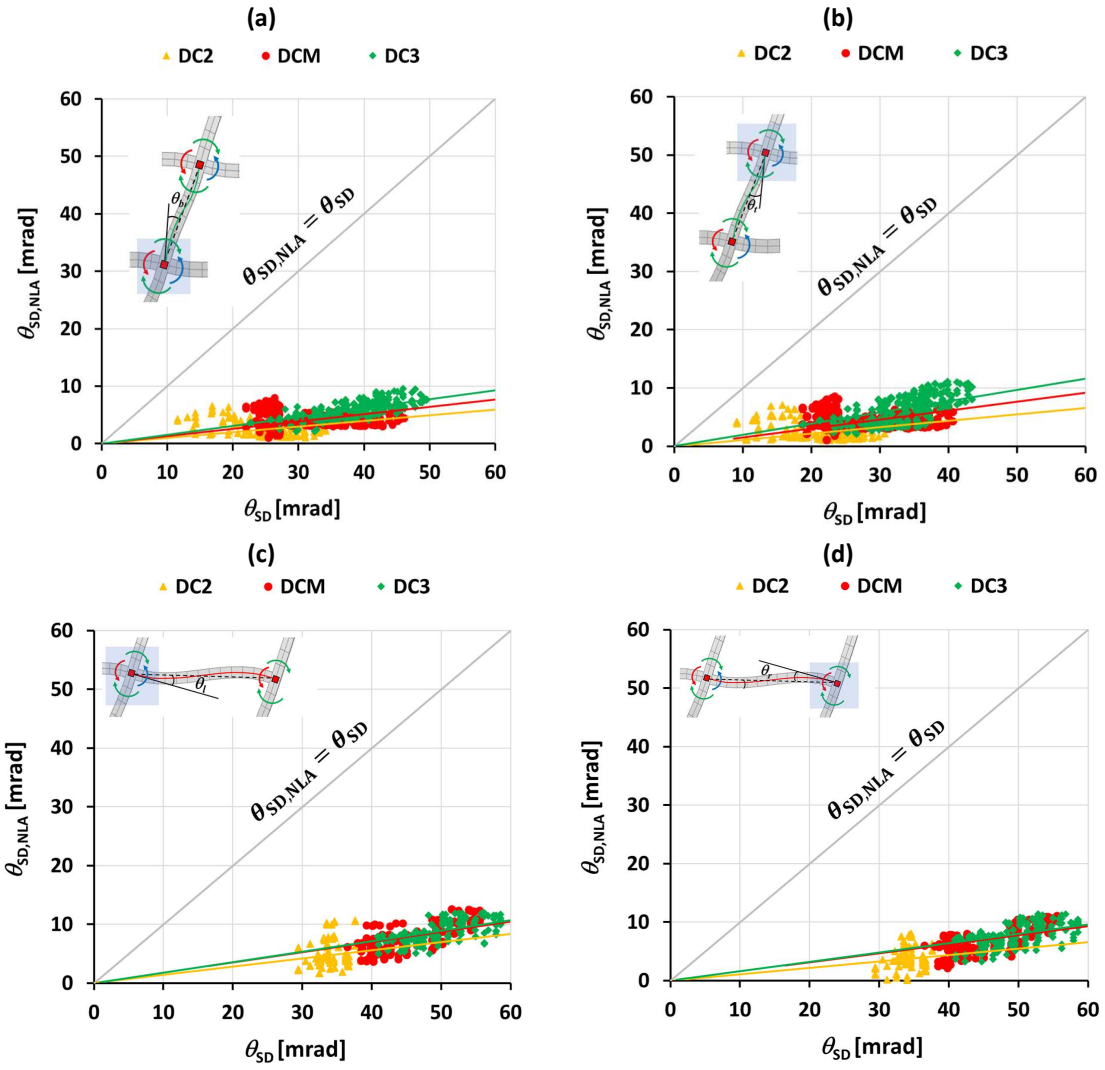


Figure 5-24: Chord rotation corresponding to the attainment of SD limit state: (a) θ_{SD} vs $\theta_{SD,\,NLA}$ at of columns bottom end; (b) θ_{SD} vs $\theta_{SD,NLA}$ of columns at the top end; (c) θ_{SD} vs $\theta_{SD,\,NLA}$ at beam left end (d) θ_{SD} vs $\theta_{SD,\,NLA}$ at beam right end

Regarding Figure 5-24, the following observations can be made:

a) The chord rotation that corresponds to the attainment of the SD limit state obtained calculated according to (θ_{SD}) FprEN 1998-1-1:2024[3] is much higher than the chord rotation that corresponds to the attainment of the SD limit state obtained by nonlinear analysis ($\theta_{SD,NLA}$). Regarding the columns, the differences between the median (θ_{SD}=$\theta_{SD,NLA}$) and the trend lines are between 85% for DC3 and 90% for DC2, with DCM in the middle with 87%, as depicted in Figure 24(a) and (b). Concerning the beams, the

differences between the median and the trend lines are 82% for DC3 and 83% for DC2, with DCM in the middle with 83%, as illustrated in Figure 24(c) and (d).

b) The observations made in item a) may be explained by the fact that the structure responds elastically ($d_{SD}<d_y$), and attending the expression (5-10), it is concluded that the FprEN 1998-1-1:2024[3] calculation of (θ_{SD}) for this type of structures is conservative. The verification outlined in expression (5-7), as recommended by prEN 1998-1-2:2023[4], is intended to ensure that structures that may exhibit inelastic behaviour meet safety provisions for chord rotation.

c) Observations of column elements presented in Figure 5-24(a) and (b) reveal that it is consistent with expectations. The ductility classes of DCM and DC3 demonstrate higher values of (θ_{SD}) compared to those of DC2. This distinction highlights the higher ductility performance of DCM and DC3 over DC2. The trend line for DC3 has a slope approximately 56% steeper than that of DC2, indicating a significantly sharper increase. On the other hand, the trend line for DCM exhibits a slope about 22% steeper than DC2's, suggesting a more moderate increase compared to DC3's steepness.

d) Concerning the beam analysis, the ductility classes of DCM and DC3 still demonstrate higher values of (θ_{SD}) compared to those of DC2, but the trend lines are less steep. The trend line for DC3 has a slope approximately 28% steeper than that of DC2, indicating a significantly sharper increase. On the other hand, the trend line for DCM exhibits a slope about 19% steeper than that of DC2. It should be noted that the difference between DC3 and DCM is about 4%.

e) Regarding the findings in items c) and d), the differences may come from the detailing, axial forces and stiffness.

f) The chord rotation at the attainment of the SD limit state (θ_{SD}) is slightly more significant at the top of the columns, as illustrated in Figure 24(a), than at the bottom of the columns, as represented in Figure 5-24 (b). This can be attributed to the fact that, during the design of the structures, there is a decrease in stiffness with increasing height.

5.7.3.2 CHORD ROTATION ANALYSIS AT YIELD

Figure 5-25 compares the chord rotation at yield obtained by nonlinear analysis ($\theta_{y,NLA}$) against the chord rotation at yield (θ_y) calculated according to FprEN 1998-1-1:2024[3] regarding the bottom and top ends of the columns for the structures in question.

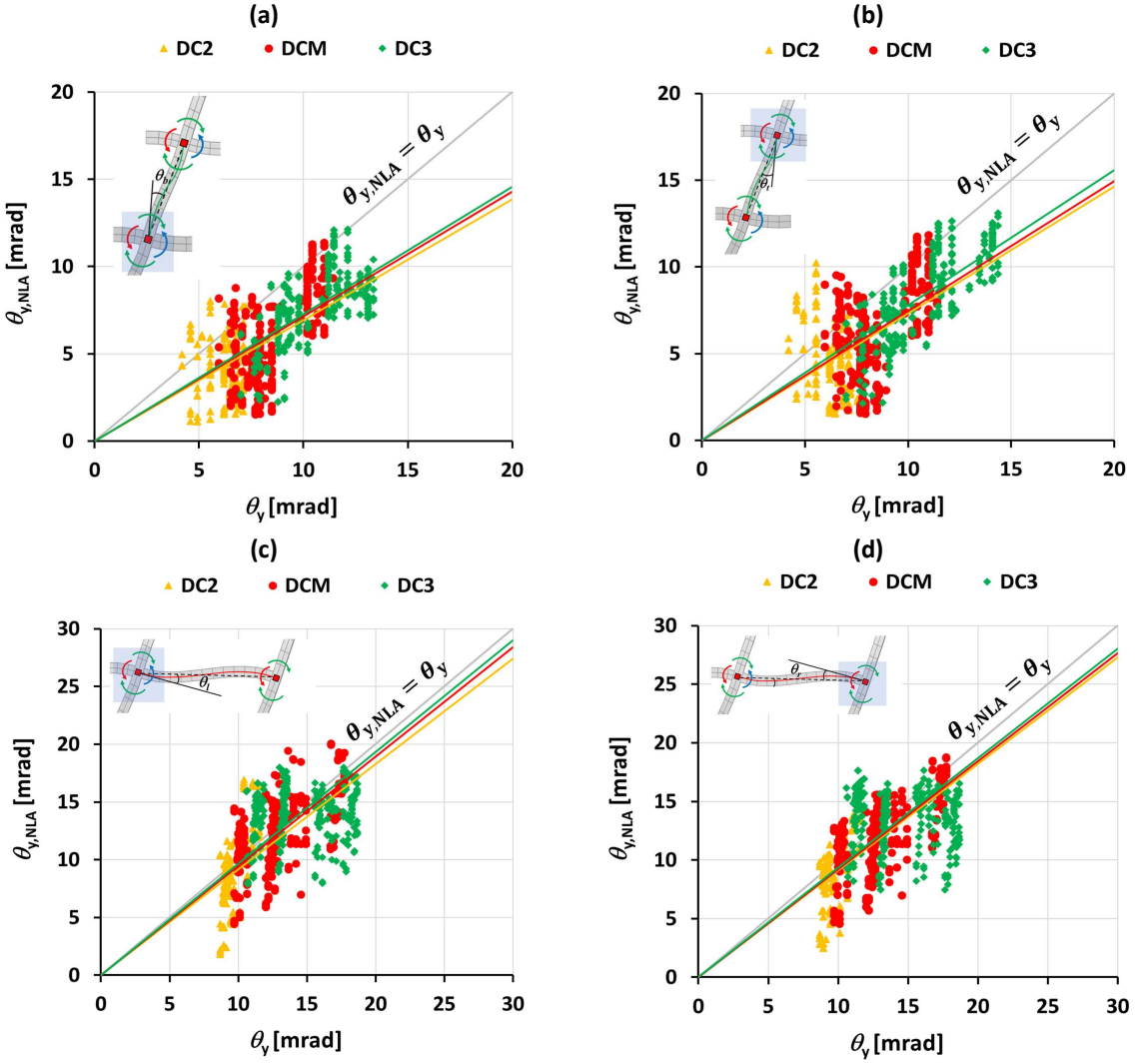


Figure 5-25: Chord rotation at yield: (a) θ_y vs $\theta_{y,NLA}$ at bottom end (b)θy vs $\theta_{y,NLA}$ at top end (c) θ_y vs $\theta_{y,NLA}$ at beam left end (d) θ_y vs $\theta_{y,NLA}$ at beam right end

Concerning Figure 5-25, the following conclusions and observations can be derived:

a) Not all elements achieve the yield point; therefore, not all element's chord rotations at yield are represented in the plots of Figure 5-25.

b) In the present analysis, the paired *t*-test, a statistical test used to determine whether there is a significant difference between the means of two related groups, such as ($\theta_{y,NLA}$) and (θ_y), was performed. It is called paired because it compares measurements or observations from the same subjects or related entities. The paired *t*-test is commonly used in experimental and pre-post-study designs. The paired *t*-test assumes that the differences between the pairs of measurements.

The *t*-test is calculated based on the differences between paired observations, considering the standard deviation of these differences. The *p*-value associated with the *t*-statistic is then compared to a significance level of 0,01 to determine whether to reject the null hypothesis. If the p-value is less than the significance level, the null hypothesis is rejected, indicating a significant difference. The t in the *t*-test stands for "Student's t-distribution". In the analysis context, the paired *t*-test was performed for each configuration (DC2, DCM, DC3) to compare the values between ($\theta_{y,NLA}$) and (θ_y). The resulting *t*-test and p-value help determine whether a statistically significant difference exists between the two scenarios for each configuration. The analysis is performed using the IBM SPSS Statistics software [34]. Table 5-4 shows the *t*-test for the columns at the end regarding the values described in Figure 5-25 (a) and (b) and for beams as illustrated in Figure 5-25(c) and (d).

Table 5-4: Paired *t*-test for the pair ($\theta_{y,NLA}-\theta_y$) for columns and beams ends [34]

Ductility class	Element	Paired Differences ($\theta_{y,NLA};\theta_y$)					*t*	Significance	
		Mean mrad	Std. Deviation mrad	Std. Error Mean	95% Confidence Interval of the Difference			One-Sided *p*	Two-Sided *p*
					Lower	Upper			
DC2	Column	-0,70	1,83	0,20	-1,11	-0,30	-3,45	<0,01	<0,01
DCM		-2,52	1,91	0,08	-2,69	-2,35	-28,93	<0,01	<0,01
DC3		-2,85	1,61	0,07	-2,99	-2,70	-38,70	<0,01	<0,01
DC2	Beam	-0,98	2,71	0,20	-1,38	-0,58	-4,84	<0,01	<0,01
DCM		-0,57	2,54	0,13	-0,84	-0,31	-4,29	<0,01	<0,01
DC3		-0,46	3,53	0,18	-0,82	-0,41	-4,46	<0,01	<0,01

A *t*-test with a p-value less than 0,01 indicates that the observed difference between the two variables is statistically significant at a 1% significance level. In other words, there is strong evidence to reject the null hypothesis, suggesting that the difference is unlikely to have occurred by random chance alone. It should be noted that the negative mean values in the paired differences for the *t*-test on ductility classes (DC2, DCM, DC3) indicate that, on average, the values of the variable ($\theta_{y,NLA}$) are lower than their corresponding values of (θ_y). The paired differences represent the change or difference between the two related measurements. In the analysis context, a negative mean suggests that ($\theta_{y,NLA}$) values are lower than (θ_y)across the paired observations for DC2, DCM, and DC3. A negative sign indicates a decline in the variable of interest when comparing ($\theta_{y,NLA}$) to (θ_y). This observation implies that the calculation, following FprEN 1998-1-1:2024[3], for (θ_y), as outlined in Equation (5-9), is conservative.

c) It is also observed that DC3 has a higher average pair ($\theta_{y,NLA};\theta_y$) with 9,13 mrad, compared to DCM with 7,27 mrad and DC2 with 5,26 mrad, suggesting a trend towards increased angular displacement in DC3, potentially indicating greater ductility in its response.

d) It is noticed also that there are some differences between the bottom and top ends of the columns. This may be explained by the difference in stiffness beams and columns concurring at the nodes and the change of the column's cross-section in height.

5.7.3.3 ULTIMATE CHORD ROTATION ANALYSIS

Figure 5-26 compares the ultimate chord rotation obtained by nonlinear analysis ($\theta_{u,NLA}$) against the ultimate chord rotation (θ_u) calculated according to FprEN 1998-1-1:2024[3] regarding the bottom and top end of the columns and left and right ends of beams.

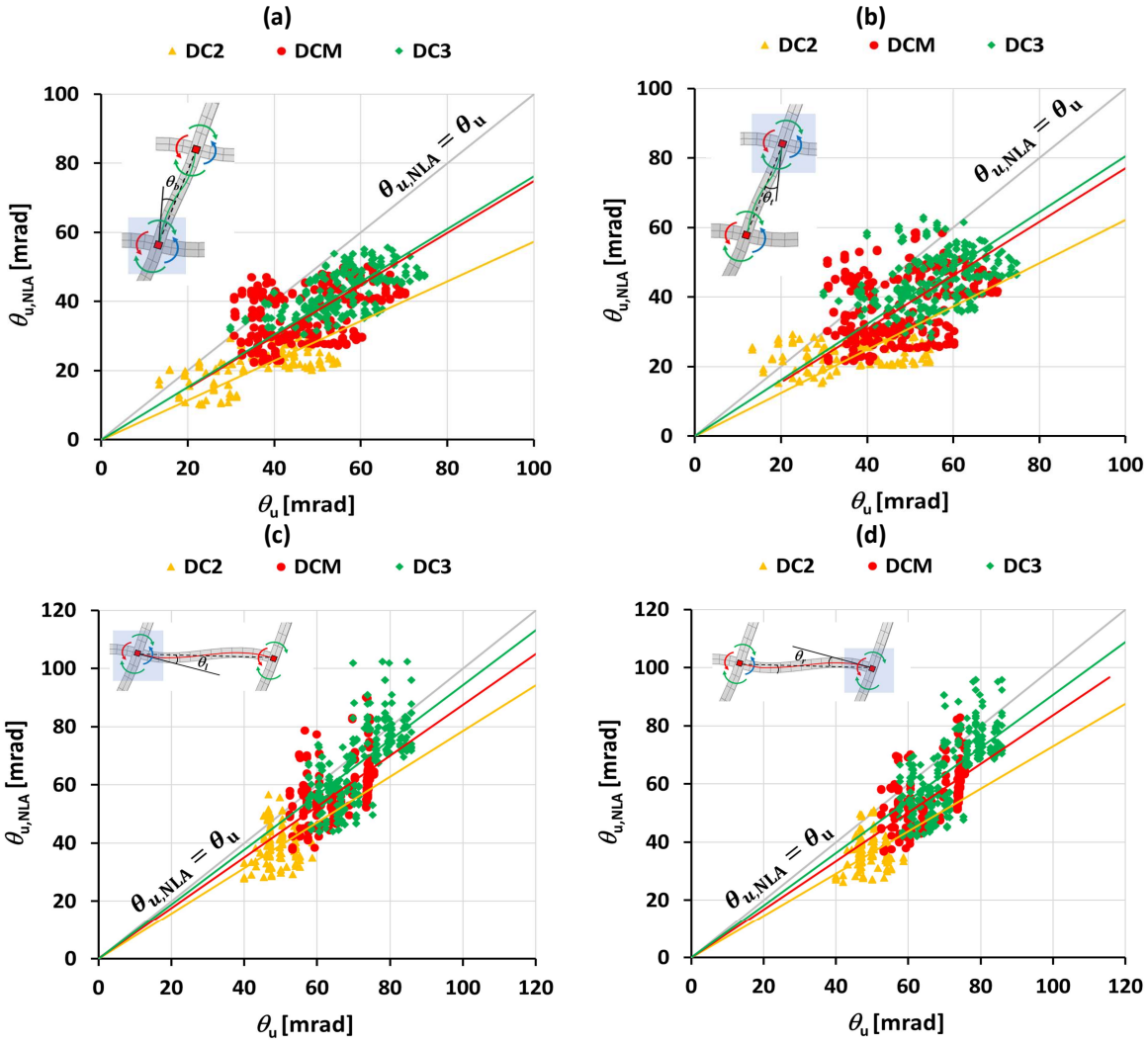


Figure 5-26: Ultimate chord rotation: (a)θ_u *vs* $\theta_{u,NLA}$ at bottom end(b) θ_u vs $\theta_{u,NLA}$ at top end(c) θ_u *vs* $\theta_{u,NLA}$ at beam left end(d) θ_u *vs* $\theta_{u,NLA}$ at beam right end

Regarding Figure 5-26, one can draw the following conclusions and observations:

a) It should be noted that not all the elements achieve the yield point and, therefore, not the ultimate state. For this reason, not all element's chord rotations at the ends are represented in the plots of Figure 5-26.

b) The analysis was also performed using the paired *t*-test to determine whether there is a significant difference between the means of two related groups, such as ($\theta_{u,NLA}$) and (θ_u), as presented in Table 5-5.

Table 5-5: Paired *t*-test for the pair ($\theta_{u,NLA}-\theta_u$) for columns and beams ends [34]

Ductility class	Element	Paired Differences ($\theta_{u,NLA};\theta_u$)					*t*	Significance	
		Mean mrad	Std. Deviation mrad	Std. Error Mean	95% Confidence Interval of the Difference			One-Sided *p*	Two-Sided *p*
					Lower	Upper			
DC2	Column	-13,16	9,83	0,63	-14,41	-11,91	-20,74	<0,01	<0,01
DCM		-10,75	10,19	0,46	-11,67	-9,84	-23,11	<0,01	<0,01
DC3		-12,53	7,31	0,33	-13,18	-11,87	-37,54	<0,01	<0,01
DC2	Beam	-10,40	7,24	0,53	-11,46	-9,33	-19,27	<0,01	<0,01
DCM		-8,17	8,63	0,45	-9,07	-7,27	-17,95	<0,01	<0,01
DC3		-4,29	9,79	0,51	-5,31	-3,28	-8,32	<0,01	<0,01

i) For the columns ends, it can be observed that for pairs ($\theta_{u,NLA},\theta_u$) for the ductility classes (DC2, DCM and DC3) from Figure 5-26(a) and (b), the p-values are low, which suggests strong evidence to reject the null hypothesis. In practical terms, it implies a statistically significant difference between the measurements in each pair based on the provided statistical analysis. In statistical terms, a "statistically significant difference" means that the observed difference between the paired measurements is unlikely to have occurred by random chance alone, implying a relationship between ($\theta_{u,NLA}$) and (θ_u). The negative values of the mean differences in each pair indicate that ($\theta_{u,NLA}$) in each pair ($\theta_u,\theta_{u,NLA}$) is, on average, lower than the (θ_u).

ii) There is a statistically significant difference for the beam ends, meaning that the observed gap between paired measurements is highly unlikely to be just a random occurrence. This suggests a relation between ($\theta_{u,NLA}$) and (θ_u). The fact that the average difference in each pair is negative indicates that, on average, ($\theta_{u,NLA}$) is lower than (θ_u) in those pairs. These findings suggest a strong relationship exists between the values of (θ_u) calculated according to FprEN 1998-1-1:2024[3].

c) It is concluded that the values of (θ_u) determined through linear analysis are higher than those obtained by nonlinear analysis. This suggests that the design provisions of FprEN 1998-1-1:2024[3] are conservative, ensuring a safe design concerning controlling the chord rotation.

d) It is observed that the values regarding the columns, the differences between the median ($\theta_u=\theta_{u,NLA}$), and the trend lines are between 24% for DC3 and 40% for DC2, with DCM in the middle with 25%, as depicted in Figure 5-26(a) and (b). Concerning the beams, the differences between the median and the trend lines are 22% for DC3 and 6% for DC2, with DCM in the middle at 13%, as illustrated in Figure 5-26(c) and (d).

e) Analysis of the column elements shown in Figure 5-26(a) and (b) indicates that, as expected, the ductility classes of DCM and DC3 register higher values of (θ_u) when compared to DC2. This difference underscores the superior ductility performance of DCM and DC3 relative to DC2. The slope of the trend line for DC3 is roughly 20% steeper than that of DC2, reflecting a considerably sharper rise. Conversely, the slope of the trend line for DCM is about 10% steeper than that of DC2, indicating a relatively moderate increase in steepness compared to DC3.

f) The disparities are less pronounced concerning the beams than in columns, as shown in Figure 5-27(c) and (d). The ductility classes of DCM and DC3 continue to show higher values of (θ_u) than those of DC2, though the trend lines are not as steep. The trend line for DC3 has a slope approximately 28% steeper than that of DC2, signifying a notably sharper increase. Meanwhile, the slope for the trend line of DCM is around 19% steeper than that of DC2. It is important to note that the difference in steepness between DC3 and DCM stands at about 4%.

g) Notably, DC3 exhibits a higher linear progression regarding the pairs ($\theta_{u,NLA}$;θ_u), while DC2 and DCM show smaller values. This discrepancy arises from the design considerations adhering to the FprEN 1998-1-1:2024[3], prEN1998-1-2:2023[4], and EN 1998-1:2004[1] standards. The evident difference in (θ_u) between DC3 and DCM underscores variations in the design provisions specified by prEN1998-1-2:2023[4] and EN 1998-1:2004[1].

5.7.3.4 PLASTIC CHORD ROTATION CAPACITY ANALYSIS

Figure 5-27 shows the plastic chord rotation capacity obtained by nonlinear analysis ($\theta_{u,NLA}^{pl}$) against the plastic chord rotation capacity (θ_u^{pl}) calculated according to FprEN 1998-1-1:2024[3] at the bottom and top ends of the columns for the structures in analysis.

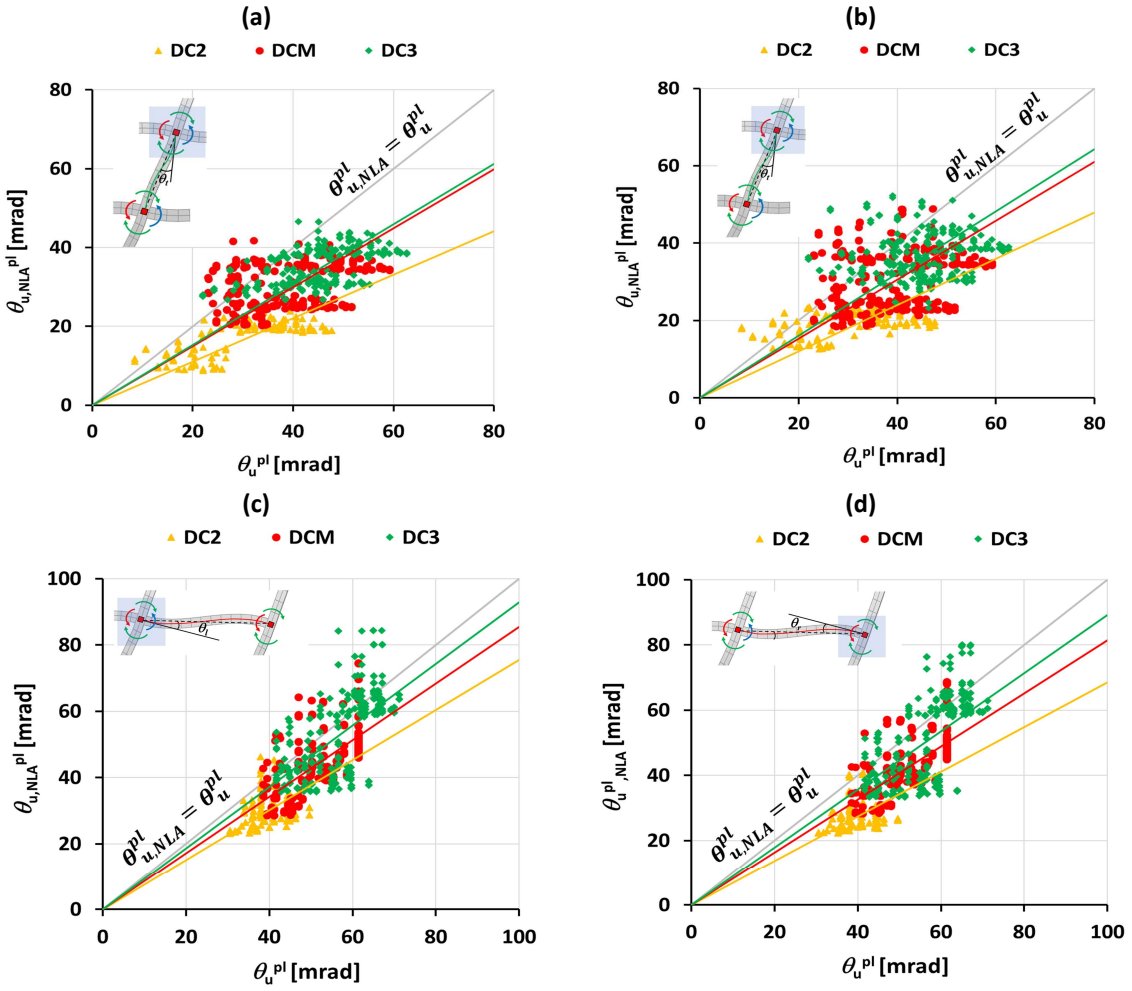


Figure 5-27: Plastic chord rotation capacity: (a) θ_u^{pl} vs $\theta_{u,NLA}^{pl}$ at bottom end(b) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at top end(c) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at beam left end (d) θ_u^{pl} *vs* $\theta_{u,NLA}^{pl}$ at beam right end

Regarding Figure 5-27, one can draw the following conclusions and observations:

a) As referred to in 5.6.1, the plastic chord rotation capacity derived from nonlinear analysis ($\theta_{u,NLA}^{pl}$) is contrasted with the plastic chord rotation capacity (θ_u^{pl}) computed according to FprEN 1998-1-1:2024[2]. The plastic chord rotation capacity (θ_u^{pl}) at each member end, defined by expression (15), is juxtaposed with the one determined through nonlinear analysis, expressed as ($\theta_{u,NLA}^{pl}=\theta_{u,NLA}-\theta_{y,NLA}$). The present analysis also performed the paired t-test to examine whether ($\theta_{u,NLA}^{pl}$) and (θ_u^{pl}) are related. Table 5-6 shows the t-test results for the columns at the end regarding the values described in Figure 5-27 (a) and (b) and for beams as illustrated in Figure 5-27(c) and (d).

Table 5-6: Paired *t*-test for the pair ($\theta_{u,NLA}^{pl} - \theta_u^{pl}$) for columns and beams ends[34]

Ductility class	Element	Paired Differences ($\theta_{u,NLA}^{pl}$;θ_u^{pl})					*t*	Significance	
		Mean mrad	Std. Deviation mrad	Std. Error Mean	95% Confidence Interval of the Difference			One-Sided *p*	Two-Sided *p*
					Lower	Upper			
DC2	Column	-11,34	8,65	0,55	-12,44	-10,24	-20,30	<0,01	<0,01
DCM		-8,43	9,31	0,42	-9,26	-7,59	-19,83	<0,01	<0,01
DC3		-9,66	7,31	0,33	-10,31	-9,00	-28,93	<0,01	<0,01
DC2	Beam	-9,48	5,64	0,42	-10,31	-8,65	-22,51	<0,01	<0,01
DCM		-7,60	7,23	0,38	-8,35	-6,85	-19,93	<0,01	<0,01
DC3		-4,22	9,46	0,49	-5,20	-3,24	-8,46	<0,01	<0,01

   i) At the ends of the columns, it is noticeable that the p-values are consistently low across the pairs ($\theta_{u,NLA}^{pl}$,θ_u^{pl}) for all ductility classes (DC2, DCM and DC3). This implies that the observed distinction between the paired measurements is unlikely to be a result of random chance alone, suggesting a meaningful relationship between and (θ_u^{pl}), as evidenced by the negative mean differences, indicating that (θ_u) is generally higher than ($\theta_{u,NLA}^{pl}$) in each pair ($\theta_{u,NLA}^{pl}$,θ_u).

   ii) For the beam ends, the same observations and conclusions can made

b) Table 5-6 shows a strong relationship between the values of (θ_u^{pl}) obtained calculated using expression (5-16) and the ones obtained by nonlinear analysis ($\theta_{u,NLA}^{pl}$) since the *p*-value is smaller than 0,01.

c) This analysis further supports the findings discussed in Figure 20. The linear progression of paired function values ($\theta_{u,NLApl}$; θ_u^{pl}) of Figure 5-27 shows that DC3 demonstrates greater ductility than DCM and DC2 in descending order, aligning with the established design provisions.

d) The column analysis in Figure 27 (a) and (b) shows that DCM and DC3 have higher ductility values (θu^{pl}) than DC2, with DC3's trend line being 32% steeper and DCM's 23% steeper, indicating a sharper and moderate increase, respectively.

e) For the beams in Figure 27(c) and (d), the differences in ductility values (θu^{pl}) between DCM, DC3, and DC2 are smaller. DC3's trend line is 23% steeper than DC2's, while DCM's is 12% steeper, with an 8% difference in steepness between DC3 and DCM.

5.7.4 ESTIMATION OF THE RELATIONSHIP BETWEEN EFFECTIVE CONCRETE CORE CONFINEMENT AND CHORD ROTATION

As referred to in 5.6.2, the prEN 1998-1-2:2023[3] considers the confinement effectiveness intrinsic to verifying the chord rotation. Figure 27 compares the effective concrete core confinement at the base of the columns ($\alpha \cdot \omega_{wd}$), following the recommendations of EN 1998-1:2004[1], with the effective concrete core confinement in the function of the plastic chord rotation capacity [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$], as defined in Equation (5-25).

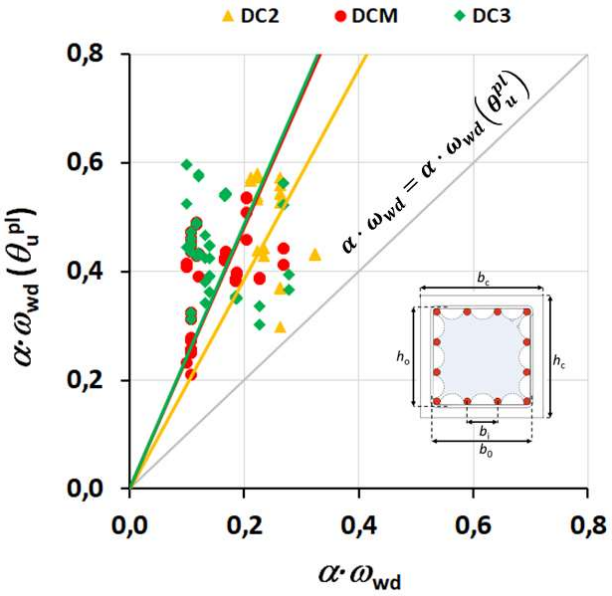


Figure 5-28: Effective mechanical volumetric ratio ($\alpha \cdot \omega_{wd}$) *vs* Mechanical volumetric ratio of the hoops as a function of the plastic chord rotation capacity [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$]

Regarding Figure 5-28, the following observations and conclusions can be made:

a) A *t*-test was performed for the abovementioned pair to verify the interconnection $\alpha \cdot \omega_{wd}(\theta_u^{pl})$ and ($\alpha \cdot \omega_{wd}$). The resulting t-test and p-value help determine whether there is a statistically significant difference between the pair [$\alpha \cdot \omega$wd(θ_u^{pl}); $\alpha \cdot \omega_{wd}$] for each ductility class (DC2, DCM and DC3). Table 7 shows the t-test for each pair [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$;$\alpha \cdot \omega_{wd}$] regarding the values illustrated in Figure 5-28.

Table 5-7: Paired *t*-test for the pair [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$,$\alpha \cdot \omega_{wd}$] for columns base [34]

Ductility class	Element	Paired Differences ($\alpha \cdot \omega_{wd}(\theta_u^{pl})$;$\alpha \cdot \omega_{wd}$)					*t*	Significance	
		Mean mrad	Std. Deviation mrad	Std. Error Mean	95% Confidence Interval of the Difference			One-Sided *p*	Two-Sided *p*
					Lower	Upper			
DC2	Column base	-0,24	0,10	0,02	-0,28	-0,21	-13,78	<0,001	<0,001
DCM		-0,24	0,08	0,01	-0,26	-0,22	-25,10	<0,001	<0,001
DC3		-0,28	0,11	0,01	-0,30	-0,25	-20,92	<0,001	<0,001

For [$\alpha \cdot \omega_{wd}(\theta_u^{pl})$; $\alpha \cdot \omega_{wd}$], the low *p*-values (both one-sided and two-sided) suggest that the differences are statistically significant. The 95% confidence intervals do not include zero, further supporting the conclusion that there is a significant difference between the measurements in each pair. Having both low p-values strengthens the evidence that the observed differences are not due to random variability but indicate a meaningful and

statistically significant distinction between the paired measurements. This means that there is indeed a relation between effective mechanical volumetric ratio ($\alpha \cdot \omega_{wd}$) and the chord plastic chord rotation capacity (θ_u^{pl}).

b) A negative mean in the context of paired differences suggests that, on average, the first measurement in each pair is lower than the second. For example, in DC3, the mean paired difference is 0,28. This implies that, on average, the value of ($\alpha \cdot \omega_{wd}$) is lower than $\alpha \cdot \omega_{wd}(\theta_u^{pl})$ by approximately 0,28. When paired measurements are subtracted in this manner, the mean difference's negative sign indicates the difference's direction (i.e., the first measurement tends to be lower than the second).

c) The differences between the median ($\alpha \cdot \omega_{wd}(\theta_u^{pl})=\alpha \cdot \omega_{wd}$) and the trend lines are between 60% for DC2 and 49% for DC3, showing that the effective confinement calculated through chord rotation is more conservative, as demonstrated in Figure 5-28. It is then demonstrated that the verification of the ($\alpha \cdot \omega$wd) prescribed by equation (5-19) can be omitted, as it is implicit in verifying the chord rotation, automatically encompassing this check.

5.8 SUMMARY AND CONCLUSION

The capacity curves of structures DC3.75.5, DC3.50.5, and DCM.75.5 exhibit more ductile behaviour, allowing more significant deformations beyond yield displacements. The ductility ratios (μ) vary even within structures of the same category, suggesting the influence of design parameters or loading conditions on ductility. DC2.50.10 shows a notable reduction in ductility ratio compared to DC2.50.5, attributed to design strategies implemented during the structural design process.

The behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures (q_R) for various structures exceeds the recommended value, indicating consistent overperformance in structural behaviour. DC2 structures have values between 1,66 and 1,68, DCM structures between 1,61 and 1,82, and DC3 structures between 1,47 and 1,76, all exceeding the recommended value of 1,3. These findings are consistent with other studies showing similar overstrength ratios in RC moment-resisting frame structures.

The behaviour factor component (q_S) accounts for overstrength due to all other sources' (q_S) values surpassing recommendations. DC2 structures range from 1,93 to 2,93, DCM structures from 2,42 to 3,30, and DC3 structures from 2,54 to 3,65. These values significantly surpass the standard recommended values of 1,48 and 1,5, indicating a conservative estimate. The design considerations include torsional resistance in beams and stress redistribution on slabs. The capacity design in shear and control of the global mechanism further enhance redundancy by facilitating controlled energy dissipation. These factors collectively contribute to an increase in the values of (q_S).

The behaviour factor component accounting for deformation capacity and energy dissipation (q_D) increases as the ductility classes increase (from DC2 to DC3) and increases with the number of stories and seismic action. The behaviour factor (q_D) averages 1,61 for DC2 structures and is higher for DCM and DC3 structures at 2,19 and 2,26, respectively. The (q_D) decreases with increasing number of stories and seismic action.

The behaviour factor (q) values from nonlinear static analysis are higher than code-prescribed values, with (q_S) being the primary contributor to the elevated values. In the

comparison between EN 1998-1:2004[1] and prEN 1998-1-2:2024[4], it is evident that the behaviour factor values (q) are notably higher for DC3 than for DCM. DC2 structures range from 5,6 to 7,2, DCM from 9,22 to 10,33, and DC3 from 9,10 to 10,71. The (q_S) factor significantly contributes to these high values, accounting for 46% of the q-factor. This observation underscores the conservative nature of Eurocode 8 behaviour factors, positioning them securely on the side of structural safety in the analysis.

Regarding the estimation of bilinear parameters, the target displacements for the limit state SD were obtained, showing that idealised elastoplastic structures remain in the elastic range. The structural design aligns with capacity design principles, allowing controlled yielding in specific parts while maintaining elasticity in others. The intentional yielding in capacity design reduces overall demand during seismic events.

For the chord rotation analysis, the dispersion in chord rotation components between nonlinear analysis and analytical equations arises from simplifications, modelling accuracy, numerical methods, and dynamic effects. Observations from Figure 24(a) and (b) show that the ductility classes DCM and DC3 have higher values of (θ_{SD}) than DC2, with DC3's trend line being 56% steeper and DCM's 22% steeper, indicating sharper and more moderate increases, respectively. In Figure 24(c) and (d), these differences are less pronounced; DC3's slope is 28% steeper, and DCM's is 19% steeper than DC2's, with a small 4% difference in steepness between DC3 and DCM. Regarding the plastic part of ultimate chord rotation capacity (θ_u^{pl}), the analysis yields higher chord rotation values than nonlinear analysis, indicating conservative design provisions in FprEN 1998-1-1:2024[3].

The analysis of columns in Figure 5-26(a) and (b) reveals that DCM and DC3 exhibit higher ductility values (θ_u^{pl}) than DC2, with DC3's trend line being 32% and DCM's 23% steeper. The differences for the beams shown in Figure 26(c) and (d) are smaller than the columns; DC3's slope is 23% steeper, and DCM's is 12% steeper than DC2's, with an 8% difference in steepness between DC3 and DCM. In the updated framework of the second generation of Eurocode 8, the high ductility class (DCH) has been phased out. Instead, the revised FprEN 1998-1-1:2024[3] introduces three new ductility classes: DC1, DC2, and DC3. Within this new classification, DC1 corresponds to the low ductility class (DCL), and DC3 aligns with the medium ductility class (DCM) as defined in the previous EN 1998-1:2004[1]. This reorganisation provides a clearer, more streamlined categorisation that includes DC2 as an intermediate ductility level. The research findings offer crucial insights, contributing significantly to the validation of DC2, ensuring that it adequately bridges the gap between the low and medium ductility classes, reflecting a more nuanced understanding of structural behaviour under seismic loads, as outlined in Maranhão et al.[7].

The analysis from Figure 5-28demonstrates that verifying the column's effective core confinement ($\alpha \cdot \omega_{wd}$) as prescribed by equation (5-19) can be safely omitted. This is because it is inherently included in the verification of chord rotation, which automatically encompasses this check. Furthermore, the comparison between the median ($\alpha \cdot \omega_{wd}(\theta_u^{pl})=\alpha \cdot \omega_{wd}$) and the trend lines reveals a difference of 60% for DC2 and 49% for DC3. This discrepancy indicates that the effective confinement encompassed through chord rotation calculation is more conservative than the direct analysis preconised in EN 1998-1:2024[1].

The findings in the present research work are applied to structures that are regular in plan and elevation and non-torsionally flexible. For future research, it is proposed that a larger and more diverse set of reinforced concrete building structures be computed, such as irregular in

plan and elevation, torsionally flexible, higher buildings with more bays and wither spans. This extensive computational effort aims to gather a robust dataset that reflects a wide spectrum of design scenarios.

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5.10 SYMBOLOGY

5.10.1 UPPER-CASE LATIN SYMBOLS

Symbol description	*Upper-case Latin symbols*		
	DC2	DCM	DC3
Cross sectional area of reinforcement	A_s		
Cross-sectional area of shear reinforcement	A_{sw}		
Roof displacement from the elastic analysis	$d_{e,roof}$		
Force at the attainment of the first plastic hinge in the SDOF system	F^*_1		
Storey forces	F_{ij}		
Maximum force in SDOF system	F^*_m		
Shear span	L_V		
Wall design value of the applied internal bending moment	M_{Ed}		
Bending (Rotation force) at yield	M^*_y		
Rotational (bending) effective ultimate strength	M^*u		
MultiDegree Of Freedom system	*MDOF*		
Near collapse limit state	*NC*		
Load vector	*P*		
Design acceleration in the function of the natural period	$S_d(T_1)$		
Acceleration of the elastic response spectrum at the period T*.	$S_e[T^*]$		
Significant damage limit state	*SD*		
Single-Degree Of Freedom system	*SDOF*		
Elastic acceleration at yield in capacity spectrum	$S^*_{e,y}$		
Constant acceleration plateau of the elastic response spectrum	S_α		
Period of the idealised equivalent SDOF system	T^*		
Natural period	T_1		
Base shear at the attainment of the first plastic hinge	V_1		
Base shear	V_b		
Elastic base shear	V_e		
Spectral base shear	V_{ed}		
Peak force	$V_{el,max}$		
Hypothetical horizontal shear force, assuming all sections are identical to the real ones but which would be made of a material with an infinite yield strength	$V^*_{e,u}$		
Horizontal shear force corresponding to the formation of a global plastic mechanism	V_u		
Bilinear yield strength [14]	V_{ud}		
Yield force	V_y		

5.10.2 LOWER CASE LATIN SYMBOLS

Symbol description	*Lower case Latin symbols*		
	DC2	DCM	DC3
Tension shift	a_l		
Cross-sectional dimension of column	b_c	b_w	
Distance between consecutive engaged bars	b_i	-	
Width of confined core a in the boundary element of a wall (to centreline of hoops)	b_0		
Concrete cover	c		
Effective depth of a cross-section	d		
Transverse reinforcement diameter	d_{bw}		
Displacement at the formation of the first plastic hinge	d_1		
Displacement at the formation of the first plastic hinge of the SDOF system	d^*_1		
Elastic displacement	d_e		
Displacement at yield	d_y		
Displacement at formation of the global plastic mechanism	d_r		
Displacement at Significant Damage limit state	d_{SD}		
Displacement at Significant Damage limit state of SDOF system	d^*_{SD}		
Displacement of top (roof) of a building	d_{top}		
Ultimate displacement of an SDOF system	d^*_u		
Design value of concrete compressive strength	f_{cd}		
Mean value of concrete compressive strength	f_c ,f_{cm}		
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}		
Mean value of axial tensile strength of concrete	f_{ctm}		
Design yield strength of reinforcement	f_{yd}		
Mean yield strength of reinforcement	f_y, f_{yd}		
Design yield strength of the shear reinforcement	f_{ywd}		
Characteristic yield strength of reinforcement	f_{yk}		
cross-section depth of a beam or column	-	h	
Equivalent stiffness of SDOF system	k^*		
Inertial mass of each storey	m_i		
mass the of the equivalent SDOF system	m^*		
Behaviour factor	q		
Behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures	-	q_R	
Behaviour factor component accounting for overstrength due to all other sources	-	q_S	
Behaviour factor component accounting for the deformation capacity and energy dissipation capacity	-	q_D	
spacing of transverse reinforcement	s		

5.10.3 GREEK LETTER

Symbol description	*Greek symbols*		
	DCM	DC2	DC3
Confinement effectiveness factor	α		
Design value of steel strain at yield	$\varepsilon_{sy,d}$		
Transformation factor	Γ		
Correction factor conforming DC1, DC3 and DC3	–	$\kappa_{conform}$	
Correction factor for an axial force different than zero	–	κ_{axial}	
Correction factor for asymmetrical reinforcement	–	κ_{reinf}	
Correction factor for concrete strength different than 25 MPa	–	$\kappa_{concrete}$	
Correction factor for a shear span-to-depth ratio different than 2,5	–	$\kappa_{shearspan}$	
Correction factor taking into account the confinement of concrete due to transverse bars	–	$\kappa_{confinement}$	
Load factor	λ		
Curvature ductility factor	μ_ϕ		
Design axial force due in the seismic design situation, normalised to $A_c \cdot f_{cd}$	ν_d		
ratio of transverse reinforcement in the direction of shear	–	ρ_{sw}	
Chord rotation corresponding to the attainment of the SD limit state according to FprEN 1998-1-1:2024[4]	θ_{SD}		
Ultimate chord rotation calculated according to FprEN 1998-1-1:2024[4]	θ_u		
Plastic chord rotation capacity calculated according to FprEN 1998-1-1:2024[4]	θ_u^{pl}		
Basic value of plastic chord rotation capacity of a member, assuming:	θ_{u0}^{pl}		
Chord rotation at yield calculated according to FprEN 1998-1-1:2024[4]	θ_y		
Chord rotation at yield, taking into account the longitudinal reinforcement contribution	$\theta_{y,M}$		
Chord rotation at yield, taking into account the reinforcement bond contribution	$\theta_{y,slip}$		
Chord rotation at yield, taking into account the transverse reinforcement contribution	$\theta_{y,v}$		
Chord rotation corresponding to the attainment of the SD limit state obtained by nonlinear analysis	$\theta_{SD,NLA}$		
Ultimate chord rotation obtained by nonlinear analysis	$\theta_{u,NLA}$		
Plastic chord rotation capacity obtained by nonlinear analysis	$\theta_{u,NLA}^{pl}$		
Chord rotation at yield obtained by nonlinear analysis	$\theta_{y,NLA}$		
Mechanical volumetric ratio of confining reinforcement	ω_{wd}		
Effective concrete core confinement at the column base	$\alpha \cdot \omega_{wd}$		
Effective concrete core confinement at columns base in the function of plastic chord rotation capacity	$\alpha \cdot \omega_{wd}(\theta_u^{pl})$		

6 Seismic design challenges: Investigating Bending Moments, Shear Actions, and Design Parameters in Concrete Walls under the second-generation of Eurocode 8

Helder Maranhão, Humberto Varum, José Melo (submitted in press) Seismic design challenges: Investigating Bending Moments, Shear Actions, and Design Parameters in Concrete Walls under the second generation of Eurocode 8

6.1 ABSTRACT

The ongoing revision of Eurocode 8 includes updates encompassing new ductility classifications, limits on damage control, conditions for local ductility, and corresponding detailing guidelines. It is crucial to thoroughly examine and compare these revisions with the existing provisions of Eurocode 8. The present research examines structural behaviour factors (q) for varying strut inclination angles (45° and 22°) and compares them against prEN 1998-1-2:2023 recommendations. Findings reveal consistent overperformance, highlighting the conservative estimations for structural redundancy offered by prEN 1998-1-2:2023. Plastic hinge lengths obtained through nonlinear analysis and FprEN 1998-1-1:2024 calculations are compared, showcasing conservative estimations prioritizing safety. Observations indicate distinct trends in tension shift (a_l) and critical height (h_{cr}) compared to plastic hinge length (l_{pl}), aligning with conservative approaches outlined in EN 1998-1:2004 and prEN 1998-1-2:2023. Additionally, the observed shear magnification factors (ε) may indicate a potential overestimation of shear forces attributed to the amplified effect of higher modes in the inelastic range combined with flexural overstrength.

6.2 INTRODUCTION

EN 1998-1:2004[1] represents the state-of-the-art seismic design in Europe, although some sections need to be updated in EN 1998-1:2004[1] regarding the last research developments and practitioners' feedback. The second generation of Eurocode 8[2][3] introduces several improvements, aiming to be state-of-the-art in seismic structural engineering. In the new code generation, the contents of the parts were revised, concentrating in FprEN 1998-1-1:2024[3] the general aspects independent of the type of structures and materials and in prEN 1998-1-2:2023[2] the specific rules for buildings.

The prEN 1998-1-2:2023[2] introduces three new ductility classes: DC1, DC2, and DC3. In these three new ductility classes, new aspects of wall structure design are introduced, such as the behaviour factor, design moment, and shear envelopes.

FprEN 1998-1-1:2024[3] defines a behaviour factor composed of the product of the partial behaviour factors (components) q_R, q_S, and q_D, where (q_R) is the behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures (q_S) is the behaviour factor component accounting for overstrength due to all other sources, and (q_D) is the behaviour factor component accounting for the deformation and energy dissipation capacity. The current study aims to analyse the behaviour factor q (and its components) proposed in the prEN 1998-1-2:2023[2] of a set of reinforced concrete (RC) wall structures designed to be regular in both plan and elevation, considering various classes of ductility concerning both the current and second-generation [2], [3] editions of Eurocode 8, through nonlinear analysis and the influence of the capacity design and the detailing for local ductility.

Another aspect crucial to the seismic safety and design of the wall structures, apart from the behaviour factors, are the bending moment and shear actions. Several parameters influence the design envelope of the bending moment and the shear actions. Fardis[4] states that for the wall to stay elastic above the plastic hinge region, Eurocode 8 requires dimensioning in bending the rest of the wall height for a linear envelope of the wall bending moments from the analysis for the design seismic action. The envelope also intends to cover a potential increase in bending moments above the base due to higher mode inelastic response after developing the plastic hinge at the base. This envelope depends on the tension shift (a_l) regarding the EN 1998-1:2004[1] and the maximum between the tension shift (al) and the critical height (h_{cr}) in what concerns the prEN 1998-1-2:2023[2].

In the case of RC walls, the critical height (h_{cr}) is intrinsically connected to the level of detailing for local ductility. According to Fardis[4], to ensure that a wall works as a solid and stiff vertical backbone, mobilising all beams into inelastic action and minimising local rotation and ductility demands for given global displacement demand, Eurocode 8 takes measures to localise wall inelastic deformations at its base. A so-designed wall is called a ductile wall. It is designed and detailed to dissipate energy in a flexural plastic hinge only at the base and remain elastic throughout its height to promote or enforce a beam-sway mechanism.

The present research aims to conduct a comprehensive analysis by comparing the plastic hinge lengths derived from nonlinear analysis ($l_{pl,NLA}$) of a series of structures with the critical tension shift height (a_l) as well as the critical height (h_{cr}). These parameters are fundamental in defining the envelope of bending moment diagrams of reinforced concrete (RC) walls. The objective is to investigate whether the plastic hinge length significantly influences the tension shift height (a_l), critical height (h_{cr}), and consequently, the bending moment distribution envelope and the minimum height required for effective energy dissipation in a flexural plastic hinge, referred to as the critical height (h_{cr}). The tension shift height (a_l), as defined by EN 1992-1-1:2004[5] and prEN 1992-1-1:2023, is known to be contingent upon the strut angle (θ). This study compares the tension shift height (al) delineating the envelope of bending moment actions for walls with the tension shift height derived from nonlinear analysis (d_{NLA}). The primary objective is to discern the optimal strut angle for structural performance.

According to EN 1998-1:2004[1] and prEN 1998-1-2:2023[2], design shear forces shall be magnified by and factor (ε). The magnification design shear forces by a factor (ε) according to EN 1998-1:2004[1] and prEN 1998-1-2:2023[1] represents a fundamental aspect of the seismic design of walls by incorporating uncertainties, variability, and dynamic effects. This approach enhances the safety, reliability, and performance of seismic structures. This research aims to evaluate and compare the magnification factor of the shear forces.

6.3 OBJECT OF RESEARCH AND ANALYSIS

To conduct this study, twenty RC wall system structures were selected from the analysis performed by Maranhão et al.[6]. All buildings were designed according to the force-based design concept (FB). It should be noted that all buildings are designed in a three-dimensional environment (3D), allowing a better idea of how the forces are transferred to the structure's elements and foundations. These structures were designed in strict adherence to the standards outlined in EN 1998-1:2004[1], FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[2]. The study encompasses a collection of twenty RC wall structures comprising:

- Five 5-storey RC walls with a strut angle (θ) of 22° and five additional 5-storey RC walls featuring a strut angle of 45°, as depicted in Figure 6-1(a), were utilised for defining the tension shift (a_l).
- Five 10-storey RC buildings with a strut angle (θ) of 22° for the tension shift definition and another set of five 10-storey RC walls featuring a strut angle of 45°, as illustrated in Figure 6-1(b).

The structures were designed for the ductility classes DCM, DC2 and DC3. All structures are regular in plan and elevation and non-torsionally flexible, according to Maranhão et al.[6].

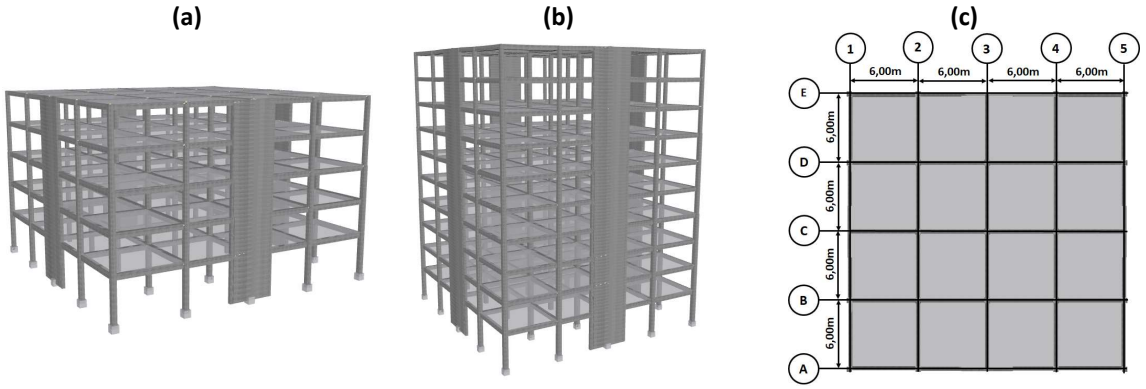


Figure 6-1: Buildings model [6] (a) 5-storey building (b) 10-storey building (c) Plan view

The structural system for the buildings consists of a series of frames with an equal spacing of 6,0 m (Figure 1(c)), typical of office buildings. The floor-to-floor height is 3,0 m for all storeys, except for the ground storey, which is 4,0 m. The solid slab thickness is 15,0 cm at each storey.

The walls are positioned centrally on each facade of the buildings, as depicted in Figure 1, with lengths (l_w) ranging from 4,0 m to 4,5 m and thicknesses (b_w) between 30 cm and 35 cm.

The building seismic responses are summarised in Table 6-1.

Table 6-1: Seismic response parameters (Adapted from Maranhão et al.[6])

Structure label	Ductility class	Number of storeys	S_α m/s²	Ductility class	q	T_1 s	$S_d(T_1)$ m/s²	V_b kN	$d_{e,roof}$ m
DC2.W.50.5	DC2	5	5,0	DC2	2,0	1,11	1,13	2478,78	0,039
DCM.W.50.5	DCM			DCM	3,0	1,16	0,72	1582,16	0,027
DC3.W.50.5	DC3			DC3	3,0	1,16	0,72	1582,16	0,027
DCM.W.75.5	DCM		7,5	DCM	3,0	1,16	1,08	2373,24	0,039
DC3.W.75.5	DC3			DC3	3,0	1,16	1,08	2373,24	0,041
DC2.W.50.10	DC2	10	5,0	DC2	2,0	1,7	0,74	3633,88	0,076
DCM.W.50.10	DCM			DCM	3,0	1,95	0,43	1993,36	0,056
DC3.W.50.10	DC3			DC3	3,0	1,9	0,44	1993,36	0,056
DCM.W.75.10	DCM		7,5	DCM	3,0	1,7	0,74	3628,89	0,076
DC3.W.75.10	DC3			DC3	3,0	1,67	0,75	3733,99	0,075

The building's primary seismic members (beams, columns, walls) for the buildings under analysis were designed with concrete C30/37 and reinforcement steel B500, according to Maranhão et al.[7]. It is important to emphasise that each structure outlined in Table 6-1 was designed for a strut angle (θ) of 22° and 45°, which defines the tension shift (a_l), essential for determining the envelope of bending moments in the wall design according to Maranhão et al.[6].

6.4 MODELLING FOR NONLINEAR ANALYSIS

According to Antoniou et al.[8] in the Nonlinear Static Procedure, the structural behaviour is no longer linear, and the elasticity rules cannot be applied. Stresses are not proportional to strains, forces are not proportional to displacement, and bending moments are not proportional to curvatures. A lateral force profile approximating the earthquake loading is applied with several types of force distributions: triangular, uniform, modal or even adaptive distributions that change from step to step.

6.4.1 NONLINEAR ANALYSIS METHOD

The adaptive load control and adaptive response control loading/solution procedures are employed instead of the load control and response control phases. Their input and functionality remain the same. However, it is noteworthy that in adaptive pushover analysis, only one adaptive phase (either load or response control) can be applied, unlike conventional pushover analysis, where multiple load or response control phases can be used simultaneously, as referred by Seismosoft[9].

According to Seismosoft[9], in the adaptive pushover approach, the lateral load distribution is not kept constant but continuously updated during the analysis, according to the modal shapes and participation factors derived by the eigenvalue analysis carried out at each analysis step. This method is fully multi-modal and accounts for the softening of the structure and its period elongation. Apart from force distributions, adaptive pushover is also able to employ deformation profiles efficiently. Due to its ability to update the lateral load patterns according to the constantly changing modal properties of the system, it overcomes the intrinsic weaknesses of fixed-pattern displacement pushover. It provides a more accurate performance-oriented tool for structural assessment, providing better response estimates than existing conventional methods, especially in cases where higher mode effects might play an essential role in its dynamic response.

Instead of utilising load control and response control phases, the adaptive load control and adaptive response control loading/solution procedures are employed. While their input and functionality remain consistent, it is essential to note that in adaptive pushover analysis, only one adaptive phase (either load or response control) can be implemented, unlike conventional pushover analysis, which allows for the simultaneous use of multiple load or response control phases, as noted by Seismosoft[9].

In the adaptive pushover approach described by Seismosoft[9], the lateral load distribution undergoes continuous updates throughout the analysis based on modal shapes and participation factors derived from eigenvalue analysis at each analysis step. This method, fully multi-modal, accounts for structural softening and period elongation. Besides accommodating various force distributions, adaptive pushover can efficiently incorporate deformation profiles. It addresses the limitations of fixed-pattern displacement pushover by dynamically updating lateral load patterns according to evolving modal properties. Consequently, it serves as a more accurate performance-oriented tool for structural assessment, providing superior response estimates compared to conventional methods, particularly in scenarios where higher mode effects significantly impact dynamic response.

6.4.2 PUSHOVER PROCEDURE

The present study employs adaptive pushover analysis using the finite element software SeismoStruct developed by Seismosoft[9] within a three-dimensional framework. In this analysis approach, similar to conventional pushover analysis, loads are applied to the structure. However, in adaptive pushover analysis, accurate modelling of the inertia mass of structures is essential to enable eigenvalue analysis, which plays a crucial role in updating the loading vector. In force-based adaptive pushover analysis, it is imperative to distribute mass appropriately across nodes where incremental loads will be applied to ensure precise determination of incremental forces. The load distribution utilised in this study follows a uniform pattern.

In the context of pushover analysis adhering to the EN 1998-3:2004[10] standard and employing a uniform load pattern, the primary objective is to evaluate the structure's seismic performance. The uniform load pattern entails the application of lateral forces uniformly across the structure's height. During the analysis, particular attention is directed towards identifying the mode of vibration that mobilises the most mass. This entails pinpointing the vibration mode with the highest effective modal mass value, highlighting its significant influence on the structural response to seismic action. The pushover analysis aims to capture the structure's behaviour under seismic loading by focusing on the most critical mode.

The nonlinear models were developed using the nonlinear analysis software SeismoStruct[9]. The analysis through the distributed fibre formulation (distributed inelasticity) approach combines the 3 integration sections with 150 fibres for beams and columns and 250 fibres for the walls in each integration section. The integration sections are located along the columns and beams in the distributed fibre formulation, as depicted in Figure 6-2.

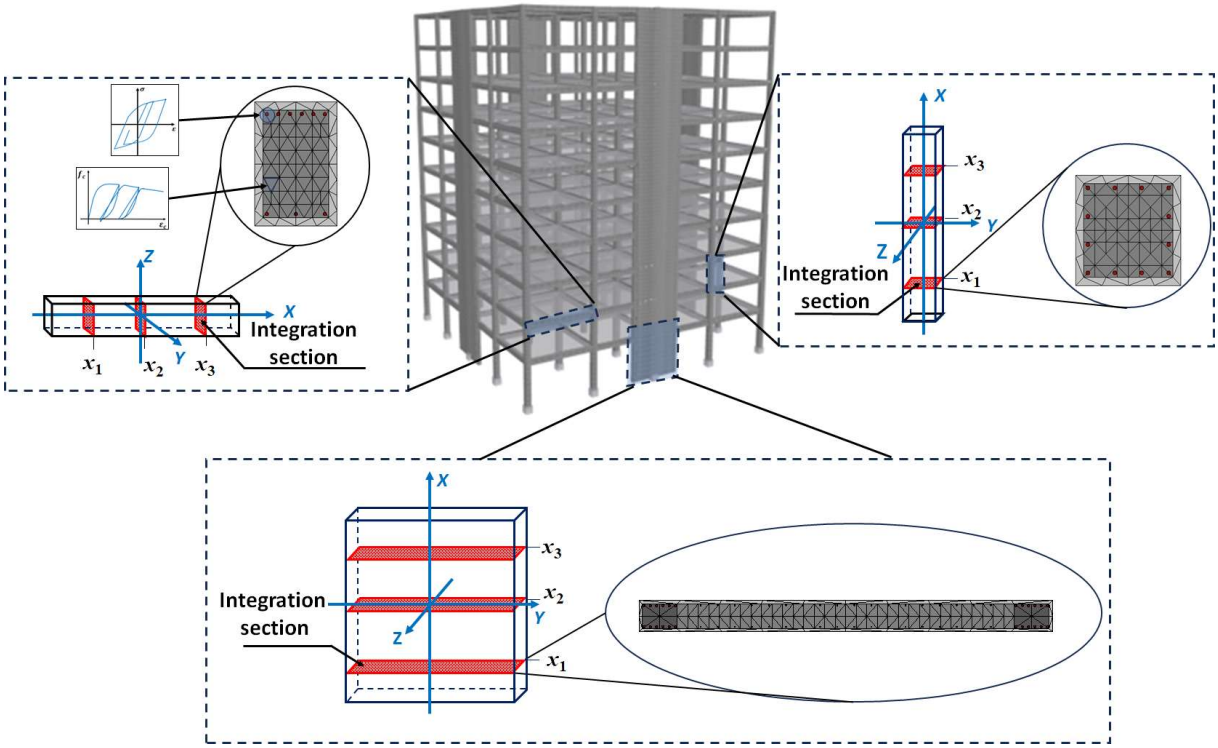


Figure 6-2: Schematic illustration of 3 integration sections and meshing for 150 fibres for beams and columns and 250 fibres for walls

The approach for implementing distributed inelasticity in elements using the force-based method was adopted based on the formulation proposed by Neuenhofer et al.[11]. In force-based elements, strict adherence to equilibrium is maintained, allowing for the development of inelastic deformations across the member without imposing any restrictions. The decision to incorporate three integration sections per element was guided by the research findings of Maranhão et al.[12]. According to Di Domenico et al.[31] is expected that the use of only three integration points introduces a limited error in element response when adopting the force-based formulation with Gauss−Lobatto integration scheme.

Using 3 integration sections instead of a higher number in seismic structural analysis in the context of the SeismoStruct[9], has several key advantages. These are derived from the handling of convergence norms, iterative strategies, and stiffness updates as described as follows:

a) Improved Convergence: with fewer integration sections, the program may find it easier to achieve convergence in challenging analysis steps. This is because convergence difficulties often arise from the use of a large number of integration sections. Reducing the number to 3 can mitigate these issues, facilitating smoother progression through analysis steps without the need for excessive adjustments to convergence norms. According to SeismoStruct[9] convergence difficulties in force-based elements are often caused by the employment of a large number of integration sections together with element discretisation (typically in elements where the reinforcement details change). In such cases, users should decrease the number of integration sections to 3.

b) Iterative Strategy: The iterative approach employed by SeismoStruct[9] can benefit from a reduced number of integration sections as it can lower the complexity of the equilibrium process within each load increment. This simplification can lead to more stable and

consistent iterations, especially when dealing with high geometric nonlinearity or material inelasticity.

c) Number of Stiffness Updates: with 3 integration sections, the recalculation and updating of the tangent stiffness matrix may become more efficient. This can lead to an optimal balance between computation time and stability. Since the ideal number of stiffness updates is between 50% and 75% of the maximum iterations, fewer integration sections can streamline this process, reducing the need for frequent stiffness updates and thus enhancing computational efficiency.

d) Reduced Computational Load: Computational Efficiency: Fewer integration sections directly translate to reduced computational load. This means faster analysis times and less computational resources required, which is particularly beneficial for complex models or when running multiple simulations.

e) Simplified Modelling and element Discretization: In elements such as beams where reinforcement details change, using 3 integration sections simplifies the element discretization process. This can prevent convergence issues that arise from intricate discretization schemes, making the modelling process more straightforward and less error prone.

Overall, using 3 integration sections can lead to a more efficient, stable, and reliable analysis process in SeismoStruct, especially when dealing with complex structures or those subjected to significant nonlinearity. This approach optimizes the balance between computational demand and analysis accuracy, facilitating better overall performance of the seismic structural analysis.

6.4.3 MATERIAL CONSTITUTIVE MODELS

The constitutive laws are defined similarly as in the work of Maranhão et al.[13] Furthermore, described in 5.4.2. The concrete constitutive relationship proposed by Mander et al.[14] is a uniaxial nonlinear constant confinement model. The concrete compressive strength considered in the analysis is the mean characteristic cylinder compressive strength, which, in the case of a concrete C30/37, the value of f_{cm}=38 MPa, and its constitutive law, used in the nonlinear FE models under analysis, is depicted in subchapter 5.4.2, Figure 5-3(b). The influence of various types of confinement is considered by defining an effective lateral confining stress, which depends on the configuration of the transverse and longitudinal reinforcement.

The constitutive law for steel utilised in the present modelling follows the formulation of Menegotto et al.[15] coupled with the isotropic hardening rules proposed by Taucer et al.[16]. The steel stress-strain curve used for nonlinear FE modelling is shown in 5.4.2, Figure 5-4(b). The yield strength of the reinforcement considered in the analysis is the mean yield strength. For B500 grade reinforcement steel, the mean value is f_{ym}=555,6 MPa.

6.5 BEHAVIOUR FACTOR ANALYSIS

As referred by Maranhão et al.[13], the various structural behaviour parameters, such as the behaviour factor (q), are associated with the reference earthquake intensity and the definition of relevant design criteria. According to Elnashai et al.[17] the q factor may be defined as the ratio of the peak force ($V_{el,max}$) that would have developed if the single-degree of freedom (SDOF) system were linear-elastic to the yield force of the system (V_y), as depicted in subchapter 5.5, Figure 5-5.

The q factor generally may be computed as depicted in subchapter 5.5 Equation (5-1), and it expresses the energy absorption and dissipation capacity of structural systems, as stated by Elnashai et al.[17] and Maranhão et al.[13].

According to EN 1998-1:2004[1], the q factor is an approximation of the ratio of the seismic forces that the structure would experience if its response were completely elastic with 5% viscous damping to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The q factor intrinsically influences the shape of the design response spectrum, affecting spectral acceleration and design force.

As referred before, FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[2] introduce a q factor composed by the product of the partial behaviour factors (components) q_R, q_S, and q_D, as depicted in the subchapter 5.5, Equation (5-2).

The behaviour factor (q) (and its components) for the different reinforced concrete wall system buildings is analysed using nonlinear analysis through the adaptive static pushover method. The evaluation of behaviour factors using the adaptive static pushover method involves the estimation of the capacity curve of the structure under a prescribed lateral load profile and crucial modelling assumptions of the strength and stiffness dependence of the member characteristics, including second-order effects at the global and local element levels.

6.5.1 ESTIMATION OF THE BEHAVIOUR FACTOR COMPONENTS

Similarly, as it was performed by Maranhão et al.[13] As explained in subchapter 5.5.1, the behaviour factor and its components are estimated, recurring to the mathematical model adapted from Plumier[18] and FprEN 1998-1-1:2024[3]. Plumier[18] proposes that the behaviour factor components q_R, q_S, and q_D are studied and analysed, recurring to the building capacity curve analysis at its key points (V_e,d_e),(V_1,d_1), (V_u,d_y) and (V_u,d_r), as shown in Figure 6-3.

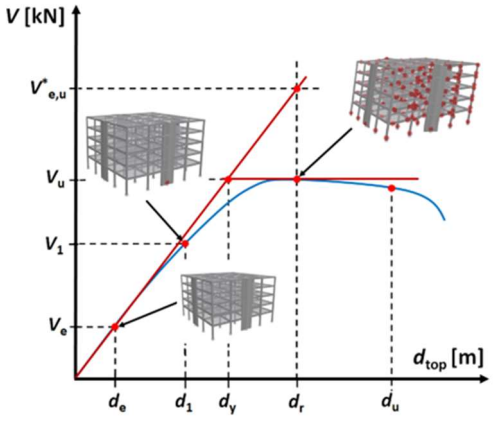


Figure 6-3: Capacity curve (Adapted from Plumier[18] and Maranhão et al.[13])

The base shear force (V_e) represents the horizontal shear force from the elastic analysis using a design spectrum reduced by the q factor, and (d_e) is the displacement corresponding to (V_e) measured at the rooftop of the building, as depicted in Figure 6-4.

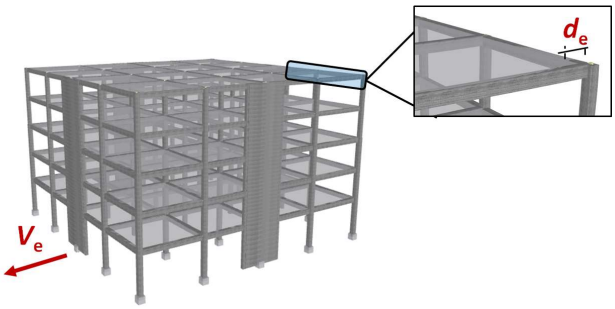


Figure 6-4: Roof displacement from elastic analysis

The parameter (V_1) is the horizontal base shear force at which the first plastic hinge is formed, and d1 is the displacement at the formation of the first plastic hinge, as shown in Figure 6-5(a).

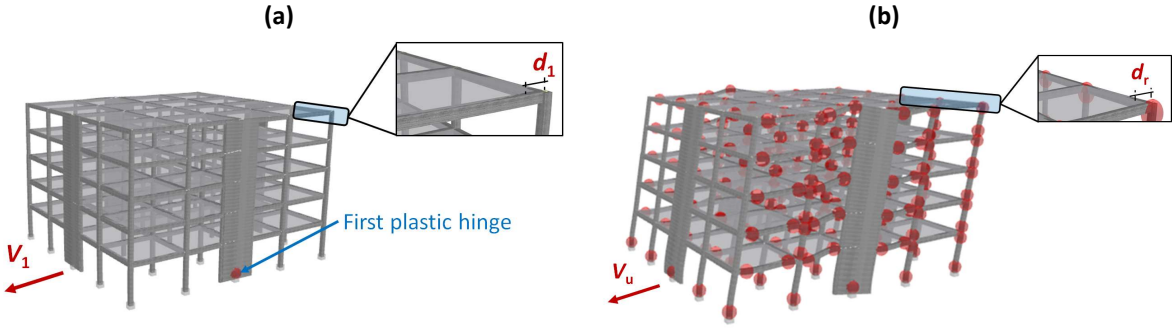


Figure 6-5: Structure damage states (a) Attainment of the first plastic hinge (b) Plastic mechanism

Similarly, as described in subchapter 5.5.1, the parameter (V_u) is the horizontal shear force corresponding to the formation of a global plastic mechanism, and (d_r) is the correspondent displacement, as defined in Figure 8(b).

