

Influence of Code Design and Detailing Requirements on Seismic Risk of Reinforced Concrete Frame Buildings

VIRGÍLIO AUGUSTO MATOS SILVA

Dissertação submetida para satisfação parcial dos requisitos do grau de
MESTRE EM ENGENHARIA CIVIL — ESPECIALIZAÇÃO EM ESTRUTURAS

Orientador: Professor Doutor Humberto Salazar Amorim Varum

Coorientador: Doutor Luís Carlos Ribeiro Martins

JANEIRO 2020

MESTRADO INTEGRADO EM ENGENHARIA CIVIL 2019/2020

DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440

✉ feup@fe.up.pt

🌐 <http://www.fe.up.pt>

Reproduções parciais deste documento serão autorizadas na condição que seja mencionado o Autor e feita referência a *Mestrado Integrado em Engenharia Civil - 2019/2020 - Departamento de Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal, 2020*.

As opiniões e informações incluídas neste documento representam unicamente o ponto de vista do respetivo Autor, não podendo o Editor aceitar qualquer responsabilidade legal ou outra em relação a erros ou omissões que possam existir.

Este documento foi produzido a partir de versão eletrónica fornecida pelo respetivo Autor.

To my beloved Parents

*“Experience does for the soul
what education does for the mind.”*
Casey Neistat

ACKNOWLEDGEMENTS

This work was only possible thanks to the exhaustless support, patience and friendship of my family, friends and colleagues who made this final year a lot easier.

Special acknowledgments must be made:

- To Professor Humberto Varum, for granting me the opportunity to develop this work abroad, for his guidance from the beginning, never-ending support, knowledge transmitted and his serenity throughout this process;
- To Doctor Luís Martins, for his constant availability since my arrival to Pavia, for the encouragement and all the knowledge transmitted;
- To Professor Vítor Silva and the GEM team for welcoming me, giving me the best conditions to work and all the advices, not only work related;
- To my friends and family for their support as well as for all the great moments which gave me the strength to always look ahead;
- To my parents, for their support on all my endeavours and for being my best example. Without them nothing of this would be possible to accomplish.

ABSTRACT

Traditional seismic design codes for structural design aim at protecting human life by preventing local or global collapse under a single level of earthquake these being introduced and enforced by the competent authorities with public safety as the overriding consideration. Earthquakes are amongst the natural hazards responsible for the highest number of human casualties and economic losses throughout mankind's history, making it an undeniable requirement that Europeans impose on their responsible authorities.

This work explores modifications to current European seismic design code EN 1998. The approach taken in this work was to define three ductility classes – deemed as improvements of the current code, regarding behaviour factor and reinforcement detailing. A 10-storey residential building was chosen as subject of experimentation; it was designed accordingly to Portuguese implementation of the European norm for the city of Lisbon, and also to its implementation in Italy for the city of Abruzzo. The design process, within the scope of this work, was automatized through Matlab scripts.

Pushover and time-history analysis were used to evaluate the performance of the buildings. The former yielded capacity curves and plastic deformation process was possible to evaluate. The later allowed to obtain average annual probability of collapse for each building in study.

Its concluded that the modifications here studied perform about what is expected to residential buildings, although some exceptions to be made and further improvements are necessary.

KEYWORDS: Eurocode 8, moment-resisting frame, reinforced concrete, seismic design, seismic risk

RESUMO

Os códigos de projeto sísmico tradicionais para projeto estrutural visam proteger a vida humana, impedindo o colapso local ou global sob um único nível de terramoto, sendo estes introduzidos e aplicados pelas autoridades competentes, com a segurança pública como consideração primordial. Os terremotos estão entre os riscos naturais responsáveis pelo maior número de perdas humanas e perdas económicas ao longo da história da humanidade, tornando este uma exigência que os europeus impõem às suas autoridades responsáveis.

Este trabalho explora modificações no atual código de projeto sísmico europeu EN 1998. A abordagem adotada neste trabalho foi definir três classes de ductilidade - consideradas como melhorias no código atual, em relação ao fator de comportamento e pormenorização de armadura. Um edifício residencial de 10 andares foi escolhido como objeto de estudo; foi concebido de acordo com a implementação portuguesa da norma europeia para a cidade de Lisboa e também a Italiana para a cidade de Abruzzo. O processo de design, no âmbito deste trabalho, foi automatizado através de scripts Matlab.

As análises de pushover e análises dinâmicas no tempo, foram usadas para avaliar o desempenho dos edifícios. A primeira produziu curvas de capacidade que permitiram avaliar o processo de deformação plástica. O último permitiu obter probabilidade anual média de colapso para cada edifício em estudo.

Concluiu-se que as modificações aqui estudadas têm o desempenho esperado para edifícios residenciais, embora sejam feitas algumas exceções e outras melhorias sejam necessárias.

PALAVRAS-CHAVE: Eurocódigo 8, sistemas porticados, betão armado, projeto sísmico, risco sísmico

TABLE OF CONTENTS

Acknowledgements	i
Abstract	iii
Resumo	v
 1 INTRODUCTION.....	 1
1.1. MOTIVATION AND FRAMING.....	1
1.2. OBJECTIVES.....	1
1.3. DOCUMENT ORGANIZATION	1
 2 LITERATURE REVIEW	 3
2.1. THE EARTHQUAKE PHENOMENON AND THE SEISMIC ACTION	3
2.1.1. HISTORICAL BACKGROUND	3
2.1.2. SEISMICITY, PLATE TECTONICS AND REID'S ELASTIC REBOUND THEORY	4
2.1.3. MAGNITUDE AND INTENSITY	4
2.1.3.1. Intensity scales	4
2.1.3.2. Magnitude scales	5
2.1.4. SEISMIC ACTION DEFINITION	5
2.1.4.1. Peak Ground Acceleration (PGA)	5
2.1.4.2. Peak Ground Velocity (PGV)	5
2.1.4.3. Peak Ground Displacement (PGD)	5
2.1.4.4. Response Spectrum.....	5
2.1.5. ACCELEROGRAMS.....	6
2.2. SEISMIC RISK.....	7
2.2.1. SEISMIC HAZARD	7
2.2.2. VULNERABILITY AND FRAGILITY.....	8
2.2.3. EXPOSURE	9
2.3. STATE-OF-THE-ART ANALYSIS	9
2.3.1. EQUIVALENT LATERAL FORCE ANALYSIS	9
2.3.2. MODAL RESPONSE SPECTRUM ANALYSIS	10
2.3.3. PUSHOVER ANALYSIS	10
2.3.4. TIME-HISTORY ANALYSIS	10
2.4. THE EN EUROCODES	11
2.4.1. EUROCODE 8.....	11
2.4.1.1. Ductility Classes (DC)	12
2.4.1.2. Behaviour factor	12
2.5. NUMERICAL MODELLING.....	13
2.5.1. FINITE ELEMENT METHOD	13
2.5.2. GENERAL MODELLING	13

2.5.3. TIMOSHENKO BEAM	13
2.5.4. GEOMETRIC NONLINEARITY	14
2.5.5. MATERIAL INELASTICITY	14
2.5.6. SEISMOSTRUCT2018	15
2.5.7. AUTODESK ROBOT STRUCTURAL ANALYSIS PROFESSIONAL	15

3 INFLUENCE OF DESIGN AND REQUIREMENTS ON THE PROBABILITY OF COLLAPSE OF RC FRAME STRUCTURES 17

3.1. PROBLEM	17
3.2. METHODOLOGY	17
3.3. DEVELOPMENT OF ROBOT MODELS	19
3.3.1. MODELLING PRINCIPLES	19
3.3.2. PRE-DESIGN PRINCIPLES	20
3.3.3. VERTICAL ACTIONS	20
3.3.4. SEISMIC ACTIONS, EQUIVALENT LATERAL FORCE ANALYSIS AND BEHAVIOUR FACTOR	21
3.3.4.1. Portugal	22
3.3.4.2. Italy	23
3.3.5. P- Δ EFFECTS	24
3.3.6. LOAD COMBINATIONS	25
3.4. DEVELOPMENT OF MATLAB TOOLS	25
3.4.1. DESIGN CAPABILITIES AND POSSIBILITIES	25
3.4.1.1. Beams	25
3.4.1.2. Columns	26
3.4.1.3. Ductility class M with participation slab script – DCMframeDesignerWSlab	26
3.4.2. WORKFLOW	27
3.5. DEVELOPMENT OF SEISMOSSTRUCT MODELS	27
3.6. GROUND MOTION RECORDS	28
3.7. RESULTS	28
3.7.1. DESIGN	28
3.7.1.1. Materials	28
3.7.2. PUSHOVER ANALYSIS AND EVIDENCE	30
3.7.2.1. Portugal	30
3.7.2.2. Italy	35
3.7.3. TIME-HISTORY ANALYSIS	38
3.7.3.1. Average Annual Probability of Collapse	42

4 FINAL REMARKS 43

4.1. CONCLUSIONS	43
4.2. IMPROVEMENTS AND FUTURE DEVELOPMENTS	43

References	45
Annexes.....	47

TABLE OF FIGURES

Figure 2.1 - Ruins of the Lisbon Cathedral following the 1755 - Jacques Philippe Le Bas (1757).....	3
Figure 2.2 – Loma Prieta Earthquake, October 18 th ,1989 – (Pacific Earthquake Engineering Research Center, 2000) and (CESMD, 2012).....	6
Figure 2.3 - Seismic Hazard Map - distribution of the PGA with a 10% probability of being exceeded in 50 years (M. Pagani et al., 2018).....	8
Figure 3.1 – Rendering of the building in study.....	18
Figure 3.2 - Horizontal Spectrum Lisbon, Portugal - Soil type D.....	23
Figure 3.3 - Horizontal Spectrum Abruzzo, Italy - Soil type D	24
Figure 3.4 - Materials cost comparison.....	29
Figure 3.5 - Pushover results for the Lisbon building, original and normalized.....	30
Figure 3.6 - Portugal building capacity curves with marked steps.....	32
Figure 3.7 - Portugal DCMslab	33
Figure 3.8 - Portugal DCMslab int.....	34
Figure 3.9 - Pushover results for the Abruzzo building, original and normalized	35
Figure 3.10 - Italy building capacity curves with marked steps.....	36
Figure 3.11 - Italy DCL.....	37
Figure 3.12 – Global drifts and probability of exceedance for Portuguese structures	39
Figure 3.13 – Global drifts and probability of exceedance for Italian structures.....	41

TABLE OF TABLES

Table 3.1 - Geometrical and reinforcement requirements for DCI and DCM	19
Table 3.2- Vertical loads	21
Table 3.3 - ψ factors	21
Table 3.4 - ϕ factor to compute ψ_{Ei}	21
Table 3.5 - Horizontal forces for seismic action, Lisbon, Portugal - Type 1 DCI	22
Table 3.6 - Torsional moments for seismic action, Lisbon, Portugal - Type 1 DCI	22
Table 3.7 – Horizontal Elastic Ground Acceleration Response Spectrum Data	22
Table 3.8 - Horizontal Elastic Ground Acceleration Response Spectrum Data.....	24
Table 3.9- Total costs of Materials [€].....	29
Table 3.10 – Average Annual Probability of Collapse	42

SYMBOLS

Latin upper-case letters

A_{column}	Gross area of a column cross section
C_c	Coefficient for subsoil
CQC	Complete Quadratic Combination
CSV	Comma Separated Value
DC	Ductility Class
$DSHA$	Deterministic Seismic Hazard Assessment
EDP	Engineering Demand Parameter
$MRSA$	Modal Response Spectrum Analysis
N_{qpc}	Quasi-permanent combination axial load
PGA	Peak Ground Acceleration
PGD	Peak Ground Displacement
PGV	Peak Ground Velocity
$PSHA$	Probabilistic Seismic Hazard Assessment
P_{tot}	Total gravity load at and above the storey considered
RC	Reinforced concrete
$S; S_{max}$	Soil factor
SA	Spectral Acceleration
S_c	Soil factor accounting for topographic amplification
$SDOF$	Single Degree of Freedom
SLS	Serviceability Limit State
$SRSS$	Square Root of the Sum of the Squares
S_s	Soil factor accounting for stratigraphic amplification
T_B	Left corner period at the upper limit of the constant acceleration region of the elastic spectrum
T_C	Right corner period at the upper limit of the constant acceleration region of the elastic spectrum
T_C^*	Beginning period of the constant velocity in the horizontal acceleration spectrum
T_D	Corner period at the beginning of the constant displacement response range of the elastic spectrum
ULS	Ultimate Limit State
V_{tot}	Total storey shear
β	Lower bound factor for the horizontal design spectrum

Latin lower-case letters

a_g	Peak ground acceleration
a_{gR}	Reference peak ground acceleration
b_0	Minimum dimension of the concrete core – to the centreline of the hoops
b_{beam}	Width of a beam cross section
$b_{ef, I, CD}$	Effective width of a slab, parallel to a beam
$d_{bl}; d_{bl, min}$	Minimum longitudinal bar diameter
d_r	Design inter-storey drift
d_w/d_x	Rotation of the cross section; slope of the element axis
f_{cd}	Design value of concrete compressive strength
h	Inter-storey height

h_{beam}	Overall depth of a beam cross section
h_c	Largest cross-sectional dimension of a column
l_{beam}	Free span of a beam
l_{cl}	Clear length of a column
l_{slab}	Free span of a slab
q	Behaviour factor
q_D	Factor accounting for the deformation capacity and energy dissipation capacity
q_R	Factor accounting for overstrength due to the redistribution of seismic action effects in redundant structures
q_S	Factor accounting for overstrength due to all other sources
s	Stirrups or hoops spacing
t_{slab}	Thickness of a slab

Greek upper-case letters

Φ	Additional rotation due to the distortion of the cross section
--------	--

Greek lower-case letters

γ_I	Importance factor
θ	Rotation of the cross section, Inter-storey drift sensitivity coefficient
ρ'	Compression reinforcement ratio in the section
ω_{wd}	Mechanical volumetric ratio of confining hoops

1

INTRODUCTION

1.1.MOTIVATION AND FRAMING

Earthquakes are amongst the natural hazards responsible for the highest number of human casualties and economic losses throughout mankind's history, making one of the core efforts of the European Union, defending its foundation values, to provide the ones under its veil: social security, protection and a right to be safe whichever the situation. And it's on this broad concept that ultimately this work is framed. Providing guidelines and a constant development of these to improve its quality, harmonization, studying new factors and new approaches, the Eurocodes are trusted, even by those who don't realize it, but ultimately benefit from it, to be a tool of safety.

The construction European codes – Eurocodes - prescribe different performance levels for different ground shaking intensity levels - performance-based seismic design – meaning that the resulted construction stock, that was designed to the provided intensity level associated with a prescribed return period, may output the expected risk level, that was previously agreed to be a sensible compromise between cost and fatality rate.

Another factor is the importance that risk assessment has in the economics of construction, losses in the insurance business due to natural disasters are quite large, being earthquakes responsible for 20% of the total monetary losses (Martins, 2018). Being able to predict the risk of building failure or even the necessity of reconstruction allows the insurance companies to be more inform about what monetary risks they want to take. If the correct risk passament is performed, the correct insurance can be made avoiding this way catastrophic losses due to natural disasters.

1.2.OBJECTIVES

This work focus on the evaluation of different design detailing requirements and conditions established for this exploratory work on reinforced concrete frame buildings, through the analysis of an example representative of its structural type.

The first step it to define the new requirements, modifying the current regulation and make new provisions when needed, these were based on some studies that have been done on similar matter. The evaluation of these are recognised through an experimental designed building following the defined rules very strictly and, in some degree, disregard some conceptual design principles. With the attained structures, their performance is evaluated against a number of seismic records.

1.3.DOCUMENT ORGANIZATION

This work starts with a revision of concepts and central knowledge for the development of it; some historical background on earthquakes and seismic engineering, focusing on intensity and seismic action definitions, scales and characterization, followed by a brief introduction to seismic risk and what makes

important it study. Being a performance seeking work, building performance methods and the underlying numerical theory is also revised. Moreover, some Eurocodes context is given.

On the third chapter, the problem is clarified, and the methodology explained. The design process, the creation of Robot models, Matlab scripts and SeismoStruct models is rationalized, developed and the produced results are evaluated.

Finally, the work ends with some final remarks and proposals for future works.

After it, further results can be found in the annexes for further information and reproducibility purposes.

2 LITERATURE REVIEW

2.1. THE EARTHQUAKE PHENOMENON AND THE SEISMIC ACTION

2.1.1. HISTORICAL BACKGROUND

Humanity have been experiencing the effects of earthquakes and trying to understand and explain its cause for millennia. The first attempts to explain seismic events were based on mythologic and religious practises, restless gods or giant creatures slumbering beneath the Earth. For almost two millennia in the search for understanding how geological features had been formed, there was controversy over whether the primary agent of geological change was the action of water - *Neptunist* - or heat - *Vulcanist*. The former would justify earthquakes through the collapse of subsurface caves formed by a primitive ocean, the latter would state that it was caused by the fire inside the planet (Lopes, 2009).

The 1755 Lisbon Earthquake created a rupture with the theories in practice and established a scientific approach to the phenomenon, up until this moment was given that God wouldn't allow such disaster and such prompted the need to find a non-religious motive, triggering the birth of modern earthquake science (B.G.S., 2017). The inquiry commissioned by Sebastião José de Carvalho e Mello is still regarded today as a reference document for its scientific rigor and (at the time) state of the art methodology (Lopes, 2009).



Figure 2.1 - Ruins of the Lisbon Cathedral following the 1755 - Jacques Philippe Le Bas (1757)

Lessons from Lisbon's earthquake inspired the physicist John Michell to formulate the hypothesis that related the earthquake phenomenon with the wave propagation through the lithosphere. Later, the addition of the *Elasticity Theory* to the phenomenon made by Robert Mallet in 1848, gave it scientific

plausibility. Nonetheless, only in the 20th century with the *Plate Tectonics Hypothesis* the origin and working mechanism of the earthquake and its effects was broadly recognized and accepted by the scientific community (Lopes, 2009).

2.1.2. SEISMICITY, PLATE TECTONICS AND REID'S ELASTIC REBOUND THEORY

The theory of plate tectonics consolidated more recently but substantiated on the *Continental Drift Theory* (1915) by Alfred Wegener stated that the outer shell of Earth's crust consists in various plates about 100km thick and that they move relatively to each other slowly to human time-scale but with relevance to geological, releasing energy suddenly and creating the ground shaking. These crustal deformations occur usually on the plate boundaries and come as consequence the stress exceeds the strength of the plate boundary material, strain energy is released and then being transmitted through the medium as Primary and Secondary waves and after reaching the surface a curious change occurs and they adopt a different behaviour resulted from the interaction of them causing Rayleigh and Love waves (Sen, 2009).

Henry Fielding Reid was the one who advanced the *Elastic Rebound Theory*, described in simple terms before, after the Great 1906 San Francisco Earthquake. On his study of the San Andreas fault he discovered that several meters of relative displacement much as 4.7m occurred along several hundred kilometres of the fault, making him to conclude that the rocks at the fault zone were bending and storing elastic energy just to release it and attain a new resting form and in between dissipate the energy partially as heat, partially cracking underground rocks and partially as elastic waves (Sen, 2009).

Regardless the focus of the seismic activity originated on the Plate Tectonics, there are other sources for seismic activity. Earthquakes have been associated with volcanoes and magma shifting events. Others associated with man-made such as explosions and detonations of nuclear devices, and consequences of that like the 1974 landslide in Peru producing seismic waves equivalent to a 4.5 magnitude earthquake. Furthermore, have been found some correlation between the filling of large reservoirs like dams and the occurrence of seismic events and the escalation of their magnitude (Kramer, 1996).

2.1.3. MAGNITUDE AND INTENSITY

The size of an earthquake may be a concept that one may find intuitively to understand through the report of an earthquake consequences, and this is in some way the definition of intensity, as described in (Sen, 2009) "intensity is the impact of ground-shaking on population, structures and the natural landscape; the impact will be greater nearer the site and less far away"; similarly, he describes magnitude as "the amount of energy released from the source or focus of the earthquake".

2.1.3.1. INTENSITY SCALES

As previously stated, intensity is a qualitative description of the effects, and has been proved useful to estimate the return rate, location and itself as an evaluation tool of the size of the earthquake whereas no seismographs were installed. Because it focuses on the effects and not on the energy, places at the same epicentral distance will have different damage outcome since it will depend largely on the soil and construction conditions. The most common scale in use nowadays is the Modified Mercalli Intensity that describes physical effects of the earthquake in twelve grades (Sen, 2009).