The parameter ($V^*_{e,u}$) is the hypothetical horizontal shear force for a displacement (d_r) of a structure with all sections identical to the real ones but which would be made of a material with an infinite yield strength. The displacement du is defined as the ultimate limit state attained locally somewhere in the structure due to excess plastic rotation inducing a resistance drop by buckling, among other phenomena.

Similar to the approach taken by Maranhão et al.[13] described in subchapter 5.5.1, the current study introduces a methodology proposed by Plumier[18]. This methodology involves developing a bilinear model of the force-deformation relationship at the member level for each building to represent ductile behaviour. Two damage levels must be established based on the member's deformation thresholds, crucial points along the force-deformation relationship. The displacement at yield (d_y), marking the achievement of effective yield strength, is linked to the elastic stiffness ($K_e=V_e/d_e$). In contrast, the displacement at the onset of the global plastic mechanism (d_r) corresponds to achieving effective ultimate strength (V_u).

To determine the displacement (d_r), at least one of the two following criteria was considered:

a) The global plastic mechanism is observed as depicted in Figure 6-6(a).

b) If the global plastic mechanism is not observed, the ultimate capacity (V_u,d_r) is defined as a 20% reduction in strength relative to the maximum resistance, as described in Figure 6-6(b).

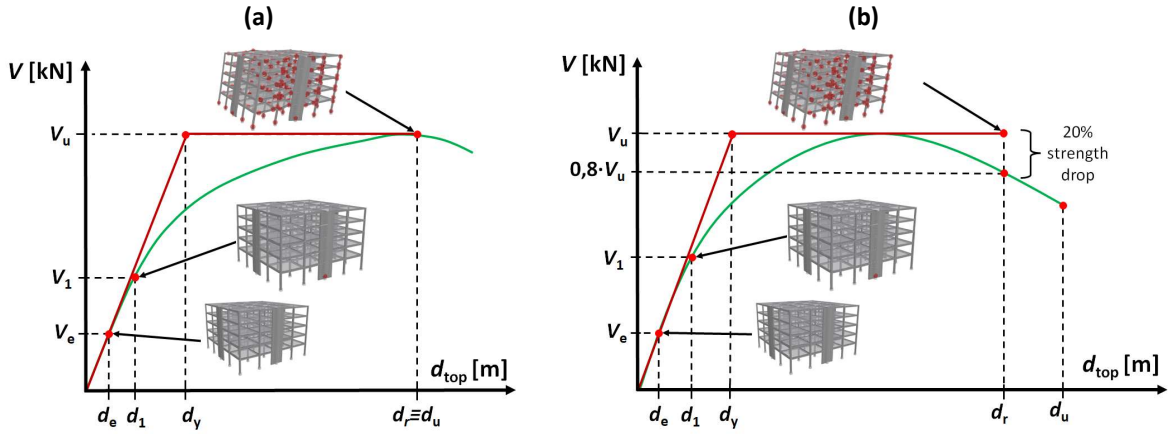


Figure 6-6: General definition of bilinear load-deformation relationship: (a) global plastic mechanism is observed; (b) strength drop of 20% relative to the maximum resistance.

The behaviour factors (components) q_R, q_S, and q_D are estimated in subchapters 5.5.1.1,5.5.1.2 and 5.5.1.3.

6.6 COMPARISON BETWEEN THE CRITICAL REGION HEIGHT, TENSION SHIFT WITH PLASTIC HINGE LENGTH OF WALLS

As referred before, the objective of this study is to analyse the behaviour factor (q) and its components, as proposed in prEN 1998-1-2:2023[2], for a series of reinforced concrete wall structures. These structures are designed to exhibit regularity both in plan and elevation. The analysis considers various classes of ductility outlined in EN 1998-1:2004[1], FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[2]. This investigation is conducted through nonlinear analysis, considering the influence of capacity design and detailing for local ductility. Additionally, the study aims to compare the plastic hinge length of the walls ($l_{pl,NLA}$) among different structures, with particular attention to critical height (h_{cr}) and tension shift height (al). These parameters are crucial in defining the design bending diagram of reinforced concrete walls. Furthermore, the study seeks to compare tension shift heights (a_l) prescribed by FprEN 1998-1-1:2024[3] and prEN 1998-1-2:2023[2], along with tension shift height determined through nonlinear analysis (d_{NLA}).

6.7 PLASTIC HINGE LENGTH

The plastic hinge concept extends the principles of ductile design employed in constructing earthquake-resistant structures. This approach involves dissipating energy by allowing specific zones at the end of a structural member to undergo plastic deformation while the remainder of the structure remains unaffected. The deformation properties of a reinforced concrete RC wall section can be portrayed by examining a given cross-section's moment-curvature (M-ϕ) characteristics. As of interest, an RC cantilever wall's moment-curvature curve follows three

distinct damage stages until the tensile rupture of reinforcing steel occurs, as shown in Figure 6-7.

The concrete cracking resistance threshold is achieved at the coordinates (ϕ_{cr},M_{cr}). Before this point, the response is elastic and linear. With an increase in the applied moment, the concrete's cracking reduces the section's flexural rigidity, which depends on the amount and configuration of the reinforcement. The tension steel begins to yield at the higher load level (ϕ_y,M_y), followed by the concrete crushing at point (ϕ_u,M_u).

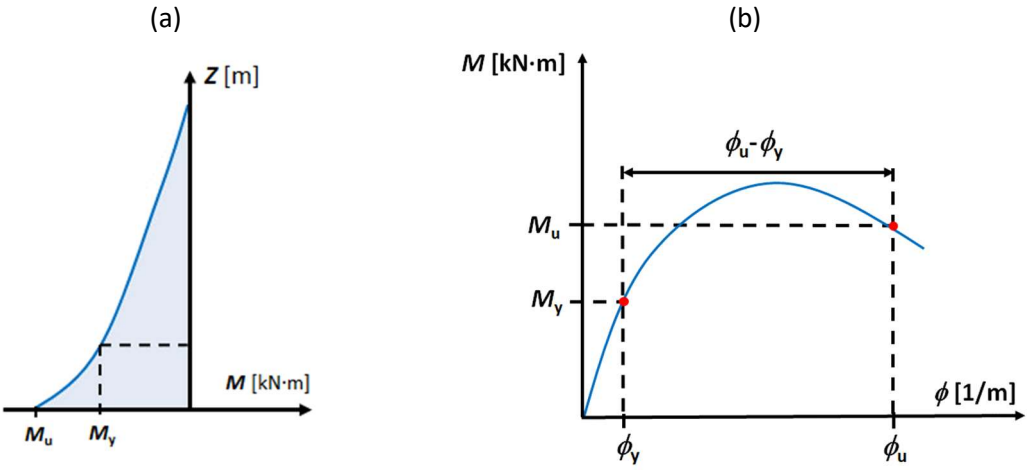


Figure 6-7: (a) Cantilevered wall bending moment diagram (b) Moment-curvature

The wall chord rotation may be estimated by analysing the curvature along the wall length.

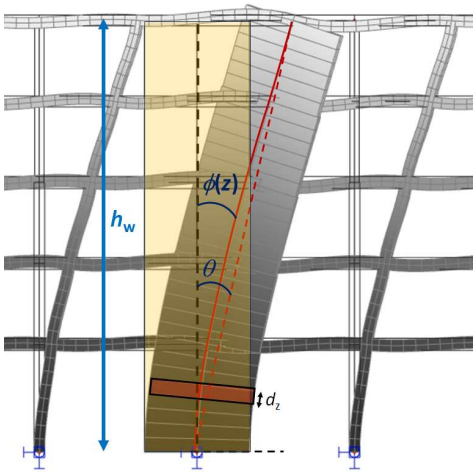


Figure 6-8: Schematic representation of chord rotation (θ) and curvature (ϕ)

According to the first-moment area theorem, the chord rotation (θ) between any two points (z=0 and z=h_w), as depicted in Figure 6-8, is equivalent to the area under the curvature diagram delineated between these two points, as illustrated in Equation (6-1):

$$\theta = \int_{o}^{h_w} \phi(z)\, dz \tag{6-1}$$

where (h_w)is the wall's total height and (ϕ) is the curvature between two points.

When loads applied to a wall are smaller than the yield bending moment (My), the amount of bending or curvature in the member increases gradually from one end (where the member is free) to the other end (where it is supported). Additionally, there is a notable increase in curvature when the tension steel in the member reaches its yield point. At the ultimate load stage, the value of the curvature at the support increases suddenly, causing a large inelastic deformation. The curvature fluctuates along the member length since the concrete between the cracks can carry some tension (tension-stiffening), as referred to by Mortezaei et al.[20]. The actual curvature distribution at the ultimate load stage can be idealised into elastic and inelastic (plastic) regions, as shown in Figure 6-9(a).

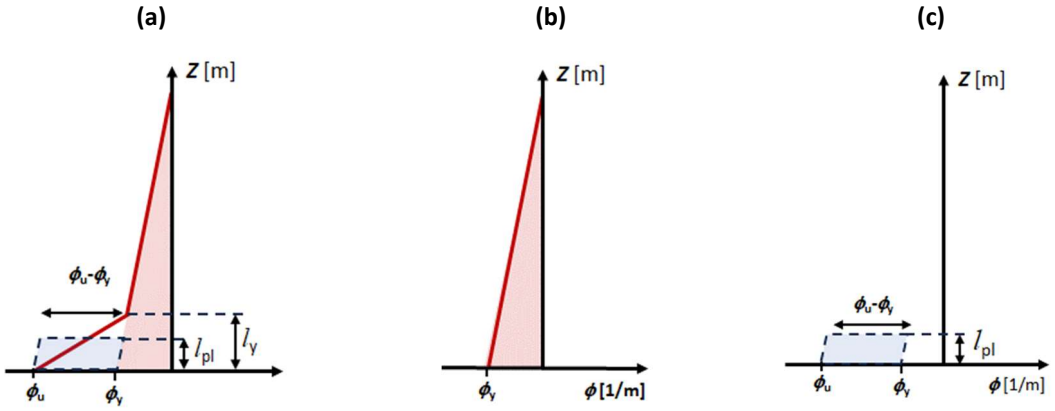


Figure 6-9: Theoretical curvature diagram (a) Distribution of the curvature along the wall (b) Curvature distribution at yield (c) Plastic part of the curvature

According to Mortezaei et al.[20], the actual distribution of curvature at the ultimate load stage can be idealised into elastic and inelastic (plastic) regions Figure 6-9 (a); thus, the total rotation over the member length can be divided into elastic (θ_e) and plastic (θ_p) rotations. The elastic rotation (θ_e), until the steel yielding, can be obtained using the curvature at yield Equation (6-1). From equation (6-1), the plastic chord rotation at capacity can be defined as depicted in the equation (6-2):

$$\theta_p = \int_0^{l_y} |\phi(z) - \phi_y| \, dz \tag{6-2}$$

in which $\phi(z)$ is the curvature at the z distance from the critical section at the ultimate load stage and (ϕ_y) is the curvature at yield. The yielding length (l_y) is defined as the length of the member segment over which the maximum moment exceeds the yield bending moment (M_y).

The plastic hinge rotation can be determined by an equivalent parallelogram of height (ϕ_u-ϕ_y) and height (l_{pl}), as shown in Figure 6-9(c). By using equation (6-2), the plastic hinge length (l_{pl}) can be defined according to expression (6-3) as defined by Kheyroddin et al.[21]:

$$l_{pl} = \frac{1}{\phi_u - \phi_y} \cdot \int_0^{l_y} |\phi(z) - \phi_y| \, dz \tag{6-3}$$

A cantilevered wall is subjected to a horizontal load, as shown in Figure 11, where the ultimate curvature (ϕ_u) has been reached at the fixed end section (z=0). According to Costa et al.[22], the ultimate rotation can be decomposed into two components:

a) The rotation corresponds to the elastic phase's limit, i.e., within the curvature field illustrated in Figure 6-9 (b) and the chord rotation at yield (θ_y).

b) The subsequent rotation beyond (θ_y), defined as the plastic chord rotation capacity denoted by (θ_u^{pl}), can be approximated by a uniform curvature distribution with value (ϕ_u-ϕ_y) in a zone called the plastic hinge region with length (l_{pl}) as depicted in Figure 11(c).

If, according to what has been mentioned, we consider in a simplified manner that the plastic chord rotation capacity (θ_u^{pl}) and results from the curvature field in Figure 6-9(c) and regarding expression (6-3), then its resultant value is defined as depicted as follows:

$$\theta_u^{pl} = (\phi_u - \phi_y) \cdot l_{pl} \tag{6-4}$$

This deformation is concentrated in plastic hinge length (l_{pl}), as illustrated in Figure 6-10.

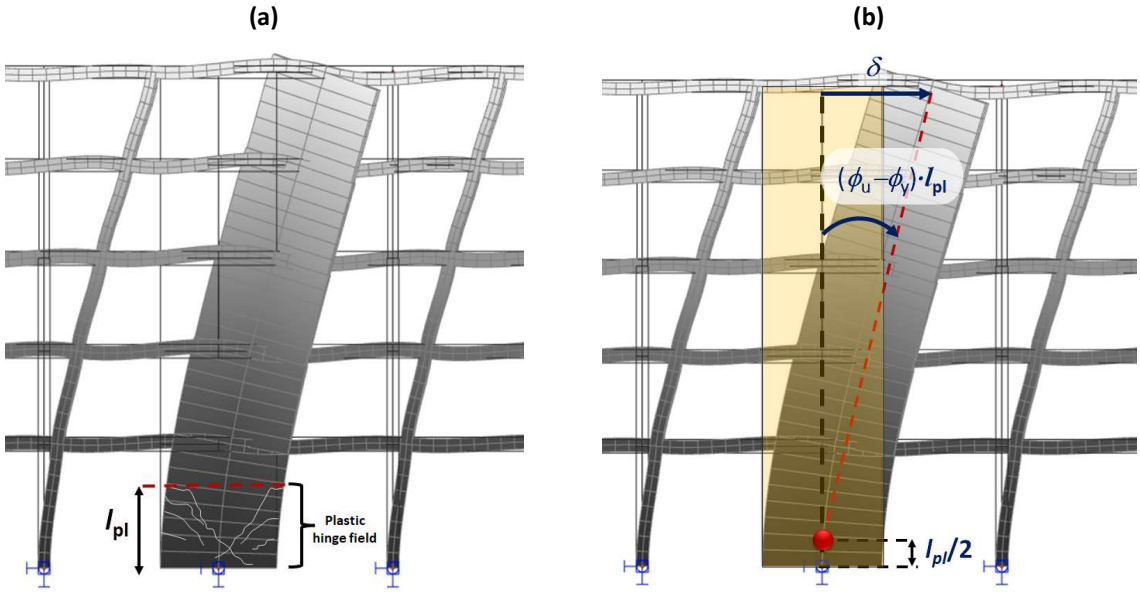


Figure 6-10: Plastic hinge length (a) Schematic representation of the plastic hinge field (b) Schematic representation (θ_u^{pl}) assuming deformation concentrated in plastic hinge length (l_{pl})

Taking into account equation (6-4), the plastic hinge length can be rewritten as depicted in the equation (6-5), which defines the plastic hinge length obtained by nonlinear analysis:

$$l_{pl,NLA} = \frac{\phi_u - \phi_y}{\theta_u - \theta_y} \tag{6-5}$$

To derive the wall plastic hinge length by nonlinear analysis ($l_{pl,NLA}$), the ultimate curvature (ϕ_u), the curvature at yield (ϕ_y) and the plastic chord rotation capacity (θ_u^{pl}) are obtained by nonlinear analysis. The curvature at yield (ϕ_y) and the ultimate curvature (ϕ_u) are estimated by nonlinear analysis according to the criteria depicted in Figure 14 by using a bilinear model adapted from FprEN 1998-1-1:2024[3]. The curvature at yield (ϕ_y) corresponds to the attainment of an effective yield rotational strength (M^*_y) linked to the elastic stiffness (K_e) by ($K_e=M^*_y/\phi_y$). The ultimate curvature of a member (ϕ_u), corresponding to the attainment of a rotational effective ultimate strength (M^*_u), is estimated considering one of two criteria:

1) The ultimate curvature (ϕ_u) at the member end corresponds to a 20% drop in strength concerning the maximum strength of the primary seismic member, as depicted in Figure 6-11(a).
2) If criteria 1) is not met, it is considered the maximum strength at the end of the pushover, as illustrated in Figure 6-11(b).

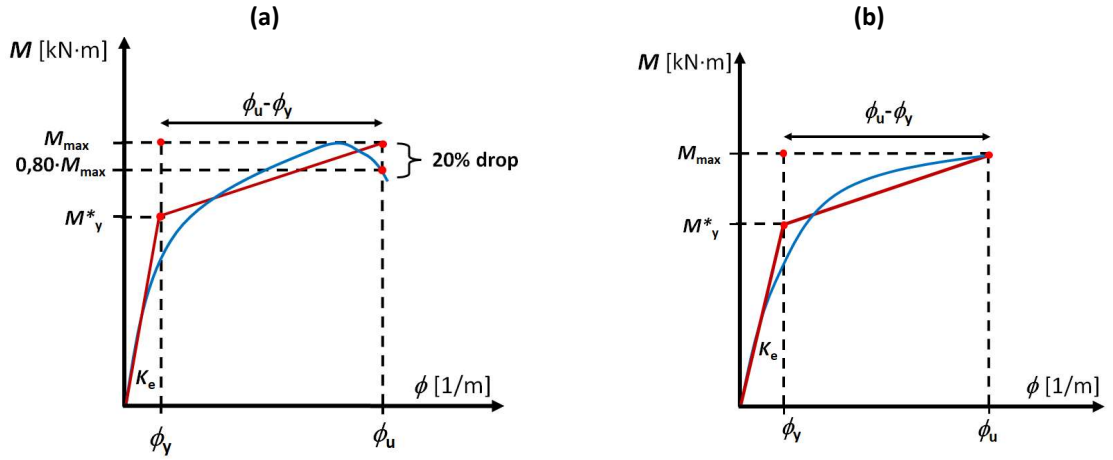


Figure 6-11: Definition of the curvature at yield (ϕ_y) and ultimate curvature(ϕ_u) (a) With verified 20% strength drop (b) Without strength drop

The plastic chord rotation capacity obtained by nonlinear analysis (θ_u^{pl}) is calculated as the difference between ultimate chord rotation (θ_u) and chord rotation at yield (θ_y) as (θ_u^{pl}= θu-θ_y). The calculation of the chord rotation (θ_u) and chord rotation at yield (θ_y) is similar as it is for ultimate curvature and the curvature at yield, using a bilinear model as depicted in Figure 6-12:

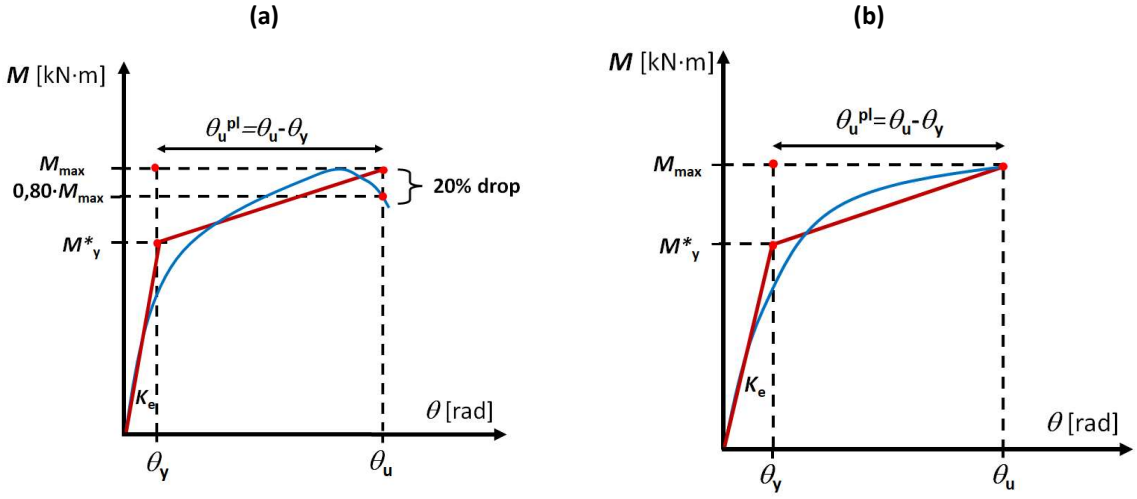


Figure 6-12: Definition of the chord rotation at yield (θ_y) and ultimate chord rotation (θ_u) (a) With verified 20% strength drop (b) Without strength drop

As referred previously, the plastic hinge length ($l_{pl,NLA}$), obtained by nonlinear analysis, is compared with the plastic hinge length obtained according to FprEN 1998-1-1:2024[3]. According to the FprEN 1998-1-1:2024[3], the plastic hinge length (l_{pl}) may be calculated according to expression (6-6)for beams or columns with sections consisting of rectangular parts and walls:

$$l_{pl} = \lambda_{section} \cdot \lambda_{shearspan} \cdot \lambda_{axial} \cdot 0{,}3 \cdot l_w \tag{6-6}$$

Where ($\lambda_{section}$) is the factor accounting for the wall section shape

$$\lambda_{section} = 1 - \frac{1}{3} \cdot \sqrt{\min\left(2{,}5; max\left(0{,}05; \frac{b_w}{l_w}\right)\right)} \tag{6-7}$$

The parameter ($\lambda_{shearspan}$) accounts for the factor for a shear span-to-depth ratio, being (b_w) and (l_w) the wall width and length, respectively, and (L_V) the shear span, as depicted in equation (6-8):

$$\lambda_{shearspan} = 1 + 0{,}4 \cdot min\left(9; \frac{L_V}{l_w}\right) \tag{6-8}$$

The parameter (λ_{axial}) is the factor that accounts for the contribution of the axial force and is defined as follows:

$$\lambda_{axial} = 1 - 0{,}45 \cdot \min\left(0{,}7; \nu\right) \tag{6-9}$$

Where (ν) is the axial force ratio being defined as depicted in expression (6-10):

$$\nu = \frac{N_{Ed}}{b_w \cdot l_w \cdot f_c} \tag{6-10}$$

6.7.1 TENSION SHIFT FOR WALL BENDING DESIGN MOMENT

According to Paulay et al.[23] once the plastic hinge zone is detailed, the question of the flexural strength for the rest of the structural wall arises. Based upon assumptions of an equivalent static force, a recommendation has been made concerning the design bending moment for the distribution of flexural strength from the plastic hinge zone to the top of the wall. This proposal is presented as depicted in Figure 6-13.

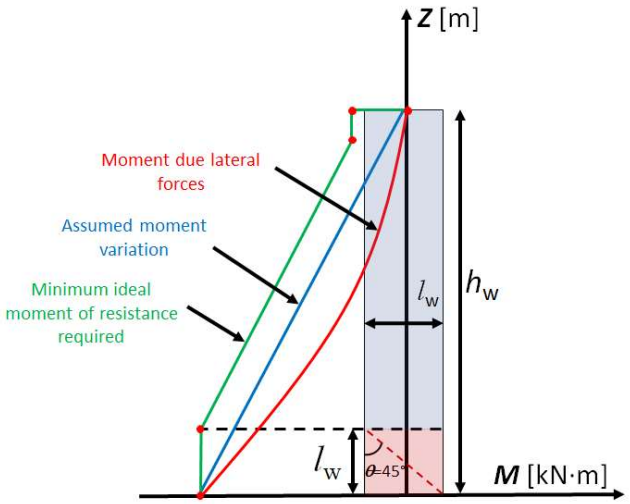


Figure 6-13: Design moment envelope proposed by Paulay et al.[23]

According to Linde[24], the flexural strength within a wall remains constant throughout the entire plastic hinge zone, extending upwards from the base of the wall. Beyond this zone, a gradual reduction in flexural strength is recommended, stretching from the upper boundary of the plastic hinge zone to the top of the wall or until reaching a flexural strength corresponding to minimum reinforcement requirements. Additionally, it is assumed that the distribution of the effective bending moment along the wall height resembles that of a cantilever subjected to a lateral force with trapezoidal distribution.

According to Fardis[4], the bending moment at the wall section above a floor level is usually larger than just below. To ensure the robustness and rigidity of a wall as a vertical solid support, ensuring all beams engage in inelastic behaviour while minimising localised rotation and demands for ductility under a specified global displacement, EN 1998-1:2004[1] and prEN 1998-1-2:2023[2] introduce measures to confine inelastic deformations of the wall to its base.

Such a wall, engineered in this manner, is commonly called a "ductile wall". It is designed and detailed to absorb energy primarily through flexural plastic hinges at its base. It maintains elastic behaviour along its height, facilitating or mandating a beam-sway mechanism. To promote the development of a flexural plastic hinge with significant ductility and energy dissipation capability at its base, a ductile wall must be firmly fixed there to prevent any relative rotation of the base concerning the remainder of the structural system, as depicted by Fardis[4]. To force the wall to stay elastic above the plastic hinge region, Eurocode 8 requires dimensioning in bending the rest of the wall height for a linear envelope of the positive and negative wall moments from the analysis for the design seismic action (Figure 6-14). The envelope also intends to cover a potential increase in bending moments above the base due to higher mode inelastic response after developing the plastic hinge at the base. The rest of the wall does not need to be detailed for high flexural ductility, and the wall design may be more straightforward, as referred by Fardis[4].

As referred by Maranhão et al.[6], to avoid yielding above the base hinge, EN 1998-1:2004[1] prescribes that the design bending moment diagram is based on an envelope derived from the bending moment diagram obtained from analysis vertically displaced, as known as tension shift (a_l), as shown in Figure 6-14(b). According to EN 1998-1:2004[1], the envelope may be assumed linear if the structure does not exhibit significant mass discontinuities, stiffness, or resistance over its height. The tension shift (al) should be consistent with the strut inclination (θ) taken in the ultimate limit states (ULS) verification for shear, as defined in EN 1992-1-1:2004[5] and prEN 1992-1-1:2021[25].

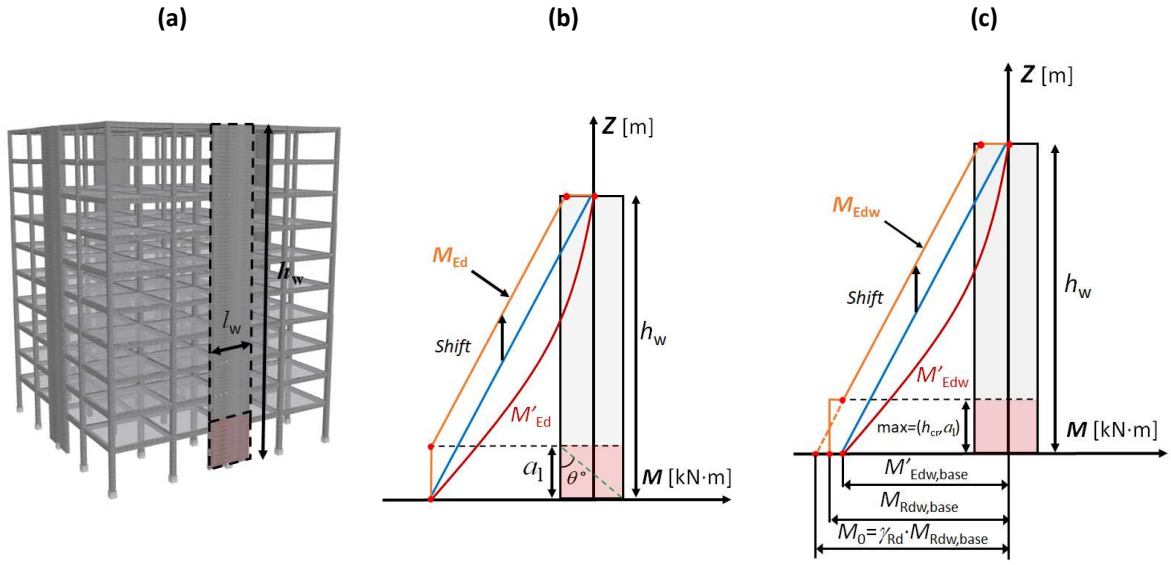


Figure 6-14: Wall envelope of bending moment (a) Design envelope for bending moments for wall systems EN 1998-1:2004[1] (b) Design envelope for bending moments for wall systems prEN 1998-1-2:2023[2] (Adapted from Maranhão et al.[6])

Maranhão et al.[6] refer that prEN 1998-1-2:2023[2] presents a distinct approach compared to EN 1998-1:2004[1]. Along the wall's height, the design bending moment (M_{Edw}) is determined by a vertically displaced (tension shift) envelope derived from the bending moment diagram obtained through analysis, wherein the value at the base section is substituted with the flexural overstrength moment, as depicted in Figure 6-14(c). According to prEN 1998-1-2:2023[2], the flexural resistance of the wall at its base must not fall below the design moment ($M_{Edw,base}$) obtained from the analysis. Throughout the wall's height, the design moment (M_{Edw}) is established by a vertically displaced (tension shift) envelope of the bending moment diagram from the analysis (M'_{Edw}), replacing the value at the base section with the flexural overstrength moment ($M_0 = \gamma_{Rd} \cdot M_{Rdw,base}$) with $\gamma_{Rd} = 1{,}2$, as illustrated in Figure 6-14(c).

Within the maximum of either the critical height of the wall or the tension shift (a_l), the design moment should equal the resistant moment ($M_{Rdw,base}$). Assuming the envelope to be linear is permissible if the structure does not exhibit notable discontinuities in mass, stiffness, or resistance along its height. It is imperative for the tension shift (a_l) to align with the strut inclination (θ) utilised in the SD limit state verification for shear.

According to Schuler et al.[26], A common approach for flexural walls is to calculate the curvature distribution over the height of a wall from the top displacement using the principle of virtual work. However, using the curvatures from a pure moment-curvature analysis of the cracked concrete cross-section results in top displacements that are too small. Larger curvatures develop only at the bottom of the wall. According to Priestley et al.[27], using plastic hinge approaches can avoid this localisation. However, empirical equations are commonly used and not based on a mechanical model. The tension shift effect significantly impacts the total displacement of inclined cracked elements and must be considered using a mechanical approach. It has to be added to the nonlinear flexural behaviour based on a moment-curvature analysis to consider deformations due to flexure and shear interaction, as Schuler et al.[26] referred. This curvature distribution significantly impacts the definition of the tension shift used to define the bending moment envelope.

According to Fardis[4], the phenomenon known as "tension stiffening", which refers to the concrete's contribution to stiffness between cracks under tension, is relatively insignificant in structural members with high longitudinal reinforcement ratios, such as those designed for seismic resistance. Additionally, this contribution heavily relies on the bond strength along the bars between cracks, which deteriorates under cyclic loading. Consequently, the member's end section yields after undergoing one or more elastic load cycles, and the influence of concrete in tension on the overall flexural deformations during yielding is minimal despite being negligible. Thus, the portion of chord rotation at yielding attributable solely to flexural deformations is insignificant. Therefore, the part of the chord rotation at yielding which is due to purely flexural deformations may be defined according to expression (6-11):

$$\theta_y = \phi_y \cdot \frac{L_v}{3} \tag{6-11}$$

According to Fardis[4], the occurrence of inclined cracking and shear deformations along the shear span contributes to an increase in the magnitude of (θ_y). Near the yielding end of the member, diagonal cracking extends the yielding of the tension bars until the first diagonal crack intersects them). This phenomenon is known as the tension shift in designing the tension reinforcement for the Ultimate Limit State (ULS) in bending depending on the axial force level. According to this rule, the force value in the tension reinforcement is transferred from the section where it is initially placed based on the moment and axial force diagrams to another section with a lower moment. The shift occurs due to diagonal cracking, and its extent depends on the angle of the diagonal cracks relative to the member axis and the amount of transverse reinforcement present. However, using a default value for the shift equal to the internal lever arm (d) is standard practice.

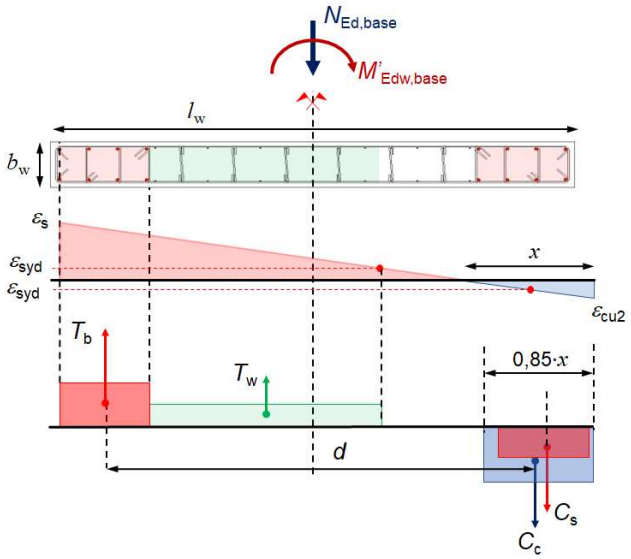


Figure 6-15: Wall strains and axial forces d tensile force and compression force resultant (Adapted from Maranhão et al.[6])

Such a shift increases the value of (θ_y), which is theoretically due to flexural deformations, from expression (6-11) to about equation (6-12):

$$\theta_y = \frac{1}{L_V} \cdot \int_0^{L_V} \phi(z) \cdot (L_V - d) dz \approx \phi_y \cdot \frac{L_v + d}{3} \quad (6\text{-}12)$$

According to Costa et al.[22], the first of these terms represents the effect of curvature associated with the distribution of the linear bending moment, including the shift of the bending moment related to the shear-resistant mechanism. The curvature distribution (ϕ(z)) in the shear span is represented in the Figure 6-16 assuming the following premises:

i) The translation of the bending moment diagram corresponds to a similar translation of the curvature diagram.

ii) The value of that translation is equal to the lever arm (d) in a wall with a rectangular section.

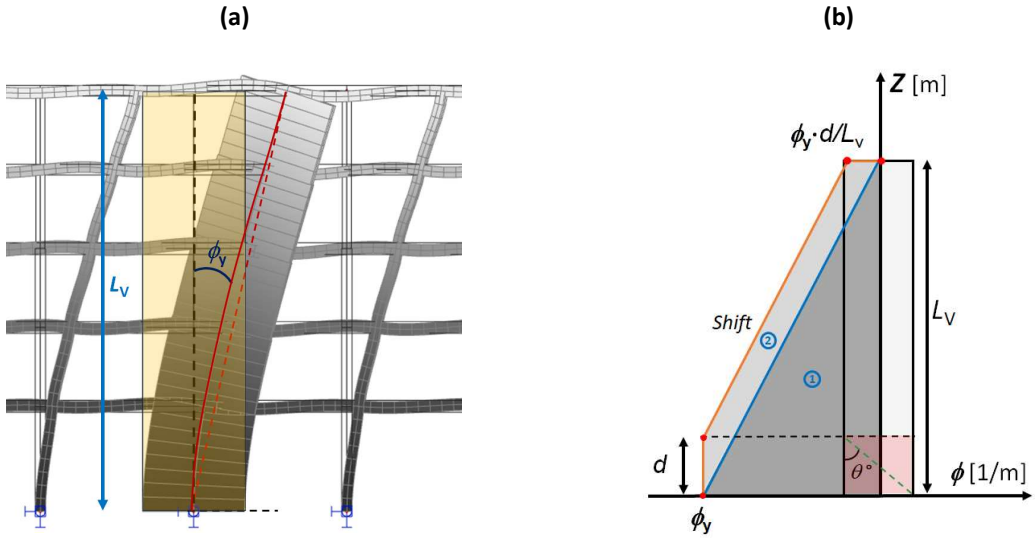


Figure 6-16: (a) Wall curvature (b) Tension shift

According to Costa et al.[22], assuming that $\tan(\theta_y)=\theta_y$, applying the unit force method to zone 1 of Figure 6-16(b), the chord rotation assumes the value of the expression (6-11) and the zone 2 of Figure 6-16(b) comes as depicted in expression (6-13):

$$\theta_y = \frac{\phi_y}{3} \cdot d \cdot \left(1{,}5 - 1{,}5\frac{L_v}{z}\right) \approx \frac{\phi_y}{3} \cdot d \quad (6\text{-}13)$$

This analysis aims to compare the tension shift (a_l) with the tension shift obtained by the nonlinear analysis (d_{NLA}), as depicted in Figure 6-17, derived from nonlinear analysis.