2.1.3.2.MAGNITUDE SCALES

Most magnitude scales are based on ground motion characteristics and this were made possible by the technology development of the last century. An objective and quantitative measurement of the earthquake size have been made, increasing our understanding of it. The first scale to be set was the Richter scale in 1935 and was valuable in the assessment of shallow and local earthquakes. From that other were developed in order to describe other earthquakes regarding epicentral distance and depth. The only magnitude scale that is able to describe the size of any earthquake is moment magnitude, given that is based on a direct measurement of the factors that produce rupture along the fault (Kramer, 1996).

2.1.4.SEISMIC ACTION DEFINITION

The definition and quantification of the seismic action is fundamental for civil engineering practise, since a good characterization of it will lead to a better understanding of building response and consequently a better and reasoned design of it.

Considering the seismic event as a very complex phenomenon using only one parameter to describe the ground motion would be of greater inaccuracy. A vast number of parameters have been proposed do characterize the amplitude, frequency content, and duration of ground motions (Kramer, 1996).

2.1.4.1.PEAK GROUND ACCELERATION (PGA)

The most common way of describing the event is through amplitude parameters during the event extent. Among these, peak ground acceleration is the most common, simply defined as the absolute value of ground acceleration per time-step. Its popularity derives from the direct relation with inertial forces and consequently with the observable damage and intensity. Greater PGA values may be associated with more destructive effects, but that now always remains true since sometimes they only last for only a very short period of time and occurred at very high frequencies (Kramer, 1996).

2.1.4.2.PEAK GROUND VELOCITY (PGV)

Defined as the absolute maximum velocity throughout the velocity-time record, due to the fact that velocity is less subjective to the high frequency influence, peak ground velocity gives information about the destructive potential of an earthquake regarding structures more sensitive to a medium frequency. The same way PGA has mathematical relations to calculate an expected intensity, PGV has its own (Kramer, 1996).

2.1.4.3.PEAK GROUND DISPLACEMENT (PGD)

Is the less used due to the information that produces and the difficulty on its acquisition concerning signal processing difficulties and long-period noise. Like the other two amplitude parameters previously mentioned it doesn't provide any frequency or duration information of the motion, making it necessary to be complemented with other information (Kramer, 1996).

2.1.4.4. RESPONSE SPECTRUM

Regarding frequency content parameters, various have been proposed such as *Fourier Spectrum*, *Power Spectrum* and *Response Spectrum*, all of them represent how the amplitude of a ground motion is distributed among different frequencies. The latter is of the important notice to structural engineering since it provides a condensed response of all possible linear SDOF systems to a particular component

of ground motion (e.g. spectral acceleration - Figure 2.2d)) enabling the development of lateral force requirements common in building codes, as a consequence of how very sensitive a structure is to the frequency at which it is loaded (Chopra, 2012; Kramer, 1996).

Another key concept is the distinction between response and design spectrum. The former is a description of all SDOF systems maximum responses to particular ground motion hence being very rough. The latter is a specification of the level of seismic design force as a function of the structure natural vibration period and damping ratio. Their similarity comes when the design one is determined on a statistical analysis of comparable response spectra (Chopra, 2012).

Is common to be able to divide the response spectrum in three intervals: acceleration-controlled (low-periods), velocity-controlled (intermediate periods), and displacement-controlled (high-periods); suggesting which of the parameters is predominant, meaning nearly constant (Kramer, 1996).

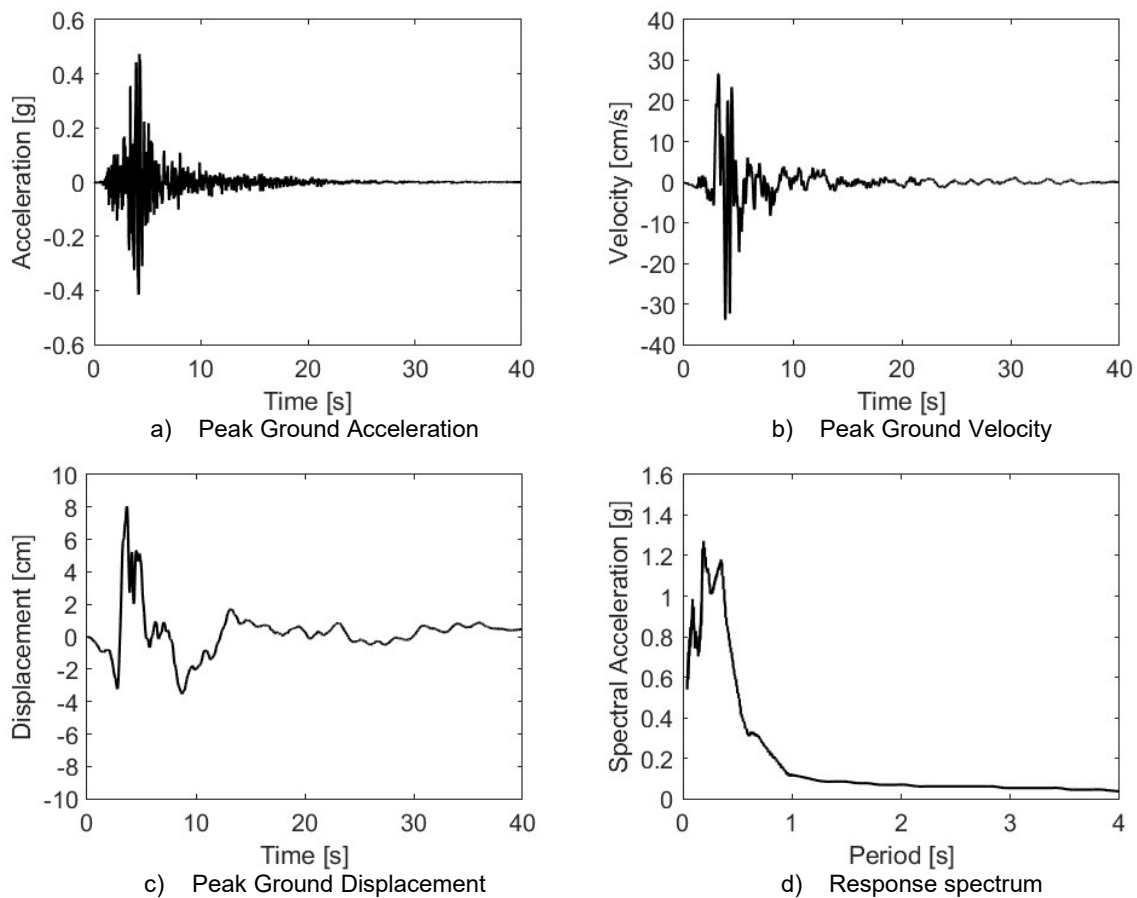


Figure 2.2 – Loma Prieta Earthquake, October 18th, 1989 – (Pacific Earthquake Engineering Research Center, 2000) and (CESMD, 2012)

2.1.5. ACCELEROGRAMS

The spectrum used for design is usually obtained by scaling the aforementioned elastic design spectrum with factors accounting for inelastic structural phenomena. May be situations that the application of this methodology be insufficient, such as buildings designed for higher ductility, structures with high degree of irregularity in plan or elevation, critical structures that which failure would be unacceptable or with special features like base isolation (Bommer & Acevedo, 2008).

The advance in computer technology and capabilities made possible dynamic time-history analysis for the study of more complex structures, enabling a better quantification of the structure behaviour

regarding energy dissipation and plastic-hinge formation, processes that lead to structure failure. However, the selection of suitable accelerograms to represent the seismic excitation is still a very complex matter with a big uncertainty associated (Katsanos, Sextos, & Manolis, 2010).

It is assumed that the use of accelerograms of real earthquakes on the same region as the one in analysis delivers valid and substantiated results. In order to select the accelerograms various techniques have been proposed being one of the most common the *record selection based on spectral matching*, in which the selection of real accelerograms is performed on the basis of compatibility between their response spectra and a corresponding 'target' spectrum as defined by code provisions or computed directly through a PSHA. Regarding the number of accelerograms selected a variety of studies present seven as the minimum threshold to valid results whilst others suggest a more generous number, between ten and twenty (Bommer & Acevedo, 2008; Katsanos et al., 2010).

2.2. SEISMIC RISK

In structural engineering, risk is the likelihood that a structural system fails to perform as intended within a reference period of time due to earthquakes. Usually risk is measured considering the convolution of three main factors: hazard (likelihood of an earthquake event), vulnerability (probability of damage, given the earthquake) and exposure (description of the assets at risk) (Pasquale, Orsini, & Romeo, 2005).

2.2.1. SEISMIC HAZARD

Natural hazard may be defined as a potentially damaging physical phenomenon that may cause loss of life or injury, property damage, social and economic disruption or environmental degradation, through naturally occurring events such as earthquakes, hurricanes, tornados and floods (Kramer, 1996).

Hazards associated with earthquakes are referred as *seismic hazards* and usually convey the following:

- Ground shaking – may be considered the most important of all seismic hazards because all the others are caused by this one. After the sudden release of energy, seismic waves radiate away from the source and travel rapidly through the earth's crust, that produce some kind of shaking when reaching the ground surface, this being determined by the size and location of the earthquake and on the characteristics of the site;
- Structural hazard – on most earthquakes, structural damage is the main cause for economic loss and human death and there is no need for a structure collapse to cause such tragedy since falling objects, parapets, shelves have caused casualties;
- Liquefaction – this phenomenon occurs when due to the ground shaking the soil deposit loses its strength and stiffness resembling a fluid behaviour unable to sustain loads and support structures. Since it only occurs in saturated soils, is most commonly observed near bodies of water;
- Landslides – sometimes due to other hazards such as the liquefaction of the soil or to the failure of marginally stable under static conditions, landslides occur in natural or man-made slopes causing damage by destroying building or disrupting other constructed facilities, sometimes burying entire towns;
- Retaining structure failures – on a static hypothesis the mass of the structure (retaining wall, bridge abutments, quay walls, anchored bulkheads, braced excavations, ...), soil pressure and external forces are in equilibrium but the disruption of an earthquake due to ground motion or liquefaction may lead to failure, whether by sliding, tilting, bending or other failure mechanism;
- Lifeline hazards – any developed area depends on a network of facilities that provide from the simpler services to more advanced ones, from commerce to public health. Networks of electrical

power, telecommunications, transportation, water and sewage, oil and gas distribution, and waste storage systems, are named lifelines. These structures and the facilities that comprise them when disrupted can cause losses that exceed the cost of repairing facilities directly damaged by earthquake shaking, and hold back emergency response and rescue efforts immediately following damaging earthquakes;

- Tsunami and Seiche hazards – fault rupture in the seafloor can produce long-period but fast waves that as approach shore, due to decreasing water depth, decrease velocity and increase height, sometimes being a near vertical wall of water that rushes to inland causing devastating damage. When earthquake induced waves occur in enclosed bodies of water are called seiches.

Seismic hazard assessment can be performed through two main approaches: deterministic or probabilistic. In the former, deterministic seismic hazard assessment (DSHA) a particular scenario is assumed, leading to a worst-case ground motion setup that can be of interest in cases that a structure failure could lead to catastrophic consequences like disruption of a nuclear power plant or large dams. It's the most subjective method since it relies on the expertise and decisions of the personnel involved creating the scenario. The latter, probabilistic seismic hazard analysis (PSHA) is a procedure that considers, by quantifying them, the uncertainties regarding size, location rate of occurrence and ground shaking effects of the earthquakes.

As a definition, in seismic engineering, seismic hazard may be defined as the likelihood of exceeding a certain amount of ground shaking, or ground motion, in the defined return period. The hazard depends on the magnitudes and locations of likely earthquakes, how often they occur, and the properties of the rocks and sediments that earthquake waves travel through. being one of its outcomes the production of hazard maps for easy consultation.

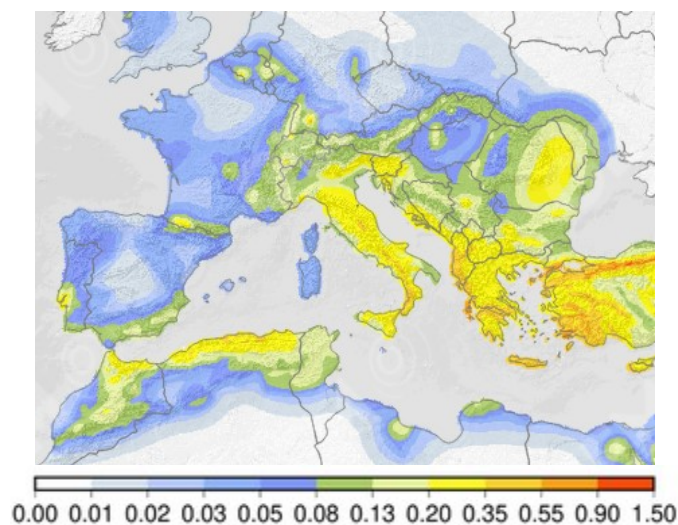


Figure 2.3 - Seismic Hazard Map - distribution of the PGA with a 10% probability of being exceeded in 50 years (M. Pagani et al., 2018)

2.2.2.VULNERABILITY AND FRAGILITY

Vulnerability is defined as the distribution of loss of a given asset conditional on the ground shaking intensity. There are two main methods of vulnerability assessment, *predicted* and *observed* vulnerability as in (Coburn & Spence, 2003) or *analytical* and *empirical* vulnerability as in (Pinho, Abrantes, Magenes, & Crowley, 2006). The observed/empirical approach is based on the statistics of past earthquakes, developed and calibrated as a function of seismic intensities, these being discrete functions

expressing, in matrices, the conditional probability of obtaining a damage level; or continuous functions (vulnerability functions) which express the probability of exceeding a damage state given the intensity. The predicted/analytical vulnerability refers to assessment based on the expected performance of the buildings based on calculations and design/code specifications generally being more detailed and transparent allowing sensitive studies being made and calibration of the various features.

Both approaches have their place since empirical is more suitable for use with non-engineered structures, whose earthquake resistance is more difficult to calculate, but for which substantial statistical damage data may be available, and analytical for engineered structures where a knowledge of the materials and the structure in general may be anticipated.

On an analytical vulnerability approach, the first step is to establish a relation between structural damage threshold and an engineering demand parameter (EDP) such as interstorey drift or floor acceleration. Secondly, the evolution of damage with increasing intensity measure level such peak ground acceleration, peak ground velocity, peak ground displacement or spectral acceleration must be evaluated to derive structural fragility functions, which provide the probability of exceeding a set of structural damage states for the given IMLs and then a damage-to-loss model is applied, thus obtaining loss depicted as a function of environmental excitation (Martins, 2018). These fragility functions that represent the probability of exceedance for any given IML, can be approximated by a two-parameter lognormal distribution function – viz median and standard deviation of the natural logarithm.

2.2.3.EXPOSURE

Exposure is description of assets at risk in a given location such as: people, buildings and infrastructure. Regarding the building stock, for a good characterization is usually applied a taxonomy that comprises the following attributes: construction material, height, age (Silva, Crowley, Pagani, Monelli, & Pinho, 2013). This information is used to attribute a vulnerability curve to a given asset.

2.3.STATE-OF-THE-ART ANALYSIS

Two main approaches to determine seismic response and design of structures are currently implemented in Eurocode. It can be either by assuming a linear or a nonlinear behaviour of the structure or in other words assuming either an elastic or inelastic behaviour respectively. The former assumes a linear stress-strain progression of the structure but incorporates a behaviour factor (q) to account for the nonlinear characteristics of the response. The nonlinear approach tends to be more computational demanding but is regarded as the best to describe the actual behaviour of a structure, in particular the time-history analysis.

Four methods are contemplated in the Eurocode 8 for the assessment of the structure, herein described in the next subchapters.

2.3.1.EQUIVALENT LATERAL FORCE ANALYSIS

The lateral force method is a linear static analysis which is performed by applying lateral forces, separately in two orthogonal horizontal directions. This way it's possible to simulate the peak inertia loads caused by the horizontal forces presented in the seismic action. This way it's assumed that the fundamental mode shape of the structure will have the most weight in its response.

Combining the natural frequency and the seismic inertia it's possible to calculate a total base shear which is then set up as a series of forces for each level of the structure. This way, the higher the level, more loads it will generate. This poses a problem when the structure doesn't present levels with the same weight. A level in the middle of the structure with significant more weight than the rest will have a

greater influence in the spectrum of the response. For this reason, a modal response spectrum analysis is recommended in Eurocode 8 since it accounts for the *all* the eigen modes instead of just considering the fundamental one.

2.3.2. MODAL RESPONSE SPECTRUM ANALYSIS

Modal analysis is based on the principle of superposition of effects and because of that can only be applied in the response to linear systems. This analysis breaks down a multiple degree of freedom system into several oscillators of one degree of freedom, the responses for each of these systems can be then calculated and combined algebraically giving in this way the response of the system.

Having the system divided into several one degree of freedom systems, the response of systems can be easily calculated. These responses, defined by the eigenvectors and eigenfrequencies, gives the displacement vector of the system when the multiplication of modal matrix (composed by the eigenvectors) to the modal vector (harmonic movement of frequency of each mode) is done.

Peak modal values of the effects of seismic action can be calculated by combining the modes of vibration using the SRSS or CQC formulation for a determined action that occurs on the building due each of the contributing modes of vibration. The number of modes should account for at least 90% of the mass of the system or consider modes each have an influence greater than 5% in the mass of the system. This type of analysis presents the advantage of considering all the different responses that the system can present given the seismic action, instead of considering that the system will vibrate like a cantilever beam which is only true by perfectly symmetric building. Nevertheless, this vibration mode will still be the fundamental one for a normal building.

2.3.3. PUSHOVER ANALYSIS

One way to assess the structure nonlinear behaviour with relatively low computational cost is the pushover. This type of analysis consists of the application of an incremental lateral load on each floor level until for a collapse mechanism develops. This method is best suited for structures with a regular distribution of stiffness and mass (i.e. symmetrical structures). This way the higher natural modes of vibration can be neglected.

The pushover analysis gives the capacity curve for the structure, this curve relates the total shear stress with the displacement of a control node. An evaluation of a seismic scenario is possible and conclusions regarding the expected structure behaviour can be reached through close estimations of forces and deformations of the structure nodes. This analysis also allows for a prediction of fracture sequence of the structural elements through the progression in the global capacity curve (Ribeiro, 2008).

This type of analysis is simpler than a nonlinear dynamic analysis and has already proven to provide good alternative to the latter (within its intended scope). Providing good estimates of the inelastic behaviour, especially in regions where significant plastic deformations occur.

The pushover analysis can be divided into two main categories. If the load profile is kept invariant analysis, it is denominated of conventional pushover algorithm. If instead the load profile varies through the structure an adaptive pushover algorithm is being used. In the latter the load profile is updated in each load increment to better account for the variation on the dynamic properties and of modes of vibration (Martins, 2013).

2.3.4. TIME-HISTORY ANALYSIS

A more accurate structural analysis should include the ductility properties of the structure, during a seismic action inelastic deformation may happen due to these properties. If these effects are to be

considered a linear approach to the problem cannot be followed. Both nonlinear responses due to material and geometry should be considered. It's worth noting that the modal analysis only considers geometric nonlinearity.

One of the solutions to include this structural behaviour in the analysis is by numerical integration, some methods are the *Newmark method* or the *Hilber-Hughes-Taylor method*. The way these methods work is by dividing the problem into time-steps. Between these time-steps the structure is treated like a linear elastic problem. Once equilibrium is reached, geometry and stiffness of the structure are updated and a new time- step is implemented.

This comes with a more severe computational cost compared to the modal analysis, although this is something to expect due to the increased complexity of the problem. To decrease this computational cost, the time-step used should be as high as possible while still allowing for numerical stability and the correct integration of the last mode of vibration with an influence in the structure. Nevertheless, smaller time-steps will provide more accurate solutions because the influence of the higher modes of vibration is better integrated (Chopra, 2012; Martins, 2018).

2.4.THE EN EUROCODES

The Eurocodes creation can be traced back to 1957 when there was an effort to create an Europe united and stronger through the Treaty of Rome and within this purpose, 18 years later the 'creation' of the Eurocodes started firstly tackling the technical obstacles to trade and the harmonisation of technical specifications by means of technical rules which, that would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them. Nine years later the first version was published (JRC). This continuum effort requires a constant contribution from the Member States for a further harmonization and development to keep with state-of-the-art procedures and obtain a high-quality standard fulfilling the original objective of the European Standards (EN) Eurocodes.

2.4.1.EUROCODE 8

The EN Eurocodes are divided in 10 standards (EN 1990 - 1999), covering from structural safety, actions on structures, design requirements for various construction materials, geotechnical design and earthquake resistant design, the latter being EN 1998 - Eurocode 8 also known as EC8, their use must be interchangeable, seamless and they should complement the other norms.