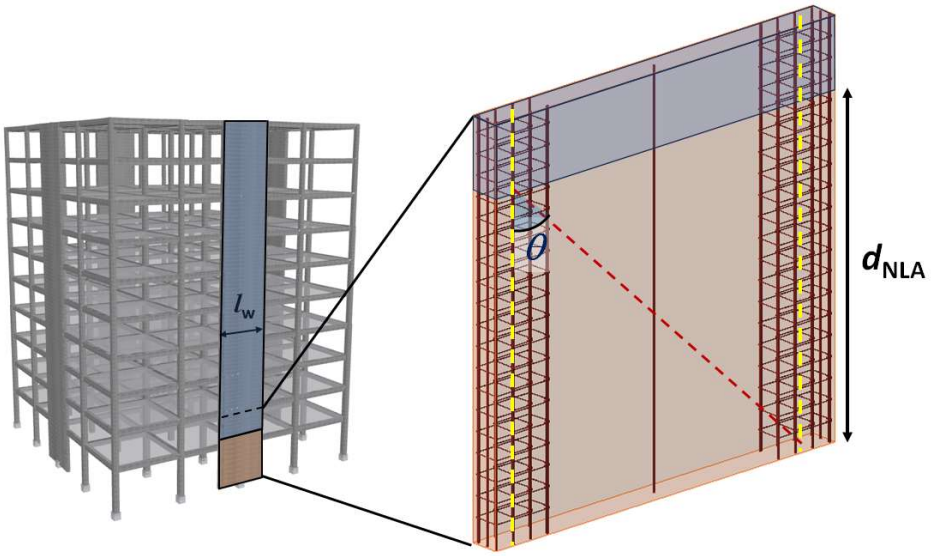


Figure 6-17: Geometrical definition of tension shift obtained by nonlinear analysis

The tension shift obtained by nonlinear analysis (d_{NLA}) can be obtained by reformulating equation (6-13) as follows:

$$d_{NLA} = \frac{3 \cdot \theta_y}{\phi_y} \tag{6-14}$$

The curvature (ϕ_y) and chord rotation (θ_y) at yield are calculated as defined in Figure 6-11 and Figure 6-12. It is essential to acknowledge that the definition of tension shift (a_l) in the present work considers the strut line, which is delineated from the centre to the centre of gravity of the confined boundaries.

6.8 CRITICAL HEIGHT DEFINITION

According to Fardis[4], energy dissipation usually occurs by alternate positive and negative bending in flexural plastic hinges at the wall's ends. The EN 1998-1:2004[1] and FprEN 1998-1-2:2024[2] define "critical regions" as the zones where energy is dissipated, for example, for the part of a concrete wall member where energy dissipation takes place by design. In RC concrete walls, the "critical region" is a conventionally defined part of the base of the wall up to a certain distance from an end section, as depicted in Figure 6-18.

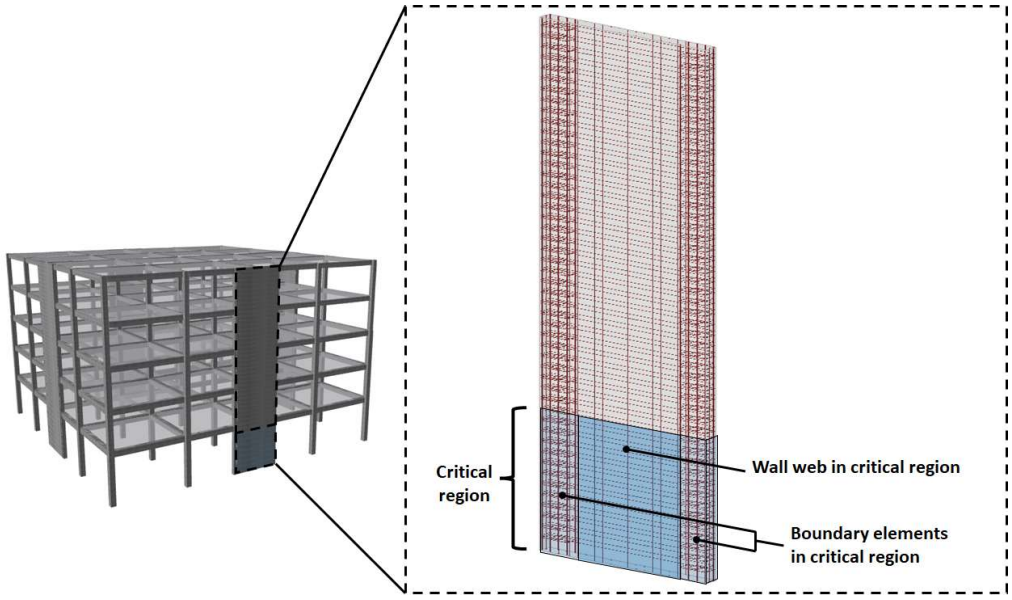


Figure 6-18: Critical region at the base of a cantilevered wall (Adapted from Maranhão et al.[6])

The EN 1998-1:2004[1] and FprEN 1998-1-2:2023[2] prescribe the length of "critical regions" and special detailing rules for them, depending on the type of members" and the Ductility Class of the building. In buildings designed for energy dissipation "critical region" is defined at the end of an element, no matter whether the relative magnitude of the moment resistances of the cross-sections around the joint suggests that a plastic hinge at that end is likely as explained by Fardis[4]. The critical height of a structural element, such as an RC wall, is not necessarily defined solely according to the plastic hinge length but also based on various considerations related to the overall structural behaviour, such as sway instability, (P-Δ) effects (geometric nonlinearity), or other factors that can significantly affect the overall structural response.

The critical region height (h_{cr}) is defined in the same way as for EN 1998-1:2004[1] and prEN 1998-1-2:2023[2] for DCM, DC2 and DC3 as defined in the expression (6-15):

$$h_{cr} = \begin{cases} \geq max\left(l_w, \frac{h_w}{6}\right) \\ \leq min(2 \cdot l_w, h_s)\ if\ wall\ \leq 6\ storeys \\ \leq min(2 \cdot l_w, 2 \cdot h_s)\ if\ wall\ > 6\ storeys \end{cases} \quad (6\text{-}15)$$

where (h_w) is the total height of the wall, (h_s) is the clear storey height, and the base is defined as the foundation level or the top of basement storeys with rigid diaphragms and perimeter walls.

As mentioned in Maranhão et al.[6], the critical height (h_{cr}) does not depend on the angle of the strut. It is also considered the tension shift (a_l) corresponding to a strut of θ=22° and θ=45°. Therefore, 20 structures were modelled for the analysis, with 10 representing a tension shift corresponding to a strut of θ=22° and another 10 representing a strut of θ=45° on the walls.

To make the distinction, Table 6-2 shows the structures' different tension shifts and critical heights, as depicted by Maranhão et al.[6].

Table 6-2: Critical height (h_{cr}) and tension shift (al) for θ=22° and θ=45° (Adapted from Maranhão et. al.[6])

Structure	h_w m	h_s m	l_w m	h_{cr} m	a_l for θ=22° m	a_l for θ=45° m
DCM.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DC2.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DC3.W.50.5	16	4,0	4,0	4,0	8,24	3,33
DCM.W.75.5	16	4,0	4,0	4,0	8,24	3,33
DC3.W.75.5	16	4,0	4,0	4,0	8,24	3,33
DCM.W.50.10	31	4,0	4,5	5,2	8,98	3,63
DC2.W.50.10	31	4,0	4,5	5,2	9,22	3,73
DC3.W.50.10	31	4,0	4,5	5,2	9,22	3,73
DCM.W.75.10	31	4,0	4,5	5,2	8,98	3,63
DC3.W.75.10	31	4,0	4,5	5,2	9,22	3,73

6.9 SHEAR MAGNIFICATION FACTOR

The EN 1998-1:2004[1] stipulates that DCM structures must account for the potential increase in shear forces after yielding at the base of a primary seismic wall. The design shear forces (V_{Ed}), as depicted in expression (6-16), are determined as 50% higher than the shear forces obtained from the analysis (V'_{Ed}). Consequently, a magnification factor (ε) of 1,5 is assumed for this specific ductility class, as referred to by Maranhão et al.[6].

$$V_{Ed}(z) = 1{,}5 \cdot V'_{Ed}(z) \tag{6-16}$$

Figure 6-19 illustrates the shift of the shear force diagram with magnification.

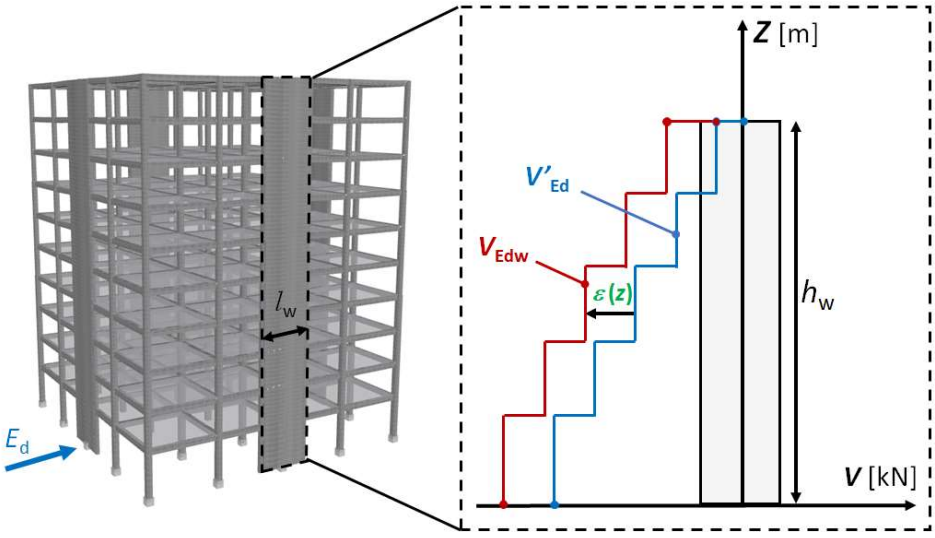


Figure 6-19: Shear force diagrams

The prEN 1998-1-2:2023[2] considers that, for ductile walls, when the force-based approach is used, the design shear force (V_{Edw}) at level (z) should be derived from expression (6-17).

$$V_{Edw}(z) = \varepsilon(z) \cdot V'_{Edw,1}(z) \leq q \cdot V'_{Edw} \tag{6-17}$$

Where (V'_{Edw} (z))is the shear force at level (z) calculated as the combination of shear force in all modes from the analysis, ($V'_{\mathrm{Edw,1}}(z)$) is the effect due to the mode with the largest participating mass in the direction of (V_{Edw}).

The shear magnification factor $\varepsilon(z)$ preconises in prEN 1998-1-2:2023[2] follows the approach Eibl [28]and Keintzel[29] as depicted in subchapter 4.4.5.2, Equation (4-11).

6.10 NUMERICAL RESULTS AND DISCUSSION

The resulting capacity curves, obtained from adaptive pushover analysis, are shown in Figure 6-20. The pushover curves were obtained for the structures with the wall designed for a strut of θ=22° and θ=45°.It is important to mention that both epistemic and aleatory uncertainties influence numerical, theoretical, and computational models of reinforced concrete (RC) structures. Epistemic uncertainties arise from gaps in knowledge and can be reduced, whereas aleatory uncertainties stem from inherent randomness. Studies such as Gino et al.[33] highlight the importance of accounting for both uncertainties

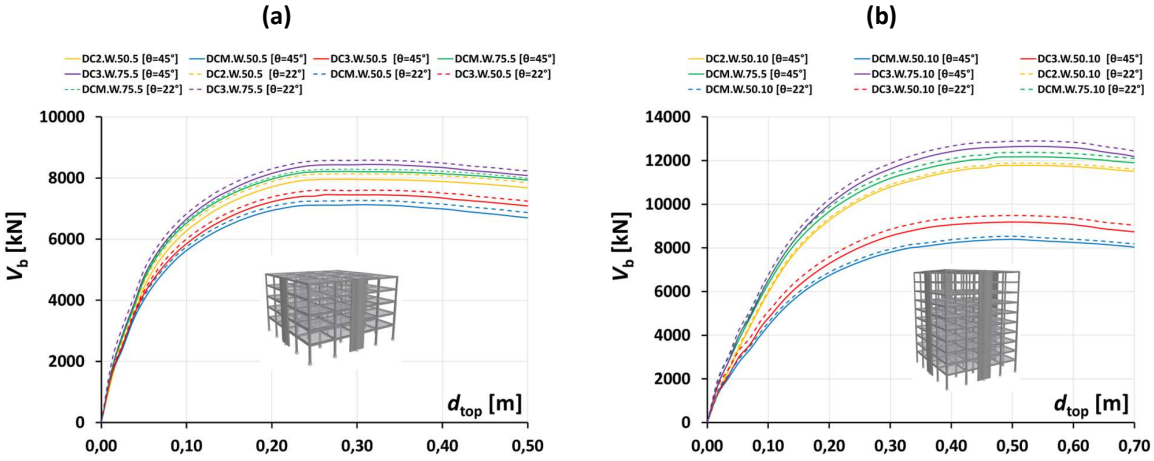


Figure 6-20: Capacity curves (a) 5-storey buildings (b) 10-storey buildings

Figure 6-21 illustrates the ductility ratio (μ=d_r/d_y), where (d_r) is the maximum displacement reached during the analysis, and (d_y) represents the yield displacement. Under identical design response spectra (S_α), the DCM and DC3 frames exhibit more ductile behaviour than DC2.

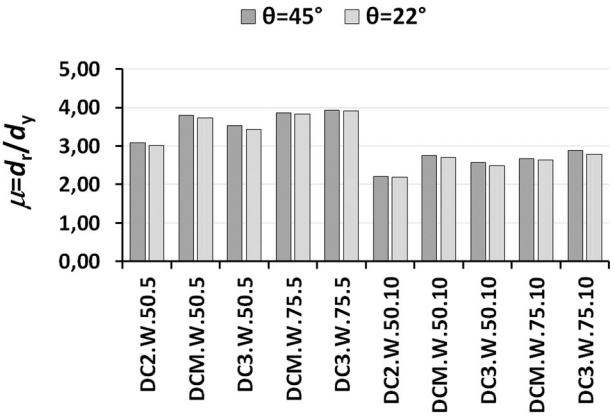


Figure 6-21: Ductility ratio (μ) for the different structures for walls designed with θ=45° and θ=22°

From Figure 6-20 and Figure 6-21, the following observations can be made:

a) As per Figure 23, the strut angle has a minimal impact on the base shear (V_b), although the structures designed with a strut of θ=22° present higher base shear than structures designed with a strut θ=45°.

b) In Figure 24, the inclination of the strut has little impact on the ductility of the structures since the difference between the structures designed with a strut inclination of θ=45° and θ=22° is less than 1%.

c) The structures DC3.W.75.5, DC3.W.50.5, and DCM.W75.5 exhibit a more ductile behaviour, indicating they can undergo more significant deformations relative to their yield displacements.

d) In the case of DC2 structures, specifically DC2.50.10, there is a notable reduction in the ductility ratio when moving from DC2.50.5 to DC2.50.10. This observed change could be attributed to the design strategies implemented during the structural design process.

Based on the analysis, it can be inferred that the ductility class DC3 is equivalent to the ductility class DCM. Furthermore, ductility class DC2 represents an intermediate level of ductility since it presents ductility lower than DC3.

6.11 BEHAVIOUR FACTOR ANALYSIS

The parameters extracted from the pushover curves are presented in Table 6-3, which shows the structures designed with a strut inclination (θ) of 45°:

Table 6-3: Nonlinear analysis parameters for structures designed with a strut of θ=45°

Structure label	V_e kN	d_e m	V_1 kN	d_1 m	V_y kN	d_y m	V_u kN	d_r m
DC2.W.50.5	2478,8	0,04	4412,3	0,05	7966,5	0,13	7966,5	0,39
DCM.W.50.5	1582,2	0,03	4008,2	0,05	7123,3	0,12	7123,3	0,46
DC3.W.50.5	1582,2	0,03	4195,7	0,05	7457,6	0,13	7457,6	0,48
DCM.W.75.5	2373,2	0,04	4662,3	0,05	8216,5	0,14	8216,5	0,49
DC3.W.75.5	2373,2	0,04	4769,3	0,05	8216,5	0,14	8216,5	0,49
DC2.W.50.10	3633,9	0,08	4384,0	0,07	11782,8	0,25	11782,8	0,54
DCM.W.50.10	1993,4	0,06	3343,1	0,07	8389,9	0,24	8389,9	0,65
DC3.W.50.10	1993,4	0,06	3564,9	0,07	9183,5	0,26	9183,5	0,67
DCM.W.75.10	3628,9	0,08	4784,0	0,07	12182,8	0,26	12182,8	0,68
DC3.W.75.10	3733,9	0,08	4870,5	0,07	12182,8	0,24	12182,8	0,68

Table 6-4 for structures designed with an inclination strut (θ) of 22°:

Table 6-4: Nonlinear analysis parameters for structures designed with a strut of θ=22°

Structure label	V_e kN	d_e m	V_1 kN	d_1 m	V_y kN	d_y m	V_u kN	d_r m
DC2.W.50.5	2478,8	0,039	4578,0	0,05	8139,7	0,13	8139,7	0,39
DCM.W.50.5	1582,2	0,027	4116,4	0,05	7267,7	0,12	7267,7	0,46
DC3.W.50.5	1582,2	0,027	4296,5	0,05	7600,1	0,13	7600,1	0,48
DCM.W.75.5	2373,2	0,039	4728,0	0,05	8289,7	0,14	8289,7	0,49
DC3.W.75.5	2373,2	0,041	4982,3	0,05	8580,9	0,15	8580,9	0,49
DC2.W.50.10	3633,9	0,076	4484,0	0,07	11882,84	0,25	11882,8	0,54
DCM.W.50.10	1993,4	0,056	3493,1	0,07	8539,9	0,24	8539,9	0,65
DC3.W.50.10	1993,4	0,056	3864,9	0,07	9483,5	0,27	9483,5	0,67
DCM.W.75.10	3628,9	0,076	4984,4	0,07	12382,8	0,26	12382,8	0,68
DC3.W.75.10	3733,9	0,075	5120,5	0,07	12903,5	0,26	12903,8	0,68

These parameters allow the evaluation of behaviour factor components q_R, q_S, and q_D.

The values of the behaviour factor components are presented in Figure 6-22. It should be noted that EN 1998-1:2004[1] does not prescribe the behaviour factor components. However,

the DCM structures' behaviour factor components were also calculated to compare the current [1] and second-generation [3] versions of Eurocode 8.

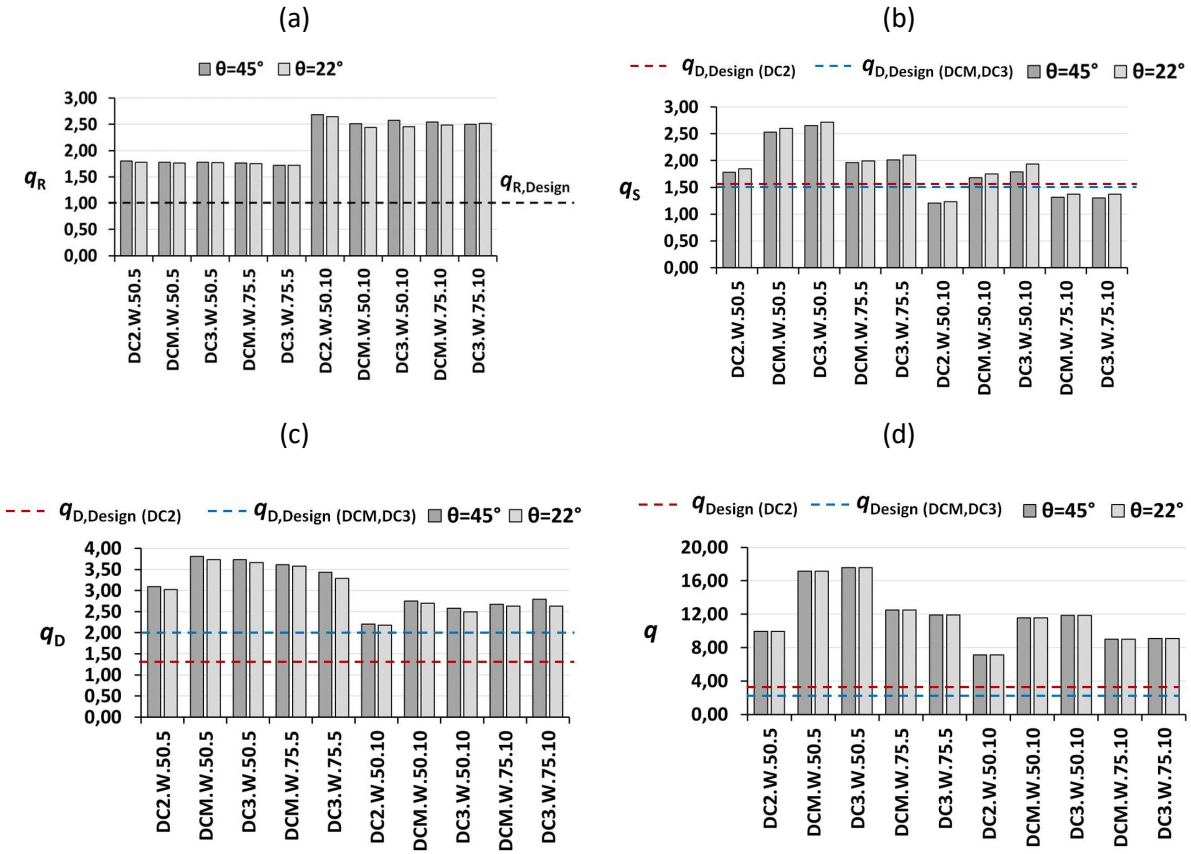


Figure 6-22: Behaviour factor and its components (a) q_R, (b) q_S, (c) q_D, and (d) q

Regarding Figure 6-22, the following observations can be drawn:

a) Upon analysis of Figure 6-22(a), the behaviour factor component accounting for the overstrength (q_R) exhibits specific ranges. For the structures designed with an inclination strut (θ) of 45°, the DCM parameter showed a maximum value of approximately 1,78 and a minimum of about 1,76, with an average of around 1,77. DC2 exhibited a maximum of approximately 2,69, a minimum of about 1,81, and an average of roughly 2,25. For DC3, the maximum value observed was approximately 1,78, while the minimum hovered around 1,72, with an average close to 1,76. Reducing the strut inclination angle to 22° resulted in slightly different outcomes. For DCM, the maximum value was approximately 1,77, with a minimum of around 1,75 and an average of roughly 1,76. DC2 presented a maximum value of approximately 2,65, a minimum of about 1,78, and an average close to 2,24. DC3 designed with a strut inclination θ=22° displayed a maximum value of roughly 1,77, a minimum of around 1,72, and an average close to 1,74. Notably, all these values surpass the recommended value (q_R=1,0) outlined in prEN 1998-1-2:2023[2], indicating a consistent overperformance in the structural behaviour across the various structures, showing that prEN 1998-1-2:2023[2] has a safe approach, considering only the 20 structures studied.

b) In Figure 6-22(b), the behaviour factor component (q_S) is defined as the factor representing overstrength from all other sources. Regarding the structures designed for a strut inclination of 45°, the DCM behaviour factor component (q_S) displayed a maximum value of approximately 2,53, a minimum value of about 1,96, and an average of around 2,13. DC2 showed a maximum of approximately 1,78, a minimum of about 1,21, and an average of roughly 1,62. For DC3, the maximum value observed was approximately 2,65, while the minimum hovered around 1,79, with an average close to 2,2. For a strut inclination of 22°, the behaviour factor component (q_S) of DCM, DC2, and DC3 parameters demonstrated slight variations compared to θ=45°. For DCM, the maximum value was approximately 2,60, with a minimum of around 1,99 and an average of roughly 2,12. DC2 presented a maximum value of approximately 1,85, a minimum of about 1,23, and an average close to 1,56. DC3 at θ=22° displayed a maximum value of approximately 2,72, a minimum of around 1,94, and an average close to 2,25. The prEN 1998-1-2:2023[2] recommends q_S=1,54 for DC2 and 1,50 for DC3 for structures regular in plan and elevation and non-torsionally flexible.

c) Consequently, it can be deduced that prEN 1998-1-2:2023 offers a cautious estimation for the overstrength stemming from various sources (structural redundancy). The analysis is conducted within a three-dimensional framework, encompassing overstrength from diverse origins. This encompasses factors like the redistribution of stresses on the orthogonal beams. By incorporating these elements, multiple load paths are established, ensuring resilience: should one path be compromised, the load can be effectively rerouted through alternative paths. Additionally, the capacity design in shear for controlling the global mechanism further enhances redundancy by facilitating controlled energy dissipation. These considerations collectively contribute to an increase in the values of (q_S).

d) The behaviour factor component (q_D) is a behaviour factor component accounting for the deformation and energy dissipation capacities. Figure 6-22(c) shows that for structures designed with a strut with an inclination angle of 45°, the parameters DCM, DC2, and DC3 exhibited distinct (q_D) values with varying maximum, minimum, and average values. The DCM parameter displayed a maximum value of approximately 3,80, a minimum value of about 3,61, and an average of around 3,69. DC2 showed a maximum of approximately 3,09, a minimum of about 2,20, and an average of roughly 2,79. For DC3, the maximum value observed was approximately 3,73, while the minimum hovered around 2,58, with an average close to 3,10. Concerning the structures designed with a strut angle of 22°, DCM, DC2, and DC3 behaviour factor (q_D)demonstrated slight variations compared to θ=45°. For DCM, the maximum value was approximately 3,73, with a minimum of around 2,63 and an average of roughly 3,11. DC2 presented a maximum value of approximately 3,03, a minimum of about 2,18, and an average close to 2,57. DC3 at θ=22° displayed a maximum value of roughly 3,66, a minimum of around 2,50, and an average close to 3,01.

e) Figure 6-22(d) shows the behaviour factor (q) calculated according to expression (5-2) in subchapter 5.5. In all cases examined here, the q-factor values determined from the nonlinear static analysis were higher than the code-prescribed values. Concerning the structures designed with a strut angle of 45°, the parameters DCM, DC2, and DC3 demonstrated distinct behaviours with varying maximum, minimum, and average values. The DCM behaviour factor (q) displayed a maximum value of approximately 17,13, a

minimum value of about 12,50, and an average of around 14,44. DC2 showed a consistent value of approximately 9,94 across all measurements. For DC3, the maximum value observed was approximately 17,59, while the minimum was around 11,88, with an average close to 14,72. For the structures designed with a strut of 22°, the behaviour factor of DCM, DC2, and DC3 parameters exhibited similar trends to θ=45°. For DCM, the maximum value was approximately 17,13, with a minimum of around 8,98 and an average of roughly 13,58. DC2 maintained a consistent value of approximately 7,14 across all measurements. DC3 for θ=22° displayed a maximum value of approximately 17,59, a minimum of around 9,10, and an average close to 13,94. From the studies of Elnashai et al.[30] The behaviour factor (q) values for RC Frame-Wall Structures range from 9,85 to 14,00, which aligns with the values found in this analysis. The behaviour factor(q_D) is the major contributor to the overall value of (q) for both strut inclination angles, with the buildings designed with a strut θ=45° having the highest average value regarding the behaviour factors components (q_R) and (q_S). The percentage contribution of (q_D) is approximately 56% based on its average value compared to (q_R) and (q_S). This means these wall system structures rely on their resistant capacity and ductility on deformation and energy dissipation capacity.

f) The (q) values presented in the plots are potential outcomes derived from non-linear analysis under various conditions. These values represent the system's potential performance. Further studies must be conducted to validate and refine these design values.

6.11.1 COMPARISON BETWEEN THE PLASTIC HINGE LENGTH OBTAINED BY NONLINEAR ANALYSIS WITH EUROCODE 8

As was referred in the objective is to compare the wall plastic hinge length obtained by nonlinear analysis ($l_{pl,NLA}$) with the critical height (h_{cr}) and the tension shift (a_l), but also to compare the wall plastic hinge length (l_{pl}) preconised by the FprEN 1998-1-1:2024[3]. As explained in 5.1, the wall plastic hinge length obtained by nonlinear analysis (l_{pl}) depends on the wall plastic chord rotation capacity (θ_u^{pl}), the yield curvature (ϕ_y) and the ultimate curvature (ϕ_u), which are obtained from Figure 6-23and Figure 6-24.

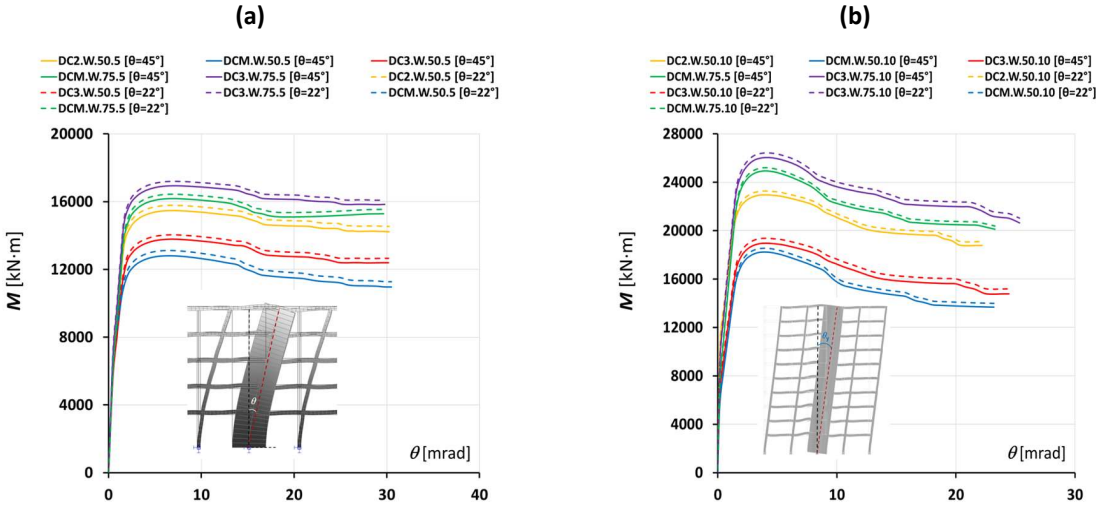


Figure 6-23: Moment vs chord rotation for (a) 5-storey buildings (b) 10-storey buildings

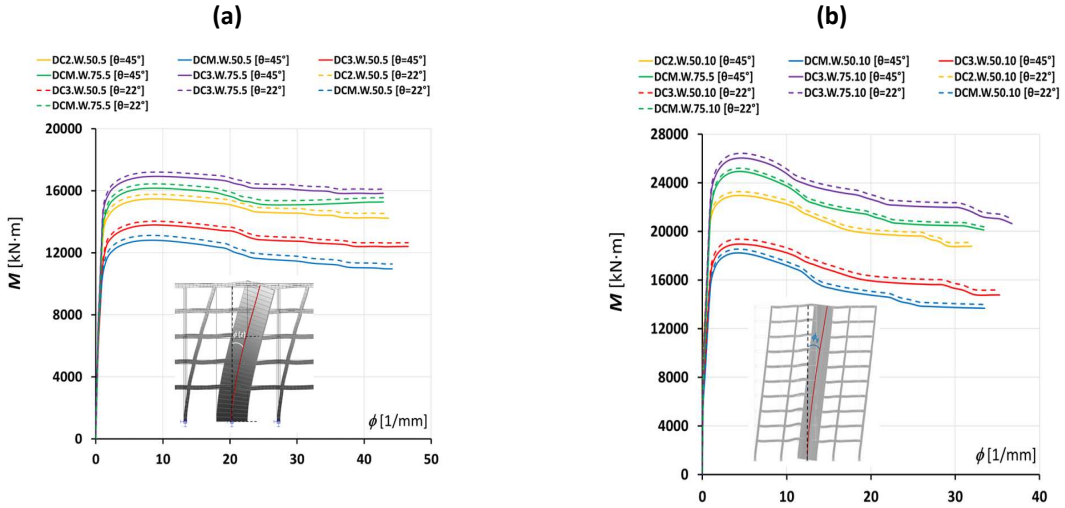


Figure 6-24: Moment vs curvature for (a) 5-storey buildings (b) 10-storey buildings

From Figure 6-23and Figure 6-24, some observations can be made:

a) The figures above show that the walls with a higher strut inclination have little impact on the values of bending moment vs the chord rotation (θ) and bending moment versus curvature (ϕ).

b) Comparing the different curves in 6-23(a) and (b) regarding the hardening (D) across the structures reveals notable differences in their hardening behaviour. The DC2 structures

exhibit the highest hardening slope on average (D=267,1), suggesting superior resistance to deformation compared to DC3 (D=263,1) and DCM (D=258,3).

c) Structures with ductility class DC3 fall between DC2 and DCM in terms of hardening slope, showcasing an intermediate level of ductility to deformation and stiffness characteristics. It should be noted that these observations are consistent with the findings of the analysis of Figure 6-21.

The length of the plastic hinge in the wall ($l_{pl,NLA}$) obtained through nonlinear analysis is calculated considering Figure 6-9 and expression (6-5)(6-5). The plots depicted in Figure 6-25 show the diagrams for the curvature at yield (ϕ_y) and the ultimate curvature (ϕ_u) for the structures under analysis.

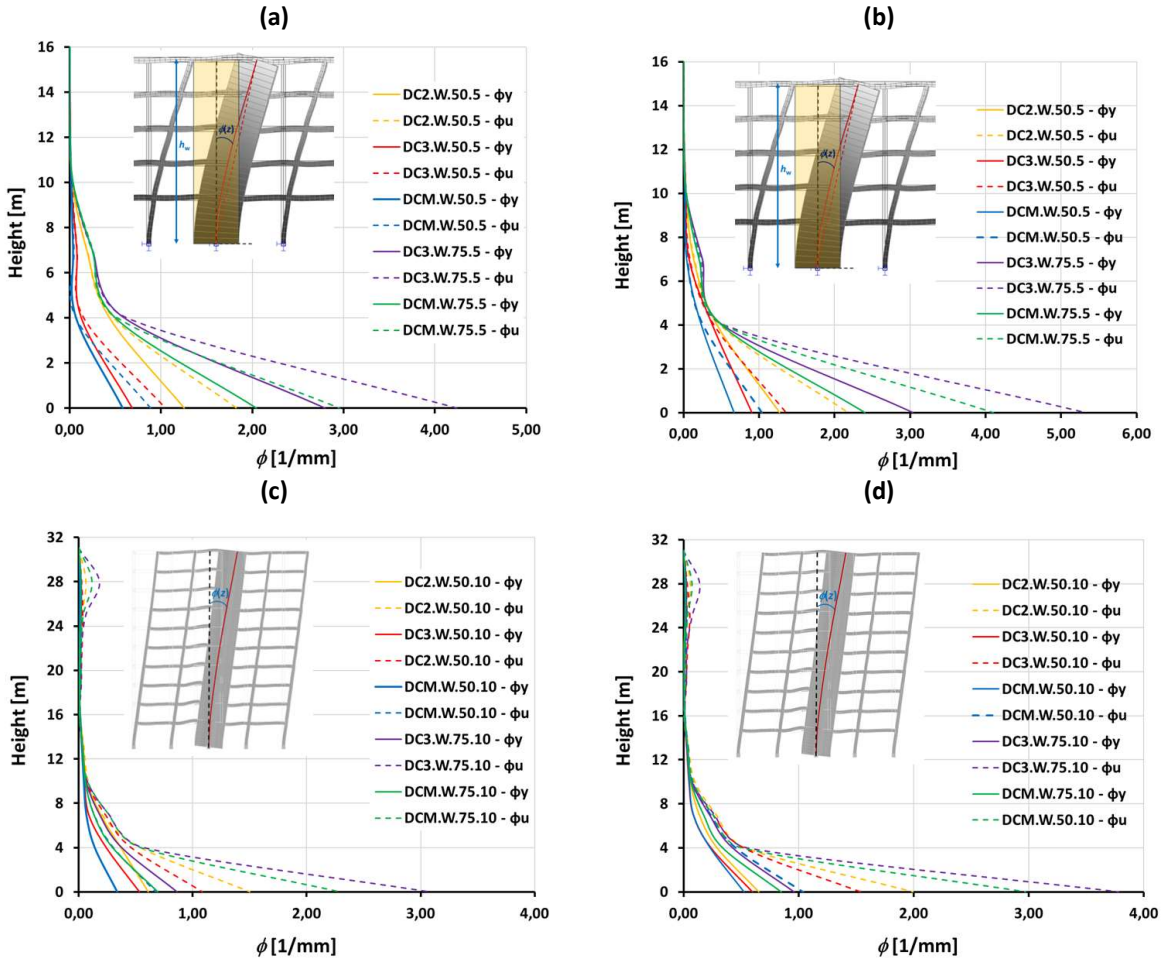


Figure 6-25: Height vs curvature (a) 5-storey buildings designed for strut θ=45° (b) 5-storey buildings designed for strut θ=22° (c) 10-storey buildings designed for strut θ=45° (d) 10-storey buildings designed for strut θ=22°

The length of the plastic hinge in the wall ($l_{pl,NLA}$) obtained through nonlinear analysis is compared with the values of the wall plastic hinge length (l_{pl}) calculated from FprEN 1998-1-1:2024[3], as depicted in Figure 6-26.

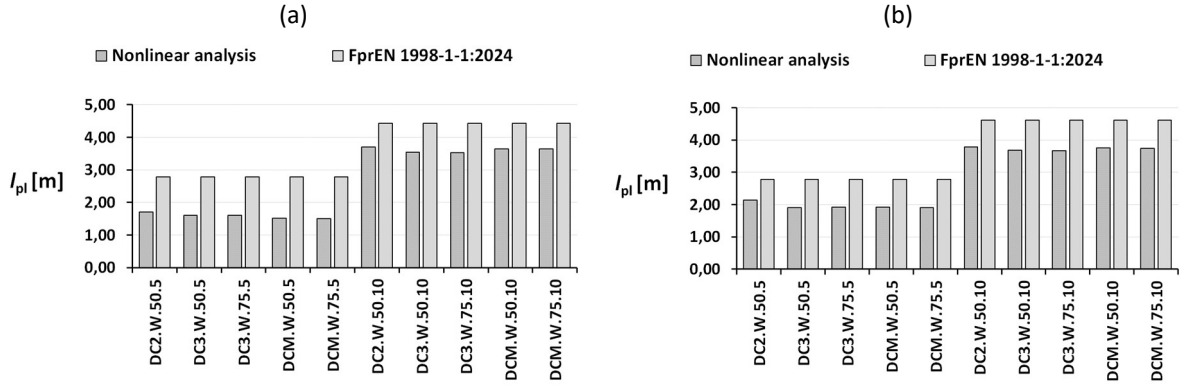


Figure 6-26: Plastic hinge length (l_{pl}) comparison (a) Structures designed with strut inclination θ=45° (b) Structures designed with strut inclination θ=22°

From Figure 6-26, the following observation can be made:

a) When comparing Figure 6-26(a) and (b), it is observed that the plastic hinge length for the wall designed with a strut inclination of 22° is marginally greater than that of walls designed with a strut inclination of 45°. This discrepancy may arise from:

   i) Stiffness Variation: The stiffness of the wall may vary with different strut inclinations, affecting the distribution of stresses and, consequently, the plastic hinge length.

   ii) Strain Concentration: The configuration of the struts and the inclination angle can cause stress concentrations in some regions of the wall. These areas may experience higher stresses, potentially resulting in the formation of longer plastic hinges.

   iii) Geometric Effects: The geometry of the wall, including its shape and aspect ratio, could influence how stresses are distributed and where plastic hinges form.

   iv) Interaction with reinforcement: The orientation and layout of reinforcement within the wall can influence stress distribution. Walls with different strut inclinations may have varying interactions between the reinforcement and applied loads, affecting stress distribution and plastic hinge formation.

b) It is observed that the plastic hinge length calculated as per FprEN 1998-1-1:2024[3] as equation (6-6) presents a higher average of 24,50% for the walls designed with a strut angle of 22° and 30,80% for the walls designed with a strut angle of 45° than the plastic hinge length by nonlinear analysis using equation (6-5).

c) Concerning the 5-storey buildings designed with a strut inclination of 45° (Figure 6-26 (a)), as depicted in Figure 29(a), the maximum difference is observed in the case of DC3.W.50. 5, where the plastic hinge length (l_{pl}) calculated using the equation (12) value exceeds the plastic hinge obtained by nonlinear analysis ($l_{pl,NLA}$) depicted in expression (11) value by 20,33%. Conversely, the minimum difference (38,98%) occurs for the structure DC2.W.50.5. Regarding the 10-storey buildings, the maximum difference is observed in the case of DC3.W.75.5, where the plastic hinge length (l_{pl}) exceeds the

plastic hinge obtained by nonlinear analysis ($l_{pl,NLA}$) value by 20,33%. Conversely, the minimum difference (38,98%) occurs for the DC2.W.50.5.

d) Concerning the 5-storey buildings designed with a strut inclination of 22° (Figure 6-26(b)), as illustrated in Figure 29(b), the maximum difference is observed in the case of DC3.W.75.5, where the plastic hinge length (l_{pl}) value exceeds the plastic hinge obtained by nonlinear value ($l_{pl,NLA}$) by 31,7%. Conversely, the minimum difference (23,24%) for DC2.W.50.5. Regarding the 10-storey buildings, the maximum difference is observed in the case of DC3.W.75.10, where the plastic hinge length (l_{pl}) exceeds the plastic hinge obtained by nonlinear analysis ($l_{pl,NLA}$) value by 20,69%. Conversely, the minimum difference (18,05%) occurs for the DC2.W.50.10.

e) These findings indicate that FprEN 1998-1-1:2024[3] provides conservative values, resulting in structural designs prioritising safety.

6.11.2 COMPUTATIONAL EVALUATION BETWEEN THE PLASTIC HINGE ($L_{PL,NLA}$), THE CRITICAL AND TENSION SHIFT (A_L) AND HEIGHT (H_{CR})

Figure 30 intends to show the comparison between the wall's plastic hinge length (l_{pl}) with tension shift (a_l) and critical height (h_{cr}) for the walls designed with a strut inclination of 45° and 22°.

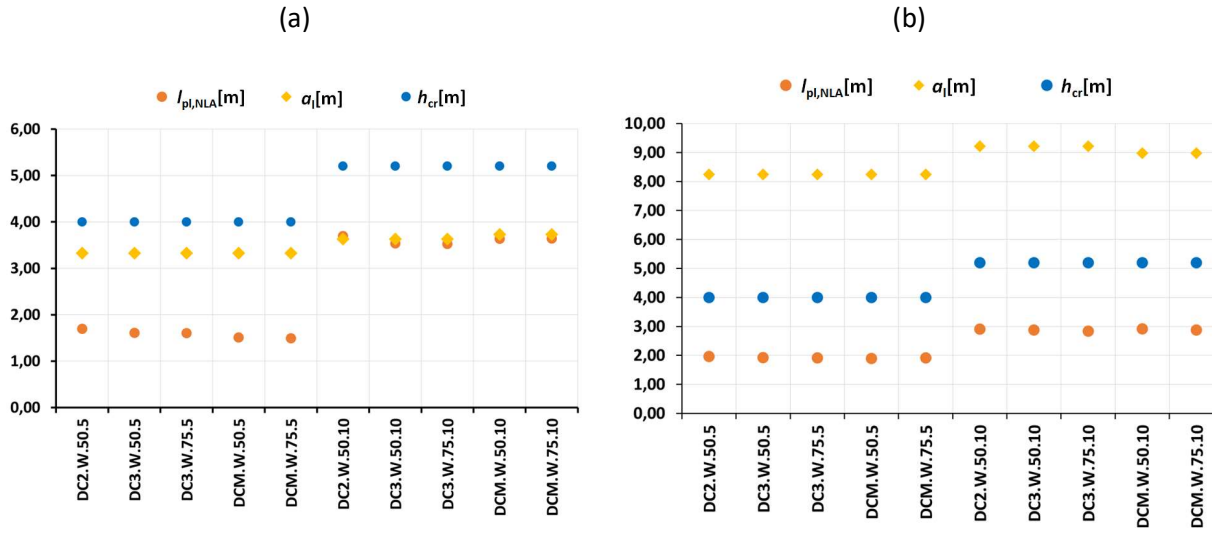


Figure 6-27: Comparison with walls plastic hinge length ($l_{pl,NLA}$) with tension shift (a_l) and critical height (h_{cr}) (a) Structures designed with strut inclination θ=45° (b) Structures designed with strut inclination θ=22°

The observations derived from Figure 6-27(a) and (b) it can be observed that the tension shift (a_l) and critical height (h_{cr}) than the plastic hinge length ($l_{pl,NLA}$) obtained by nonlinear analysis, the following conclusions can be made:

a) Regarding Figure 6-27 (a), analysing the plastic hinge length ($l_{pl,NLA}$) for walls designed with a strut θ=45°, the percentage differences (a_l-$l_{pl,NLA}$)/a_l reveal distinct trends among the structures analysed. Notably, DCM.W.75.5 displays the highest percentage difference, approximately 55%, indicating a significant emphasis on tension resistance relative to plastic deformation. For the 5-storey buildings, the differences range from 48% to 55%. For the 10-storey building, the differences are relatively small, ranging from 1,80% to 2,88%.

b) For designs with a strut θ=22°, comparing percentage differences$(a_l\text{-}l_{pl,NLA})/a_l$ among the structures demonstrates relatively high differences. The differences range from 68% to 76% across all structures, with the buildings presenting the highest differences.

c) Regarding the critical height (h_{cr}) for walls design with a strut inclination θ=45°, when considering the percentage differences $(h_{cr}\text{-}l_{pl,NLA})/h_{cr}$), the values indicate the extent to which critical height exceeds plastic hinge length relative to critical height. For example, DCM.W.75.5 exhibits a percentage difference of approximately 63%. The 5-storey building presents a difference ranging from 57% to 63%, and the 10-storey building presents a difference ranging from 28% to 32%. This suggests a significant emphasis on critical height over plastic hinge length.

d) Concerning the critical height (h_{cr}) for walls designed with a strut inclination θ=22°, the percentage differences $(h_{cr}\text{-}l_{pl,NLA})/h_{cr}$) maintain a comparable pattern, with generally high values across the analysed structures, although smaller than those designed with a strut inclination θ=45°. The 5-storey building presents a difference ranging from 50% to 52%, and the 10-storey building presents a difference ranging from 43% to 44%. This suggests a significant emphasis on critical height over plastic hinge length.

The difference between the tension shift height (a_l) and the plastic hinge length ($l_{pl,NLA}$) is positive; the tension shift height surpasses the plastic hinge length. This observation aligns with the provisions of EN 1998-1:2004[1]and prEN 1998-1-2:2023[2], indicating a conservative approach towards structural safety.

6.11.3 COMPUTATIONAL EVALUATION OF BETWEEN THE TENSION SHIFT (A_L) AND TENSION SHIFT OBTAINED BY NONLINEAR ANALYSIS (D_{NLA})

As referred to in 6.2, it is intended to compare the tension shift height (a_l) that defines the walls envelope bending moment preconised by EN 1998-1:2004[1] and the prEN EN 1998-1-2:2023[2] and the tension shift defined by nonlinear analysis (d_{NLA}).

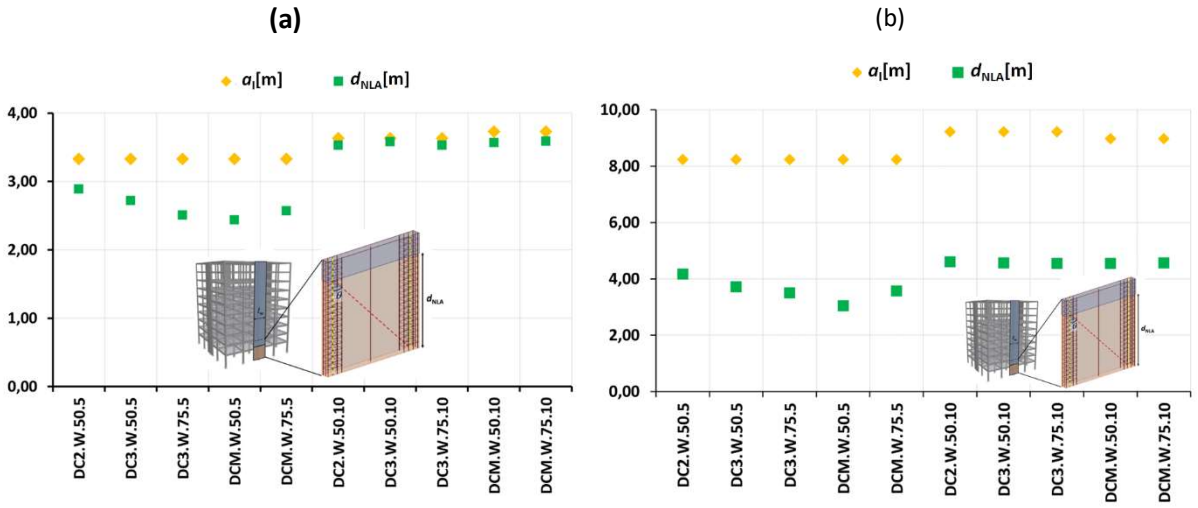


Figure 6-28: Comparison with walls tension shift (d_{NLA}) with tension shift (a_l) (a) Structures designed with strut inclination θ=45° (b) Structures designed with strut inclination θ=22°

From Figure 6-28, some observations and conclusions can be made:

a) For the structures designed with a strut θ=45° DC2.W.50.5, DC3.W.50.5, DC3.W.75.5, DCM.W.50.5, and DCM.W.75.5 the tension shift values (al) remain constant at 3,33m. In contrast, the tension shift values obtained from nonlinear analysis (d_{NLA}) vary slightly, showing differences ranging from 2,44m to 2,89m. This indicates that the nonlinear analysis predicts tension shifts deviating from the initial tension shift values. Similarly, for structures DC2.W.50.10, DC3.W.50.10, DC3.W.75.10, DCM.W.50.10, and DCM.W.75.10, tension shift values (a_l) range from 3,63 to 3,73, while (d_{NLA})values vary between 3,53m and 3,59m.

b) For structures designed with a strut angle of θ=22°, such as DC2.W.50.5, DC3.W.50.5, DC3.W.75.5, DCM.W.50.5, and DCM.W.75.5, the tension shift values (a_l) remain constant at 8,24m. However, tension shift values obtained from nonlinear analysis (d_{NLA}) show slight variations, ranging from 3,57m to 4,17m. This indicates that nonlinear analysis predicts tension shifts that deviate from the initial values. Similarly, for structures DC2.W.50.10, DC3.W.50.10, DC3.W.75.10, DCM.W.50.10, and DCM.W.75.10, tension shift values (al) range from 8,98m to 9,22m, while (d_{NLA}) values vary between 4,56m and 4,61m.

Based on the analysis above and considering Figure 6-28, it can be concluded that performing the structural design with a strut angle of θ=45° is more economically viable. This conclusion arises from observing that the provided tension shift value (a_l) consistently exceeds the tension shift obtained from nonlinear analysis (d_{NLA}) at this inclination. This discrepancy between (a_l) and (d_{NLA}) values suggests a conservative approach that enhances structural safety. Designing the strut angle of θ=22° ensures a higher safety margin, contributing to overall structural robustness and reliability but less economical.

6.12 SHEAR MAGNIFICATION FACTOR COMPUTATIONAL ANALYSIS

To evaluate the shear magnification factor (ε), it can be derived from nonlinear analysis at any point along the wall height (z) as the ratio between the shear force at failure for significant damage limit state (SD) at the base of the wall ($V_{R,NLA}$) and the shear forces obtained from the analysis (V'_{Ed}), as depicted in Figure 32.

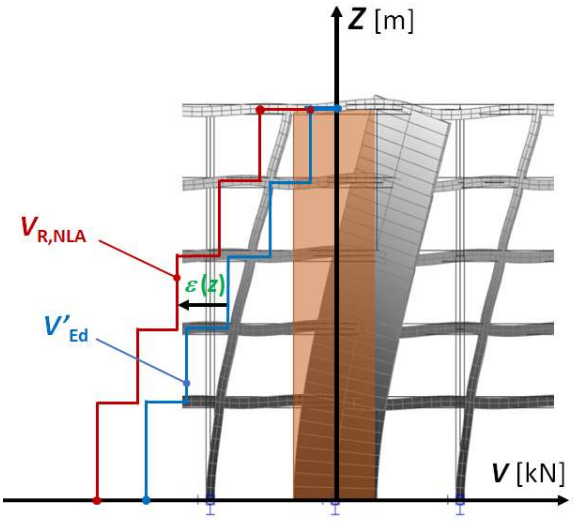


Figure 6-29: Shear force for linear analysis (V'_{Ed}) and shear force at shear failure at the base of the wall ($V_{R,NLA}$)

The magnification factor to be compared may be derived from nonlinear analysis (ε_{NLA}) and can be calculated using the following equation (6-18):

$$\varepsilon_{NLA}(z) = \frac{V_{R,NLA}}{V'_{Edw}} \tag{6-18}$$

Furthermore, compared with the magnification factor (ε), derived from linear elastic analysis from the work of Maranhão et al.[6] calculated according to EN 1998-1:2004[1] and prEN 1998-1-2:2023[2] for the buildings under analysis.

Figure 6-30 represents a triparty plot where the magnification factor to be compared may be derived from nonlinear analysis (ε_{NLA}) with magnification factor (ε) derived from linear elastic analysis and the q-factor.

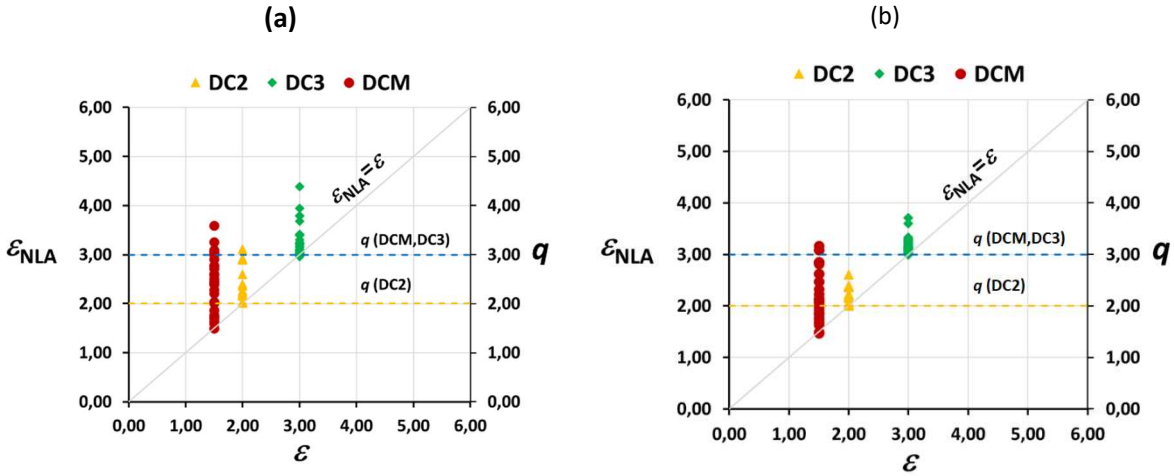


Figure 6-30: Shear magnification factor derived by nonlinear analysis (ε_{NLA}) and behaviour factor (q) *vs* shear magnification factor (ε) derived from Maranhão et al.[7] (a) Structures designed with a strut inclination θ=45° (b) Structures designed with a strut inclination θ=22°

From Figure 6-30, the following observation can be made:

a) In structures designed with a strut inclination of 45°, as depicted in Figure 6-30 (a), varying deviation levels from the median ($\varepsilon_{NLA}=\varepsilon$) discrepancy across different ductility classes are observed. For DC2 structures, the percentage differences range from approximately 0%, meaning that ($\varepsilon_{NLA}=\varepsilon$) to 67,9% ($\varepsilon_{NLA}=1{,}67\varepsilon$) compared to the median difference, indicating significant variability in predicted behaviours. Conversely, DCM structures exhibit differences ranging from approximately 0% to 22,3%, suggesting relatively more consistent alignment with the median differences. For DC3 structures, the differences range from approximately 0% to 38,9% relative to the median, reflecting mixed levels of accuracy across different cases.

b) Similarly, in structures designed with a strut inclination of 22°, as illustrated in Figure 6-30 (b), we observe discrepancies from the median discrepancy in predicted and actual behaviours. However, these discrepancies tend to be lower in magnitude than structures with a 45° strut inclination. For DC2 structures, discrepancies range from approximately 0% ($\varepsilon_{NLA}=\varepsilon$) to 39,2% ($\varepsilon_{NLA}=1{,}39\varepsilon$) relative to the median discrepancy, indicating generally closer agreement with median predictions. DCM structures show discrepancies ranging from approximately 0% to 55,4%, while DC3 structures exhibit discrepancies

ranging from approximately 0% to 16%, implying varying levels of accuracy relative to the median.

c) The observations described in a) align with Rejec et al.'s observations [31], which refer to significantly overestimated shear forces (by up to 40%).

d) Figure 6-30 (a) and (b) reveal shear magnification factors, which are higher than behaviour factor (q) for DC2 and DC3. According to Rejec et. al.[31] these magnifications may be attributed to the amplified effect of higher modes in the inelastic range, combined with flexural overstrength inherent prEN 1998-1-2:2023[2].

6.13 SUMMARY AND CONCLUSIONS

The structural behaviour factors (q) analysis for different strut inclination angles (45° and 22°) reveals consistent overperformance concerning recommended values outlined in prEN 1998-1-2:2023[2].

Results indicate that prEN 1998-1-2:2023[2] offers cautious estimations for structural redundancy. The analysis encompasses various load paths and controlled energy dissipation mechanisms, contributing to increased (q_S) values. The behaviour factor component (q_D) is the major contributor to the overall behaviour factor (q), indicating reliance on structural resistance and ductility for deformation and energy dissipation. The study underscores the effectiveness of prEN 1998-1-2:2023[2] in ensuring structural safety, with observed overperformance in structural behaviour factors. The analysis highlights the importance of considering diverse sources of overstrength and energy dissipation capacities in structural design. These findings provide valuable insights into the robustness of the analysed structures, emphasising the significance of incorporating redundancy and controlled energy dissipation mechanisms for enhanced structural resilience.

Regarding the plastic hinge obtained by nonlinear analysis ($l_{pl,NLA}$) and calculated by the expression preconised by FprEN 1998-1-1:2024[3] (l_{pl}), it is evident that walls designed with a strut angle of 22° strut inclination exhibit slightly longer plastic hinged lengths compared to those with a 45° inclination. This difference can be attributed to various factors, including stiffness variation, strain concentration, geometric effects, and interaction with reinforcement. These factors influence stress distribution and consequently affect plastic hinge formation within the wall structure. Notably, plastic hinge lengths calculated according to FprEN 1998-1-1:2024 yield higher averages than those obtained through nonlinear analysis. Specifically, the plastic hinge length averages are 24,5% higher for walls designed with a strut 22° strut angle and 30,8% higher for walls with a 45° angle. This indicates a conservative estimation provided by FprEN 1998-1-1:2024, prioritising safety in structural designs. Analysing the differences in plastic hinge lengths between FprEN 1998-1-1:2024[3] and nonlinear analysis for both 5-storey and 10-storey buildings with different strut inclinations reveals varying degrees of difference. These differences range from 18,1% to 31,7%, with walls designed with DC3 exhibiting the highest differences. Conversely, walls designed with DC2 demonstrate the lowest differences, indicating potential variations in structural behaviour based on design parameters. Overall, the findings suggest that FprEN 1998-1-1:2024[3] provides conservative values for plastic hinge lengths, leading to structural designs prioritising safety. Understanding these variations is crucial for ensuring appropriate design considerations and maintaining structural integrity in building projects.

Observations from Figure 6-27(a) and (b) reveal differences between tension shift (a_l) and critical height (hcr) compared to plastic hinge length (l_{pl}) obtained through nonlinear analysis. For walls with a 45° strut inclination, the percentage differences between (a_l) and (l_{pl}) exhibit distinct trends. Structures like DCM.W.75.5 display significant differences, emphasising tension resistance over plastic deformation. The percentage differences range from 48% to 55% for 5-storey buildings, while for the 10-storey structure, they range from 1,80% to 2,88%, indicating varying design priorities. In contrast, for walls designed with a strut angle of 22° strut, the differences between (a_l) and ($l_{pl,NLA}$) show relatively higher values, ranging from 68% to 76% across all structures. This suggests a stronger emphasis on tension resistance than plastic deformation, particularly in buildings with the highest differences. Analysing critical height (h_{cr}) relative to plastic hinge length ($l_{pl,NLA}$) for both strut inclinations, similar patterns emerge, with generally high percentage differences observed. For walls with a 45° inclination, the percentage differences range from 57% to 63% for 5-storey buildings and 28% to 32% for 10-storey buildings. Meanwhile, for walls with a 22° inclination, the percentage differences range from 50% to 52% for 5-storey buildings and 43% to 44% for 10-storey buildings, indicating a consistent prioritisation of critical height over plastic hinge length. These observations align with conservative approaches towards structural safety outlined in EN 1998-1-1:2004[1] and prEN 1998-1-2:2023[2].

Observations from Figure 6-28 highlight the consistency of tension shift values (al) for structures designed with a strut inclination of 45°. In contrast, values obtained from nonlinear analysis (dNLA) show slight variations, indicating deviations from the initial tension shift values. Similarly, walls designed with a 22° strut inclination exhibit consistent tension shift values, albeit with slight variations in values obtained from nonlinear analysis. Considering the provided data and the accompanying Figure 6-28(a) and (b), it can be inferred that structural design with a 45° strut inclination is more economically viable. This inference stems from the consistent observation that the provided tension shift values (al) consistently exceed those obtained from nonlinear analysis (d_{NLA}) at this inclination. The discrepancy between these values suggests a conservative approach, enhancing structural safety. In contrast, designing the strut angle θ=22° ensures a higher safety margin, contributing to overall structural robustness and reliability, albeit at a potentially higher cost.

Significant deviations from the median discrepancy across different ductility classes are observed in structures featuring a strut inclination of 45°, illustrated in Figure 33(a). For DC2 structures, the deviations range from 0% to 67,9% compared to the median difference, indicating substantial variability in predicted behaviours. Conversely, DCM structures show differences ranging from 0% to 22,3%, suggesting relatively more consistent alignment with the median differences. DC3 structures exhibit differences ranging from 0% to 38,9%, reflecting mixed levels of accuracy across different cases.

Similarly, in structures designed with a 22° strut inclination, as depicted in Figure 33(b), discrepancies from the median in predicted behaviours are observed, albeit with lower magnitudes than wall structures with a 45° strut inclination. For DC2 structures, discrepancies range from 0% to 39,2% relative to the median, indicating generally closer agreement with median predictions. DCM structures show discrepancies ranging from 0% to 55,4%, while DC3 structures exhibit discrepancies ranging from 0% to 16%, implying varying levels of accuracy relative to the median.

These observations align with findings from Rejec et al.[31], suggesting that structures' shear forces may be overestimated by up to 40%. Furthermore, Figure 6-30(a) and (b)

demonstrate shear magnification factors exceeding the behaviour factor (q) for DC2 and DC3. Rejec et al.[31] highlight that these magnifications are likely attributed to the amplified effect of higher modes in the inelastic range, combined with the flexural overstrength inherent in prEN 1998-1-2:2023.

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6.15 SYMBOLOGY

6.15.1 UPPER CASE LATIN SYMBOLS

Symbol description	*Upper case Latin symbols*		
	DC2	DCM	DC3
Reinforcement steel hardening slope	D		
Shear span	L_V		
Bending moment at the formation of the first crack	M_{cr}		
Wall design value of the applied internal bending moment	M_{Ed}		
Bending (Rotation force) at yield	M^*_y		
Rotational (bending) effective ultimate strength	M^*u		
Near collapse limit state	NC		
Load vector	P		
Design acceleration in the function of the natural period	$S_d(T_1)$		
Significant damage limit state	SD		
Single-Degree Of Freedom system	$SDOF$		
Constant acceleration plateau of the elastic response spectrum	S_α		
Period of the idealised equivalent SDOF system	T^*		
Natural period	T_1		
Base shear at the attainment of the first plastic hinge	V_1		
Base shear	V_b		
Elastic base shear	V_e		
Shear forces obtained from the analysis	V'_{Ed}		
Design shear force	V_{Edw}		
Spectral base shear	V_{ed}		
Peak force	$V_{el,max}$		
Hypothetical horizontal shear force, assuming all sections are identical to the real ones but which would be made of a material with an infinite yield strength.	$V^*_{e,u}$		
Horizontal shear force corresponding to the formation of a global plastic mechanism	V_u		
Bilinear yield strength[14]	V_{ud}		
Shear force at yield	V_y		

6.15.2 LOWER CASE LATIN SYMBOLS

Symbol description	*Lower case Latin symbols*		
	DC2	**DCM**	**DC3**
Tension shift	a_l		
Cross-sectional dimension of column	b_c	b_w	
Distance between consecutive engaged bars	b_i	-	
Width of confined core in the boundary element of a wall (to the centreline of hoops)	b_0		
Concrete cover	c		
Effective depth of a cross-section	d		
Roof displacement from the elastic analysis	$d_{e,roof}$		
Transverse reinforcement diameter	d_{bw}		
Displacement at the formation of the first plastic hinge	d_1		
Displacement at the formation of the first plastic hinge of the SDOF system	d^*_1		
Elastic displacement	d_e		
Displacement at yield	d_y		
Displacement at formation of the global plastic mechanism	d_r		
Displacement at Significant Damage limit state	d_{SD}		
Displacement of top (roof) of a building	d_{top}		
Ultimate displacement of an SDOF system	d^*_u		
Design value of concrete compressive strength	f_{cd}		
Mean value of concrete compressive strength	f_c, f_{cm}		
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}		
Mean value of axial tensile strength of concrete	f_{ctm}		
Design yield strength of reinforcement	f_{yd}		
Mean yield strength of reinforcement	f_y, f_{yd}		
Design yield strength of the shear reinforcement	f_{ywd}		
Characteristic yield strength of reinforcement	f_{yk}		
cross-section depth of a beam or column	-	h	
Behaviour factor	q		
Behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures	-	q_R	
Behaviour factor component accounting for overstrength due to all other sources	-	q_S	
Behaviour factor component accounting for the deformation capacity and energy dissipation capacity	-	q_D	
Spacing of transverse reinforcement	s		

6.15.3 GREEK LETTERS

Symbol description	Greek symbols		
	DCM	DC2	DC3
Shear action magnification factor	ε	$\varepsilon(z)$	
Load factor	λ		
Factor accounting for the wall section shape	$\lambda_{section}$		
Factor for a shear span-to-depth ratio	$\lambda_{shearspan}$		
Factor that accounts for the contribution of the axial force	λ_{axial}		
Design axial force due in the seismic design situation, normalised to $A_c \cdot f_{cd}$	ν		
Strut inclination/chord rotation	θ		
Chord rotation corresponding to the attainment of the SD limit state according to FprEN 1998-1-1:2024[4]	θ_{SD}		
Ultimate chord rotation calculated according to FprEN 1998-1-1:2024[4]	θ_u		
Plastic chord rotation capacity calculated according to FprEN 1998-1-1:2024[4]	θ_u^{pl}		
Basic value of plastic chord rotation capacity of a member, assuming:	θ_{u0}^{pl}		
Chord rotation at yield calculated according to FprEN 1998-1-1:2024[4]	θ_y		
Chord rotation at yield considering the longitudinal reinforcement contribution	$\theta_{y,M}$		
Chord rotation at yield considering the reinforcement bond contribution	$\theta_{y,slip}$		
Chord rotation at yield considering the transverse reinforcement contribution	$\theta_{y,V}$		
Chord rotation corresponding to the attainment of the SD limit state obtained by nonlinear analysis	$\theta_{SD,NLA}$		
Ultimate chord rotation obtained by nonlinear analysis	$\theta_{u,NLA}$		
Plastic chord rotation capacity obtained by nonlinear analysis	$\theta_{u,NLA}^{pl}$		
Chord rotation at yield obtained by nonlinear analysis	$\theta_{y,NLA}$		

7 Closure and future research

In closing, the work encapsulated within this thesis represents a comprehensive exploration of the seismic design landscape as sculpted by the emerging provisions of the second generation of Eurocode 8. Through meticulous investigation and analysis, the study provides fresh insights into the design, safety, and performance of reinforced concrete structures in the face of seismic adversity. The conclusions drawn herein contribute to the academic discourse and resonate with practical implications for structural engineering.

The narrative of this thesis traverses through the complex and evolving terrain of seismic structural engineering, unpacking the nuances of the second generation of Eurocode 8. The research delves into the interplay between new ductility classes and the behaviour factor's components, evaluating their influence on structural resilience and integrity. This work highlights pivotal changes, including the pivot towards displacement-based design, the streamlining of ductility classifications, and the recalibration of the behaviour factor - each element carefully weighed for its seismic significance.

As we stand at the cusp of these evolutionary code modifications, the findings urge the industry to re-examine traditional methodologies, embrace innovation, and recalibrate existing practices to align with the new seismic design paradigm. This body of work has laid down a gauntlet, challenging practitioners and researchers to scrutinize, adapt, and further refine the approaches proposed by the latest Eurocode iteration.

The thesis highlights introducing new ductility classes—DC1, DC2, and DC3—within the second generation of Eurocode 8, simplifying previous classifications and aiming to address various seismic intensities better. This allows for more tailored seismic design approaches based on regional seismic activity levels.

Behavior Factor Components: The research emphasizes the redefined behaviour factor, now split into components (q_R, q_S, q_D). This change enhances the design's flexibility and accuracy by accounting for the overstrength, energy dissipation, and deformation capacities of structures, making the seismic design more responsive to the actual behaviour of structures under earthquake loads.

Integration of the Displacement-Based Design (DBD): A major shift has been identified, with the movement towards displacement-based design becoming more prominent compared to traditional force-based methods. This approach focuses on the structure's performance under seismic loading, particularly through controlling displacements and rotations, ensuring greater structural integrity and safety.

Impact of Code Changes on Structural Safety and Economy: The conclusions suggest that the updated code provisions potentially increase seismic designs' safety and economic efficiency. The thesis proposes that future buildings designed under these new regulations could achieve higher resilience against earthquakes, reducing overall costs related to damage and reconstruction.

The thesis identifies several areas for future research, including the need for empirical validation of the new design approaches, further refinement of behaviour factor components, and exploring the economic impacts of the updated codes. This future work will help to confirm the efficacy of the new Eurocode 8 provisions and refine them for practical application.

7.1 GENERAL CONCLUSIONS

This summary encapsulates the key findings and implications of the seismic design advances presented in the thesis, emphasizing the transformative potential of the second generation of Eurocode 8 for building safer, more cost-effective structures in seismic zones.

The comparison of the behaviour factor (q) values across the ductility classes and MRF and wall structures from the document reveals several insights into their structural behaviour under seismic loads. The nonlinear analysis presents different behaviour factor (q) components (qR, qS, qD) for each structure type and ductility class.

The behaviour factor components of the Moment-Resistant frames (MRF) and wall structures differ notably across the DC2, DCM, and DC3 classifications. The overstrength component (q_R) generally shows values surpassing the typical recommended values across all ductility classes, indicating a strong performance in terms of structural overstrength. From the examination of the behaviour factors, there is a noticeable differentiation in the behaviour of MRFs and wall structures, particularly when dissecting components like q_R (overstrength), q_S (overstrength from other sources), q_D (deformation and energy dissipation), and the overall behaviour factor (q).

The q_R-factor, which represents the overstrength, has notably higher ranges in the MRF structures (Figure 5-21) than in wall structures (Figure 6-22). In MRFs, for instance, DC2 structures exhibit (q_R)values from 1,66 to 1,68, significantly higher than those in wall structures where DC2 spans only 1,81 to 2,69. This variance indicates the inherent robustness in MRFs due to their flexibility and ductility, which contrasts with the rigidity of wall structures, thus affecting the overstrength values. The (q_S) values, reflecting overstrength from sources other than the primary resistance mechanism, also vary. In MRFs, (q_S) values are substantially higher, indicating that factors like torsional resistance and stress redistribution through slabs are more pronounced. This contrasts with wall structures, where qS values, though varied, reflect more concentrated structural integrity with less reliance on auxiliary strength sources. The (q_D) values, indicating deformation and energy dissipation capabilities, are consistently higher in wall structures compared to MRFs. For example, in DC2 of wall structures, (q_D) can reach up to 3,09, significantly surpassing the maximum of 2,9 in MRFs. This highlights that wall structures, designed with specific inclination angles, are optimized for higher energy dissipation and deformation capacity, which is crucial in seismic regions.

The overall behaviour factor (q), a culmination of all the individual components, shows higher values in MRFs than in wall structures. This discrepancy is largely influenced by the qS component, contributing substantially to the total q value in MRFs. This indicates a complex interaction of structural elements in MRFs, contributing to a higher overall behaviour factor, thereby suggesting a greater capacity to withstand seismic forces through multiple mechanisms of action. Across different DCs, both structures often exceed the values recommended for the current and second generation of Eurocode 8. This overperformance might suggest that current design practices and materials capabilities allow for safer, more robust structures than those minimally required by existing standards. This is particularly evident in the significant variance in (q_S) and (q_D) components across different DCs and structural types.

The comparative analysis between MRF and wall structures in light of their behaviour factors offers significant insights into the design and enhancement of seismic resilience. The higher (q) values in MRFs suggest a design that inherently accounts for multiple failure modes and paths for stress redistribution, which is crucial in earthquake-prone areas.

In conclusion, both structural systems offer distinct advantages depending on the seismic design requirements and the specific performance objectives engineers target. Such insights can guide future revisions of structural design codes, ensuring they reflect the latest understanding of behaviour factors in varied structural configurations.

For the MRF, there is a specific focus on longitudinal and transverse reinforcement, which are crucial for structural integrity, especially in seismic zones. Similarly, Wall structures, characterized by different strut inclinations (22° and 45°), demonstrate variation in reinforcement needs, reflecting the structural responses to lateral forces.

The bill of quantities for MRF structures shows differences in the material usage based on the design code variations (DCM, DC2, DC3) and seismic demands. For 5-storey buildings, DCM to DC2 increases longitudinal and transverse reinforcements, indicating a stronger design to withstand higher seismic impacts. Conversely, transitioning from DC2 to DC3 reveals a reduction in reinforcement weights, suggesting a design optimization or different assumptions in structural load handling. As observed, seismic considerations (denoted by S_α) significantly influence the reinforcement strategy. The data underlines the importance of tailored structural reinforcements to address specific seismic activities, evident from the variable reinforcement increments between the structures designed for 5,00 m/s² and 7,50 m/s² seismic velocities.