The EN 1998 is divided in 6 parts regarding design general rules, buildings and other civil engineering works. The standard is created on the basis that in the case of a seismic event:

- Human lives are protected;
- Damage is limited;
- Lifeline structures for civil protection remain operational.

These safety concerns are implemented in the code through *performance-based seismic design*, meaning its focus is to fulfil its intended purpose, considering the consequences of its failure to meet it, contrasting with more traditional codes, being more process-oriented, where the means and standard, easy to execute rules may surpass the pursuit of satisfactory performance. This approach tries to maximise the utility from the use of a facility by minimising its expected total cost, including the short-term cost of the work and the expected value of the loss in future earthquakes (in terms of casualties, cost of repair or replacement, loss of use)(Fardis, 2009).

Performance levels are associated with Limit States of the structure. These are divided according to its concern:

- Ultimate Limit States – safety, either people or structure;
- Serviceability Limit States – normal function of the structure, comfort of its users and damage to non-structural elements.

For new constructions, each one of them is associated with one performance level: no-collapse (NC) and damage limitation (DL). The former entails the protection of life under a rare seismic event, prevention of collapse of any structural member and retention of structural integrity and residual load capacity after the event; the latter aims to mitigation of property loss in frequent earthquakes.

The damage limitation performance level is evaluated by limitation of the lateral displacement of the building storeys to levels of adequate for the integrity of all its elements – including non-structural – viz., interstorey drift ratios, defined as the difference between the average lateral displacements of consecutive storeys divided by the interstorey height. The no-collapse performance level is obtained by designing and detailing structural elements for a combination of strength and ductility that provides a safety factor against substantial loss of lateral load resistance.

The seismic action under which collapse should be prevented is the *design seismic action*. The one for which damage limitation is pursued is called the *damage limitation seismic action*. Within the Eurocode philosophy of national competence on issues of safety and economy, the hazard levels for these two seismic actions are left to national determination.

For structures of ordinary importance (II), Eurocode 8 commends:

1. Design seismic action – with 10% exceedance probability in 50 years – a mean return period of 475 years;
2. Damage limitation seismic action – having a 10% probability of being exceeded in 10 years – a mean return period of 95 years.

2.4.1.1. DUCTILITY CLASSES (DC)

The current Eurocode enlists three ductility classes: low (L), medium (M) and high (H) dissipative structural behaviour. The choice of the ductility class of concrete structures is one of the key design parameters since it will heavily influence design criteria and requirements, the choice of it depends on technical and economic reasons and geographical limitations. For example, the current version states that DC-L is only recommended for low seismicity situations enabling less demanding design requirements. In contrast, structures in DCM and DCH need to satisfy specific requirements primarily related to ensuring enough ductility in the main dissipative zones (critical zones), mainly by means of ductile hysteretic behaviour under repeated reversed loading, without suffering brittle failures.

It's also needed to refer that for each ductility class the final safety of the designed structure would necessarily be the same, its behaviour under seismic loading and consequent use of the structure may differ.

2.4.1.2. BEHAVIOUR FACTOR

The behaviour factor q is fundamentally an approximation of the ratio of the seismic forces that the structure would withstand for an elastic response, with 5% viscous damping, regarding the forces used in seismic design on a conventional elastic analysis model, still ensuring an adequate response of the structure. The factor itself and the damping ratios are given for different materials and structural systems according to which ductility class.

It reflects the capacity of a structure to deform plastically, and is used for design purposes to reduce the forces obtained from a linear analysis in order to account for the non-linear response of a structure

associated with the material, the structural system and the design procedures; it also gives a sense of how much of the structure ductility is exploited; as stated in (Matejčková-Farhat & Ároch, 2013) "is an approximation of the ratio of the seismic forces that a structure with a completely elastic response would experience to the seismic forces that may be used in the design to still ensure a satisfactory response of the structure.". On the design procedure is used, for example, reducing the forces on a lateral force method of analysis.

On the current code the behaviour factor depends on the structural system and ductility class. Its value is higher on DCH, meaning that is expected that for a more demanding DC will exploit more of its ductility.

2.5. NUMERICAL MODELLING

2.5.1. FINITE ELEMENT METHOD

The first reference to the term *finite element* is attributed to Ray Clough, in the 1960 titled article "The Finite Element in Plane Stress Analysis" (Azevedo, 2003). The first real on-field test to this theory was the Norfork Dam project, where the gravity concrete dam had a temperature induced vertical crack. The University of California, Berkeley professor and his team concluded that the cracked dam was safe, using an IBM 704; and the results documented in "The Stress Distribution of Norfork Dam" presented at the Symposium on the Use of Computers in Civil Engineering, held in Lisbon, Portugal, in 1962. In that symposium was also presented an important paper on the step-by-step dynamic response analysis of finite element systems, co-authored by Edward Wilson (Clough & Wilson, 1999).

The 1970-80's have seen significant developments in the finite element method and nowadays is very diversified with different approaches and fields of application such as heat transfer phenomena, fluid flow, electric and magnetic fields as others, but as aforementioned it first developed on the context of civil engineering, parallel to the development of more powerful computational machines. In its core, it subdivides the complex system - continuum - into smaller and finite – as opposing to the classical term "infinitesimal" - simpler elements, with simpler governing equations and boundary conditions, then the assembly of its elements following a well-defined procedure of establishing local equilibrium at each node or connecting point of the structure, resulting in equations that can be solved for the unknowns, typically displacements and rotations (Zienkiewicz, 1989).

2.5.2. GENERAL MODELLING

Numerical models have to be conceived and calibrated to provide the best estimates of the real problem. When developing a FEM model, the modeller should consider the desired outcome, which physical phenomena may influence the problem, the material properties, the targeted accuracy level and any computational constraints that may exist.

2.5.3. TIMOSHENKO BEAM

One of the most relevant elements for structural engineering applications is the *Timoshenko beam*. Timoshenko's formulation shares hypotheses with other more conventional theories, but it states the following for the normal kinematics: "cross-sections normal to the beam axis before the deformation remain plane but not necessarily orthogonal to the beam axis after deformation".

$$\theta = \frac{dw}{dx} + \phi \quad (2.1)$$

Where θ denotes the rotation of the section, that is equal to the slope of the beam axis plus the additional rotation due to the distortion of the cross section - shear component (Oñate, 2010).

The contribution of shear deformation may be considered trivial in slender beams, as one can imagine that the tangential forces on the cross-section don't account much, however, in deeper beams (height over span greater than 0.10), shear deformation plays a significant role in the flexural behaviour. The major drawback of his theory is the assumption of the transverse shear strain distribution to be uniform across the beam cross-section at any point, thus requiring problem dependent shear modification factors to accurately represent the strain energy of deformation (Chinwuba Ike, 2019).

2.5.4.GEOMETRIC NONLINEARITY

The geometric nonlinearity is comprised by three factors: large displacements/rotations, second-order effects and beam-column effects. The two former effects are taken in account through the implementation of a total co-rotational formulation developed in (Correia & B. E. Virtuoso, 2006). The dominant motion in the large displacement analysis of a slender-bar space frame is attributable to finite rotations, the elastic strains remaining small. This method considers three-dimensional finite rotations; these rotations are considered making use of the Euler's finite rotation formula obtaining the correct relationship of the rotational degrees of freedom with large displacements/rotations. If the rigid body motion is eliminated from the total displacements, the deformational part which remains is always a small quantity relative to the local element axes, thus, the elastic deformation can be isolated. Within this interpretation, a local Cartesian coordinate system attached to each finite element continuously translates and rotates with the element as the deformations proceeds and then this is translated to the global coordinate system unveiling the final global response of the structure (Sandhu, Stevens, & Davies, 1990).

The beam-column effect is considered calculating the transverse displacement as a function of the end-rotations of the element as proposed in (Elnashai & Izzuddin, 1991).

2.5.5.MATERIAL INELASTICITY

The study of a non-linear analysis requires a definition of the stress-strain relation. To describe the nonlinear behaviour of concrete and steel, different models can be found in the literature to represent the hysteretic behaviour of the materials to cyclic loads, in general, the difference between models is how many stages the progression is divided and, for instance, in the concrete case, if tensile force is considered or not.

The material inelasticity can be modelled on the element through a distributed plasticity, where the whole element is considered inelastic, or with concentrated plasticity -as was used in this work -, where the element is considered elastic and plastic hinges are considered in the end nodes – plastic-hinge formulation - with this approach is assumed that all the inelastic deformation may occur on the length previously defined, being of special attention upon its definition on the possibility of compromising the results. The hinge rotation capacity depends on the aforementioned hinge-length and of the element chord rotation, inferring the necessity and possibility of different criteria for its definition.

The software implements these assumptions through a formulation of “fibre approach”, meaning each section is discretized into a large number of “fibres”, typically between 100 and 200, where each fibre is associated with an individual stress-strain relationship and the boundary sections follow the Navier-Bernoulli approximations of plane sections remain plane; the section response is determined by the numerical integration of the individual fibre's response between two “Gaussian sections”; the number of Gaussian sections can vary between 2 up to 10, usually 5 being sufficient for a good characterization of the element (Scott, Franchin, Fennes, & Filippou, 2004), thus being used in this work.

2.5.6. SEISMOSTRUCT 2018

SeismoStruct 2018 is a software of finite element method developed by SeismoSoft Ltd with the main objective to assess the behaviour of structures under seismic loads. The software accounts for both material and geometric nonlinearities in the analyses (SeismoSoft, 2018).

2.5.7. AUTODESK ROBOT STRUCTURAL ANALYSIS PROFESSIONAL

Commonly referred simply as *Robot*, is a finite element method program capable of model, analyse and design various types of structures that also accounts for nonlinearities. The program allows the user to create structures, carry out structural analysis, verify obtained results, perform code check calculations of structural members and to prepare documentation for a calculated and designed structure.

3

INFLUENCE OF DESIGN AND REQUIREMENTS ON THE PROBABILITY OF COLLAPSE OF RC FRAME STRUCTURES

3.1. PROBLEM

The design of a structure, in particular, a moment resisting frame in RC, takes advantage of well thought conceptual design, and its design is governed by earthquake resistance, the structural layout is the key factor that determines the seismic performance of it. Damage in strong earthquakes shows that, all other design conditions being the same (design code, computational methods and tools, professional skill and design effort), irregular and geometrically complex structures perform on average worse than simple and regular ones as (Fardis, 2009) states.

The challenge here is to assess and validate the proposed requirements in an actual structure. In order to obtain results with significance in addition to the exploratory character of this work, which attempts to validate the requirements previously stated through the analysis of eight buildings designed accordingly to them

3.2. METHODOLOGY

The case study structures are residential 10-storey (i.e. 28.5 metres) reinforced concrete buildings. The buildings have a moment resisting frame structure with a 18x21 m² plan divided in 3 bays in both horizontal dimensions with 6 and 7m span.

The structure falls is classified as category 4, meaning is projected for an expected lifespan of 50 years (EN 1990/Table 2.1) and is classified as importance class II per EN 1998-1/Table 4.3.

For seismic design purposes the buildings were considered to be located in Lisbon, Portugal, and in Abruzzo, Italy, both on a type D soil and constructed with concrete from the C30/37 class and S400 rebar, the class concrete being chosen because of its common use for residential buildings and the rebar class because a *weaker* class would allow a more detailed strength characterization of the reinforced sections.

The aim of the study was to evaluate the effect of EC8 design demands in seismic risk. Being this an exploratory study, it was decided to not include irregularities (both in height and in plan) as it would introduce extra variables and potential bias in the results.

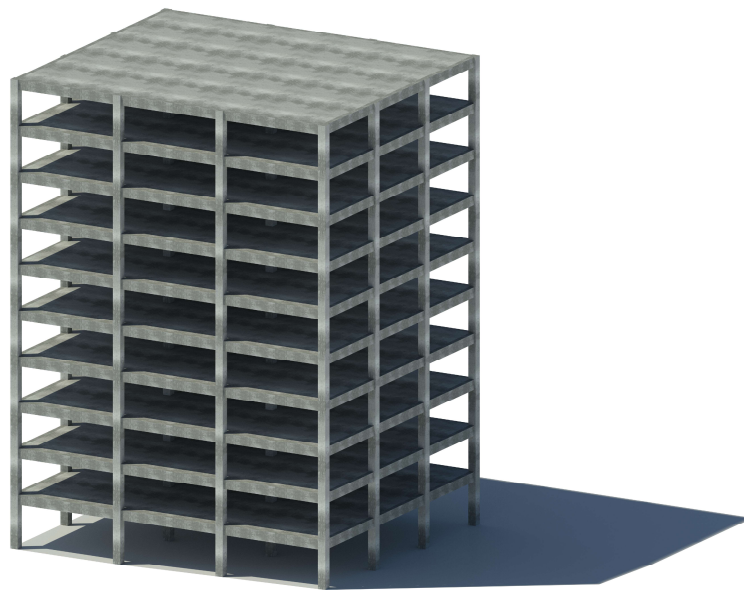


Figure 3.1 – Rendering of the building in study

Three ductility classes were considered for new structures: DCL, DCI and DCM; DCL corresponding to low seismicity class where the overstrength capacity is taken into account, whilst the deformation and energy dissipation capacity are disregarded, this DC corresponds to the DCL in current code; DCM is the more demanding DC where its local overstrength, local deformation and local energy dissipation capacity are taken into account consequently the ability of the structure to form a global plastic mechanism at Significant Damage limit state, corresponding in great degree to the present DCM; evidently, DCI is an **intermediate** DC where the local overstrength, the local deformation and the local energy dissipation capacity are taken into account. Control of global plastic mechanisms is made through limitation of drift and of second order effects.

Regarding the first ductility class, the design code sets the behaviour factor to 1.5 regardless of structural type, regularity and some particular global behaviour aspects. For DCI and DCM the behaviour factor is the multiplication of three factors:

- i. Factor accounting for overstrength due to the redistribution of seismic action effects in redundant structures – q_R ;
- ii. Factor accounting for the deformation capacity and energy dissipation capacity – q_D ;
- iii. Factor accounting for overstrength due to all other sources – q_S .

For the structure in study the behaviour factor amounts 2.5 for DCI and 3.9 for DCL.

Capacity design is a fundamental principle in Eurocode 8, thus, in DCI and DCM, structural elements must be designed on the basis of its equilibrium under the effects of the load in the seismic design situation and the equilibrium of the end moments corresponding to plastic-hinge formation, either for beams or columns – shear capacity design. In DCM, for frame-buildings, moment capacity design must also be applied, in order to avoid formation of a soft-storey plastic mechanism, employing Eq. (3.9) – regarding the sum of the resisting bending moments of the beams, two scenarios were taken into account: the simple resisting bending moment of the beam and the resisting bending moment accounting the **slab participation**, meaning that the slab reinforcement parallel to the beam within an effective width ($b_{eff,CD}$) would account for it; this effective width was defined as up to one-quarter of the beam's clear span, or up to mid-distance to the nearest parallel beam, or to the edge of the slab, whichever is smallest.

After forces, the other component of the capacity design are the geometrical requirements and reinforcement detailing, especially on the dissipative - critical zones. Below are presented the requirements necessary for the development of this work. No rules regarding eccentricities and non-symmetrical reinforcement are expressed since it goes beyond the scope of it.

Table 3.1 - Geometrical and reinforcement requirements for DCI and DCM

Criteria		DCI	DCM
Beams	Critical length, l_{cr}	depth of the beam, h	
	Minimum longitudinal reinforcement ratio of the tension zone, ρ_{min}	$0.5 \cdot f_{ctm} / f_{yk}$	
	Maximum longitudinal reinforcement ratio of the tension zone*, ρ_{max}	2.5%	$\rho' + 1.0\%$
	Minimum diameter of stirrups, d_{bw}	6mm	
	Maximum spacing of stirrups on critical length, s	$\min(h/4; 30d_{bw}; 12d_{bl})$	$\min(h/4; 24d_{bw}; 8d_{bl})$
	Minimum cross-sectional dimension	200mm	250mm
Columns	Critical length, l_{cr}	$\max(h_c; l_c/6; 0.45m)$	
	Minimum longitudinal reinforcement ratio	1%	
	Maximum longitudinal reinforcement ratio	4%	
	Minimum diameter of the longitudinal bars	12mm	
	Maximum distance between consecutive longitudinal bars engaged by hoops	250mm	200mm
	Maximum spacing of stirrups on critical length, s	$\min(b_d/2; 200mm; 9d_{bL,min})$	$\min(b_d/2; 175mm; 8d_{bL,min})$
	Minimum value of mechanical volumetric ratio of confining hoops within the critical regions, ω_{wd}	0.05	0.08
	Maximum normalised axial force, v_d	0.65	0.55

*depends on the concrete and steel grade, value given for C30/37 and S400

3.3.DEVELOPMENT OF ROBOT MODELS

An initial elastic finite element model of the structures was created in Robot to estimate the stresses caused by the vertical loads (i.e. permanent and live loads) and the horizontal loads used for seismic design. In addition, these models were later used to perform an equivalent lateral force analysis and compute the seismic loads with the support of a spreadsheet.

3.3.1.MODELLING PRINCIPLES

The structural models were developed to comply with all the requirements of EN 1998-1/4.3.1, their basic characteristics are listed below:

- All elements were modelled as bar elements;
- All degrees of freedom were restricted at the foundation level;
- Masses and moments of inertia of each floor were considered as in the real situation. The vertical loads corresponding to the seismic design situation were calculated and considered as per EN 1998-1/4.3.1(10);
- The effects of cracked elements were considered as in prescribed. The elastic flexural and shear stiffness properties were taken to be one half of the uncracked section, and Poisson's ratio was taken equal to zero, according to EN 1992/3.1.3 as for cracked concrete.
- No infills were considered as elements, just the corresponding loads.

3.3.2.PRE-DESIGN PRINCIPLES

To define the preliminary design of the structures, simplified expressions commonly found in guidelines and textbooks (ACI Committee 318, 2005), were used to calculate an approximate cross-section for the elements.

The initial slab thickness was assumed to be a function of largest span - Eq. (3.1). When applied to the case study structures this yielded a slab thickness of 230mm corresponding to a dead load of 5.75 kNm².

$$t_{slab} = \frac{l_{slab}}{25 \text{ to } 30} = \frac{6.0m}{25 \text{ to } 30} \quad (3.1)$$

For the beams, the height was taken from Eq. (3.2):

$$h_{beam} = \frac{l_{beam}}{10 \text{ to } 12} \quad (3.2)$$

For the first iteration, and even though the buildings have two different spans lengths, the height of the beams was kept at a constant 500mm. The beam width was considered a function of the height through Eq. (3.3):

$$b_{beam} = (0.3 \text{ to } 0.5)h_{beam} \quad (3.3)$$

Three main column sections were considered according to their influence area (i.e. the columns in the centre, in the corners and the ones in the edges). For the quasi-permanent combination, considering only vertical loads, the critical columns were the ones in the centre on ground floor. This one was used to estimate the area of the columns cross-section through Eq. (3.4).

$$A_{column} = \frac{N_{qpc}}{0.5f_{cd}} \quad (3.4)$$

These assumptions were used for the creation of the model in the first iteration of the buildings in order to estimate forces to feed to Matlab scripts and not used as starting point for the automatic design of cross-sections; the following iterations used the sections outputted by the scripts.

3.3.3.VERTICAL ACTIONS

For the vertical actions, both the permanent loads and the live load were considered. The infills load and an eventual staircase well were not considered explicitly; however, it should be noted the added permanent load value chosen was an upper value.

The first permanent load considered was the self-weight for the columns, beams and slabs. The total weight was automatically accounted by the software from the density of reinforced concrete. An additional permanent load of 3.5kN/m² was included following the recommendations of EN 1991-1/6.3.1.2.

Being residential buildings, EN 1991-1/Table 6.1 classifies the case study structures in category A. Thus, a uniformly distributed live load of 2kN/m² has to be included. In seismic design situation this must be reduced by a factor ψ_2 equal to 0.3 (EN 1990/Table A.1.1). The roof was considered H category (i.e. non-accessible rooftop except for maintenance) reducing the characteristic variable-load to 1 kN/m² and the ψ_2 factor to zero.

The floor masses and moments of inertia were determined considering the possibility of the variable-loads not being present over the entire structure during the seismic event and for the reduced

participation of these masses in the motion of the structure, multiplying to the aforementioned factor ψ_2 another one ϕ , defined in EN1998-1/Table 4.2, as 1 to roof levels and 0.5 to the other storeys.