The wall structures with varied strut inclinations show different reinforcement patterns. The longitudinal reinforcement is greater at a 22° inclination, possibly due to the increased axial forces and bending moments at lower angles. The 45° inclination sees a higher transverse reinforcement, attributed to the increased shear forces at steeper angles. These distinctions underline engineering strategies to optimize wall responses to lateral forces effectively. Significant variances are noticed in reinforcement changes when comparing structures from DCM to DC2 and DC3 for wall structures under seismic action. The results highlight how seismic forces necessitate adjustments in structural reinforcements, notably at higher storey counts and different strut inclinations, reflecting a strategic enhancement in structural resilience against seismic activities.

The data from both MRF and wall structures suggest a complex interplay of structural dimensions, material quantities, and seismic requirements. Engineers and architects must consider these variables to design buildings that meet the statutory requirements and ensure safety and durability. The observed trends and variations in the reinforcements point towards a nuanced understanding of how different structural components react under varied seismic loads, informing better, more efficient structural designs.

Chapter 3 and Chapter 5 of the study primarily explore various aspects of building design under seismic influences according to Eurocode 8, focusing on ductility classifications, design strategies, and the behaviour of structures under seismic loading. Both chapters delve into the detailed behaviour of structural components but from slightly different angles.

Chapter 3 discusses the design provisions and requirements for different ductility classes (DCM, DC2, DC3) and seismic action intensity levels. It emphasizes the seismic response, stability, and detailing of local ductility in building structures, particularly how these elements influence the overall seismic performance. In contrast, Chapter 5 looks at the variability in ductility ratios within the same structural category, highlighting how design parameters and loading conditions influence ductility. It extends this discussion to the behaviour factor components which account for overstrength and energy dissipation during seismic events. Both chapters provide a granular analysis of structural components such as beams, columns, and

reinforcements. Chapter 3 examines how different ductility classes affect the behaviour of beams in terms of bending moments and shear forces. Similarly, Chapter 5 explores how these structural elements respond regarding chord rotation capacity and confinement, providing insights into the conservative nature of design considerations.

Chapter 3 and Chapter 5 discuss the role of overstrength and redistribution in enhancing structural performance during seismic events. Chapter 3 highlights this by comparing current and previous versions of Eurocode 8, focusing on how overstrength affects the design of columns and beams. Chapter 5 continues this discussion by providing quantitative data on behaviour factors that exceed recommended values, indicating a conservative approach to seismic design that factors in extra capacity for energy dissipation and overstrength.

Both chapters use comparative analysis to demonstrate the implications of different design choices. Chapter 3 directly compares different ductility classes and their impact on structural responses, such as roof displacements and base shear. Chapter 5 compares behaviour factor values across different structures, demonstrating variations in ductility and overstrength influenced by design decisions. The safety and economic implications of structural design choices are a common theme in both chapters. Chapter 3 discusses how different ductility requirements can affect buildings' overall cost and safety, providing a comprehensive summary of findings that offer insights into economic implications. Chapter 5, while focusing more on structural behaviour under seismic loading, also touches on the safety margins incorporated through design strategies that exceed standard recommendations.

Comparing Chapter 4 and Chapter 6 allows us to draw connections and identify common themes regarding structural engineering design for seismic resilience according to different iterations of the Eurocode provisions. Both chapters focus on analysing the impact of various design parameters, such as ductility classes, strut inclinations, and building heights, on seismic performance. However, each emphasizes different aspects of the structural design and response.

Chapter 4 emphasizes how buildings of different heights and ductility classes (DCM, DC2, DC3) respond to seismic actions regarding base shear values and roof displacements. Similarly, Chapter 6 discusses how strut inclination impacts the plastic hinge lengths and emphasizes the conservative nature of the second generation of the Eurocode 8 in predicting structural behaviour, focusing on overperformance in terms of behaviour factors (q). Both chapters thoroughly examine structural responses to seismic loads, indicating that base shear and ductility significantly influence building performance during earthquakes. Chapter 4 outlines the performance of different ductility classes under seismic stress, showing minor differences in interstorey drift ratios (IDR) between similar ductility classes, indicating consistent displacement patterns. Chapter 6, conversely, focuses on calculating plastic hinge lengths and their conservative estimations in design, which directly relate to ductility and energy dissipation capacities. Both chapters highlight the critical role of ductility in managing seismic forces, with Chapter 6 providing a more detailed analysis of the mechanical behaviour contributing to the conservative design approach. Both chapters discuss the effect of strut inclination on seismic performance. Chapter 4 delves into how different angles (22° and 45°) affect design bending moments and torsional deformation, suggesting that lower angles increase bending moments due to a prolonged lever arm effect, which aligns with findings in Chapter 6 where strut inclination significantly affects plastic hinge lengths and tension shifts, implying that angle adjustments can considerably influence structural integrity.

The discussions in Chapter 4 about the necessity to optimize reinforcement and the bending moments under varying seismic conditions resonate with findings in Chapter 6, where the safety margins are emphasized through the conservative estimates of plastic hinge lengths and behaviour factors. Both chapters suggest an approach that prioritizes code compliance and an enhanced understanding of how design adjustments can affect long-term structural performance and safety. Chapter 4 notes the material economy and the design trade-offs in choosing different ductility classes and structural configurations. It suggests that higher ductility might require more material, impacting economic factors. Chapter 6 does not explicitly discuss economic factors but highlights the conservative nature of design standards, which could imply higher costs due to increased safety margins. The alignment here is on the trade-off between material use and safety, a central theme in structural engineering.

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

7.2 FUTURE RESEARCH

To further advance the research initiated in this thesis on seismic design according to the second generation of Eurocode 8, it is imperative to expand the scope of the study to include a broader array of structures. This expansion will enable a more comprehensive analysis of the variables influencing seismic design and the performance of reinforced concrete buildings under seismic loads.

For future research, it is proposed that a larger and more diverse set of reinforced concrete building structures be computed. This extensive computational effort aims to gather a robust dataset that reflects a wide spectrum of design scenarios and seismic responses. By grouping these structures into distinct families based on key defining characteristics, the research can systematically address variations in structural behaviour and optimize design practices for specific conditions, as follows:

a) Ductility Classes: Different ductility levels (e.g., DCL, DCM, DC1, DC2, DC3) provide insights into how varying degrees of ductility affect the structural capacity to withstand and dissipate seismic energy. This variable will help evaluate the effectiveness of the new ductility classes introduced in the updated Eurocode 8.

b) Seismicity: Buildings must be analysed in terms of their geographic location's seismic activity. Grouping structures by several spectra

c) Structural Regularity: This variable assesses the complexity of structural configurations, from simple regular forms to more complex irregular designs. Understanding how irregularities influence seismic behaviour is vital for developing effective seismic design strategies.

d) Structural Type: The type of structure, whether a frame system, wall system, or dual system, significantly impacts how forces are distributed and managed during seismic events. Analysing different structural types will show their advantages and vulnerabilities in earthquake scenarios.

e) Building Height: The height of a building influences its dynamic characteristics, such as natural frequency and mode shapes, which in turn affect its seismic response. By studying a range of building heights, the research can develop more nuanced design guidelines that consider the interplay between height and seismic performance.

f) Number of Spans: The number of spans in a building affects its lateral stiffness and mass distribution, which are crucial factors in seismic design. Analysing buildings with varying spans will help understand how the span number impacts seismic resilience and structural behaviour under earthquake loading.

Future research should employ various finite element modelling techniques to enhance our understanding of structural behaviours under seismic loads. Specifically, for Moment-Resistant structures, it is crucial to model the nodes to verify if the structures behave analogously to those in the current study. Walls should be modelled using shell or plate elements, and different calculation models should be applied to assess the impact on design according to the second generation of Eurocode 8.

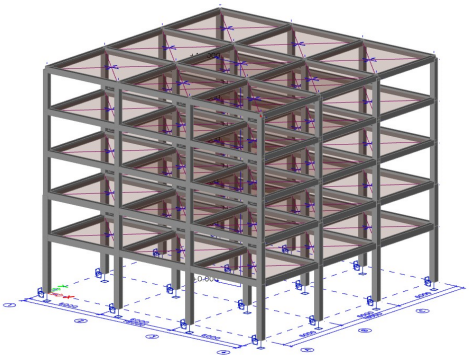
Additionally, another research avenue involves designing several structures using seismic codes from other regions, such as the United States, New Zealand, Chile, and Mexico. These designs can then be compared to those based on the second generation of Eurocode 8 to evaluate differences and potential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

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

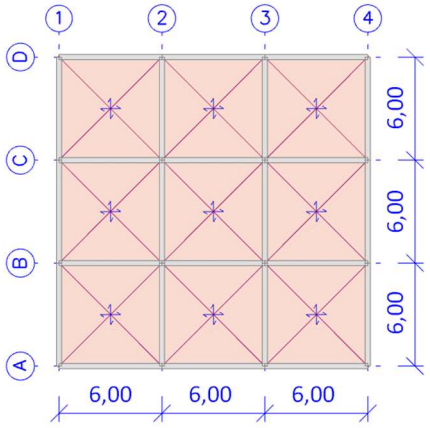
Appendix A

A.1 Moment Resisting Frames – 5-storey building

3D model



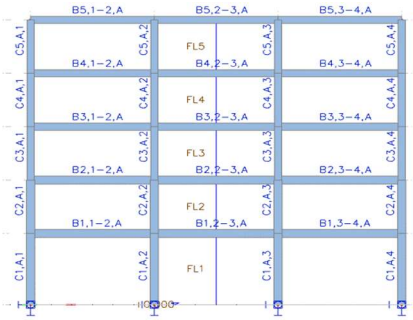
Plan

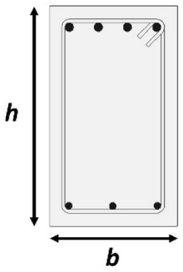


The tables below display the cross-sections of the beams and columns, where (b) represents the width of the beams and (h) represents their height.

A.1.1 Beams

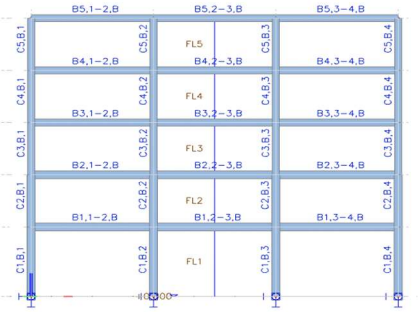
Frame A

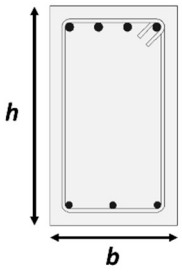




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,A	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,2-3,A	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,3-4,A	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,1-2,A	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,2-3,A	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,3-4,A	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,1-2,A	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,2-3,A	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,3-4,A	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,1-2,A	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,2-3,A	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,3-4,A	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,1-2,A	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,2-3,A	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,3-4,A	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

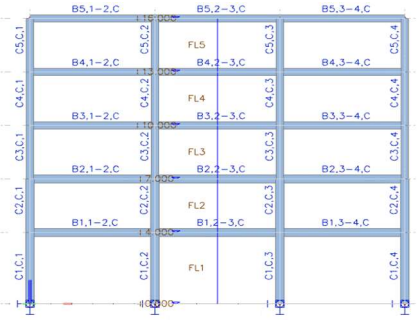
Frame B

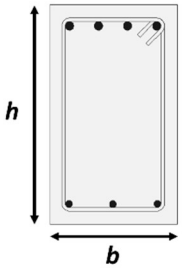




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,1-2,B	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,2-3,B	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,3-4,B	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,1-2,B	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,2-3,B	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,3-4,B	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,1-2,B	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,2-3,B	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,3-4,B	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,1-2,B	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,2-3,B	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,3-4,B	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,1-2,B	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,2-3,B	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,3-4,B	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

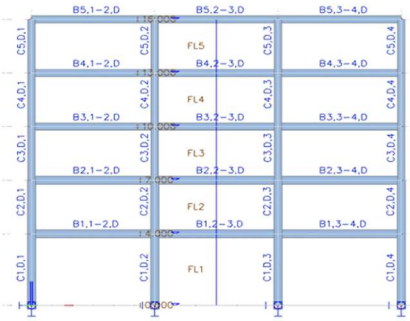
Frame C

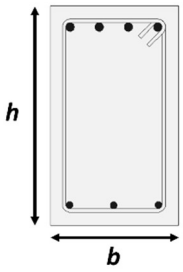




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,1-2,C	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,2-3,C	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,3-4,C	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,1-2,C	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,2-3,C	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,3-4,C	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,1-2,C	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,2-3,C	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,3-4,C	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,1-2,C	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,2-3,C	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,3-4,C	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,1-2,C	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,2-3,C	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,3-4,C	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

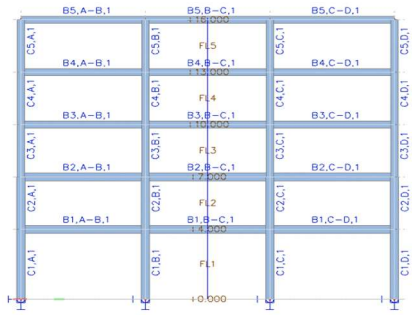
Frame D

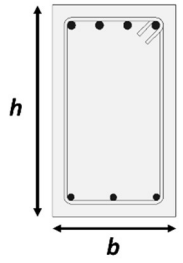




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,1-2,D	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,2-3,D	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,3-4,D	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,1-2,D	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,2-3,D	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,3-4,D	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,1-2,D	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,2-3,D	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,3-4,D	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,1-2,D	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,2-3,D	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,3-4,D	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,1-2,D	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,2-3,D	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,3-4,D	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

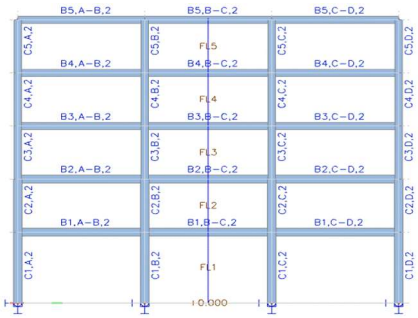
Frame 1

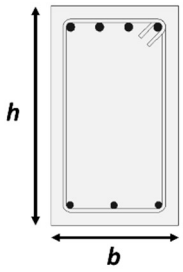




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,A-B,1	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,B-C,1	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,C-D,1	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,A-B,1	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,B-C,1	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,C-D,1	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,A-B,1	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,B-C,1	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,C-D,1	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,A-B,1	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,B-C,1	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,C-D,1	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,A-B,1	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,B-C,1	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,C-D,1	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

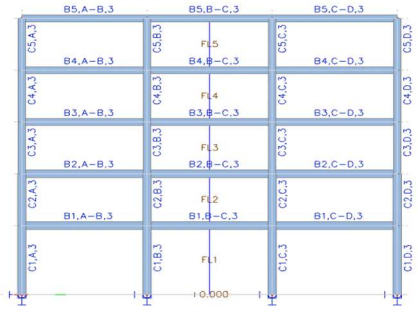
Frame 2

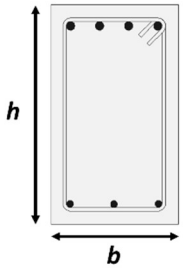




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,A-B,2	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,B-C,2	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,C-D,2	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,A-B,2	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,B-C,2	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,C-D,2	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,A-B,2	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,B-C,2	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,C-D,2	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,A-B,2	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,B-C,2	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,C-D,2	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,A-B,2	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,B-C,2	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,C-D,2	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

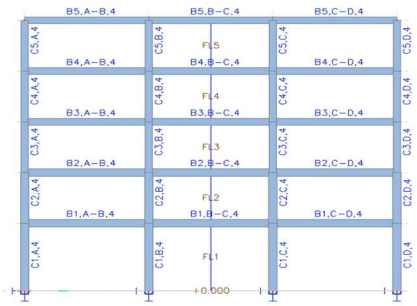
Frame 3

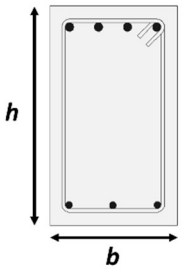




Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,A-B,3	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,B-C,3	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,C-D,3	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,A-B,3	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,B-C,3	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,C-D,3	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,A-B,3	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,B-C,3	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,C-D,3	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,A-B,3	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,B-C,3	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,C-D,3	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,A-B,3	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,B-C,3	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,C-D,3	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

Frame 4

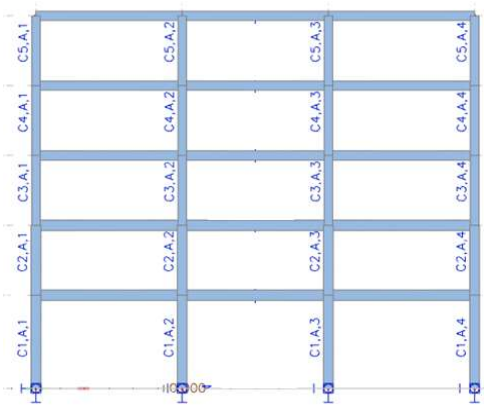


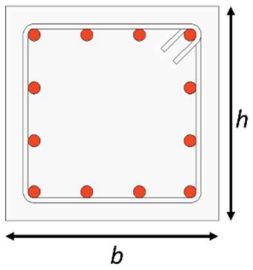


Beam	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,4	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,B-C,4	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B1,C-D,4	1	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,A-B,4	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,B-C,4	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B2,C-D,4	2	0,30×0,45	0,45×0,30	0,45×0,30	0,50×0,40	0,50×0,40
B3,A-B,4	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,B-C,4	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B3,C-D,4	3	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,A-B,4	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,B-C,4	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B4,C-D,4	4	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,A-B,4	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,B-C,4	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35
B5,C-D,4	5	0,30×0,40	0,40×0,30	0,40×0,30	0,45×0,35	0,45×0,35

A.1.2 Columns

Frames A-1/A-2/A-3/A-4

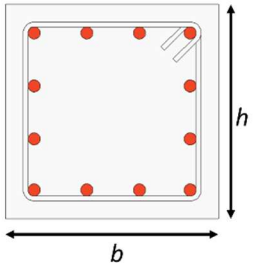




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
C1,A,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C2,A,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C3,A,1	3	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C4,A,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C5,A,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C1,A,2	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,A,2	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,A,2	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,A,2	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,A,2	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,A,3	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,A,3	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,A,3	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,A,3	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,A,3	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,A,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C2,A,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C3,A,4	3	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C4,A,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C5,A,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45

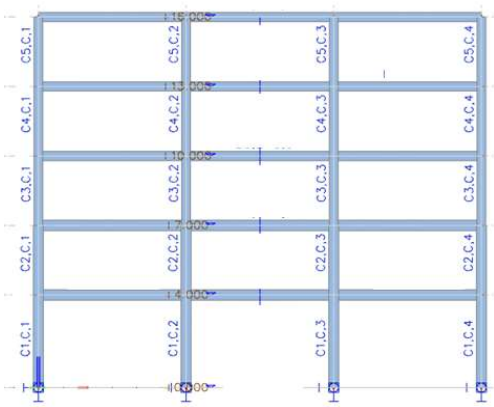
Frames B-1/B-2/B-3/B-4

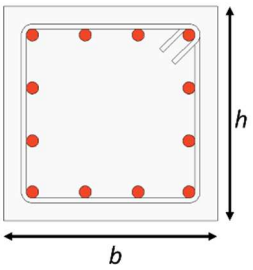




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
C1,B,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,B,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,B,1	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,B,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,B,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,B,2	1	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,B,2	2	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,B,2	3	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,B,2	4	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,B,2	5	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,B,3	1	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,B,3	2	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,B,3	3	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,B,3	4	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,B,3	5	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,B,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,B,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,B,4	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,B,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,B,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50

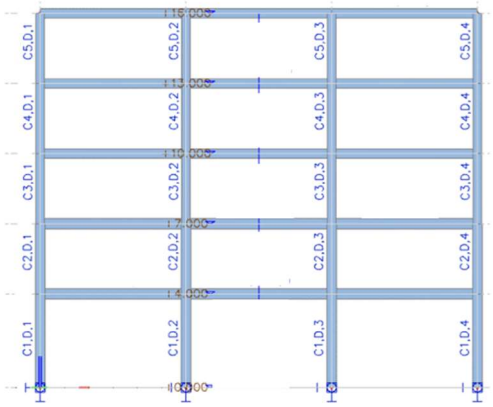
Frames C-1/C-2/C-3/C-4

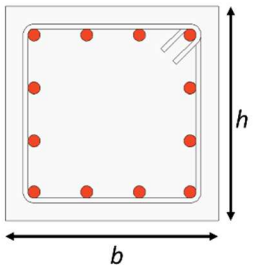




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
C1,C,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,C,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,C,1	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,C,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,C,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,C,2	1	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,C,2	2	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,C,2	3	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,C,2	4	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,C,2	5	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,C,3	1	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,C,3	2	0,45×0,45	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,C,3	3	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,C,3	4	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,C,3	5	0,40×0,40	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,C,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,C,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,C,4	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,C,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,C,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50

Frames D-1/D-2/D-3/D-4

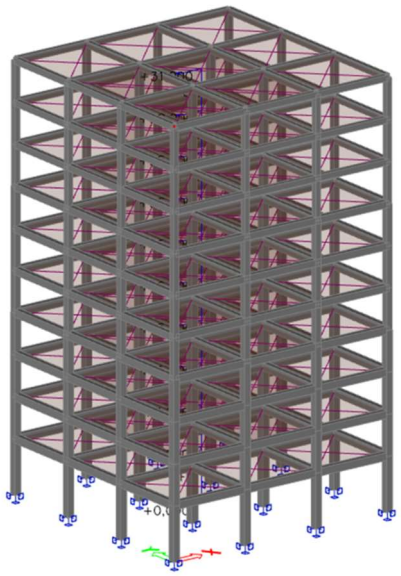




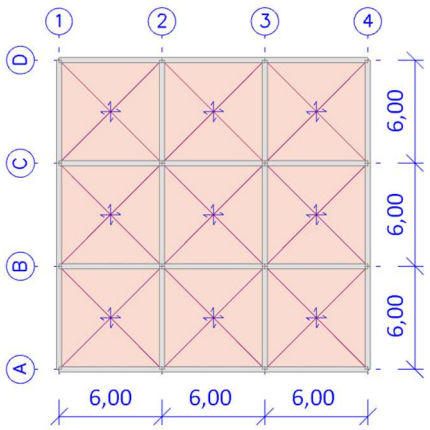
Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,D,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C2,D,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C3,D,1	3	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C4,D,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C5,D,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C1,D,2	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,D,2	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,D,2	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,D,2	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,D,2	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,D,3	1	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C2,D,3	2	0,40×0,40	0,40×0,40	0,40×0,40	0,55×0,55	0,55×0,55
C3,D,3	3	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C4,D,3	4	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C5,D,3	5	0,35×0,35	0,35×0,35	0,35×0,35	0,50×0,50	0,50×0,50
C1,D,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C2,D,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,50×0,50	0,50×0,50
C3,D,4	3	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C4,D,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45
C5,D,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,45×0,45	0,45×0,45

A.2 Moment Resisting Frames – 10-storey building

3D model



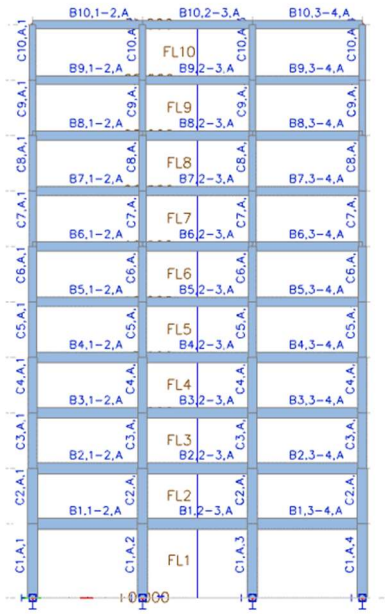
Plan

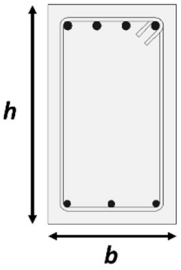


The tables below display the cross-sections of the beams, where (b) represents the width and (h) represents the height. The columns have square cross-sections, meaning the dimensions are the same in both orthogonal directions

A.2.1 Beams

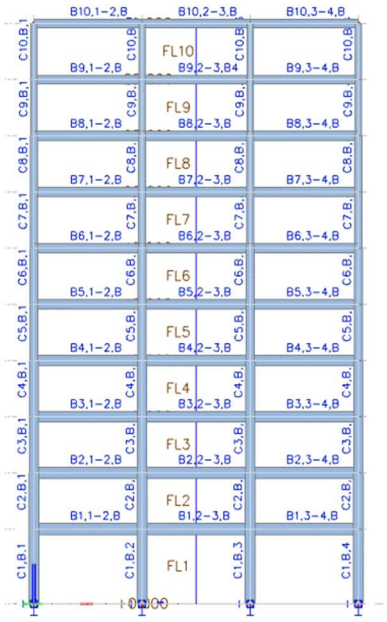
Frame A

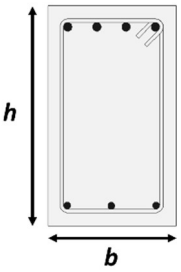




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

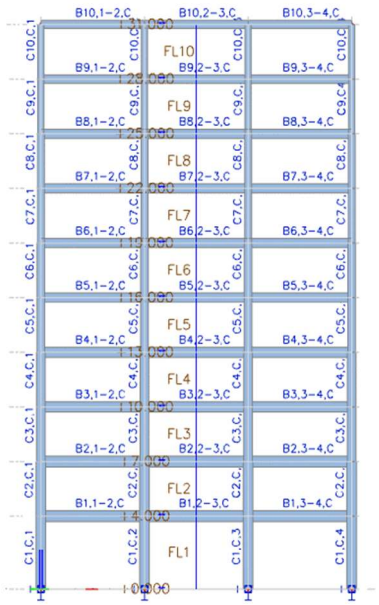
Frame B

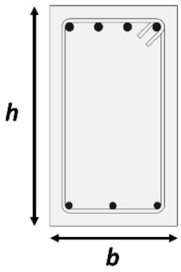




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,1-2,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

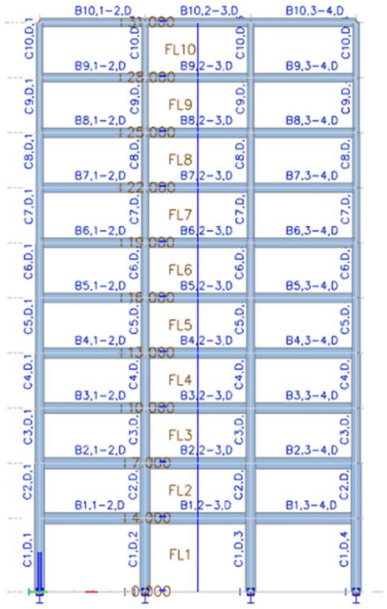
Frame C

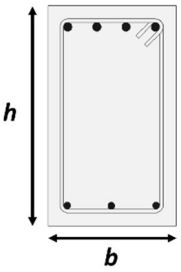




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B1,1-2,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

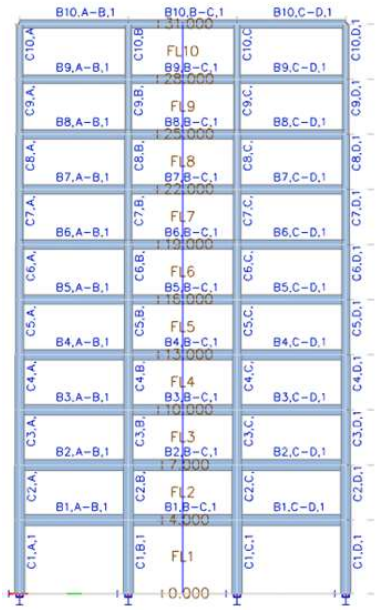
Frame D

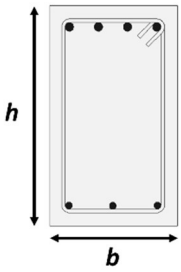




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
B1,1-2,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

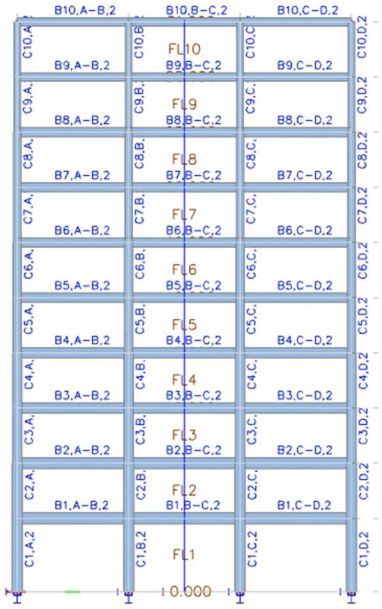
Frame 1

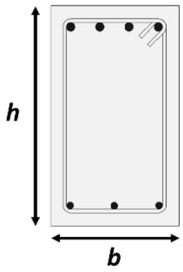




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
B1,1-2,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,A	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,A	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,A	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,A	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,A	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,A	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,A	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,A	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,A	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,A	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

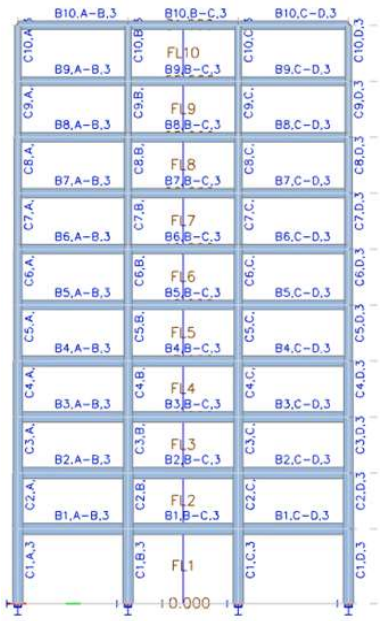
Frame 2

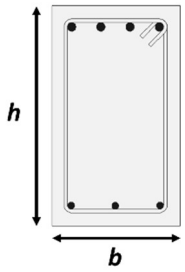




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
B1,1-2,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,B	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,B	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,B	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,B	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,B	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,B	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,B	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,B	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,B	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,B	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

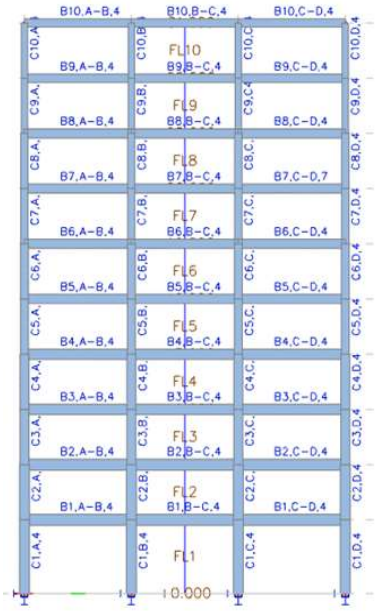
Frame 3

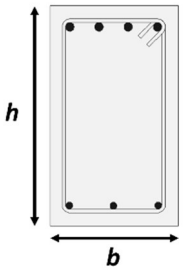




Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
B1,1-2,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,C	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,C	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,C	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,C	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,C	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,C	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,C	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,C	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,C	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,C	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

Frame 4

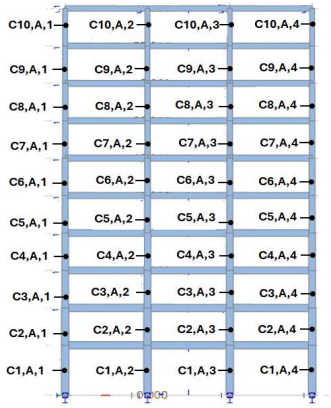


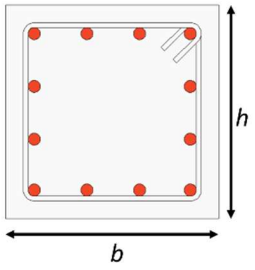


Beam	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
B1,1-2,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,2-3,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B1,3-4,D	1	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,1-2,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,2-3,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B2,3-4,D	2	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B3,1-2,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,2-3,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B3,3-4,D	3	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55
B4,1-2,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,2-3,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B4,3-4,D	4	0,40×0,50	0,40×0,50	0,40×0,50	0,40×0,55	0,40×0,55
B5,1-2,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,2-3,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,3-4,D	5	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,1-2,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,2-3,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B6,3-4,D	6	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,1-2,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,2-3,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B7,3-4,D	7	0,35×0,45	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B8,1-2,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,2-3,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B8,3-4,D	8	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,1-2,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,2-3,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B9,3-4,D	9	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,1-2,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,2-3,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45
B10,3-4,D	10	0,30×0,40	0,30×0,40	0,30×0,40	0,35×0,45	0,35×0,45

A.2.2 Columns

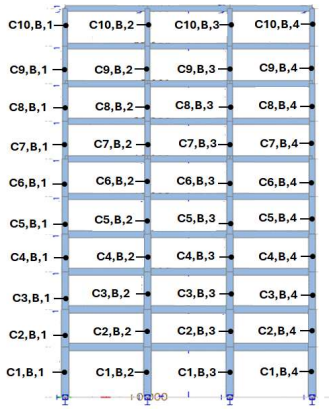
Frames A-1/A-2/A-3/A-4

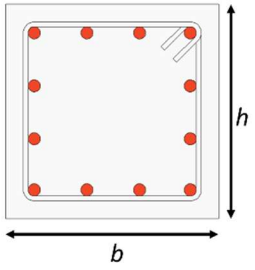




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m	*b*×*h* m	*b*×*h* m	*b*×*h* m	*b*×*h* m
C1,A,1	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,1	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,1	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,1	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,1	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,1	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,1	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,1	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,1	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,1	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,A,2	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,2	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,2	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,2	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,2	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,2	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,2	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,2	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,2	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,2	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,A,3	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,3	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,3	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,3	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,3	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,3	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,3	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,3	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,3	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,3	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,A,4	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,4	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,4	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,4	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,4	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,4	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,4	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,4	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,4	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,4	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40

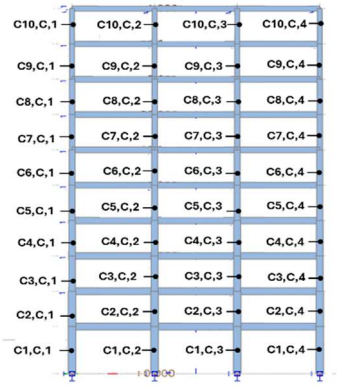
Frames B-1/B-2/B-3/B-4

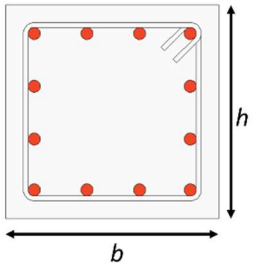




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
C1,B,1	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,B,1	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,B,1	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,B,1	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,B,1	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,B,1	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,B,1	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,B,1	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,B,1	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,B,1	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,B,2	1	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C2,B,2	2	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C3,B,2	3	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C4,B,2	4	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C5,B,2	5	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C6,B,2	6	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C7,B,2	7	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C8,B,2	8	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C9,B,2	9	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C10,B,2	10	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C1,B,3	1	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C2,B,3	2	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C3,B,3	3	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C4,B,3	4	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C5,B,3	5	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C6,B,3	6	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C7,B,3	7	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C8,B,3	8	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C9,B,3	9	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C10,B,3	10	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C1,B,4	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,B,4	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,B,4	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,B,4	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,B,4	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,B,4	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,B,4	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,B,4	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,B,4	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,B,4	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40

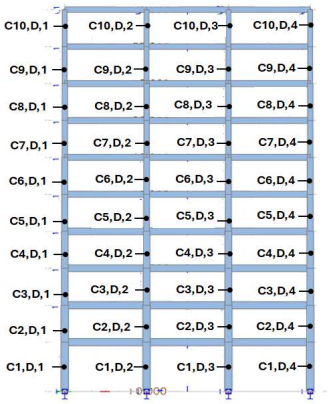
Frames C-1/C-2/C-3/C-4

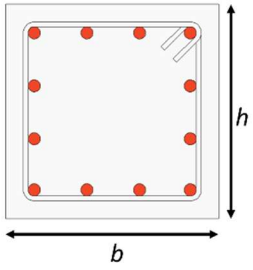




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
C1,C,1	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,C,1	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,C,1	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,C,1	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,C,1	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,C,1	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,C,1	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,C,1	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,C,1	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,C,1	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,C,2	1	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C2,C,2	2	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C3,C,2	3	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C4,C,2	4	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C5,C,2	5	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C6,C,2	6	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C7,C,2	7	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C8,C,2	8	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C9,C,2	9	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C10,C,2	10	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C1,C,3	1	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C2,C,3	2	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C3,C,3	3	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C4,C,3	4	0,55×0,55	0,50×0,50	0,50×0,50	0,55×0,55	0,55×0,55
C5,C,3	5	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C6,C,3	6	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C7,C,3	7	0,50×0,50	0,45×0,45	0,45×0,45	0,50×0,50	0,50×0,50
C8,C,3	8	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C9,C,3	9	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C10,C,3	10	0,45×0,45	0,40×0,40	0,40×0,40	0,45×0,45	0,45×0,45
C1,C,4	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,C,4	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,C,4	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,C,4	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,C,4	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,C,4	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,C,4	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,C,4	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,C,4	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,C,4	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40

Frames D-1/D-2/D-3/D-4

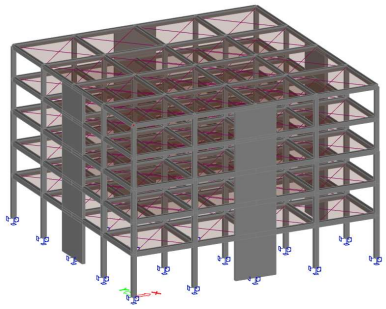




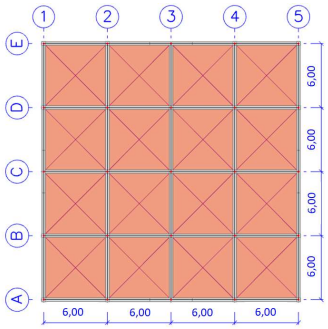
Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m	*b×h* m	*b×h* m	*b×h* m	*b×h* m
C1,D,1	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,1	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,1	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,1	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,1	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,1	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,1	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,1	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,1	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,1	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,D,2	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,2	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,2	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,2	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,2	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,2	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,2	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,2	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,2	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,2	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,D,3	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,3	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,3	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,3	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,3	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,3	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,3	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,3	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,3	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,3	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C1,D,4	1	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,4	2	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,4	3	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,4	4	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,4	5	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,4	6	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,4	7	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,4	8	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,4	9	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,4	10	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40

A.3 Wall structures – 5 storey buildings

3D model



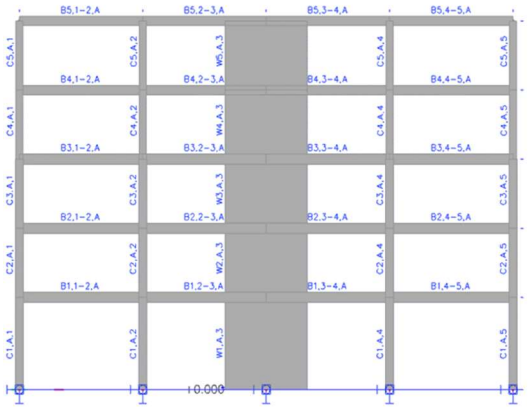
Plan

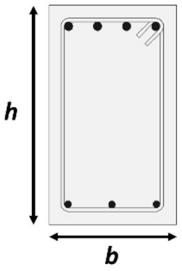


The tables below display the cross-sections of the beams and columns, where (b) represents the width of the beams and (h) represents their height.

A.3.1 Beams

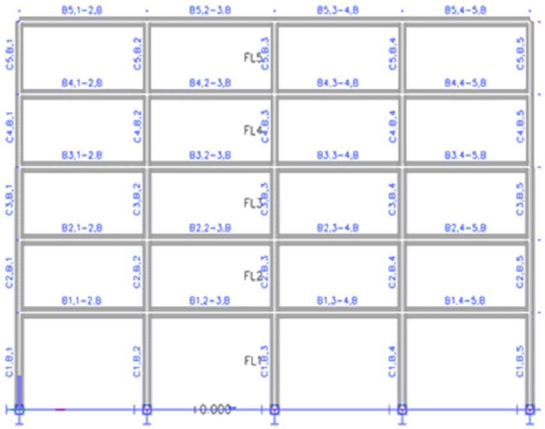
Frame A

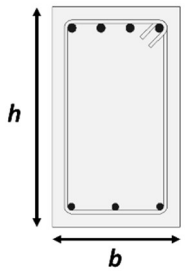




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,A	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,2-3,A	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,3-4,A	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,4-5,A	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,1-2,A	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,2-3,A	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,3-4,A	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,4-5,A	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,1-2,A	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,2-3,A	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,3-4,A	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,4-5,A	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,1-2,A	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,2-3,A	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,3-4,A	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,4-5,A	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,1-2,A	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,2-3,A	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,3-4,A	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,4-5,A	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

Frame B

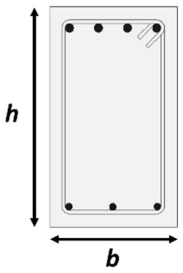




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,B	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,2-3,B	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,3-4,B	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,4-5,B	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,1-2,B	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,2-3,B	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,3-4,B	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,4-5,B	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,1-2,B	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,2-3,B	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,3-4,B	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,4-5,B	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,1-2,B	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,2-3,B	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,3-4,B	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,4-5,B	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,1-2,B	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,2-3,B	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,3-4,B	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,4-5,B	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

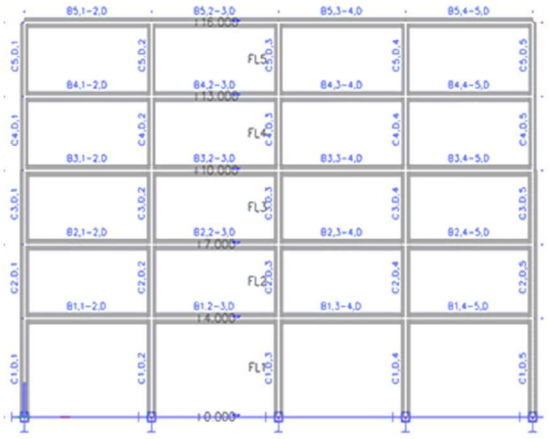
Frame C

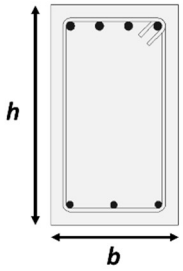




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,C	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,2-3,C	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,3-4,C	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,4-5,C	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,1-2,C	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,2-3,C	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,3-4,C	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,4-5,C	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,1-2,C	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,2-3,C	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,3-4,C	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,4-5,C	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,1-2,C	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,2-3,C	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,3-4,C	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,4-5,C	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,1-2,C	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,2-3,C	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,3-4,C	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,4-5,C	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

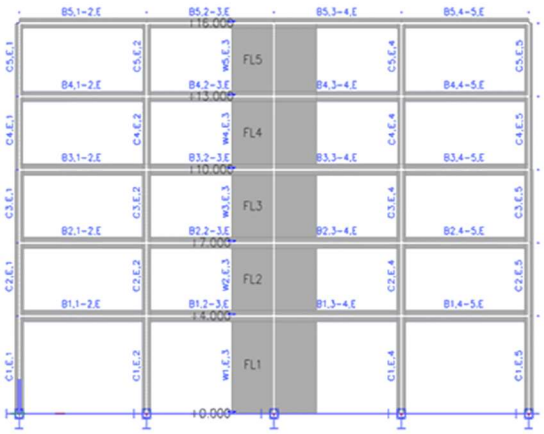
Frame D

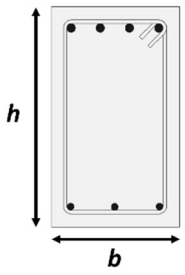




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,D	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,2-3,D	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,3-4,D	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,4-5,D	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,1-2,D	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,2-3,D	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,3-4,D	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,4-5,D	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,1-2,D	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,2-3,D	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,3-4,D	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,4-5,D	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,1-2,D	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,2-3,D	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,3-4,D	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,4-5,D	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,1-2,D	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,2-3,D	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,3-4,D	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,4-5,D	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

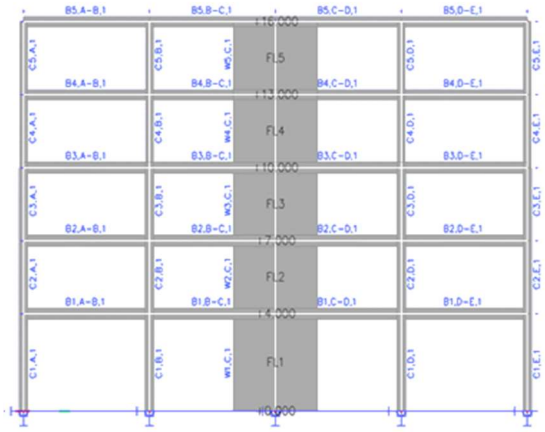
Frame E

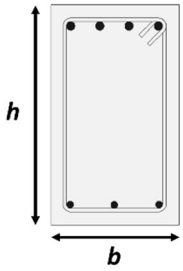




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,E	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,2-3,E	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,3-4,E	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,4-5,E	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,1-2,E	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,2-3,E	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,3-4,E	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,4-5,E	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,1-2,E	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,2-3,E	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,3-4,E	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,4-5,E	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,1-2,E	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,2-3,E	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,3-4,E	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,4-5,E	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,1-2,E	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,2-3,E	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,3-4,E	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,4-5,E	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

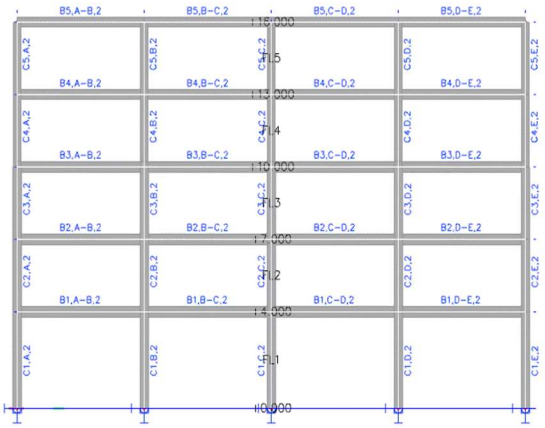
Frame 1

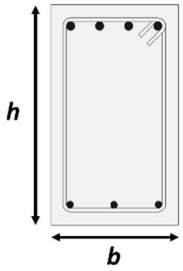




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,1	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,B-C,1	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,C-D,1	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,D-E,1	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,A-B,1	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,B-C,1	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,C-D,1	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,D-E,1	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,A-B,1	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,B-C,1	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,C-D,1	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,D-E,1	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,A-B,1	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,B-C,1	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,C-D,1	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,D-E,1	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,A-B,1	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,B-C,1	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,C-D,1	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,D-E,1	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

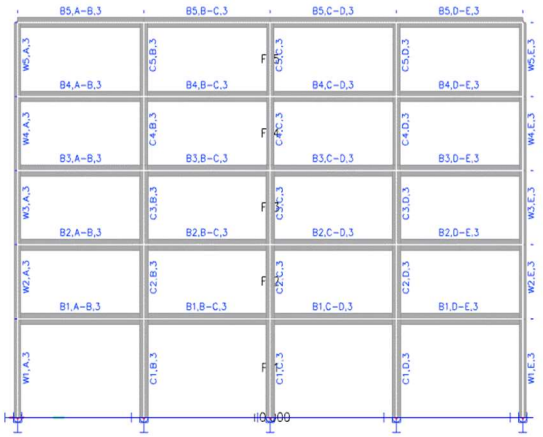
Frame 2

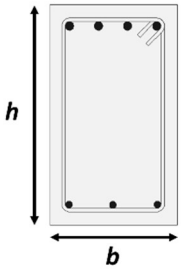




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,2	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,B-C,2	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,C-D,2	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,D-E,2	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,A-B,2	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,B-C,2	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,C-D,2	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,D-E,2	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,A-B,2	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,B-C,2	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,C-D,2	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,D-E,2	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,A-B,2	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,B-C,2	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,C-D,2	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,D-E,2	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,A-B,2	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,B-C,2	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,C-D,2	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,D-E,2	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

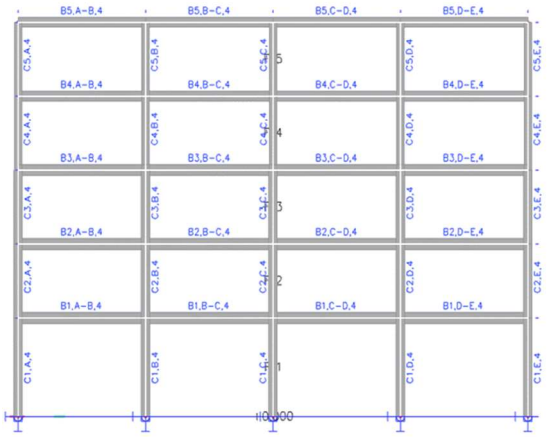
Frame 3

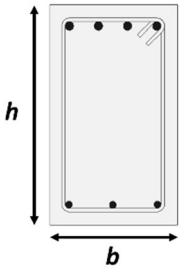




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,3	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,B-C,3	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,C-D,3	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,D-E,3	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,A-B,3	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,B-C,3	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,C-D,3	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,D-E,3	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,A-B,3	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,B-C,3	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,C-D,3	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,D-E,3	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,A-B,3	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,B-C,3	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,C-D,3	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,D-E,3	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,A-B,3	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,B-C,3	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,C-D,3	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,D-E,3	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

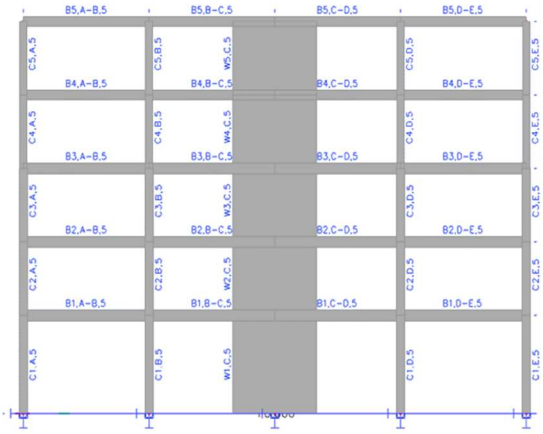
Frame 4

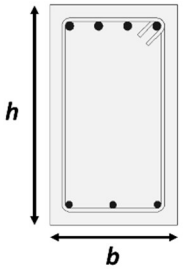




Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,B-C,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,C-D,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,D-E,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,A-B,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,B-C,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,C-D,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,D-E,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,A-B,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,B-C,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,C-D,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,D-E,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,A-B,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,B-C,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,C-D,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,D-E,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,A-B,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,B-C,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,C-D,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,D-E,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

Frame 5

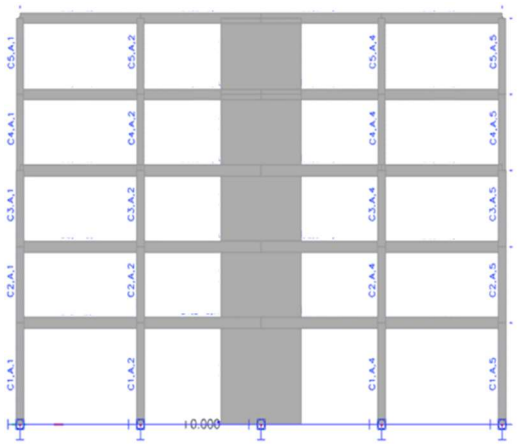


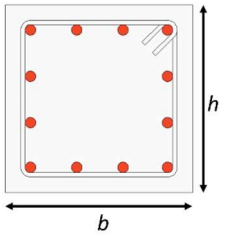


Beam	Storey	DC2.W.50.5	DCM.W.50.5	DC3.W.50.5	DCM.W.75.5	DC3.W.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,B-C,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,C-D,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B1,D-E,4	1	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,A-B,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,B-C,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,C-D,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B2,D-E,4	2	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,A-B,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,B-C,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,C-D,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B3,D-E,4	3	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45	0,35×0,45
B4,A-B,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,B-C,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,C-D,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B4,D-E,4	4	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,A-B,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,B-C,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,C-D,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40
B5,D-E,4	5	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40	0,30×0,40

A.3.2 Columns

Frames A-1/A-2/A-4/A-5

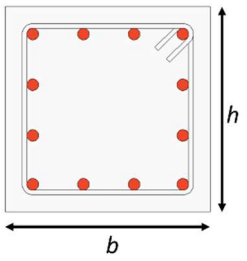




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,A,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,A,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,A,1	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,A,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,A,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,A,2	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,A,2	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,A,2	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,A,2	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,A,2	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,A,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,A,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,A,4	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,A,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,A,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,A,5	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,A,5	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,A,5	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,A,5	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,A,5	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35

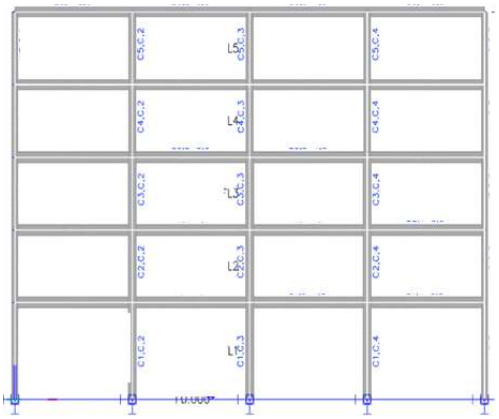
Frames B-1/B-2/B-3/B-4/B-5

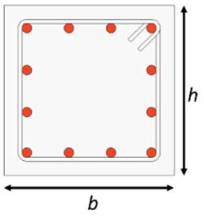




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,B,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,B,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,B,1	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,B,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,B,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,B,2	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,B,2	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,B,2	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,B,2	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,B,2	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,B,3	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,B,3	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,B,3	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,B,3	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,B,3	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,B,4	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,B,4	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,B,4	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,B,4	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,B,4	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,B,5	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,B,5	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,B,5	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,B,5	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,B,5	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35

Frames C-2/C-3/C-4

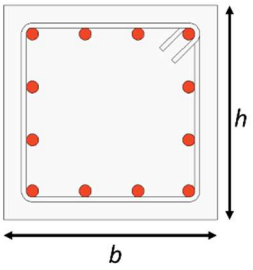




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,C,2	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,C,2	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,C,2	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,C,2	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,C,2	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,C,3	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,C,3	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,C,3	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,C,3	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,C,3	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,C,4	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,C,4	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,C,4	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,C,4	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,C,4	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35

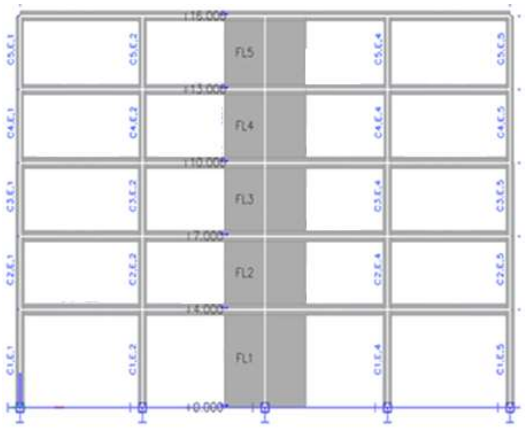
Frames D-1/D-2/D-3/D-4/D-5

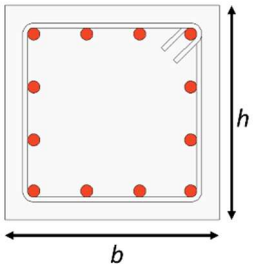




Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
C1,D,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,D,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,D,1	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,D,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,D,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,D,2	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,D,2	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,D,2	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,D,2	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,D,2	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,D,3	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,D,3	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,D,3	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,D,3	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,D,3	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,D,4	1	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,D,4	2	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,D,4	3	0,45×0,45	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,D,4	4	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,D,4	5	0,40×0,40	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,D,5	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,D,5	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,D,5	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,D,5	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,D,5	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35

Frames E-1/E-2/E-4/E-5



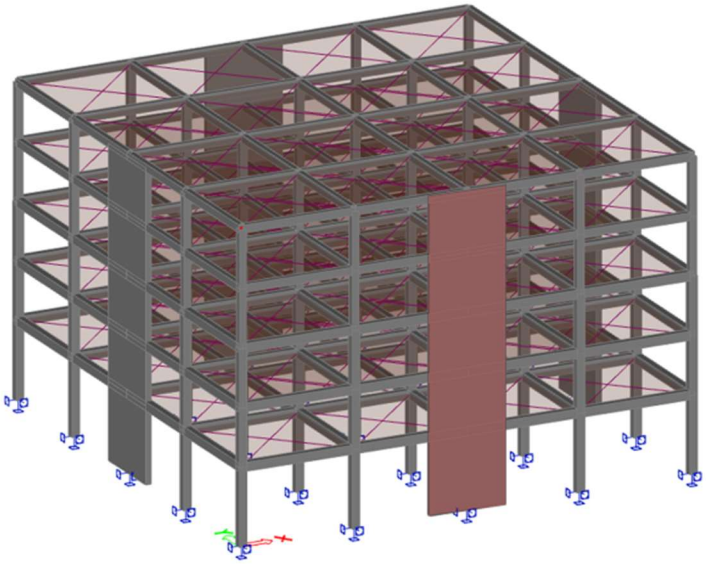


Column	Storey	DC2.50.5	DCM.50.5	DC3.50.5	DCM.75.5	DC3.75.5
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
C1,E,1	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,E,1	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,E,1	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,E,1	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,E,1	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,E,2	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,E,2	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,E,2	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,E,2	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,E,2	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,E,4	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,E,4	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,E,4	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,E,4	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,E,4	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C1,E,5	1	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C2,E,5	2	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C3,E,5	3	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40	0,40×0,40
C4,E,5	4	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35
C5,E,5	5	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35	0,35×0,35

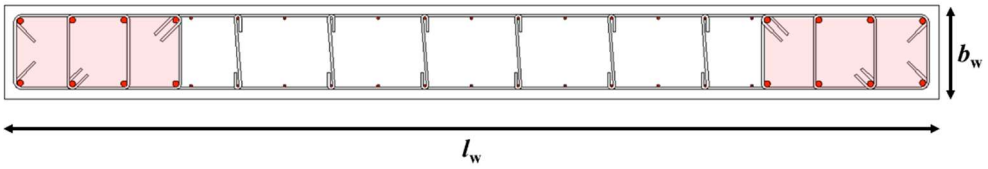
A.3.3 Walls

All the walls have the same cross section along the total height, and are represented in the following table.

(a)



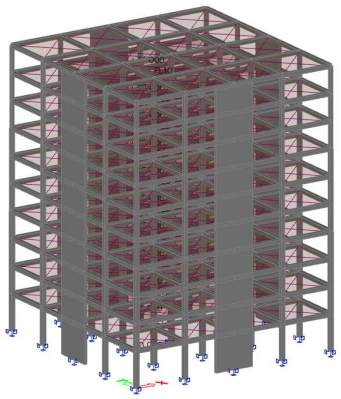
(b)



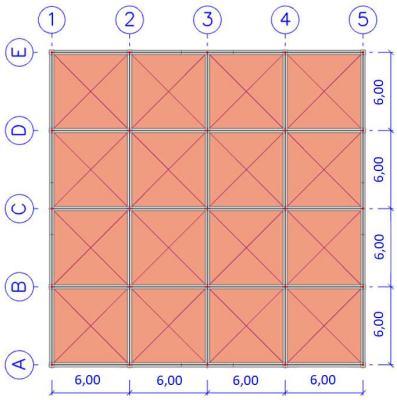
Structure	Wall width b_w (m)	Wall length l_w (m)
DCM.W.50.5	0,30	4,00
DC2.W.50.5	0,30	4,00
DC3.W.50.5	0,30	4,00
DCM.W.75.5	0,30	4,00
DC3.W.75.5	0,30	4,00

A.4 Wall structures – 10 storey buildings

3D model



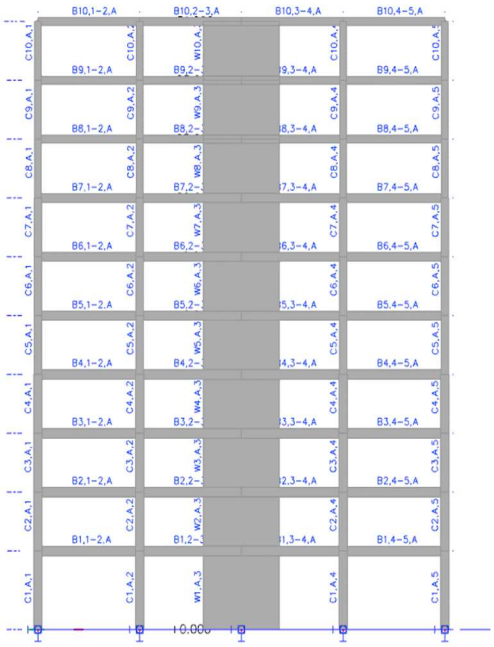
Plan

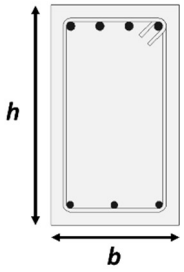


The tables below display the cross-sections of the beams and columns, where (b) represents the width of the beams and (h) represents their height.

A.4.1 Beams

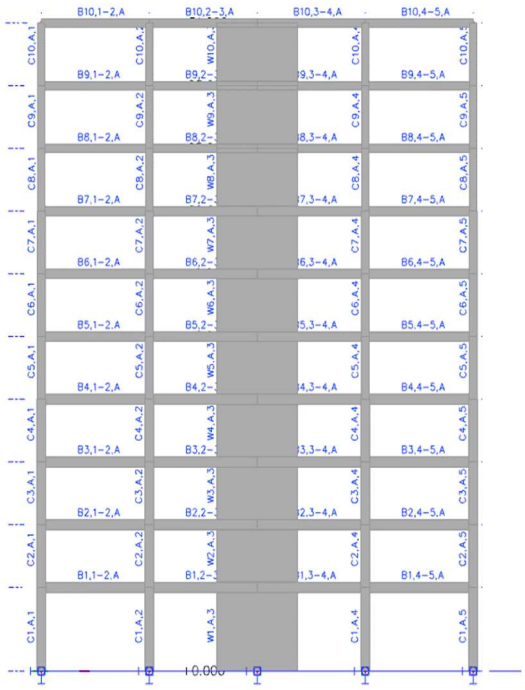
Frame A

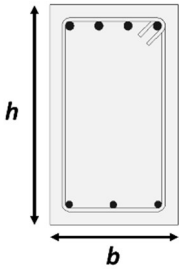




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,A	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,2-3,A	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,3-4,A	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,4-5,A	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,1-2,A	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,2-3,A	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,3-4,A	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,4-5,A	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,1-2,A	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,2-3,A	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,3-4,A	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,4-5,A	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,1-2,A	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,2-3,A	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,3-4,A	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,4-5,A	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,1-2,A	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,2-3,A	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,3-4,A	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,4-5,A	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

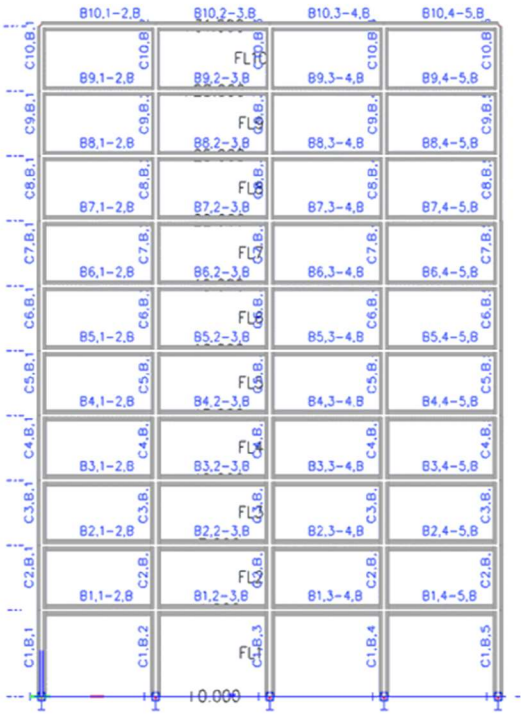
Frame A

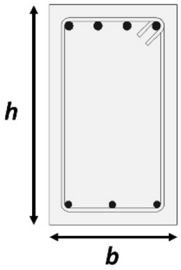




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,1-2,A	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,2-3,A	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,3-4,A	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,4-5,A	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,1-2,A	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,2-3,A	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,3-4,A	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,4-5,A	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,1-2,A	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,2-3,A	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,3-4,A	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,4-5,A	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,1-2,A	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,2-3,A	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,3-4,A	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,4-5,A	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,1-2,A	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,2-3,A	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,3-4,A	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,4-5,A	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

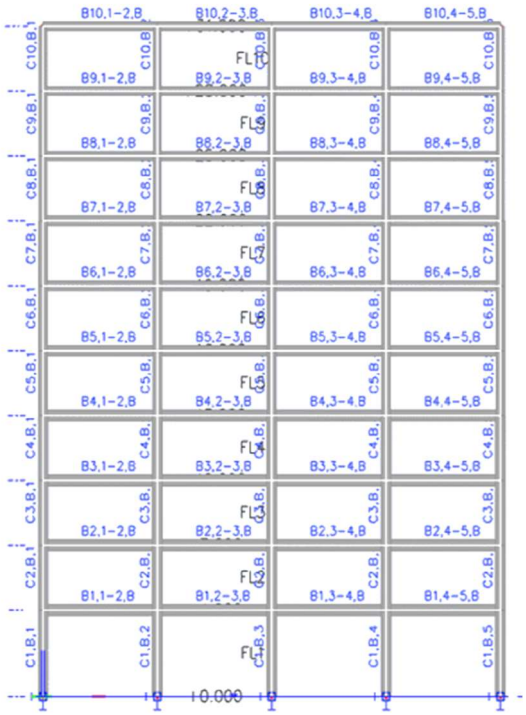
Frame B

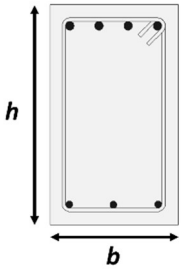




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,B	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,2-3,B	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,3-4,B	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,4-5,B	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,1-2,B	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,2-3,B	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,3-4,B	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,4-5,B	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,1-2,B	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,2-3,B	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,3-4,B	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,4-5,B	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,1-2,B	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,2-3,B	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,3-4,B	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,4-5,B	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,1-2,B	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,2-3,B	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,3-4,B	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,4-5,B	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

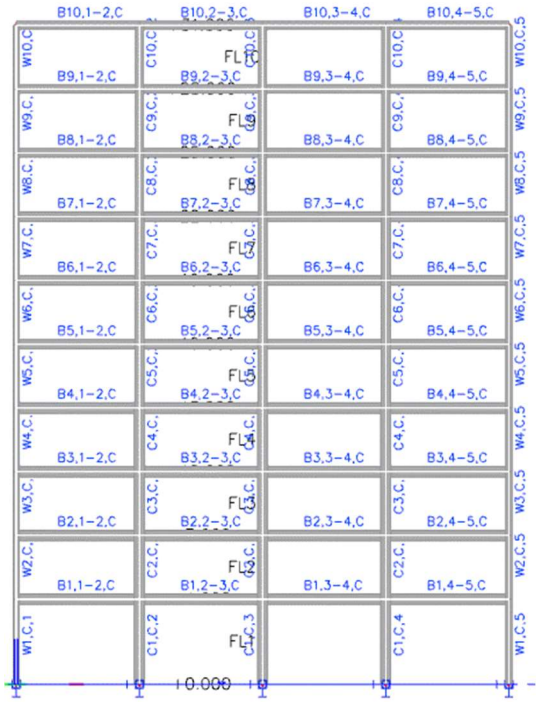
Frame B

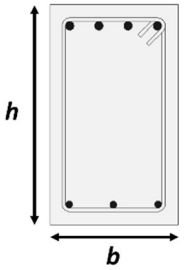




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,1-2,B	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,2-3,B	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,3-4,B	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,4-5,B	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,1-2,B	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,2-3,B	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,3-4,B	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,4-5,B	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,1-2,B	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,2-3,B	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,3-4,B	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,4-5,B	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,1-2,B	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,2-3,B	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,3-4,B	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,4-5,B	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,1-2,B	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,2-3,B	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,3-4,B	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,4-5,B	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

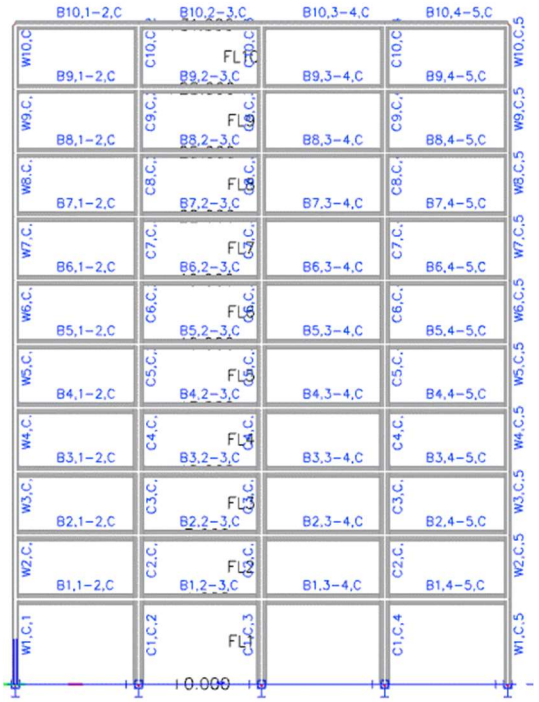
Frame C

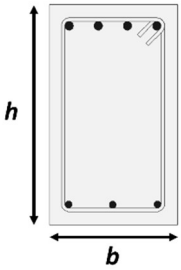




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,C	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,2-3,C	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,3-4,C	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,4-5,C	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,1-2,C	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,2-3,C	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,3-4,C	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,4-5,C	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,1-2,C	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,2-3,C	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,3-4,C	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,4-5,C	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,1-2,C	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,2-3,C	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,3-4,C	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,4-5,C	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,1-2,C	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,2-3,C	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,3-4,C	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,4-5,C	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

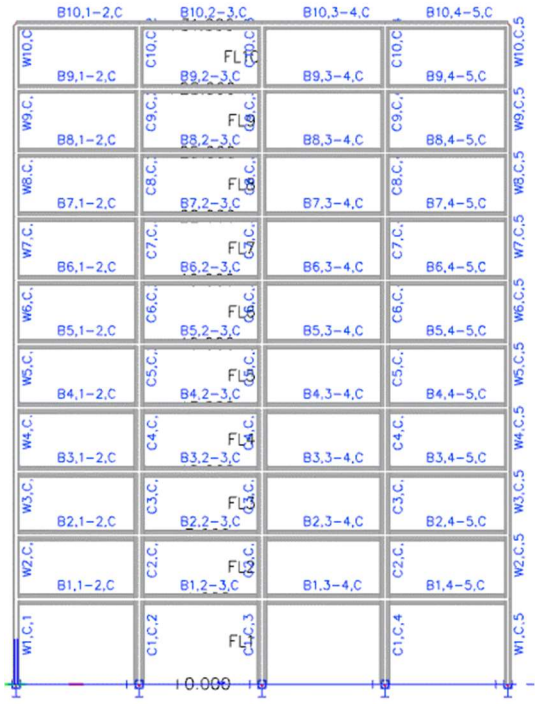
Frame C

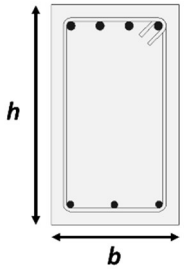




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
B6,1-2,C	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,2-3,C	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,3-4,C	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,4-5,C	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,1-2,C	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,2-3,C	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,3-4,C	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,4-5,C	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,1-2,C	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,2-3,C	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,3-4,C	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,4-5,C	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,1-2,C	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,2-3,C	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,3-4,C	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,4-5,C	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,1-2,C	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,2-3,C	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,3-4,C	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,4-5,C	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

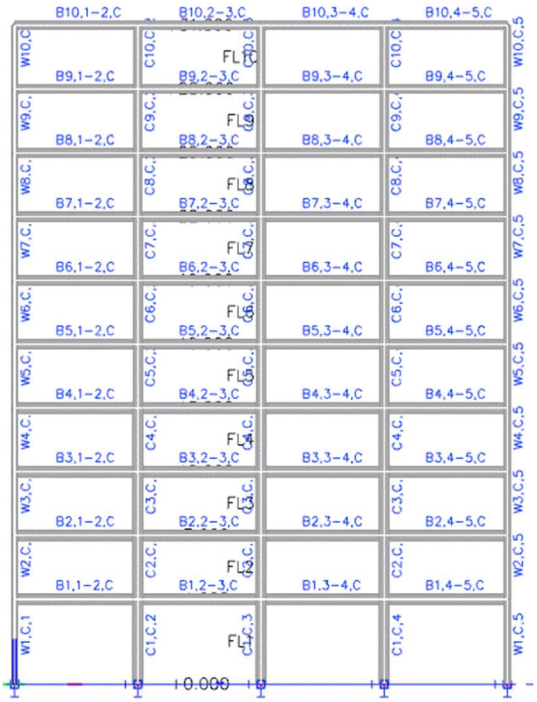
Frame D

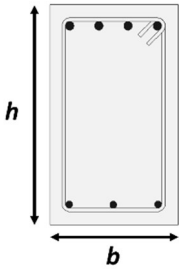




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,D	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,2-3,D	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,3-4,D	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,4-5,D	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,1-2,D	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,2-3,D	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,3-4,D	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,4-5,D	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,1-2,D	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,2-3,D	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,3-4,D	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,4-5,D	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,1-2,D	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,2-3,D	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,3-4,D	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,4-5,D	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,1-2,D	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,2-3,D	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,3-4,D	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,4-5,D	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