Table 3.2- Vertical loads

Description	Value	Reference
Dead load [kN/m ³]	25	EN 1991-1/Table A.1
Permanent load [kN/m ²]	3.5	EN 1991-1/Table 6.1 EN 1991-1/6.3.1.2
Live load – stories [kN/m ²]	2	EN 1991-1/Table 6.2
Live load – roof [kN/m ²]	1	EN 1991-1/Table 6.10

Table 3.3 - ψ factors

Description	ψ factor	Reference
Dead load	$\psi_0 = 0.7$	EN 1990/Table A1.1
Permanent load	$\psi_1 = 0.5$	
Live load - stories	$\psi_2 = 0.3$	
Live load – roof	$\psi_0 = \psi_1 = \psi_2 = 0$	

Table 3.4 - ϕ factor to compute ψ_{Ei}

Type of variable action	Storey	ϕ	Reference
Category A Habitational Buildings	Roof	1.0	EN 1998-1/Table 4.2
	Storeys	Correlated occupancies	
		Independent occupied	
		0.5	

3.3.4. SEISMIC ACTIONS, EQUIVALENT LATERAL FORCE ANALYSIS AND BEHAVIOUR FACTOR

An equivalent lateral force method was performed according to EN1998/4.3.3.2. For each direction the fundamental period of vibration was estimated from the numerical model. Seismic loads are applied to the structure through lateral loads computed from the floor masses and the pseudo-spectral acceleration as prescribed in Eurocode 8 (EN 1998-1/3.2.2.2). The distribution of the horizontal forces along the height was determined according to the following expression:

$$F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_i \cdot m_i} \quad (3.5)$$

where m_i are the storey masses and z_i are the heights of the masses.

The eccentricity of the loads was considered through a torsional moment on the floor computed with the square root of the sum of the squares rule (*SRSS*) of the moments created from the orthogonal lateral loads with an eccentricity of 5% of the perpendicular length size, as prescribed.

It is worth noting, several iterations were considered during the design stage in order to obtain an optimized structural system and minimize the concrete sections and rebar, thus, several Robot models were designed for the same ductility class. In the below table can be found the values to characterize the seismic action for the type 1 seismic event in Lisbon for the DCI. Further results may be found in the Annex A.

Table 3.5 - Horizontal forces for seismic action, Lisbon, Portugal - Type 1 DCI

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			376			382
9			352			358
8			328			333
7			303			307
6	1.90	2344	262	1.87	2381	266
5			224			228
4			189			192
3			149			151
2			104			105
1			57			57

Table 3.6 - Torsional moments for seismic action, Lisbon, Portugal - Type 1 DCI

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			338	401	525
9			317	376	492
8			295	350	458
7			272	323	422
6	1.05	0.90	236	279	366
5			202	239	313
4			170	202	264
3			134	159	207
2			93	111	145
1			51	60	79

3.3.4.1. PORTUGAL

Being an importance II building the importance factor (γ_I) amounts to 1. In order to compute horizontal load for each earthquake type let us take the reference peak ground acceleration (a_{gR}), the three period demarcations (T_B , T_C and T_D), the vibration period of a linear single-degree-of-freedom (SDOF) system and a soil factor (S) to estimate the spectral acceleration.

The seismic zone was defined matching to the National Annex 3.2.1 of the Portuguese code.

The values taken are as follows:

Table 3.7 – Horizontal Elastic Ground Acceleration Response Spectrum Data

Soil type D – Lisbon, Portugal		
Earthquake	Type 1	Type 2
Seismic Zone	1.3	2.3
$a_g = a_{gR} \gamma_I$ [m/s ²]	1.5	1.7
$S = f(S_{max})$	1.83	1.77
T_B [s]	0.1	0.1
T_C [s]	0.6	0.25
T_D [s]	2.0	2.0

The design spectra are computed from the elastic spectrum (EN 1998-1/3.2.2.5) reduced through the behaviour factor (q).

The behaviour factor purpose being to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, and is relative to the material, the structural system and the design procedure, it was defined according to the provisions here studied.

For ductility class 1 the design code sets the behaviour factor to 1.5 regardless of structural type, regularity and some particular global behaviour aspects. The modifications here studied define for multi-storey, multi-bay moment resisting frame the resulted behaviour factor, being 2.5 and 3.9 for DCI and DCM respectively. For the elastic response spectrum, EN 1998-1/3.2.2.5(4) defines a lower bound factor for the horizontal design spectrum (β).

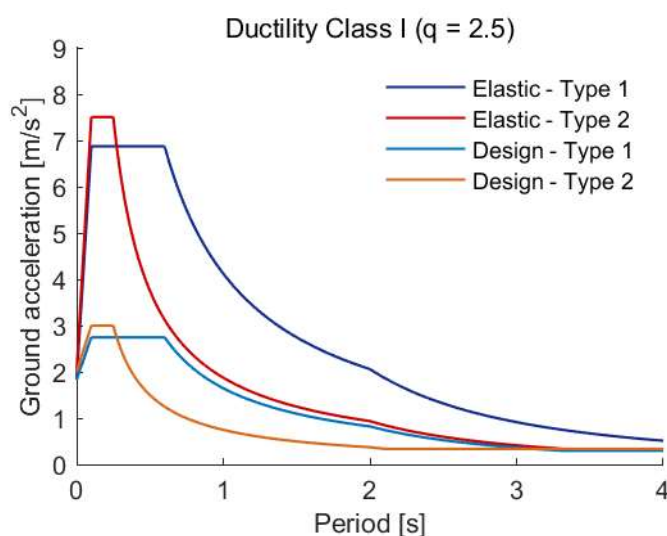


Figure 3.2 - Horizontal Spectrum Lisbon, Portugal - Soil type D

Worth of mention that no vertical seismic action was considered according to EN 1998-1/4.3.3.5.2.

3.3.4.2. ITALY

The determination of the horizontal elastic ground acceleration response spectrum for the Abruzzo region per *Norme Tecniche per le Costruzioni* - the Italian national construction code - is very similar to the previous, since it complies with the Eurocode provisions, having some differences regarding the determination of the constants:

- Ground acceleration, a_g ; maximum spectral amplification factor, F_o ; and the beginning period of the constant velocity in the horizontal acceleration spectrum, T_C^* are sampled values around the country in 10751 different locations for 9 different return periods;
- The corner periods are computed using the previous factors;
- Soil parameter, S – is the product of two other parameters, for taking into account the effect of stratigraphy and topography amplification;
- Subsoil parameter, C_C – function of geography and T_C^* ;

The values obtained to compute the spectrum are as follows:

Table 3.8 - Horizontal Elastic Ground Acceleration Response Spectrum Data

Soil type D – Abruzzo, Italy	
a_g [m/s ²]	2.61
F_o	2.364
T_C^*	0.346
$C_C = f(T_C^*)$	2.13
S_S	1.46
S_T	1
$S = f(S_S, S_T)$	1.83
$T_B = f(T_C)$ [s]	0.1
$T_C = f(T_C^*, C_C)$ [s]	0.6
$T_D = f(a_g)$ [s]	2.0

Yielding the following:

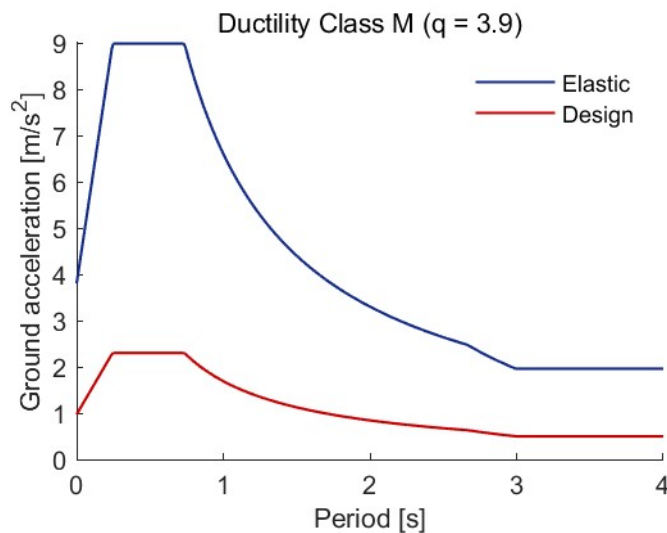


Figure 3.3 - Horizontal Spectrum Abruzzo, Italy - Soil type D

For the behaviour factor the same values were adopted.

3.3.5.P-Δ EFFECTS

The P-Δ effects (second-order effects) were evaluated according to the interstorey drift sensitivity coefficient, presented in EN 1998/4.4.2.2:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h} \quad (3.6)$$

Where h is the interstorey height, P_{tot} and V_{tot} are the total gravity load and the total seismic shear at the storey; d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements at the top and bottom storey under consideration. This coefficient has the following boundaries:

- It shall not exceed 0.3;
- If between 0.1 and 0.2 the P-Δ effects may approximately be taken into account by multiplying the seismic action effects by a factor correlated to the coefficient;
- If less than 0.1 the second-order effects shall be disregarded.

This verification was made for the seismic design situation.

3.3.6. LOAD COMBINATIONS

Eurocode requires that two limit states must be verified to ensure the safety and good performance of a structure: Serviceability Limit State (*SLS*) and Ultimate Limit State (*ULS*). The former refers to the conditions under which a building is still considered usable, regarding maximum deflections, vibrations and cracking, those from the structural perspective. The latter refers to structural integrity as a whole and requires more demanding combinations to be verified in order to comply with the norm.

The Eurocode foresees the use of the *verification by the partial factor method*, and enlists three combinations of actions for the ULS, namely:

- Persistent or transient design situations - Eq. (3.7);
- Accidental design situations;
- Seismic design situations - Eq. (3.8).

In this work were only used the first and the last and none of the *SLS* combinations, since the last was assumed that would never compromise the design procedure. and to proper design on the limit of the code herein study. The equations presented were adapted from EN 1990/6.4.3 to the case in study.

$$\sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i \geq 2} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \quad (3.7)$$

$$\sum_{j \geq 1} G_{k,j} + A_{Ed} + \sum_{i \geq 2} \psi_{2,i} \cdot Q_{k,i} \quad (3.8)$$

3.4. DEVELOPMENT OF MATLAB TOOLS

In the context of this work, several Matlab scripts were developed, from the automatic design of the structures to the post-processing of the models results, being those for design the bulk of this work.

3.4.1. DESIGN CAPABILITIES AND POSSIBILITIES

For the Matlab script developed for design purposes, a straightforward approach was taken: input three CSV files exported from Robot – nodes coordinates, elements connectivity and forces on each element (axial, shear and bending moment) – materials and concrete cover. The script was organized to iteratively find the optimal cross-section considering all the load combinations upon each element considering the conditions imposed by Eurocodes. A total of four scripts were developed in order to perform the design, one for each ductility class plus an extra one for the third ductility class that accounts with slab participation – *DC#frameDesigner*.

The approach was to design each element (beam or column) individually on the global context, meaning that the element would be designed in order to withstand the forces computed by the Robot complying the provisions from both Eurocode 2 and Eurocode 8 but considering the geometry of the connecting elements and the resisting forces in the connection.

3.4.1.1. BEAMS

A current beam and as stated in Eurocode is an horizontal element with an axial force lower than 10% of the compressive concrete strength and must be designed to withstand bending moment and shear

through the vertical plane of the cross-section. On this premise, the script would assume a starting section of 200 by 250 mm² and evaluate the needed reinforcement, being said that the solutions outputted by the program were always with symmetric reinforcement on top and bottom. If the solution of reinforcement fitted in the section and the resisting bending moment was superior to the needed it would proceed to the next step, otherwise another iteration would be required with a bigger section or reinforcement area. Worth of note that the section dimensions would be at maximum of a 0.8 ratio (width over height), criteria that if not required would force another iteration, and that the starting section and reinforcement patterns available could be chosen previously to match geometric constraints.

For the stirrups design, it would compute firstly the minimum required by the code and its resisting value, compute the resisting value of the concrete alone and compare with action, if needed, regarding strength or minimum requirements, would compute a solution with more shear strength.

Either for stirrups and longitudinal rebars, maximum and minimum steel areas/reinforcement ratios were always implemented (for all ductility classes) and if not fulfilled would require another iteration.

3.4.1.2. COLUMNS

Similarly, the column design would start with a square section of 200 by 200 mm² or if given, another square section and then compute the needed reinforcement based on (J. D'Arga e Lima, Vitor Monteiro, & Mun, 1997) abacus of evenly distributed reinforced on the four sides of it. Analogously, it would verify minimum and maximum ratios, and the possibility of a reinforcement solution fit.

To compute the hoops, a similar approach was programmed respecting the rules specified for columns.

3.4.1.3. DUCTILITY CLASS M WITH PARTICIPATION SLAB SCRIPT – DCMFRAMEDESIGNERWSLAB

In order to exemplify the full designing process, the script developed to design accordingly the third DC will be described since it's the one with more designing steps and the others may be considered as sub-parts of these. The full script can be found at Annex B.

1. To initiate the program 10 inputs must be given:
 - *buildingName* – the directory of the input spreadsheets from Robot Structural Analysis;
 - *folder* – the destination directory of the output results;
 - *fck* – the characteristic compressive strength of concrete in MPa;
 - *fyk* – characteristic yield stress of steel in MPa;
 - *cover* – the concrete cover;
 - *seismicVerticalLoadCase* – the number of the load combination in Robot with only the vertical loads of the combination for the seismic design situation;
 - *seismicCases* – the numbers of the load combinations in Robot considered for the seismic design situation;
 - *nonSeismicCases* – the number of the load combination in Robot for persistent or transient design situations;
 - *slabTopReinf* – top reinforcement of the slab in m²/m;
 - *flag* – a Boolean *yes* or *no* to employ a routine to make the reinforcement patterns of the columns of the same vertical alignment well-matched;
2. The first step of the script is to import the spreadsheet's data and sort it, yielding various matrices that resume the data;
3. In the next routine beams are designed individually. First, regarding all load combinations, the longitudinal reinforcement, being the selecting criteria uniquely the resisting bending moment and successively the stirrups chosen by the biggest resisting value. Despite in DCM design the

- stirrups must be defined according capacity design rules, these values are of interest for other analysis, likely the shear reinforcement that is in the non-critical length also being calculated;
4. The columns are designed per vertical alignment. Firstly, for each of them, iteratively will find the more appropriate side length of each column, then, for those dimensions will find the load combination more critical and design the reinforcement appropriately. Like on the beams, the hoops, on and off the critical length, are being designed for further analysis;
 5. Next step is the bending capacity design, meaning, that for each node, in both orthogonal directions, the Eq. (3.9) must be respected:

$$\sum M_{Rc} \geq 1.3 \cdot \sum M_{Rb} \quad (3.9)$$

Where on the left is the sum of the design absolute values of the moments of resistance of the columns framing the joint. The minimum absolute value of column moments of resistance within the range of column axial forces produced by the **seismic design situation** should be used; and on the right is the sum of the design absolute values of the moments of resistance of the beams framing the joint;

For each joint, it will find the beams joining, and for each of them calculate the resisting bending moment considering the participation slab. After that, for each of the seismic load combination, it will do the comparison and find the critical combination and save the value of the needed resisting bending moment to comply with. In another cycle, it will iterate through the nodes that didn't fulfilled and update the columns connected to it;

6. For the shear capacity design, the beams shear forces must be evaluated in the equilibrium in the transverse load acting on it in the seismic design situation and in the equilibrium of the end moments corresponding to plastic hinge formation for positive and negative directions of seismic loading at the ends of the beams;

The final step of the routine is to export CSV file with all the input information for SeismoStruct

3.4.2.WORKFLOW

With the purpose of obtaining an optimal structure, meaning a structure that would work near the maximum capacity and that followed the codes, making representative in the results the influence of it, a few iterations were made using the Matlab script; after obtaining the original forces from the predesigned building - 3.3.2 - and made use of the script, the yielded structure would be assembled in Robot to obtain forces to be computed again in Matlab.

After a pair of iterations, the third one would produce a very similar structure, being this the stop criterion.

3.5.DEVELOPMENT OF SEISMOSSTRUCT MODELS

For the pushover and the time-history analyses the inputs needed for the construction of the model were facilitated by the Matlab scripts since it would output CSV files easily copy and pasted for the interface of the program, hence the only need was for the user to perform this linking task of the process.

For the materials it was chosen the (Mander, Priestley, & Park, 1988) nonlinear concrete model for being regarded the one that characterizes better the material behaviour among the ones available in the software, this is a nonlinear uniaxial model that assumes a constant confinement value provided by the section. For the steel was applied also a nonlinear uniaxial model, proposed by (Menegotto & Pinto, 1973) with some modifications to account for the hardening of the steel.

The sections and geometric definitions like nodes, element connectivity and restraints would come directly from CSV, including the added masses on the nodes to account for the loads not computed automatically by the software, being live loads and other permanent masses (slab and other permanent loads); these were applied as lumped masses on the nodes.

The support conditions were modelled as perfect fixed (all six degrees of freedom restrained) and a rigid diaphragm was considered for all floors, both current assumptions on studies of this nature; refer that the rigid diaphragms assumption implies that the beams never carry axial force, thus non benefiting of the compressive effect of the concrete.

3.6. GROUND MOTION RECORDS

Ground shaking is one of the most influential sources of uncertainty in fragility and vulnerability analysis. With many variables to consider (e.g. tectonic environment, duration, magnitude, frequency content) when selecting records for seismic design and assessment of structures one must be thoughtful during the selection process. For this work the Conditional Spectrum Method (CSM) by (Baker, 2011) was considered for the selection process. To this end a seismic hazard disaggregation analysis (i.e. a distribution of magnitude, distance and epsilon) was performed to the city of Lisbon.

The spectral acceleration at 1s was selected as the intensity measure and 10 levels (from 0.05g to 2g) were considered for the record selection. For each intensity level the first 20 ground motion records that minimized the distance to the conditional spectrum were selected, meaning a total of 200 records were used to perform the nonlinear dynamic analyses.

3.7. RESULTS

The results section is divided on three main parts: the results obtained from the design process, the pushover analysis results and the time-history analysis processing. with the aforementioned

3.7.1. DESIGN

3.7.1.1. MATERIALS

For a decision maker, the cost of a structure is often a decisive condition, or at least, is part of the decision.

The volumes of steel longitudinal rebar, either in beams and columns, were given the length of the element in the model without anchor or lapping lengths being considered. The stirrups length was measured considering only the length necessary to do the concrete confinement, again without anchor lengths; for its quantity, two lengths were considered per element, with different stirrup spacings, making possible estimate the total number of stirrups per element, and consequently its volume.

In order to estimate the total concrete volume, the length of the element was simply multiplied by the area of the cross section and subtracted the volume of steel.

For an estimation of costs and comparison, the prices used were obtained from a real database (CYPE Ingenieros) for construction in Portugal with concrete prepared in a concrete central. The value used for concrete was 90.76 €/m³ and 0.78 €/kg of steel.

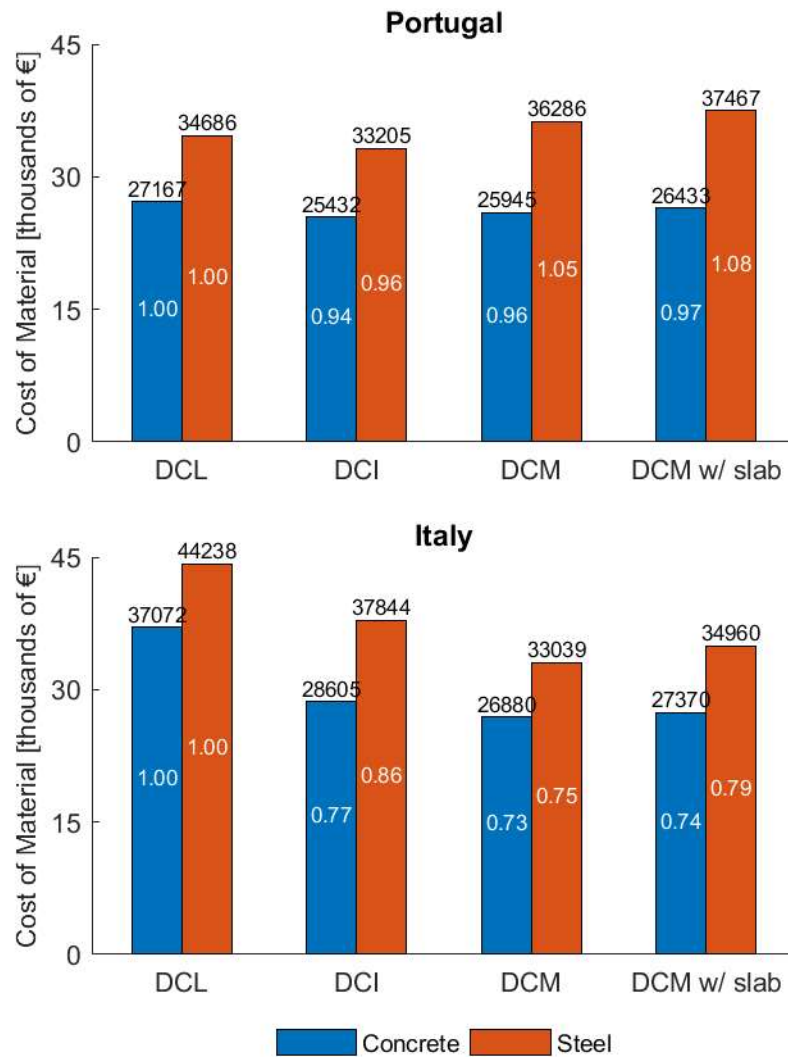


Figure 3.4 - Materials cost comparison

The value in the bars is the price normalized to DCL cost, being the top graph the Portuguese buildings and the bottom Italian. Regarding the former, one may conclude that the total cost of a frame building slightly increases progressing on the ductility increasing classes granting that the other set implies the opposite assumption, in addition that total cost of the structure doesn't vary in great value, as (Fardis, 2009) states, for the current code, that the total quantity of steel and concrete is essentially independent of the ductility class adopted for the design.