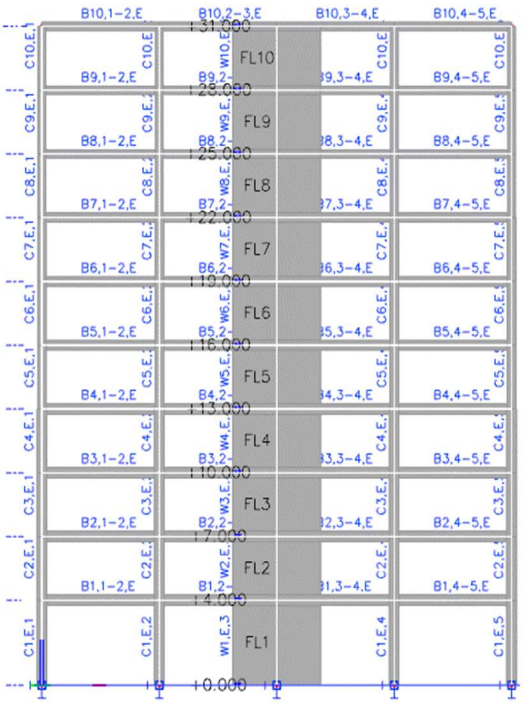
Frame D

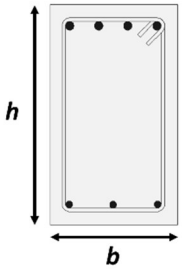




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,1-2,D	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,2-3,D	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,3-4,D	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,4-5,D	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,1-2,D	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,2-3,D	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,3-4,D	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,4-5,D	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,1-2,D	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,2-3,D	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,3-4,D	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,4-5,D	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,1-2,D	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,2-3,D	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,3-4,D	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,4-5,D	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,1-2,D	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,2-3,D	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,3-4,D	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,4-5,D	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

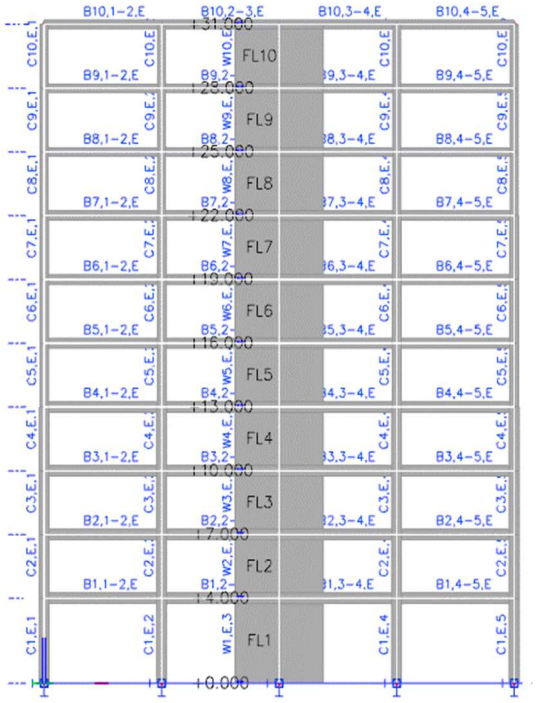
Frame E

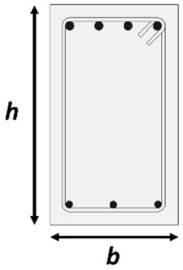




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,1-2,E	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,2-3,E	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,3-4,E	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,4-5,E	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,1-2,E	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,2-3,E	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,3-4,E	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,4-5,E	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,1-2,E	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,2-3,E	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,3-4,E	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,4-5,E	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,1-2,E	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,2-3,E	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,3-4,E	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,4-5,E	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,1-2,E	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,2-3,E	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,3-4,E	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,4-5,E	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

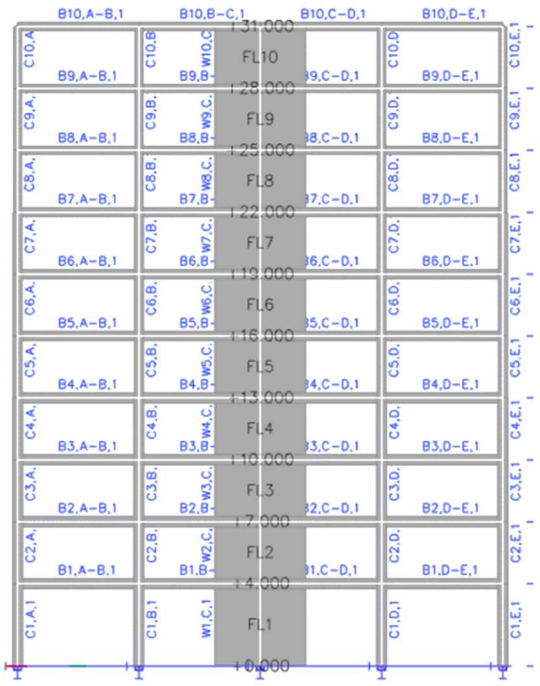
Frame E

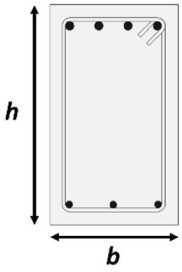




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,1-2,E	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,2-3,E	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,3-4,E	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,4-5,E	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,1-2,E	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,2-3,E	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,3-4,E	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,4-5,E	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,1-2,E	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,2-3,E	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,3-4,E	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,4-5,E	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,1-2,E	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,2-3,E	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,3-4,E	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,4-5,E	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,1-2,E	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,2-3,E	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,3-4,E	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,4-5,E	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

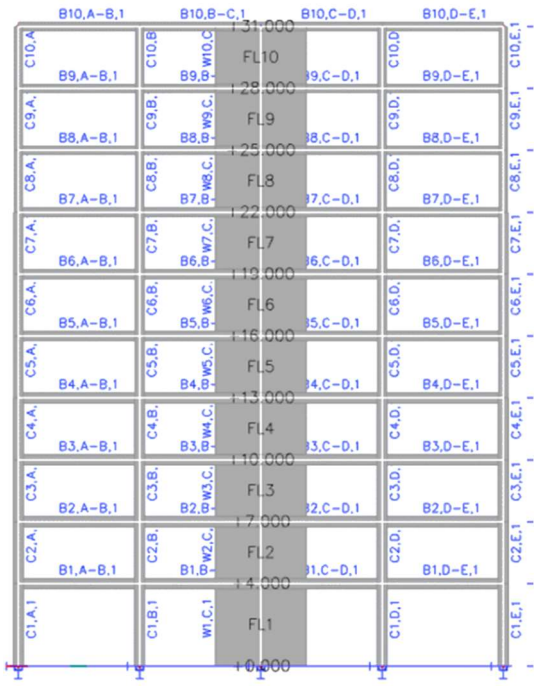
Frame 1

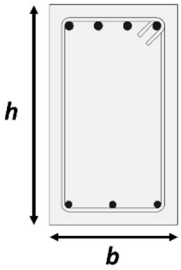




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,1	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,B-C,1	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,C-D,1	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,D-E,1	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,A-B,1	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,B-C,1	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,C-D,1	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,D-E,1	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,A-B,1	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,B-C,1	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,C-D,1	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,D-E,1	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,A-B,1	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,B-C,1	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,C-D,1	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,D-E,1	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,A-B,1	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,B-C,1	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,C-D,1	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,D-E,1	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

Frame 1

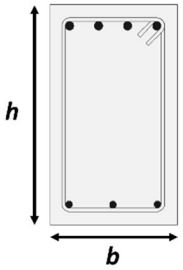




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,A-B,1	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,B-C,1	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,C-D,1	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,D-E,1	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,A-B,1	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,B-C,1	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,C-D,1	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,D-E,1	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,A-B,1	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,B-C,1	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,C-D,1	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,D-E,1	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,A-B,1	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,B-C,1	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,C-D,1	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,D-E,1	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,A-B,1	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,B-C,1	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,C-D,1	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,D-E,1	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

Frame 2

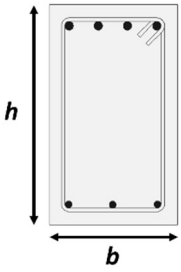




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,2	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,B-C,2	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,C-D,2	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,D-E,2	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,A-B,2	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,B-C,2	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,C-D,2	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,D-E,2	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,A-B,2	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,B-C,2	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,C-D,2	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,D-E,2	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,A-B,2	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,B-C,2	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,C-D,2	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,D-E,2	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,A-B,2	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,B-C,2	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,C-D,2	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,D-E,2	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

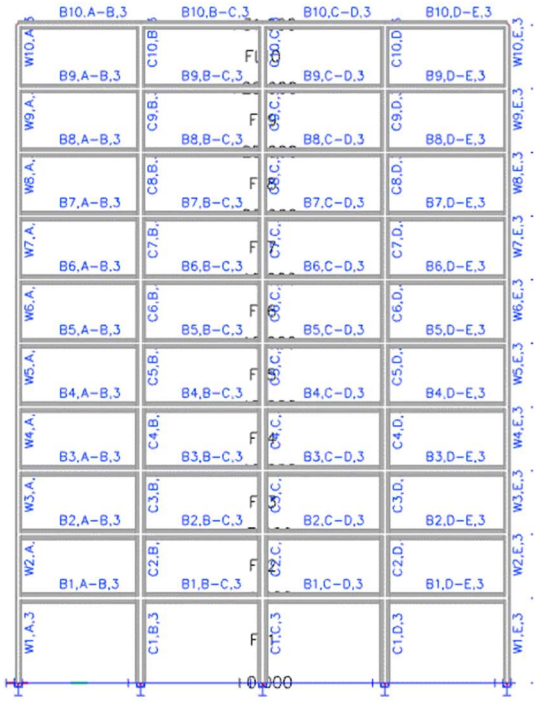
Frame 2

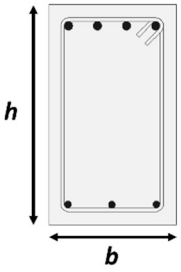




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,A-B,2	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,B-C,2	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,C-D,2	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,D-E,2	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,A-B,2	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,B-C,2	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,C-D,2	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,D-E,2	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,A-B,2	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,B-C,2	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,C-D,2	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,D-E,2	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,A-B,2	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,B-C,2	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,C-D,2	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,D-E,2	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,A-B,2	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,B-C,2	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,C-D,2	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,D-E,2	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

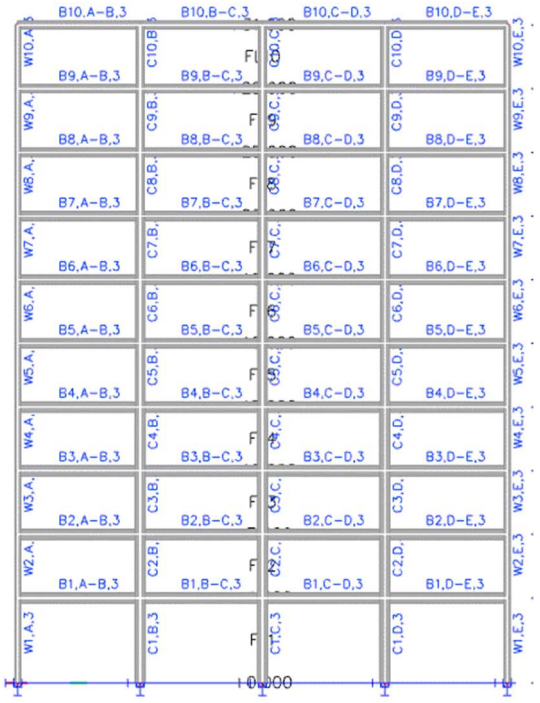
Frame 3

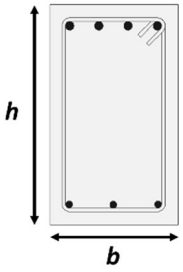




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,3	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,B-C,3	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,C-D,3	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,D-E,3	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,A-B,3	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,B-C,3	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,C-D,3	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,D-E,3	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,A-B,3	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,B-C,3	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,C-D,3	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,D-E,3	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,A-B,3	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,B-C,3	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,C-D,3	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,D-E,3	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,A-B,3	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,B-C,3	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,C-D,3	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,D-E,3	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

Frame 3

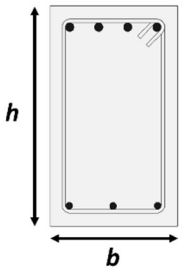




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,A-B,3	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,B-C,3	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,C-D,3	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,D-E,3	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,A-B,3	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,B-C,3	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,C-D,3	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,D-E,3	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,A-B,3	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,B-C,3	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,C-D,3	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,D-E,3	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,A-B,3	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,B-C,3	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,C-D,3	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,D-E,3	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,A-B,3	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,B-C,3	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,C-D,3	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,D-E,3	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

Frame 4

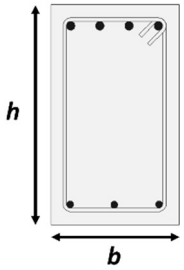




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,4	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,B-C,4	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,C-D,4	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,D-E,4	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,A-B,4	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,B-C,4	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,C-D,4	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,D-E,4	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,A-B,4	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,B-C,4	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,C-D,4	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,D-E,4	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,A-B,4	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,B-C,4	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,C-D,4	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,D-E,4	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,A-B,4	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,B-C,4	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,C-D,4	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,D-E,4	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

Frame 4

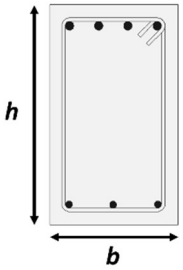




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,A-B,4	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,B-C,4	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,C-D,4	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,D-E,4	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,A-B,4	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,B-C,4	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,C-D,4	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,D-E,4	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,A-B,4	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,B-C,4	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,C-D,4	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,D-E,4	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,A-B,4	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,B-C,4	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,C-D,4	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,D-E,4	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,A-B,4	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,B-C,4	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,C-D,4	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,D-E,4	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

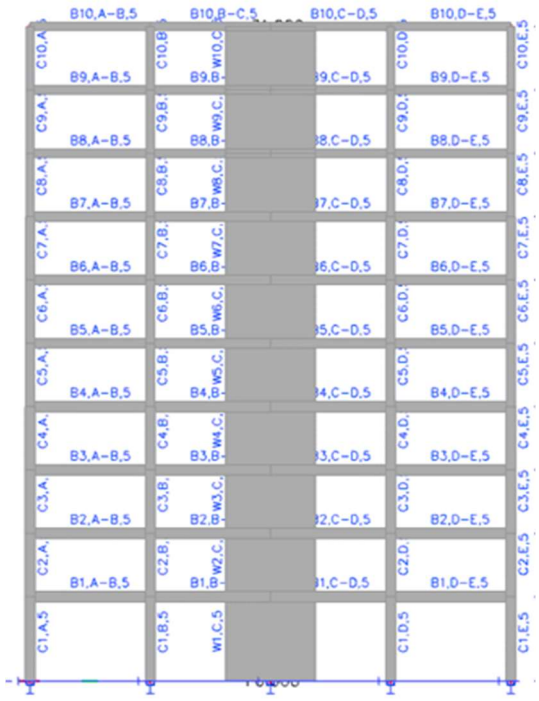
Frame 5

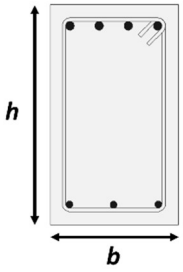




Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B1,A-B,5	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,B-C,5	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,C-D,5	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B1,D-E,5	1	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,A-B,5	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,B-C,5	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,C-D,5	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B2,D-E,5	2	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,A-B,5	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,B-C,5	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,C-D,5	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B3,D-E,5	3	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,A-B,5	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,B-C,5	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,C-D,5	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B4,D-E,5	4	0,40×0,50	0,35×0,45	0,35×0,45	0,40×0,50	0,40×0,50
B5,A-B,5	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,B-C,5	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,C-D,5	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B5,D-E,5	5	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50

Frame 5

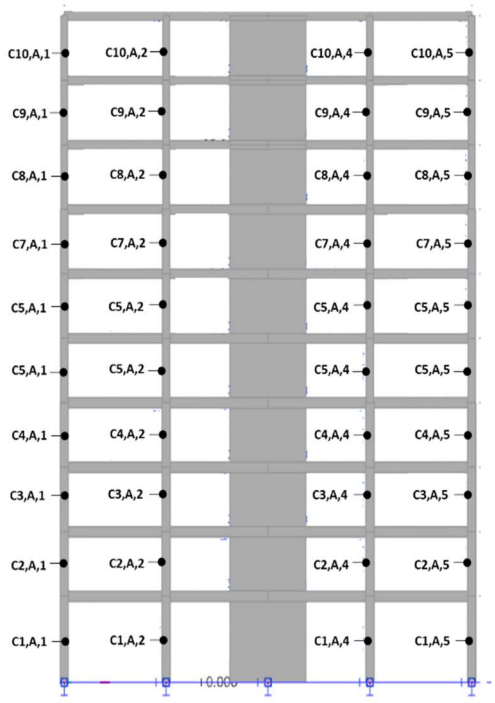


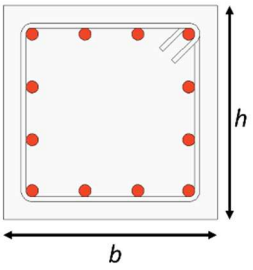


Beam	Storey	DC2.W.50.10	DCM.W.50.10	DC3.W.50.10	DCM.W.75.10	DC3.W.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
B6,A-B,5	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,B-C,5	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,C-D,5	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B6,D-E,5	6	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,A-B,5	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,B-C,5	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,C-D,5	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B7,D-E,5	7	0,40×0,50	0,30×0,45	0,30×0,45	0,40×0,50	0,40×0,50
B8,A-B,5	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,B-C,5	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,C-D,5	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B8,D-E,5	8	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,A-B,5	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,B-C,5	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,C-D,5	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B9,D-E,5	9	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,A-B,5	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,B-C,5	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,C-D,5	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45
B10,D-E,5	10	0,35×0,45	0,30×0,45	0,30×0,45	0,35×0,45	0,35×0,45

A.4.2 Columns

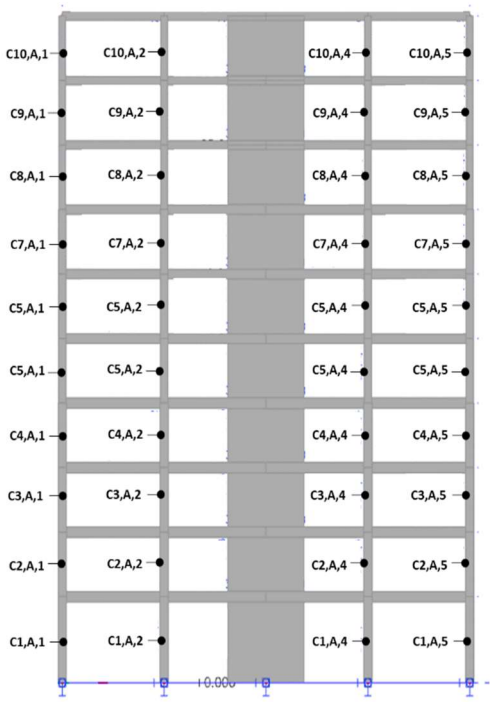
Frames A-1/A-2/A-4/A-5

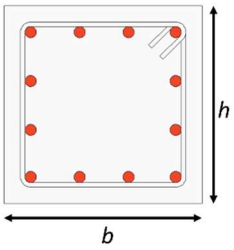




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×h m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m	*b×h* m×m
C1,A,1	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,1	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,1	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,1	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,1	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,1	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,1	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,1	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,1	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,1	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,A,2	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,2	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,2	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,2	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,2	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,2	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,2	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,2	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,2	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,2	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

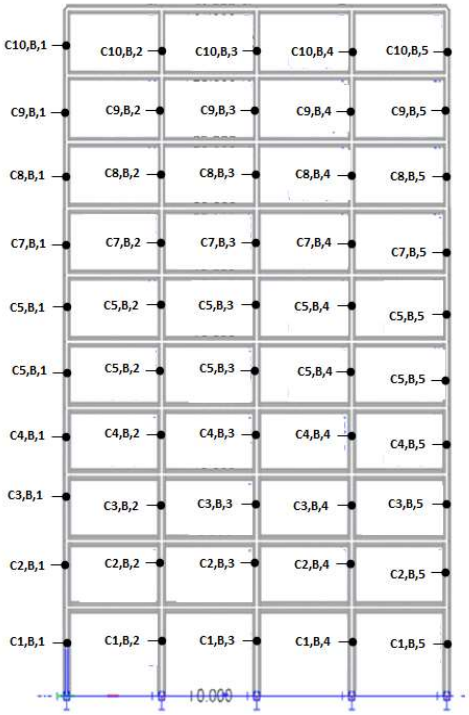
Frames A-1/A-2/A-4/A-5

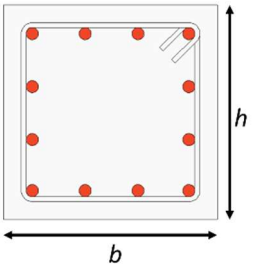




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,A,4	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,4	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,4	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,4	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,4	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,4	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,4	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,4	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,4	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,4	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,A,5	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,A,5	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,A,5	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,A,5	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,A,5	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,A,5	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,A,5	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,A,5	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,A,5	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,A,5	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

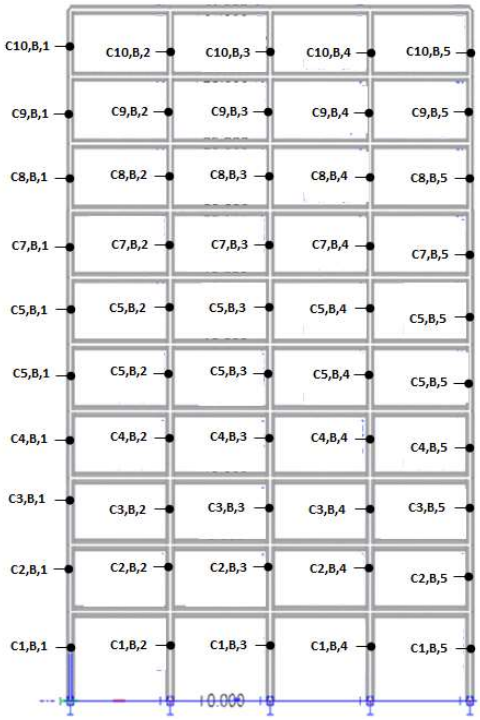
Frames B-1/B-2/B-3/B-4/B-5

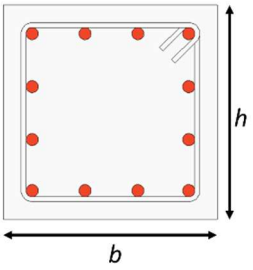




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,B,1	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,B,1	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,B,1	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,B,1	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,B,1	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,B,1	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,B,1	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,B,1	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,B,1	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,B,1	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,B,2	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,B,2	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,B,2	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,B,2	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,B,2	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,B,2	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,B,2	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,B,2	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,B,2	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,B,2	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,B,3	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,B,3	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,B,3	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,B,3	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,B,3	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,B,3	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,B,3	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,B,3	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,B,3	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,B,3	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45

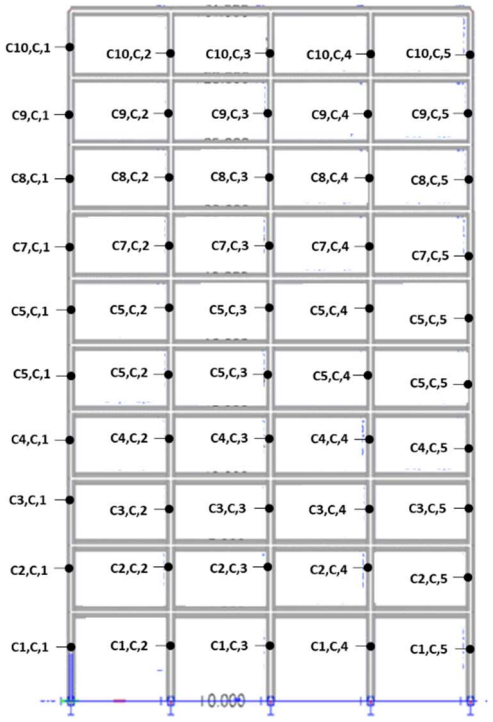
Frames B-1/B-2/B-3/B-4/B-5

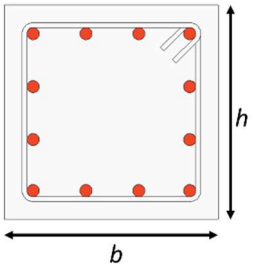




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,B,4	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,B,4	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,B,4	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,B,4	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,B,4	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,B,4	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,B,4	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,B,4	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,B,4	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,B,4	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,B,5	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,B,5	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,B,5	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,B,5	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,B,5	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,B,5	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,B,5	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,B,5	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,B,5	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,B,5	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

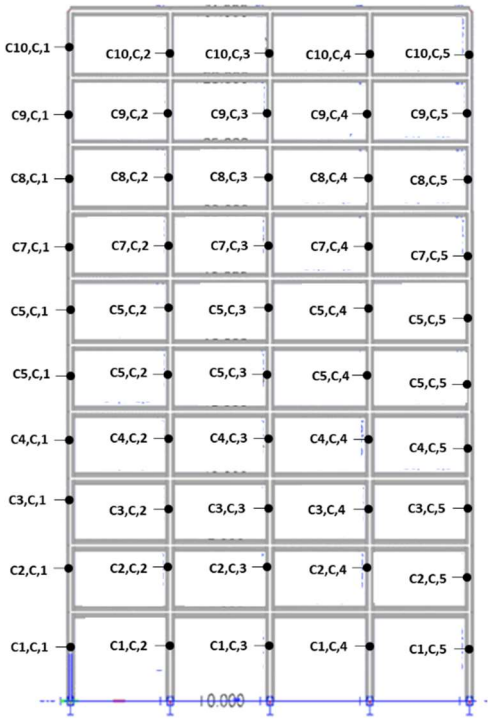
Frames C-1/C-2/C-3/C-4/C-5

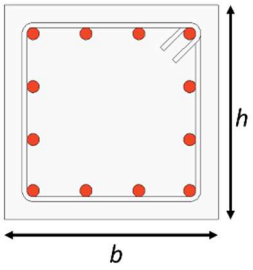




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,C,1	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,C,1	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,C,1	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,C,1	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,C,1	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,C,1	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,C,1	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,C,1	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,C,1	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,C,1	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,C,2	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,C,2	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,C,2	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,C,2	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,C,2	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,C,2	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,C,2	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,C,2	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,C,2	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,C,2	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,C,3	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,C,3	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,C,3	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,C,3	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,C,3	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,C,3	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,C,3	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,C,3	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,C,3	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,C,3	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45

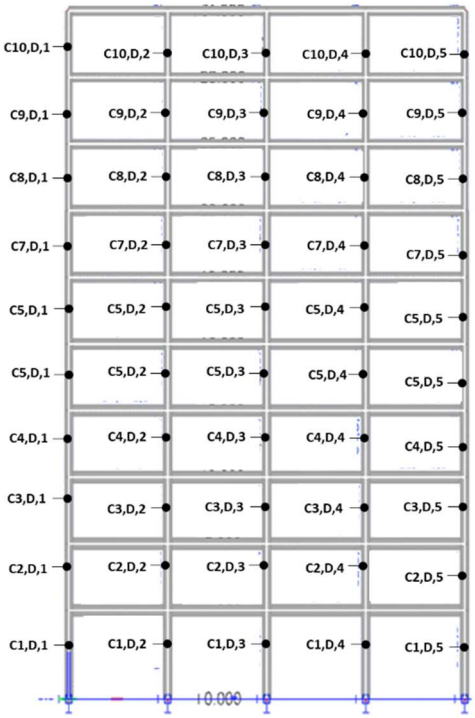
Frames C-1/C-2/C-3/C-4/C-5

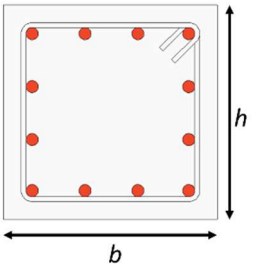




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,C,4	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,C,4	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,C,4	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,C,4	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,C,4	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,C,4	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,C,4	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,C,4	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,C,4	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,C,4	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,C,5	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,C,5	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,C,5	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,C,5	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,C,5	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,C,5	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,C,5	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,C,5	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,C,5	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,C,5	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45

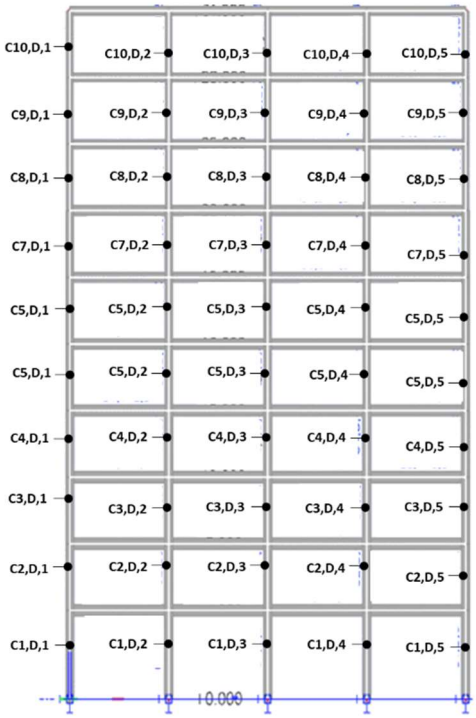
Frames D-1/D-2/D-3/D-4/D-5

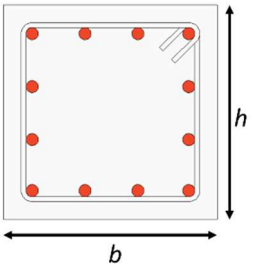




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,D,1	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,D,1	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,D,1	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,D,1	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,D,1	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,D,1	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,D,1	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,D,1	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,D,1	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,D,1	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,D,2	1	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C2,D,2	2	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C3,D,2	3	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C4,D,2	4	0,50×0,50	0,50×0,50	0,55×0,55	0,50×0,50	0,55×0,55
C5,D,2	5	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C6,D,2	6	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C7,D,2	7	0,45×0,45	0,45×0,45	0,50×0,50	0,45×0,45	0,50×0,50
C8,D,2	8	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C9,D,2	9	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C10,D,2	10	0,40×0,40	0,40×0,40	0,45×0,45	0,40×0,40	0,45×0,45
C1,D,3	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,3	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,3	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,3	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,3	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,3	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,3	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,3	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,3	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,3	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

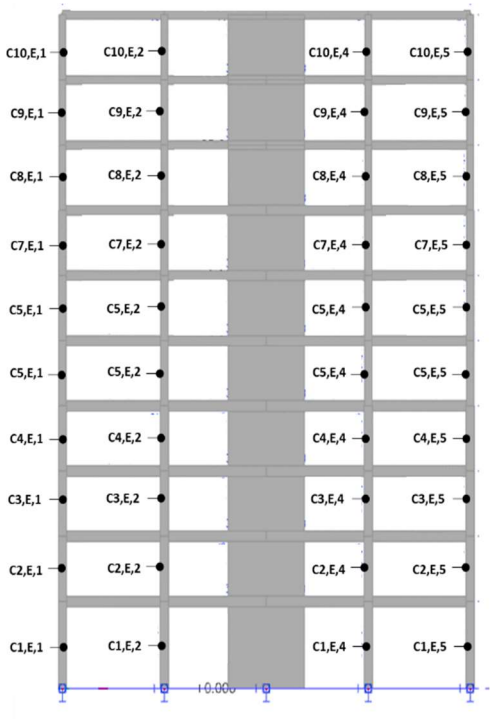
Frames D-1/D-2/D-3/D-4/D-5

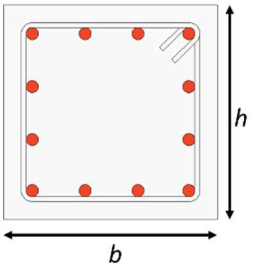




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,D,4	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,4	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,4	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,4	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,4	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,4	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,4	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,4	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,4	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,4	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,D,5	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,D,5	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,D,5	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,D,5	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,D,5	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,D,5	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,D,5	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,D,5	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,D,5	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,D,5	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

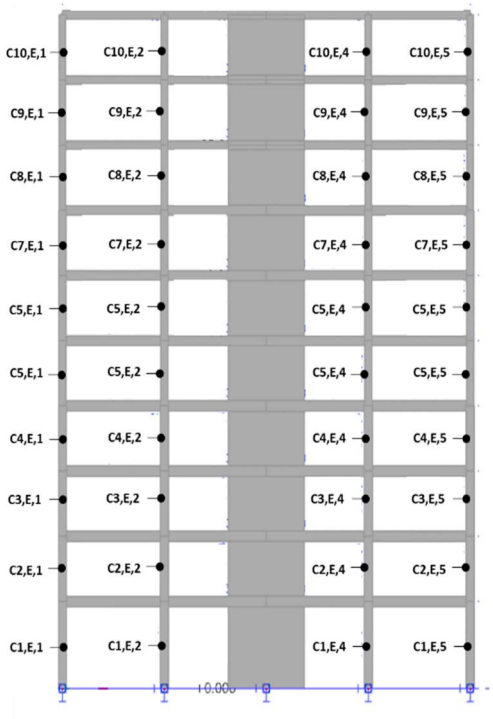
Frames E-1/E-2/E-4/E-5

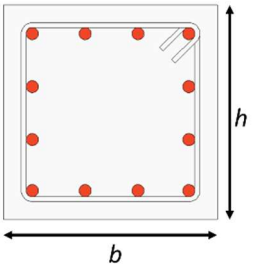




Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,E,1	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,E,1	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,E,1	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,E,1	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,E,1	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,E,1	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,E,1	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,E,1	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,E,1	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,E,1	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,E,2	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,E,2	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,E,2	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,E,2	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,E,2	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,E,2	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,E,2	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,E,2	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,E,2	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,E,2	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

Frames E-1/E-2/E-4/E-5



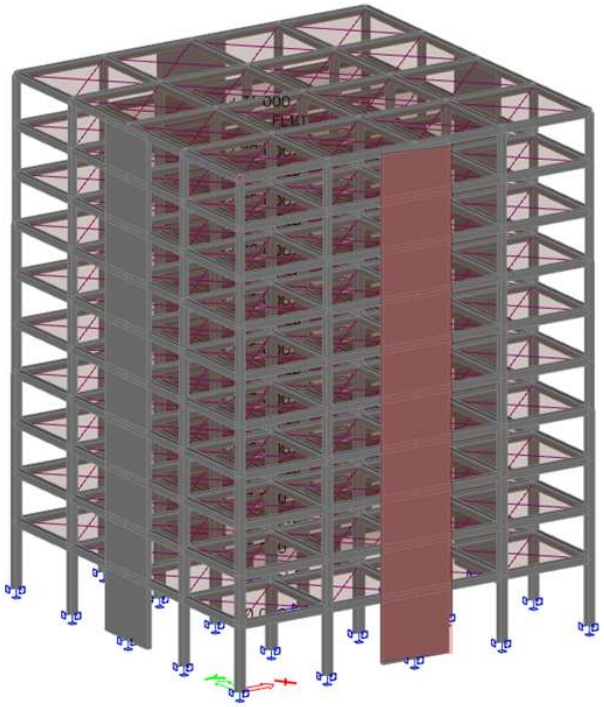


Column	Storey	DC2.50.10	DCM.50.10	DC3.50.10	DCM.75.10	DC3.75.10
		b×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m	*b*×*h* m×m
C1,E,4	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,E,4	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,E,4	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,E,4	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,E,4	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,E,4	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,E,4	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,E,4	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,E,4	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,E,4	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C1,E,5	1	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C2,E,5	2	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C3,E,5	3	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C4,E,5	4	0,50×0,50	0,45×0,45	0,50×0,50	0,50×0,50	0,50×0,50
C5,E,5	5	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C6,E,5	6	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C7,E,5	7	0,45×0,45	0,40×0,40	0,45×0,45	0,45×0,45	0,45×0,45
C8,E,5	8	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C9,E,5	9	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40
C10,E,5	10	0,40×0,40	0,35×0,35	0,40×0,40	0,40×0,40	0,40×0,40

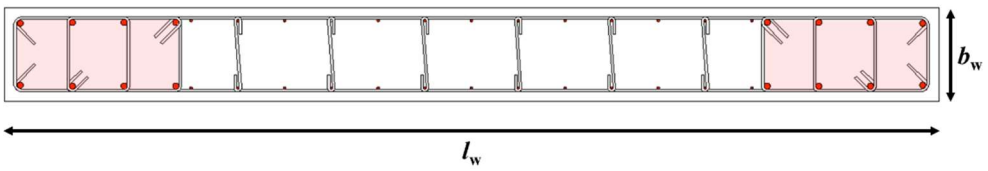
A.4.3 Walls

All the walls have the same cross section along the total height and are represented in the following table.

(a)



(b)



Structure	Wall width b_w (m)	Wall length l_w (m)
DCM.W.50.10	0,30	4,50
DC2.W.50.10	0,30	4,50
DC3.W.50.10	0,30	4,50
DCM.W.75.10	0,30	4,50
DC3.W.75.10	0,30	4,50