Table 3.9- Total costs of Materials [€]

Ductility Class	Portugal	Italy
DCL	61853	81310
DCI	58637	66449
DCM	62231	59919
DCM w/ Slab	63900	62330

This is of particular interest even though recent studies regarding the typical construction practices in Portugal has revealed that, on average, structural components account only for one-third of the building's total value, moreover, recent events have indicated that a large portion of the direct losses are due to damage in the non-structural components and not in the structural elements (Martins, Silva, Marques, Crowley, & Delgado, 2016).

3.7.2. PUSHOVER ANALYSIS AND EVIDENCE

Conventional pushover analyses were performed with a triangular load profile to assess the nonlinear characteristics of the models.

The pushover analyses yielded capacity curves in which some evidenced convergence problems that couldn't be surpassed with convergence criteria tuning. Thus, on the resulted curves was applied a locally weighted scatterplot smoothing (Cleveland, 1979) with a variable span accordingly to the case. The following capacity curves, and further results consequently, have only a solid line represented up to a reduction of 20% on the lateral load bearing capacity, this being regarded suffice condition to consider a structure as collapsed (Silva, Crowley, Varum, Pinho, & Sousa, 2014).

3.7.2.1. PORTUGAL

The Portuguese building models produced the following capacity curves.

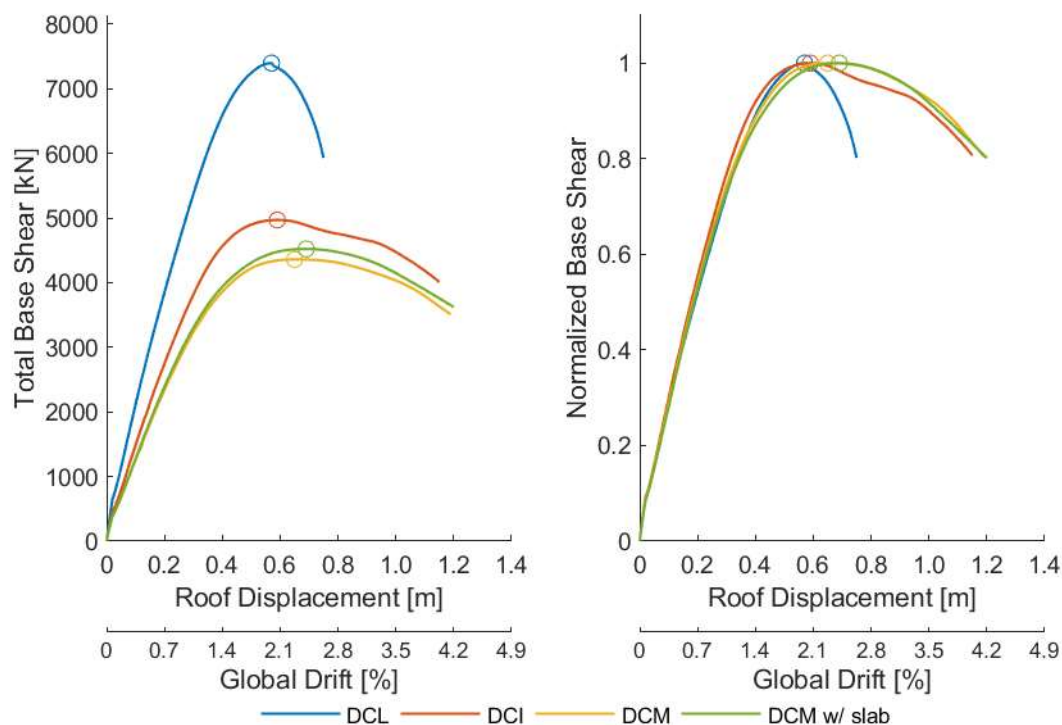


Figure 3.5 - Pushover results for the Lisbon building, original and normalized

The reduction of total base shear goes accordingly of what is expected due to the increase of ductility expected in higher classes as of more sustained ductility after reaching the peak total base shear.

The DCI, DCM and DCM w/ slab yielded a very similar evolution through all the analysis, obtaining similar maximum *Total Base Shear* and residual resistance, still being clear that DCI has less ductility

and a less smooth *evolution*. The DCL model showed the expected more fragile behaviour, reaching a bigger value for the *Total Base Shear* and a more pronounced drop-down after it.

Below can be found the four capacity curves on which were marked evolution steps regarding structure behaviour. A horizontal red line indicates the design total base shear; the dashed line represents the decrease of total base shear from 80 to 50%.

- Development of first plastic-hinge on a beam
- ☆ 10% of beams have developed at least one plastic-hinge
- ✧ 20% of beams have developed at least one plastic-hinge
- Development of first plastic-hinge on a column
- ◁ 10% of columns have developed at least one plastic-hinge
- ▷ 20% of columns have developed at least one plastic-hinge
- First plastic-hinge on a beam to reach ultimate capacity
- ☆ 10% of beams have reached ultimate capacity
- ✧ 20% of beams have reached ultimate capacity
- First plastic-hinge on a column to reach ultimate capacity
- ◁ 10% of columns have reached ultimate capacity
- ▷ 20% of columns have reached ultimate capacity

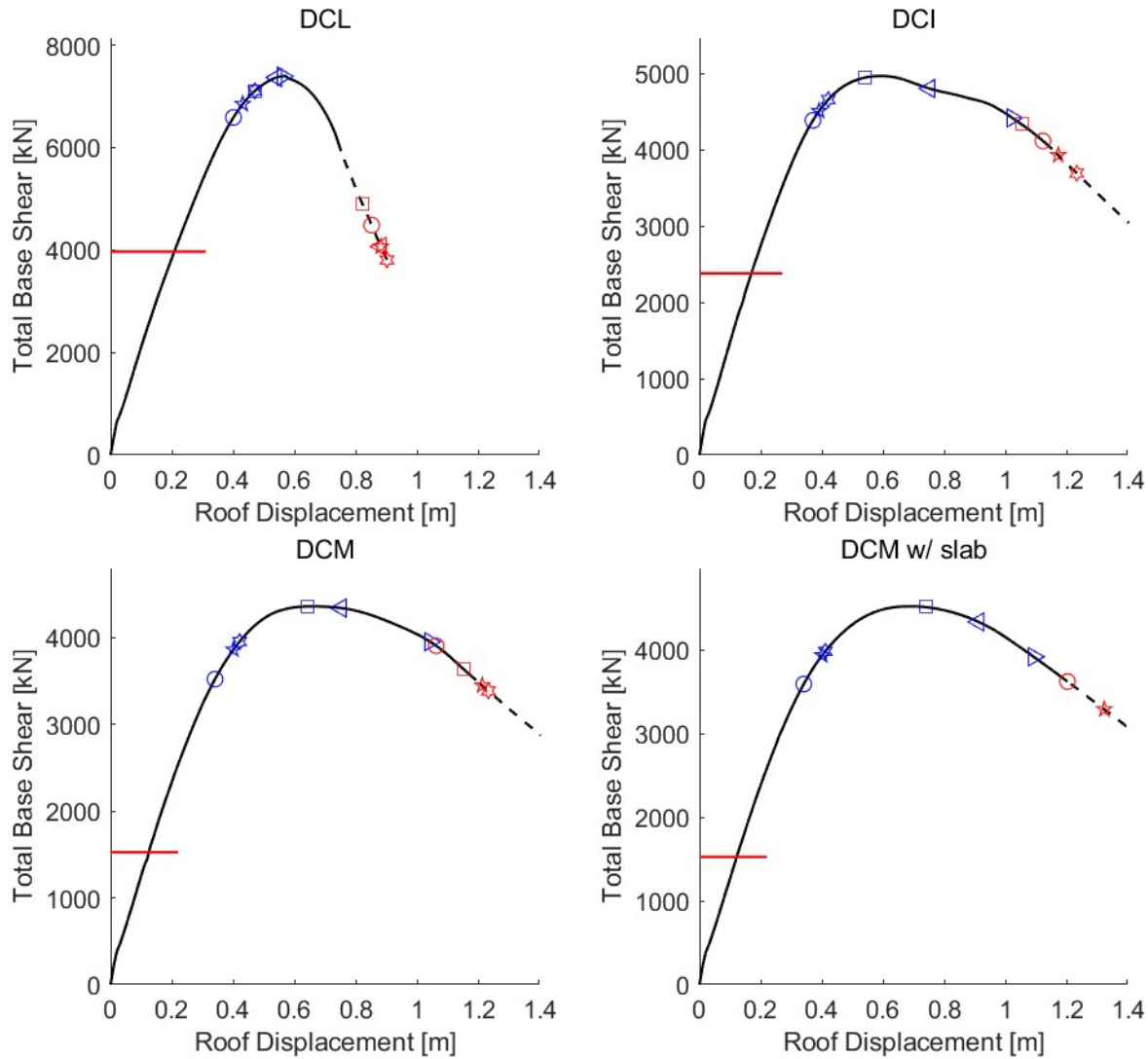
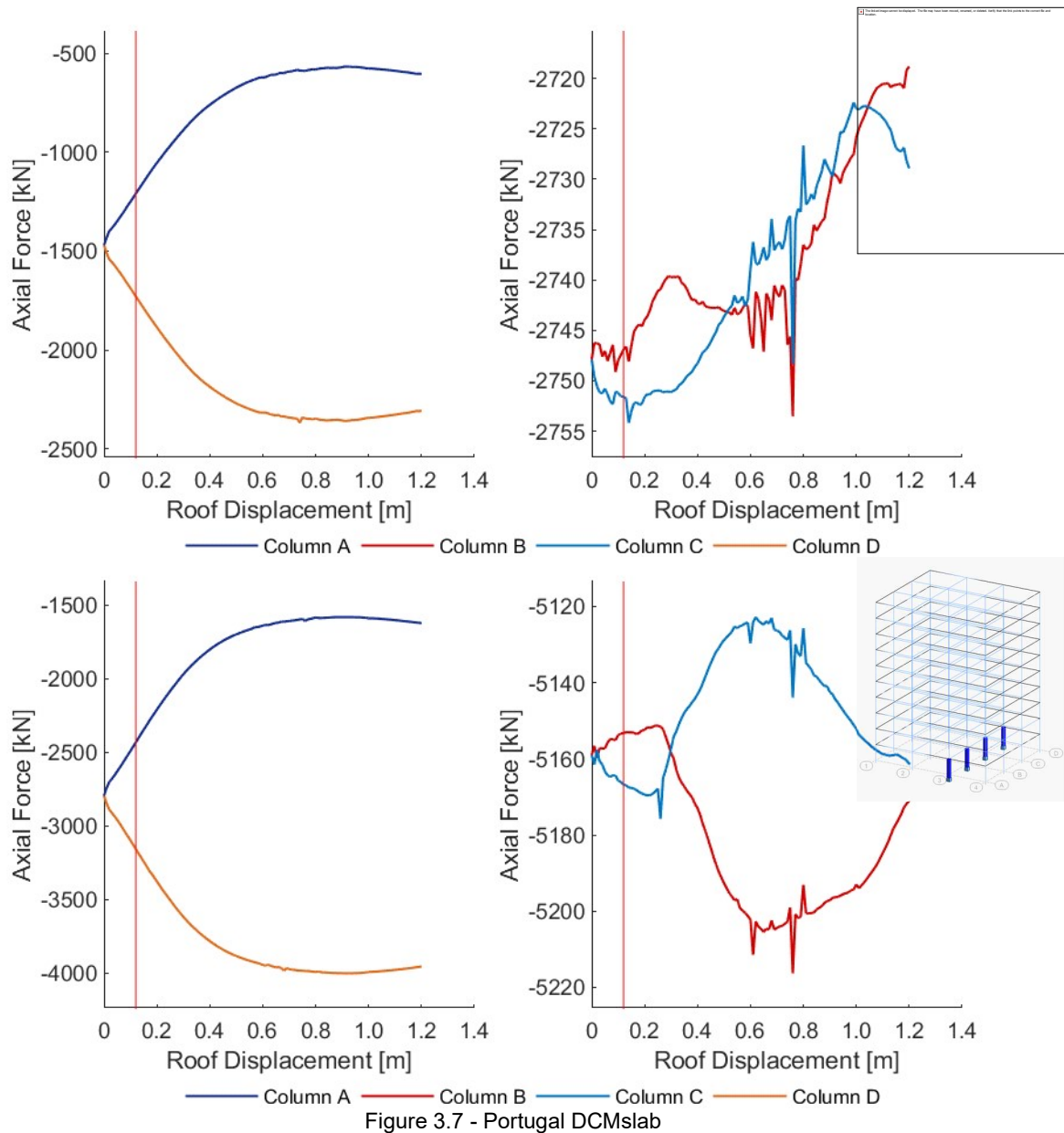


Figure 3.6 - Portugal building capacity curves with marked steps

Is evident in all four that the design procedure was fitting given that after reaching the design shear it still has a reserve of elastic deformation – apparent linear deformation. Still rendering that even when a structure is designed on its bear limits, the redundancy of the codes

The *plastification* process indicators show that although some plastic-hinges are formed below the peak total shear, the vast majority happens after it and, in some way, validates the criteria later used to define collapse of structure.

Regarding the base columns, two effects were evaluated: the evolution of axial forces and the curvature on the base of it against bending moment. In the first, for the outer columns in the both frames - exterior and interior – smoother results are observed contrary to the inside pair of columns, this may be explained by the difficult of numerical convergence in fibre models, albeit this observation, a trend can still be observed. For all DC, the outside columns, show a reserve or potential deformation in the elastic domain, in various degrees regarding the various models, with a great level of similarity to the capacity curves presented before, where for higher ductility smoother progression towards the complete damage criteria.



The interior pair of columns deeming its performance can be divided into two groups: the first two ductility classes where up to the maximum total shear displacement have the expected axial force variation, columns B reducing and columns C increasing, indicative of an overall overturning whereas the other models showed a more hectic performance. The interior pair of columns on the exterior frame keep a similar axial force up to the design base shear and after it are unloaded though the interior pair carries the expected load variation up to a point where it reverses that trend as it is shown in Figure 3.7. Further results can be found in Annex C.

Regarding the moment-rotation relations, these were evaluated using the proposed method in EN 1998-3/A.3.2 – where it is suggested to evaluate the chord rotation of the structural elements which is defined as the angle between the tangent to the axis at the yielding end and the chord connecting that end with the point of contraflexure. The damage limitation state is evaluated to the corresponding capacity given

by the chord rotation at yielding, and the near collapse limit state is represented by the total chord rotation capacity of the element.

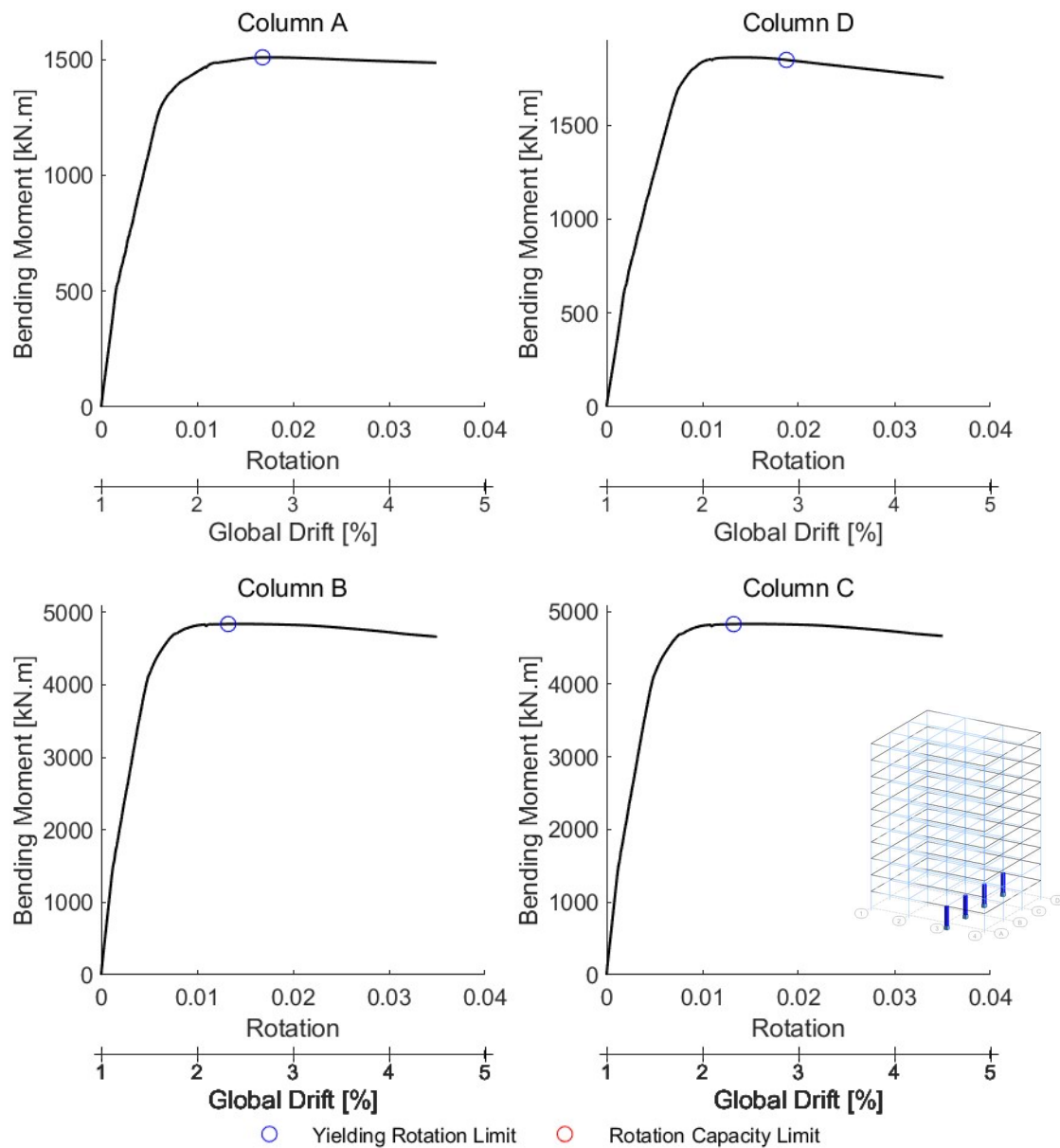


Figure 3.8 - Portugal DCMslab int

Concerning these results, one may state that the overall performance goes as expected, since it follows a smooth curve and on a high ductility construction it doesn't reach the ultimate rotation capacity.

The models for Portuguese DCI and DCM buildings experienced a soft-storey phenomenon on the upper floors, removing significance from the results obtained for this feature.

3.7.2.2. ITALY

Similarly, for the Italian building the following capacity curves were obtained:

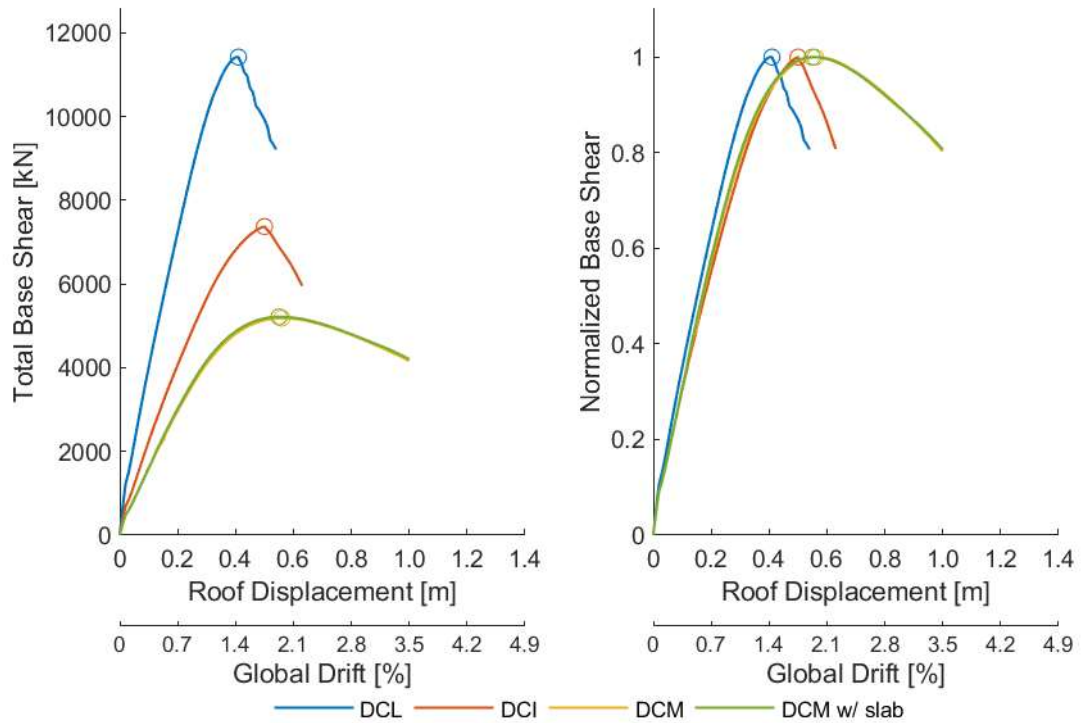


Figure 3.9 - Pushover results for the Abruzzo building, original and normalized

An higher total shear design of the procedure yielded

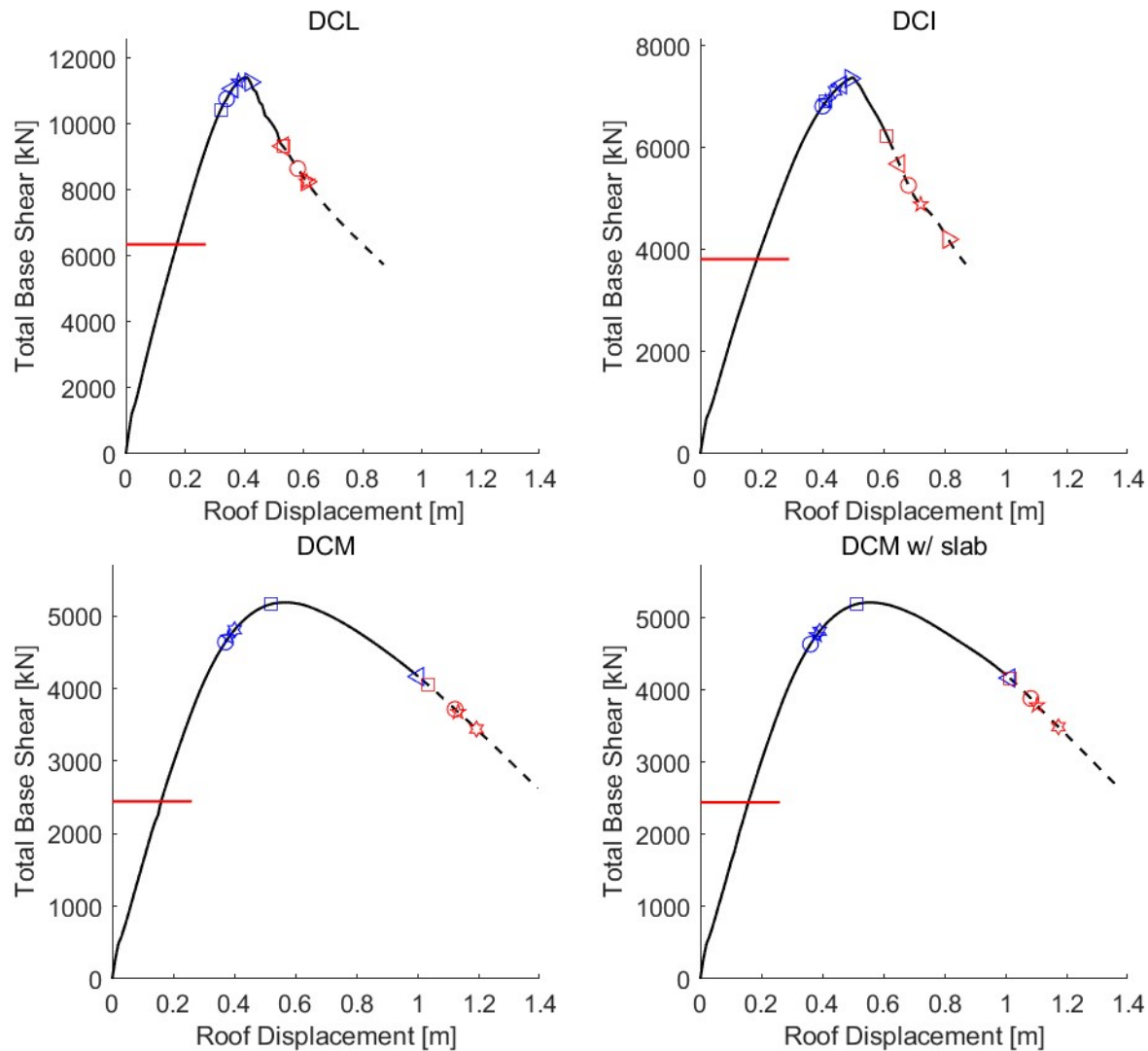
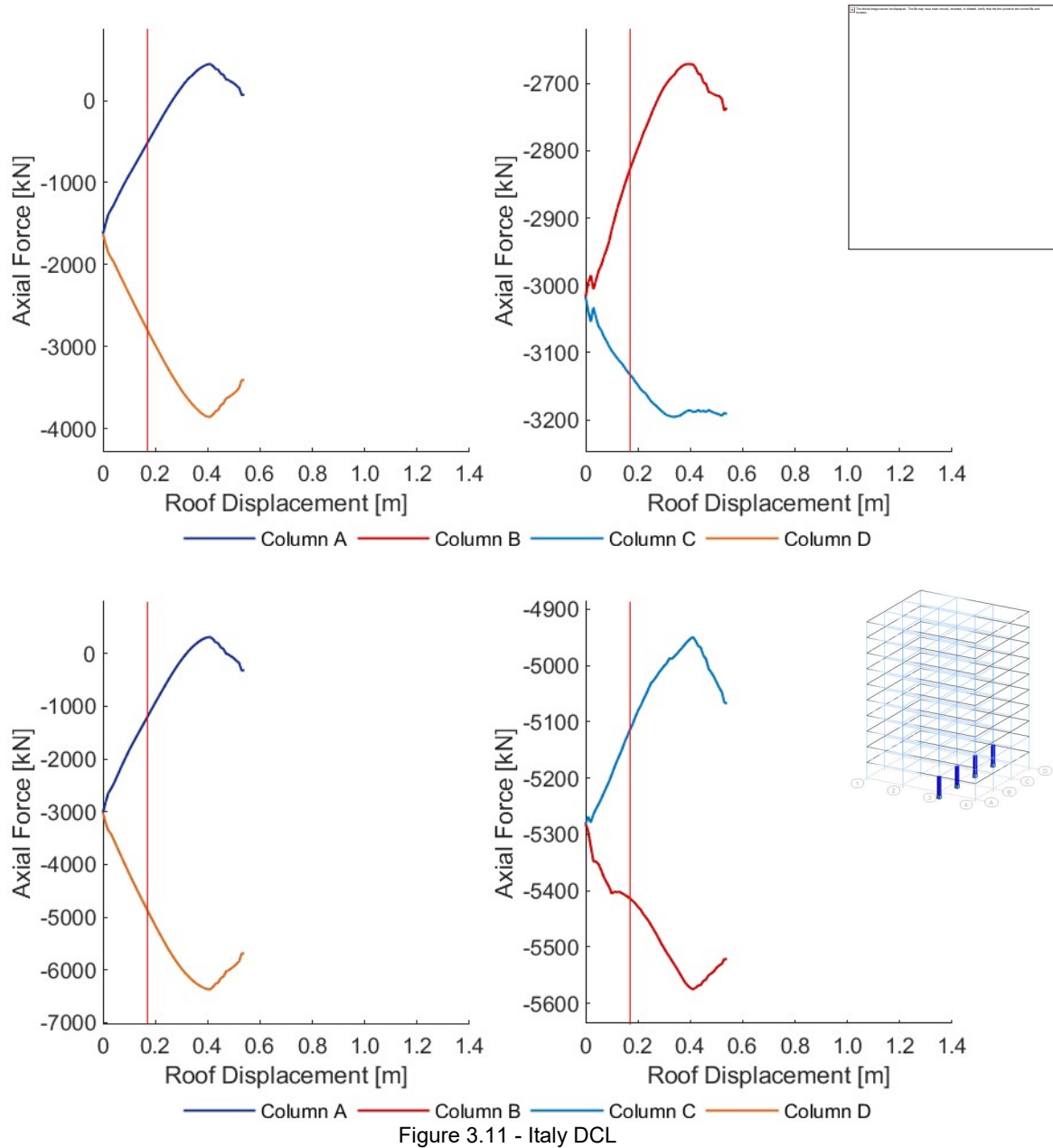


Figure 3.10 - Italy building capacity curves with marked steps

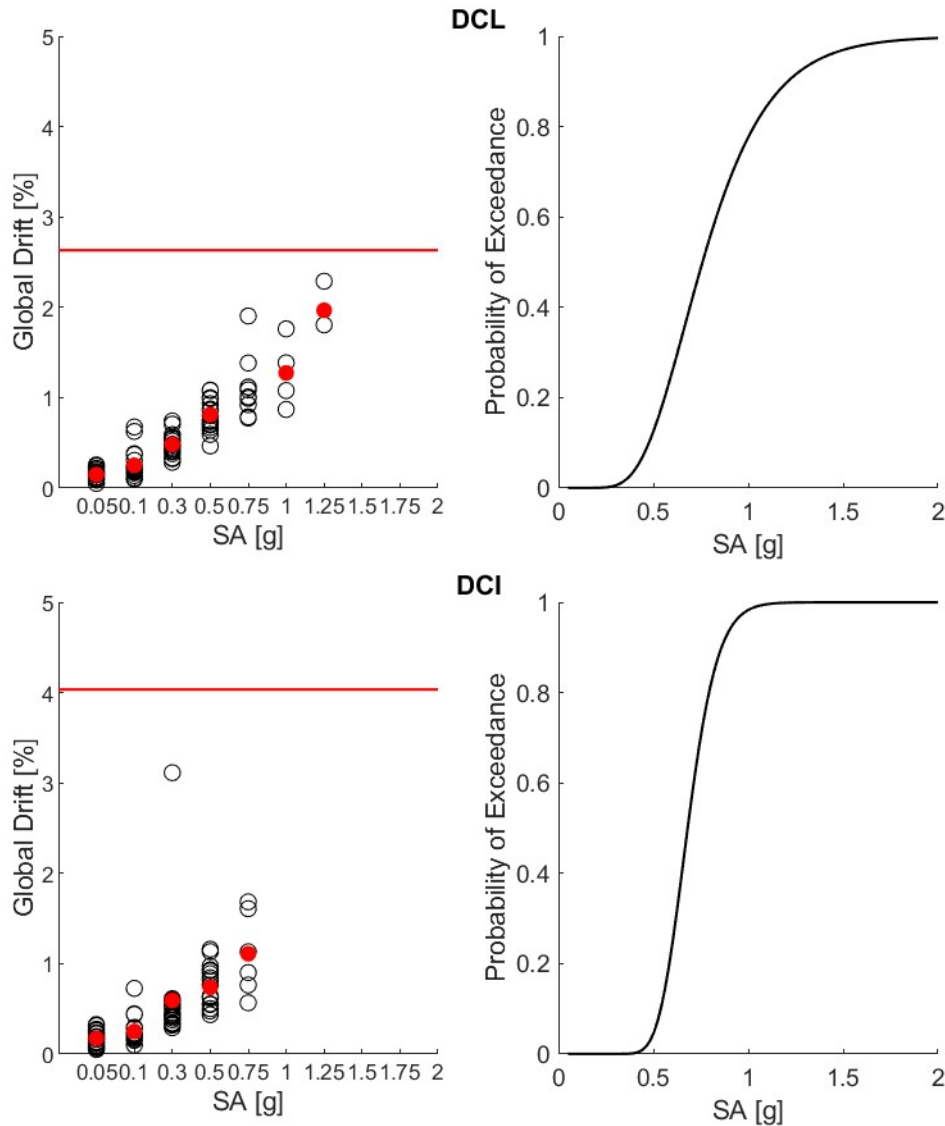
The higher design lateral forces may have deemed a resilient and overall well-behaved structure, given that the axial forces distribution on the base columns load and unload according to the load being applied: a general overturn of the building, compressing the posterior columns and reducing the load on the anterior ones - apart from the model DCM, whereas the middle pair of the exterior frame both reduce their axial load.



Although the performance of axial load distribution seems to be better than the previous models, the Italian models also experienced soft storey phenomena on the DCL and DCI models, although it was not so noticeable in the former, not influencing further results apart from the ones regarding the moment-rotation.

3.7.3. TIME-HISTORY ANALYSIS

The previous analysis allowed to consolidate the criterion of the reduction of 20% on the lateral load bearing capacity as a condition of collapse, being the correspondent global drift deemed as the EDP. Below are presented the maximum global drifts and the probability of exceedance for each DC. On the left, black circles represent the maximum global drift and the red dot represents the mean for that SA, of the models that finished, meaning that the models that didn't finish the length of the record, were considered as collapsed, the horizontal red line represents the global drift threshold for collapse. On the right, the lognormal fitting for the discrete probability of collapses determined from the exceedance on the left.



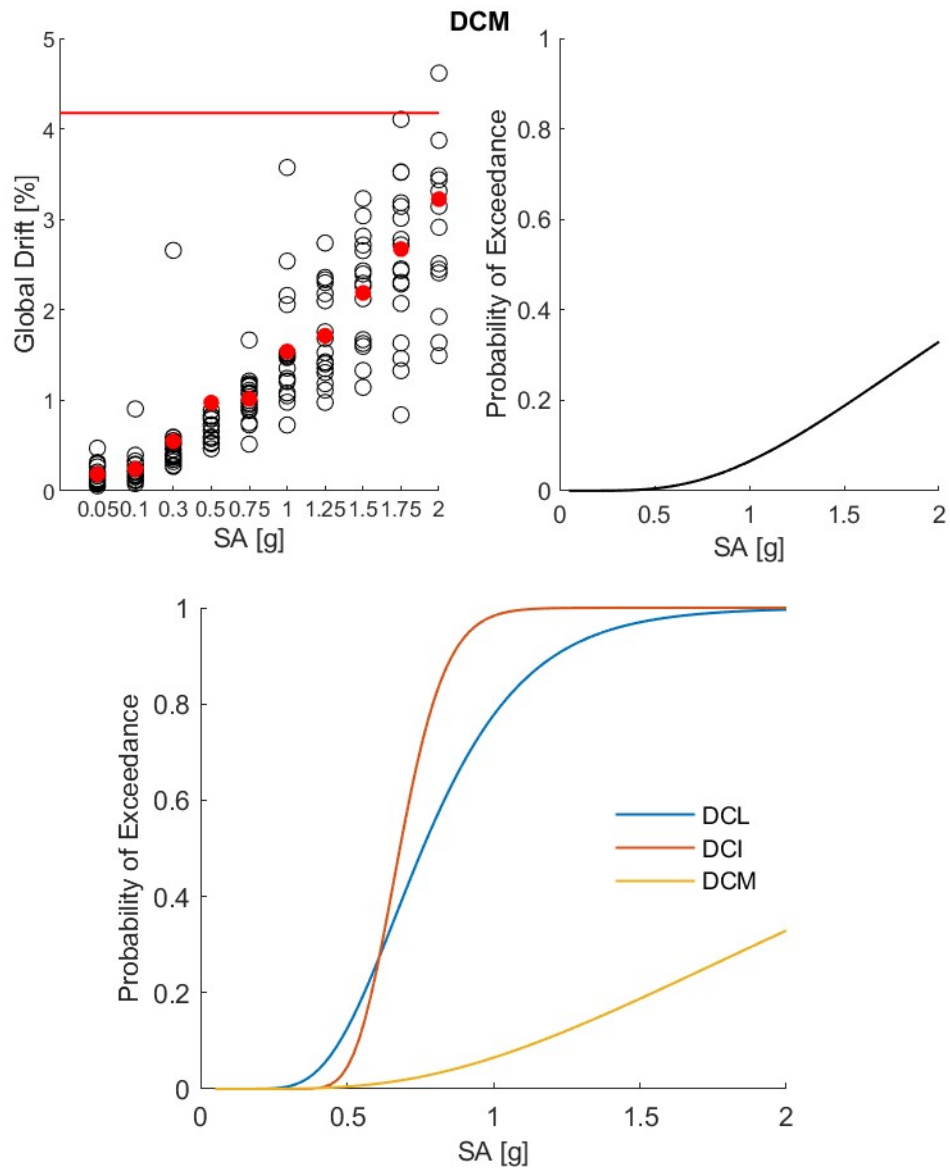
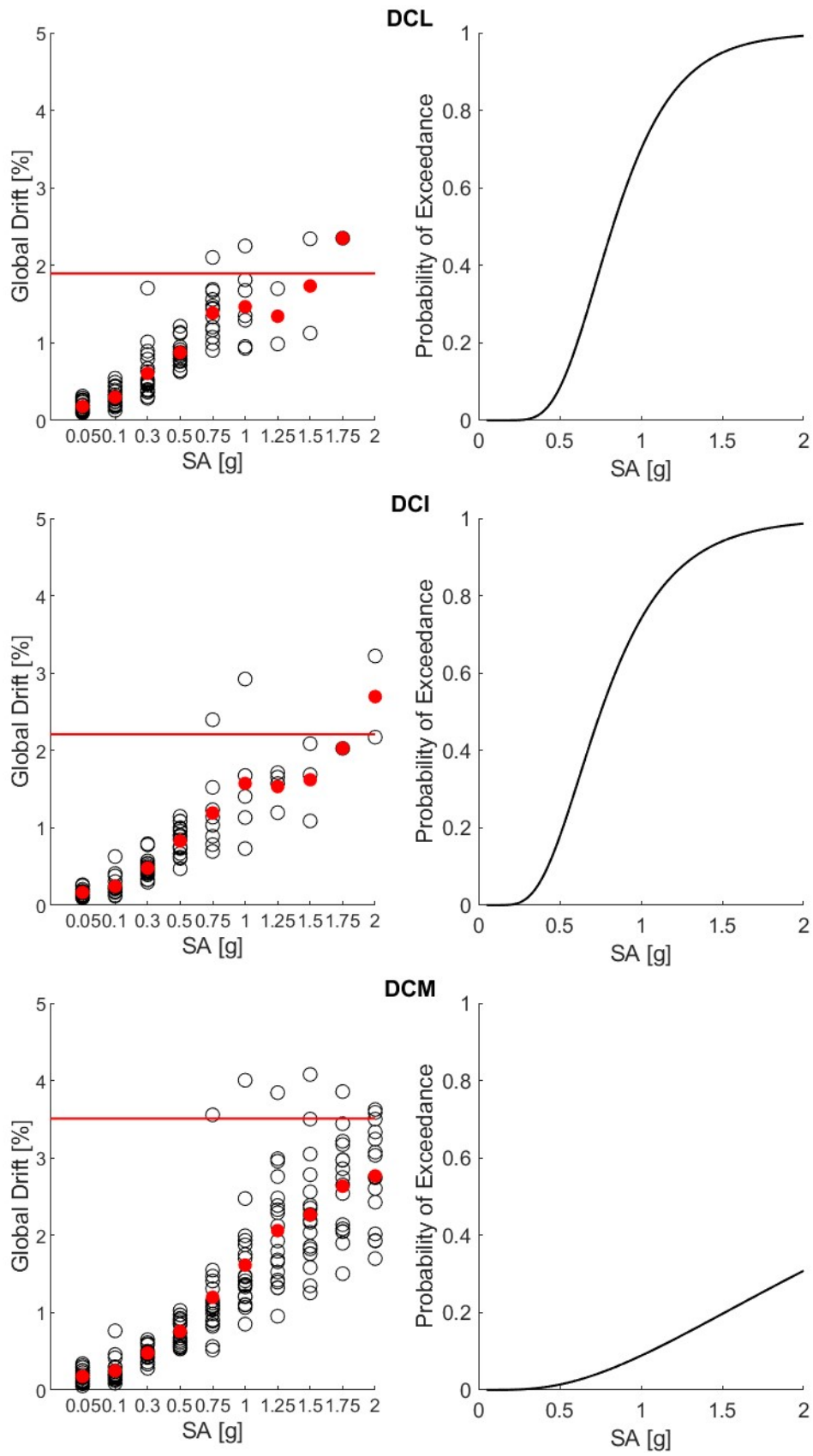


Figure 3.12 – Global drifts and probability of exceedance for Portuguese structures



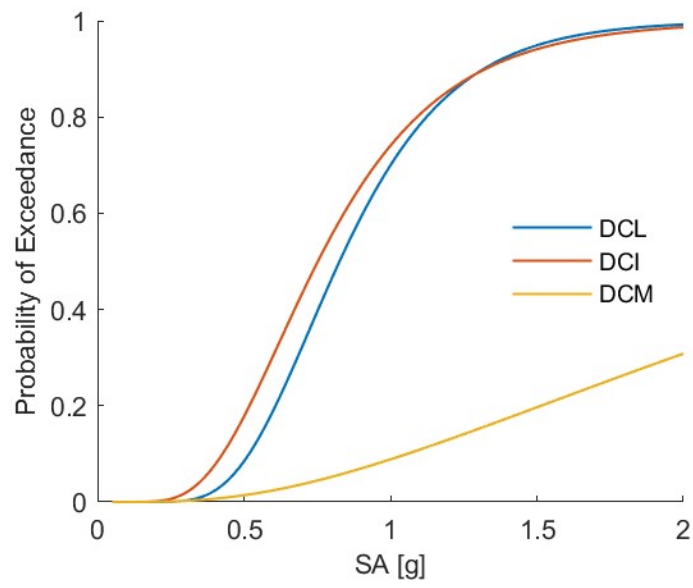


Figure 3.13 – Global drifts and probability of exceedance for Italian structures

3.7.3.1.AVERAGE ANNUAL PROBABILITY OF COLLAPSE

From the previous probabilities of exceedance, the average annual probability of collapse (AAPC) was calculated with the hazard curve for Lisbon.

Table 3.10 – Average Annual Probability of Collapse

Ductility Class	Portugal	Italy
DCL	$3.4 \cdot 10^{-5}$	$2.9 \cdot 10^{-5}$
DCI	$3.6 \cdot 10^{-5}$	$3.8 \cdot 10^{-5}$
DCM	$3.7 \cdot 10^{-6}$	$5.1 \cdot 10^{-6}$

Following the results for the probability of exceedance, the AAPC for the second DC are higher than for DCL, that goes against the expected results. Considering the $5 \cdot 10^{-5}$ probability as an acceptable reference threshold proposed in (Martins, Silva, Bazzurro, & Marques, 2018) the values obtained for the last DC look rather conservative, suggesting an adjustment of the code to obtains a more uniform risk.

4

FINAL REMARKS

4.1. CONCLUSIONS

The objective of this exploratory work was to define and assess the effect of different code provisions on RC frame buildings, this endeavour was accomplished through the investigation of a 10-storey residential building, envisioned in the areas of Lisbon and Abruzzo. Four models were designed per city, one for each ductility class in study, making a total of eight buildings to analyse.

Regarding the design component, one may conclude that the conditions evaluated deemed results as expected by other authors; the total cost of the structural component of a building doesn't vary in great amount across different ductility classes. The observable exception is the Italian DCL building.

The pushover analysis deemed capacity curves that envisioned an improvement of overall performance throughout the ductility classes, even that so slightly in some cases. Is observable a decrease of total base shear and an increase of total displacement on their peaks; the behaviour after it also shows an increase of ductility as expected. Relating its design total base shear and its peak shear, a fair amount of displacement can still be accommodated on an almost linear behaviour, moreover, the process from which plastic deformation occur on its elements, shows that the models only-develop plastic-hinges near the maximum total base shear and the elements that reached the total capacity only happen near the collapse, ultimately leading to it. Regarding the axial forces in base columns, one may observe that a changing from compression to tensile only occurs in DCL buildings – overall more stiff buildings – and it happens for greater displacement, after the maximum total base shear. From the rotation-bending moment results, evaluated by the chord rotation criteria given in EN1998-3, it shows, substantiating previous statements, that base columns don't develop plastic-hinges until later, when collapse is impending.

Two soft-storeys were observed during the analysis and its occurrence is almost imperceptible in these results. Both in DCI models, deeming that the requirements enforced on this DC may not be adequate – an imbalance between behaviour factor and detailing.

Even though some models showed this failure mechanism, the resulted average annual probability of collapse is within the expected values. However, the models for the third ductility class presented very conservative values, suggesting a finer tuning of the DCM requirements.

4.2. IMPROVEMENTS AND FUTURE DEVELOPMENTS

Because knowledge is an on-going process, and the author is aware of his difficulties and imperfections, herein proposes some improvements and suggestions for future works.

On the overall process of structure design, the following is proposed:

- As a replacement of an equivalent lateral force analysis, perform a modal response spectrum analysis, since it's the preferred method for design as per EN1992, and in tall buildings, the higher vibration modes have a greater contribution on the overall response;

- Relatively to the beams design, is proposed to perform a redistribution of the bending moments, to better capture a real design process;
- Improve the single beam design: having the possibility of non-symmetrical reinforcement and more rational cross-sections – ratio width/depth.

Regarding future developments and giving answer to some problems that were found (i.e. soft- storey mechanisms), is suggested to:

- Attempt a fine tuning on DCI requirements and/or design of DCI buildings with capacity design to bending moments;
- Different structures, higher and lower rise, different typology, with irregularities.

REFERENCES

- ACI 318, Building Code Requirements for Structural Concrete, (2005).
- Azevedo, A. F. M. (2003). *Método dos Elementos Finitos*. In.
- B.G.S. (2017, January 25th, 2017). British Geological Survey - Earthquakes: myths and legends. Retrieved from <https://www.bgs.ac.uk/discoveringGeology/hazards/earthquakes/mythsAndLegends.html>
- Baker, J. W. (2011). Conditional Mean Spectrum: Tool for Ground-Motion Selection. *Journal of Structural Engineering*, 137(3), 322-331. doi:10.1061/(asce)st.1943-541x.0000215
- Bommer, J. J., & Acevedo, A. B. (2008). The Use of Real Earthquake Accelerograms as Input to Dynamic Analysis. *Journal of Earthquake Engineering*, 8(sup001), 43-91. doi:10.1080/13632460409350521
- CESMD. (2012). Center for Engineering Strong Motion Data. Retrieved from <https://www.strongmotioncenter.org/index.html>
- Chinwuba Ike, C. (2019). Timoshenko Beam Theory for the Flexural Analysis of Moderately Thick Beams – Variational Formulation, and Closed Form Solution. *TECNICA ITALIANA-Italian Journal of Engineering Science*, 63(1), 34-45. doi:10.18280/ti-ijes.630105
- Chopra, A. K. (2012). *Dynamics of structures: Theory and applications to earthquake engineering*. Englewood Cliffs, N.J.: Prentice Hall.
- Cleveland, W. S. (1979). Robust Locally Weighted Regression and Smoothing Scatterplots. *Journal of the American Statistical Association*, 74(368), 829-836. doi:10.1080/01621459.1979.10481038
- Clough, R. W., & Wilson, E. L. (1999). *Early Finite Element Research at Berkeley*. Paper presented at the Fifth U.S. National Conference on Computational Mechanics.
- Coburn, A., & Spence, R. (2003). *Earthquake protection*: John Wiley & Sons.
- Correia, A., & B. E. Virtuoso, F. (2006). Nonlinear Analysis of Space Frames. doi:10.1007/1-4020-5370-3_107
- CYPE Ingenieros, S. A. Gerador de Preços. Portugal. Retrieved from <http://www.geradordeprecos.info>
- Elnashai, A. S., & Izzuddin, B. A. (1991). Dynamic characteristics and analysis of steel jacket structures with nonlinear flexible joints.
- Fardis, M. N. (2009). General Principles for the Design of Concrete Buildings for Earthquake Resistance. In *Seismic Design, Assessment and Retrofitting of Concrete Buildings* (pp. 1-46). Dordrecht: Springer Netherlands.
- Betão Armado Esforços Normais e de Flexão (REBAP-83), (1997).
- JRC. (2007). JOINT RESEARCH CENTRE - Eurocodes: Building the Future - Time line. Retrieved from <https://eurocodes.jrc.ec.europa.eu/showpage.php?id=12>
- Katsanos, E. I., Sextos, A. G., & Manolis, G. D. (2010). Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective. *Soil Dynamics and Earthquake Engineering*, 30(4), 157-169. doi:10.1016/j.soildyn.2009.10.005
- Kramer, S. L. (1996). *Geotechnical earthquake engineering*. Upper Saddle River, N.J.: Prentice Hall.
- Lopes, M. (2009). *Sismos e Edifícios* (E. ORION Ed.).
- M. Pagani, J., Garcia-Pelaez, R., Gee, K., Johnson, V., Poggi, R., Styron, G., . . . Danciu, D. M. (2018). Global Earthquake Model (GEM) Seismic Hazard Map (version 2018.1 - December 2018). In.
- Mander, J. B., Priestley, M. J. N., & Park, R. (1988). Theoretical Stress-Strain Model for Confined Concrete. *Journal of Structural Engineering*, 114(8), 1804-1826. doi:10.1061/(asce)0733-9445(1988)114:8(1804)
- Martins, L. (2013). *Avaliação das Perdas e Danos Estruturais em Edifícios de Betão Armado Devidos à Acção Sísmica*. Retrieved from
- Martins, L. (2018). *Earthquake damage and loss assessment of reinforced concrete buildings*. (Doctor), University of Porto,
- Martins, L., Silva, V., Bazzurro, P., & Marques, M. (2018). Advances in the derivation of fragility functions for the development of risk-targeted hazard maps. *Engineering Structures*, 173, 669-680. doi:10.1016/j.engstruct.2018.07.028

- Martins, L., Silva, V., Marques, M., Crowley, H., & Delgado, R. (2016). Development and assessment of damage-to-loss models for moment-frame reinforced concrete buildings. *Earthquake Engineering & Structural Dynamics*, 45(5), 797-817. doi:10.1002/eqe.2687
- Matejčková-Farhat, M., & Ároch, R. (2013). Some Remarks on the Choice of Ductility Class for Earthquake-Resistant Steel Structures. *Slovak Journal of Civil Engineering*, 21(3), 1-10. doi:10.2478/sjce-2013-0011
- Menegotto, M., & Pinto, P. E. (1973). Method of analysis for cyclically loaded R.C. plane frames including changes in geometry and non elastic behaviour of elements under combined normal force and bending. *IABSE reports of the working commissions*, 13. doi:10.5169/seals-13741
- Oñate, E. (2010). *Structural Analysis with the Finite Element Method : Linear Statics* (Vol. Vol II: Beams, Plates and Shells). Dordrecht: Springer Netherlands.
- Pacific Earthquake Engineering Research Center. (2000). PEER Strong Motion Database. Retrieved from <https://peer.berkeley.edu/>
- Pasquale, G. D., Orsini, G., & Romeo, R. W. (2005). New Developments in Seismic Risk Assessment in Italy. *Bulletin of Earthquake Engineering*, 3(1), 101-128. doi:10.1007/s10518-005-0202-1
- Pinho, R., Abrantes, M., Magenes, G., & Crowley, H. (2006). *DEVELOPMENT OF SEISMIC VULNERABILITY ASSESSMENT METHODOLOGIES OVER THE PAST 30 YEARS*.
- Ribeiro, R. A. V. (2008). *Análise Pushover de Pontes em Betão Armado*. (MSc), Faculty of Engineering, University of Porto, Porto.
- Sandhu, J. S., Stevens, K. A., & Davies, G. A. O. (1990). A 3-D, co-rotational, curved and twisted beam element. *Computers & Structures*, 35(1), 69-79. doi:10.1016/0045-7949(90)90257-3
- Scott, M. H., Franchin, P., Fenves, G. L., & Filippou, F. C. (2004). Response Sensitivity for Nonlinear Beam–Column Elements. *Journal of Structural Engineering*, 130(9), 1281-1288. doi:10.1061/(asce)0733-9445(2004)130:9(1281)
- SeismoSoft. (2018). SeismoStruct 2018 – A computer program for static and dynamic nonlinear analysis of framed structures.
- Sen, T. K. (2009). *Fundamentals of Seismic Loading on Structures*.
- Silva, V., Crowley, H., Pagani, M., Monelli, D., & Pinho, R. (2013). Development of the OpenQuake engine, the Global Earthquake Model's open-source software for seismic risk assessment. *Natural Hazards*, 72(3), 1409-1427. doi:10.1007/s11069-013-0618-x
- Silva, V., Crowley, H., Varum, H., Pinho, R., & Sousa, L. (2014). Investigation of the characteristics of Portuguese regular moment-frame RC buildings and development of a vulnerability model. *Bulletin of Earthquake Engineering*, 13(5), 1455-1490. doi:10.1007/s10518-014-9669-y
- Zienkiewicz, O. C. T., RL. (1989). *The Finite Element Method* (Vol. Volume1: The basis).

ANNEXES

Annex A SEISMIC DESIGN FORCES

Table A.1 - Horizontal forces for seismic action, Lisbon, Portugal - Type 1 DCL

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			627			637
9			587			597
8			547			556
7			504			512
6	1.90	3906	437	1.87	3968	444
5			374			380
4			315			320
3			248			252
2			173			176
1			94			96

Table A.2 - Torsional moments for seismic action, Lisbon, Portugal - Type 1 DCL

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			564	669	875
9			529	626	820
8			492	583	763
7			454	538	704
6	1.05	0.90	393	466	609
5			336	399	522
4			284	336	440
3			223	264	346
2			156	184	241
1			85	101	132

Table A.3 - Horizontal forces for seismic action, Lisbon, Portugal - Type 2 DCL

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			285			290
9			267			271
8			249			253
7			229			233
6	1.90	1777	199	1.87	1806	202
5			170			173
4			144			146
3			113			115
2			79			80

Table A.4 - Torsional moments for seismic action, Lisbon, Portugal - Type 2 DCL

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			257	304	398
9			241	285	373
8			224	266	347
7			206	245	320
6	1.05	0.90	179	212	277
5			153	181	237
4			129	153	200
3			101	120	157
2			71	84	110
1			39	46	60

Table A.5 - Horizontal forces for seismic action, Lisbon, Portugal - Type 1 DCI

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			376			382
9			352			358
8			328			333
7			303			307
6	1.90	2344	262	1.87	2381	266
5			224			228
4			189			192
3			149			151
2			104			105
1			57			57

Table A.6 - Torsional moments for seismic action, Lisbon, Portugal - Type 1 DCI

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			338	401	525
9			317	376	492
8			295	350	458
7			272	323	422
6	1.05	0.90	236	279	366
5			202	239	313
4			170	202	264
3			134	159	207
2			93	111	145
1			51	60	79

Table A.7 - Horizontal forces for seismic action, Lisbon, Portugal - Type 2 DCI

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			171			174
9			160			163
8			149			152
7			138			140
6	1.90	1066	119	1.87	1083	121
5			102			104
4			86			87
3			68			69
2			47			48

Table A.8 - Torsional moments for seismic action, Lisbon, Portugal - Type 2 DCI

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			154	183	239
9			144	171	224
8			134	159	208
7			124	147	192
6	1.05	0.90	107	127	166
5			92	109	142
4			77	92	120
3			61	72	94
2			42	50	66
1			23	27	36

Table A.9 - Horizontal forces for seismic action, Lisbon, Portugal - Type 1 DCM

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			241			245
9			226			229
8			210			214
7			194			197
6	1.90	1502	168	1.87	1526	171
5			144			146
4			121			123
3			95			97
2			66			68
1			36			37

Table A.10 - Torsional moments for seismic action, Lisbon, Portugal - Type 1 DCM

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			217	257	336
9			203	241	315
8			189	224	294
7			175	207	271
6	1.05	0.90	151	179	234
5			129	153	201
4			109	129	169
3			86	102	133
2			60	71	93
1			33	39	51

Table A.11 - Horizontal forces for seismic action, Lisbon, Portugal - Type 2 DCM

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			147			147
9			138			138
8			128			128
7			118			118
6	1.90	917	103	1.87	917	103
5			88			88
4			74			74
3			58			58
2			41			41

Table A.12 - Torsional moments for seismic action, Lisbon, Portugal - Type 2 DCM

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			132	155	204
9			124	145	191
8			116	135	178
7			107	124	164
6	1.05	0.90	92	108	142
5			79	92	121
4			67	78	102
3			52	61	80
2			37	43	56
1			20	23	31

Table A.13 - Horizontal forces for seismic action, Abruzzo, Italy - DCL

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			1004			1020
9			941			956
8			876			890
7			808			821
6	1.90	6257	700	1.87	6356	711
5			599			608
4			505			513
3			397			403
2			277			281
1			151			154

Table A.14 - Torsional moments for seismic action, Abruzzo, Italy - DCL

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			904	1071	1401
9			847	1003	1313
8			789	935	1223
7			727	862	1127
6	1.05	0.90	630	746	976
5			539	639	836
4			455	539	705
3			357	423	554
2			249	295	386
1			136	161	211

Table A.15 - Horizontal forces for seismic action, Abruzzo, Italy - DCI

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			602			612
9			564			573
8			526			534
7			485			492
6	1.90	3754	420	1.87	3814	426
5			359			365
4			303			308
3			238			242
2			166			169
1			91			92

Table A.16 - Torsional moments for seismic action, Abruzzo, Italy – DCI

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			542	643	841
9			508	602	788
8			473	561	734
7			436	517	676
6	1.05	0.90	378	448	586
5			323	383	501
4			273	323	423
3			214	254	332
2			150	177	232
1			82	97	127

Table A.17 - Horizontal forces for seismic action, Abruzzo, Italy – DCM

Storey	XX direction (parallel to 21m frame)			YY direction (parallel to 18m frame)		
	Fundamental period [s]	Total base shear [kN]	FX_i [kN]	Fundamental period [s]	Total base shear [kN]	FY_i [kN]
Roof			386			392
9			362			368
8			337			342
7			311			316
6	1.90	2407	269	1.87	2445	273
5			230			234
4			194			197
3			153			155
2			106			108
1			58			59

Table A.18 - Torsional moments for seismic action, Abruzzo, Italy – DCM

Storey	eX_i [m]	eY_i [m]	$MX_i = FX_i \cdot eY_i$ [kN·m]	$MY_i = FY_i \cdot eX_i$ [kN·m]	M_i (SRSS) [kN·m]
Roof			348	412	539
9			326	386	505
8			303	359	470
7			280	331	434
6	1.05	0.90	242	287	376
5			207	246	321
4			175	207	271
3			137	163	213
2			96	114	149
1			52	62	81

Annex B MATLAB SCRIPT - DCMFRAMEDESIGNERWSLAB

```
function [] = DC3frameDesignerWSlab(buildingName, fck, fyk, cover,
seismicVerticalLoadCase, seismicCases, nonSeismicCases, folder, flag,
slabTopReinf)
%factor accounting for overstrength due to steel strain hardening and
confinement of the concrete of the compression zone of the section
G_RD = 1.1;

tic; disp('Started DC3 WSlab');

%% STEP 1 - GETTING DATA
loading = waitbar(0, 'Reading data', 'Name', 'DC3Slab: Step 1 of 9');

fnData = ['data\' buildingName '\dataset.csv'] ;
fnNodes = ['data\' buildingName '\nodes.csv'] ;
fnElement = ['data\' buildingName '\connectivity.csv'] ;

[~, barsOfColumns, beamDesiOrd, ~, ~, DataDesignMax, DataDesignMin,
element, ~, stories, nodes, cases] = dataTransformer (fnData, fnElement,
fnNodes);
clear buildingName fnData fnElement fnNodes

allCasesIdx = [1:length(cases)];
seismicVerticalLoadCaseIdx = find(ismember(cases,
seismicVerticalLoadCase));
seismicCasesIdx = find(ismember(cases, seismicCases));
nonSeismicCasesIdx = find(ismember(cases, nonSeismicCases));

loading = waitbar(1, loading, 'Reading data', 'Name', 'DC3Slab: Step 1 of 9');
pause(.5);

%% STEP 2 - DESIGN BEAMS
close(loading); loading = waitbar(0, 'Initializing beams', 'Name', 'DC3Slab:
Step 2 of 9'); pause(1);
beams = []; beamsMid = [];
for i = 1 : length(beamDesiOrd)
    barIndex = find(DataDesignMax(:,1,1) == beamDesiOrd(i));
    %longitudinal rebar
    mAux = [];
    for j = allCasesIdx
        M_Ed = DataDesignMax(barIndex, 5, j);
        [sec_h, sec_b, longReinfNo, longReinfPhi, longReinfArea, M_Rd,
roMinCondition] = DC3beamDesign(fck, fyk , cover, M_Ed, 0);
        mAux = [mAux;[sec_h, sec_b, longReinfNo, longReinfPhi,
longReinfArea, M_Rd, roMinCondition]];

        M_Ed = DataDesignMin(barIndex, 5, j);
        [sec_h, sec_b, longReinfNo, longReinfPhi, longReinfArea, M_Rd,
roMinCondition] = DC3beamDesign(fck, fyk , cover, M_Ed, 0);
        mAux = [mAux;[sec_h, sec_b, longReinfNo, longReinfPhi,
longReinfArea, M_Rd, roMinCondition]];
    end
    mAux = sortrows(mAux, [2 1 5]);
    [M_rd, conIndex] = max(mAux(:,6)); %best M_rd

    %stirrups
    given_h = mAux(conIndex, 1);
```

```

given_b = mAux(conIndex, 2);
longRebarN = mAux(conIndex, 3);
longRebarPh = mAux(conIndex, 4);

Fz_Ed = max(max(abs(DataDesignMax(barIndex, 4,
allCasesIdx)),abs(DataDesignMin(barIndex, 4, allCasesIdx))));

[~,~,~,~,~,~,~, shearPhi, shearSpac, shearLegs, V_Rd_it,
sCondition] = DC3beamDesign(fck, fyk , cover, M_Ed, Fz_Ed, given_b,
given_h, longRebarN, longRebarPh);
%midStirrups
[shearPhiMid, shearSpacMid, shearLegsMid, V_RdMid] =
DC3beamDesignMidShear(fck, fyk , cover, Fz_Ed, given_b, given_h,
longRebarN, longRebarPh);

beams(end+1,:) = [DataDesignMax(barIndex,1,1), given_h, given_b,
longRebarN, longRebarPh, M_rd, mAux(conIndex, 7), shearPhi, shearSpac,
shearLegs, V_Rd_it, sCondition, Fz_Ed];
beamsMid(end+1,:) = [DataDesignMax(barIndex,1,1), given_h, given_b,
longRebarN, longRebarPh, M_rd, 0, shearPhiMid, shearSpacMid, shearLegsMid,
V_RdMid, Fz_Ed];

waitbar(size(beams,1) / length(beanDesiOrd),loading,'Beams
progress','Name', 'DCL: Step 2 of 4');
end
save([folder '\DC3beamsIt1.mat'],'beams');
save([folder '\DC3beamsIt1mid.mat'],'beamsMid');

%% STEP 3 - DESIGN COLUMNS
close(loading); loading = waitbar(0,'Initializing columns','Name',
'DC3Slab: Step 3 of 9'); pause(1);
noStories = max(stories(:,1)); columns = []; columnsMid = []; count = 0;
for i = 1 : size(barsOfColumns,1)
    % #1 design individually bars of a column
    barNames = [];
    for j = 1 : noStories
        barName = barsOfColumns(i,j); barNames = [barNames; barName];
        barIndex = find(DataDesignMax(:,1,1) == barName);
        try [minWidth] = minWidFind(barName, element, beams); catch
minWidth = .2; end

        mAux1 = [];
        for k = nonSeismicCasesIdx
            N_axial = DataDesignMax(barIndex, 2, k);
            My_h = DataDesignMax(barIndex, 5, k);
            Mz_b = DataDesignMax(barIndex, 6, k);
            V_Ed = DataDesignMax(barIndex, 4, k);
            [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesignIts(fck, fyk , cover, N_axial, My_h, Mz_b,
minWidth);
            mAux1 = [mAux1;[sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed]];

            N_axial = DataDesignMin(barIndex, 2, k);
            My_h = DataDesignMin(barIndex, 5, k);

```

```
Mz_b = DataDesignMin(barIndex, 6, k);
V_Ed = DataDesignMin(barIndex, 4, k);
[sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesignIts(fck, fyk, cover, N_axial, My_h, Mz_b,
minWidth);
    mAux1 = [mAux1; sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed];
    end

    mAux1 = sortrows(mAux1, [5 1], {'descend' 'ascend'});
    [~, index] = max(mAux1(:, 1));
    bestIndi(j, :) = mAux1(index, :);
end
clear areaRebar barIndex barName index M_Rd mAux1 minWidth My_h Mz_b
N_axial ...
    noRebar phiRebar reinfPercFin sCondition sec_b sec_h shearReinfArea
...
    shearReinfLoops shearReinfPhi shearReinfSpac V_Ed V_Rd j k

% #2 design from bottom to top - based on the biggest width of the best
individuals
% bottom bar
bigOrigWidth = max(bestIndi(:, 1)); %biggest width on the individual
iteration
barName = barsOfColumns(i, 1);
barIndex = find(DataDesignMax(:, 1, 1) == barName);
areaMinIt = bestIndi(1, 5);
givenLong = columnLongMin(areaMinIt);
mAux3 = [];
for p = allCasesIdx
    N_axial = DataDesignMax(barIndex, 2, p);
    My_h = DataDesignMax(barIndex, 5, p);
    Mz_b = DataDesignMax(barIndex, 6, p);
    V_Ed = DataDesignMax(barIndex, 4, p);
    try [minWidth] = minWidFind(barName, element, beams); catch
minWidth = .2; end
    gWidth = max(minWidth, bigOrigWidth);
    [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin, M_Rd,
shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesign(fck, fyk, cover, N_axial, My_h, Mz_b,
gWidth, givenLong);
    mAux3 = [mAux3; sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed];

    N_axial = DataDesignMin(barIndex, 2, p);
    My_h = DataDesignMin(barIndex, 5, p);
    Mz_b = DataDesignMin(barIndex, 6, p);
    V_Ed = DataDesignMin(barIndex, 4, p);
    try [minWidth] = minWidFind(barName, element, beams); catch
minWidth = .2; end
    gWidth = max(minWidth, bigOrigWidth);
    [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin, M_Rd,
shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesign(fck, fyk, cover, N_axial, My_h, Mz_b,
gWidth, givenLong);
```

```

        mAux3 = [mAux3;[sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed]];
    end
    mAux3 = sortrows(mAux3,[6 1],{'descend' 'ascend'});
    [~, index] = max(mAux3(:,6));%best iteration for that bar
    auxColumns = mAux3(index,:);
    clear areaRebar barIndex barName bigOrigWidth gWidth M_Rd minWidth ...
        areaMinIt My_h Mz_b N_axial noRebar p phiRebar reinfPercFin
sCondition sec_b sec_h ...
        givenLong shearReinfArea shearReinfLoops shearReinfPhi
shearReinfSpac V_Ed V_Rd ...
        barName index mAux3 minWidth
    % other bars

    for j = 2 : noStories
        bigOrigWidth = bestIndi(j,1);
        barName = barsOfColumns(i,j);
        barIndex = find(DataDesignMax(:,1,1) == barName);
        areaMinIt = bestIndi(j, 5);
        givenLong = columnLongMin(areaMinIt);
        mAux3 = [];
        for p = allCasesIdx
            N_axial = DataDesignMax(barIndex, 2, p);
            My_h = DataDesignMax(barIndex, 5, p);
            Mz_b = DataDesignMax(barIndex, 6, p);
            V_Ed = DataDesignMax(barIndex, 4, p);
            try [minWidth] = minWidFind(barName, element, beams); catch
minWidth = .2; end
            gWidth = max(minWidth, bigOrigWidth);
            [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesign(fck, fyk , cover, N_axial, My_h, Mz_b,
gWidth, givenLong);
            mAux3 = [mAux3; [sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed]];

            N_axial = DataDesignMin(barIndex, 2, p);
            My_h = DataDesignMin(barIndex, 5, p);
            Mz_b = DataDesignMin(barIndex, 6, p);
            V_Ed = DataDesignMin(barIndex, 4, p);
            try [minWidth] = minWidFind(barName, element, beams); catch
minWidth = .2; end
            gWidth = max(minWidth, bigOrigWidth);
            [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesign(fck, fyk , cover, N_axial, My_h, Mz_b,
gWidth, givenLong);
            mAux3 = [mAux3; [sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, V_Ed]];
        end
        %best iteration for that bar
        mAux3 = sortrows(mAux3,[6 1],{'descend' 'ascend'});
        [~, index] = max(mAux3(:,6));
        auxColumns(j,:) = mAux3(index,:);
    end
end

```

```
clear areaRebar barIndex barName bestIndi bigOrigWidth gWidth j M_Rd
mAux3 minWidth ...
    areaMinIt My_h Mz_b N_axial noRebar p phiRebar reinfPercFin
sCondition sec_b sec_h ...
    givenLong shearReinfArea shearReinfLoops shearReinfPhi
shearReinfSpac V_Ed V_Rd index

% check if dimensions agree (no bigger dimension on top of the column)
% if it fails will try another method to design it
sorted = issorted(auxColumns(:,1), 'descend');
if ~sorted
    disp('First design method failed')

    for t = noStories : -1 : 2
        if auxColumns(t-1,2) < auxColumns(t,2)
            gWidth = auxColumns(t,2);
            areaMinIt = auxColumns(t-1, 5);
            givenLong = columnLongMin(areaMinIt);
            [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd] = DC3columnDesign(fck, fyk, cover, 10, 10, 10, gWidth, givenLong);
            auxColumns(t-1,[1:7]) = [sec_h, sec_b, noRebar, phiRebar,
areaRebar, reinfPercFin, M_Rd];
        end
        clear b h areaRebar barID N_AxialN M_RdN mAuxN p
    end
end

for j = 1 : size(auxColumns,1)
    sec_h = auxColumns(j, 1);
    noRebar = auxColumns(j, 3);
    phiRebar = auxColumns(j, 4);
    [shearReinfPhiMid, shearReinfSpacMid, shearReinfLoopsMid,
shearReinfAreaMid] = DC3columnDesignMidShear(fck, fyk, cover, sec_h,
noRebar, phiRebar);
    auxColumnsMid(j,:) = [shearReinfPhiMid, shearReinfSpacMid,
shearReinfLoopsMid, shearReinfAreaMid];
end

columns = [columns; [barNames, auxColumns]];
columnsMid = [columnsMid; [barNames, auxColumns(:, [1:7])],
auxColumnsMid]];

clear auxColumns auxColumnsMid barNames bestIndi noRebar phiRebar ...
    sec_h shearReinfAreaMid shearReinfLoopsMid shearReinfPhiMid ...
    shearReinfSpacMid sorted j

count = count + 1;
loading = waitbar(count / size(barsOfColumns,1), loading, 'Columns
progress', 'Name', 'DC3Slab: Step 3 of 9');
end

save([folder '\DC3columnsIt1.mat'], 'columns');
save([folder '\DC3columnsIt1mid.mat'], 'columnsMid');

%% STEP 4 - UPDATE COLUMNS RESISTING BENDING MOMENTS
close(loading); loading = waitbar(0, 'Initializing columns resisting bending
moments update', 'Name', 'DC3Slab: Step 4 of 9'); pause(1);
```

```

for i = 1 : size(columns,1)
    b = columns(i, 3);
    h = columns(i, 2);
    areaRebar = columns(i, 6);

    mAux = [];
    for j = seismicCasesIdx
        N_Axial = DataDesignMax(DataDesignMax(:,1,1) == columns(i,1) , 2,
j);
        M_Rd = MrdColumn(fck, fyk, b, h, areaRebar, N_Axial);
        mAux(end+1) = M_Rd;
    end

    columns(i, 8) = min(mAux);
    loading = waitbar(i / size(columns,1),loading,'Columns resisting
bending moments update progress','Name', 'DC3Slab: Step 4 of 9');
end
clear areaRebar b h i j M_Rd mAux N_Axial

%% STEP 5 - DO THE RESISTING BENDING MOMENTS [1.3] COMPARISON
close(loading); loading = waitbar(0,'Initializing bending
comparisons','Name', 'DC3Slab: Step 5 of 9'); pause(1);
minX = min(nodes(:,2)); maxX = max(nodes(:,2));
minY = min(nodes(:,3)); maxY = max(nodes(:,3));
borders = [minX,maxX,minY,maxY];

increNeed = [];
for i = 1 : size(nodes,1)
    %skip base points and on the top floor the verification is not needed
    if nodes(i, 5) == 0
        continue
    end

    %get bars on that point
    [row, ~] = find(element(:,[2 3]) == nodes(i,1) & element(:,4) == 1);
    barsX = element(row,1);
    [row, ~] = find(element(:,[2 3]) == nodes(i,1) & element(:,4) == 2);
    barsY = element(row,1);
    [row, ~] = find(element(:,[2 3]) == nodes(i,1) & element(:,4) == 3);
    barsZ = element(row,1);

    %get design bending moments on those bars
    bendRdX = 0;
    for j = 1 : length(barsX)
        selfLength = DataDesignMax(DataDesignMax(:,1,1) == barsX(j) , 7,
1);
        plusLength = 0;
        for k = 1 : length(barsY)
            auxLength = DataDesignMax(DataDesignMax(:,1,1) == barsY(k) , 7,
1);
            plusLengthAux = min(selfLength/4, auxLength/2);
            plusLength = plusLength + plusLengthAux;
        end
        extraArea = plusLength * slabTopReinf;
        h = beams(beams(:,1) == barsX(j), 2);
        b = beams(beams(:,1) == barsX(j), 3);
        num = beams(beams(:,1) == barsX(j), 4);
        phi = beams(beams(:,1) == barsX(j), 5);

```

```
A1 = pi * (phi/2000)^2 * num; A2 = A1;
A1 = A1 + extraArea;

MRD_beam = MrdBeam (fck, fyk, cover, b, h, A1, A2);
bendRdX = bendRdX + MRD_beam;
end

bendRdY = 0;
for j = 1 : length(barsY)
    selfLength = DataDesignMax(DataDesignMax(:,1,1) == barsY(j) , 7,
1);
    plusLength = 0;
    for k = 1 : length(barsX)
        auxLength = DataDesignMax(DataDesignMax(:,1,1) == barsX(k) , 7,
1);
        plusLengthAux = min(selfLength/4, auxLength/2);
        plusLength = plusLength + plusLengthAux;
    end
    extraArea = plusLength * slabTopReinf;
    h = beams(beams(:,1) == barsY(j), 2);
    b = beams(beams(:,1) == barsY(j), 3);
    num = beams(beams(:,1) == barsY(j), 4);
    phi = beams(beams(:,1) == barsY(j), 5);
    A1 = pi * (phi/2000)^2 * num; A2 = A1;
    A1 = A1 + extraArea;

    MRD_beam = MrdBeam (fck, fyk, cover, b, h, A1, A2);
    bendRdY = bendRdY + MRD_beam;
end

bendRdZ = 0;
for j = 1 : length(barsZ)
    [columnRow, ~] = find(columns(:,1) == barsZ(j));
    bendRdZ = bendRdZ + columns(columnRow, 8);
end

%do the sums and comparisons
ratioXZ = bendRdZ / bendRdX;
if ratioXZ < 1.3
    auxBendMomX = bendRdX * 1.3;
else
    auxBendMomX = 0;
end

ratioYZ = bendRdZ / bendRdY;
if ratioYZ < 1.3
    auxBendMomY = bendRdY * 1.3;
else
    auxBendMomY = 0;
end
increNeed(end+1,:) = [nodes(i,1), max(auxBendMomX, auxBendMomY)];

clear auxBendMomX auxBendMomY barsX barsY barsZ beamRow bendRdX bendRdY
...
    bendRdZ columnRow j ratioXZ ratioYZ row
```

```

loading = waitbar(i / (size(nodes,1)),loading,'Comparisons
progress','Name', 'DC3Slab: Step 5 of 9');
end
save([folder '\increNeed.mat'],'increNeed');
save([folder '\columnsDel.mat'],'columns');

%% STEP 6 - REINFORCE COLUMNS FOR THE 1.3 RULE
close(loading); loading = waitbar(0,'Initializing column re-
reinforcement','Name', 'DC3Slab: Step 6 of 9'); pause(1);
newColumns = []; columns13 = [];
for i = 1 : size(barsOfColumns,1)
    % build table
    aux1 = barsOfColumns(i,:);
    for j = 1 : length(aux1)
        auxElement(j,:) = element(element(:,1) == aux1(j),[1:3]);
    end

    %--NODE--%--BAR 1--%--MED 1--%--BAR 2--%--MED2--%--SUM MED--%--MED
    NEEDED--%--Diff--%
    table(:,1) = intersect([auxElement(:,2); auxElement(:,3)],
increNeed(:,1));
    for j = 1 : size(table,1)
        [row, ~] = find(table(j,1) == auxElement(:, [2 3]));
        table(j, [2 4]) = auxElement(row,1);
        table(j, 3) = columns(columns(:,1) == table(j, 2), 8);
        table(j, 5) = columns(columns(:,1) == table(j, 4), 8);
        table(j, 6) = table(j, 3) + table(j, 5);
        table(j, 7) = increNeed(table(j,1) == increNeed(:,1) , 2);
        table(j, 8) = table(j, 7) - table(j, 6);
    end

    clear auxElement

    % iterations
    while ~all(table(:,8) < 3)%while there is a "needed" bigger than what
it is
        [~, index] = max(table(:, 8));
        toGive(:,1) = [table(index, 2); table(index,4)];
        toGive(:,2) = [table(index, 7)*(table(index, 3)/table(index, 6));
table(index, 7)*(table(index, 5)/table(index, 6))];

        for j = 1 : size(toGive,1)
            matAux = [];
            for k = seismicCasesIdx
                N_axial = DataDesignMax(DataDesignMax(:,1,1) ==
toGive(j,1), 2, k);
                My_h = toGive(j, 2); Mz_b = 0;
                [columnsRow,~] = find(columns(:,1) == toGive(j,1));
                gWidth = columns(columnsRow, 2);
                areaOld = columns(columnsRow, 6);
                givLong = columnLongMin(areaOld);

                [sec_h, sec_b, noRebar, phiRebar, areaRebar, reinfPercFin,
M_Rd, shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea, V_Rd,
sCondition] = DC3columnDesign(fck, fyk , cover, N_axial, My_h, Mz_b,
gWidth, givLong);

```

```
V_Ed = columns(columnsRow, 15);
matAux = [matAux;toGive(j, 1), sec_h, sec_b, noRebar,
phiRebar, areaRebar, reinfPercFin, M_Rd, shearReinfPhi, shearReinfSpac,
shearReinfLoops, shearReinfArea, V_Rd, sCondition, V_Ed];
end
matAux = sortrows(matAux,[6 2],{'descend' 'ascend'});
[~, indexu] = max(matAux(:, 2));
finalColumn = matAux(indexu,:);
newColumns = [newColumns; finalColumn];

%update my table!
[updateRow, updateCol] = find(table(:,[2,4]) == toGive(j,1));
updateCol = updateCol * 2;

for k = 1 : length(updateCol)
    table(updateRow(k), [updateCol(k), updateCol(k) + 1]) =
[toGive(j, 1), max(matAux(:, 8))];
end

table(:, 6) = table(:, 3) + table(:, 5);
table(:, 8) = table(:, 7) - table(:, 6);
end

clear toGive areaOld areaRebar finalColumn givLong gWidth improve
...
index indexu k longReinforce M_Rd My_h Mz_b N_axial noRebar ...
phiRebar reinfPercFin row sCondition sec_b sec_h col mImprove
...
shearReinfArea shearReinfLoops shearReinfPhi shearReinfSpac ...
updateCol updateRow V_Ed V_Rd columnsRow

end

clear matAux

%add the ones without changes and clean extra bars
if ~isempty(newColumns)
    nonReReinf = setxor(aux1,newColumns(:,1));
    for j = 1 : length(nonReReinf)
        mAux = columns(columns(:,1) == nonReReinf(j),:);
        newColumns = [newColumns; mAux];
    end

    for j = 1 : noStories
        lastIndex = find(aux1(j) == newColumns(:,1), 1, 'last');
        primeColumns(j,:) = newColumns(lastIndex,:);
    end
else
    primeColumns = newColumns;
end
clear auxElement aux1 table toGive newColumns lastIndex mAux nonReReinf

%give concrete dimensions
for z = 1 : (noStories) % check if superior dimesions are the same or
smaller, if is it, add it to possDesigns
    aCheck = issorted(primeColumns(:,2),'descend');
    if ~aCheck
```

```

        for t = noStories : -1 : 2
            if primeColumns(t-1,2) < primeColumns(t,2)
                gWidth = primeColumns(t,2);
                areaMinIt = primeColumns(t-1, 6);
                givenLong = columnLongMin(areaMinIt);
                [sec_h, sec_b, noRebar, phiRebar, areaRebar,
reinfPercFin, M_Rd] = DC3columnDesign(fck, fyk , cover, 10, 10, 10, gWidth,
givenLong);
                primeColumns(t-1,[2:8]) = [sec_h, sec_b, noRebar,
phiRebar, areaRebar, reinfPercFin, M_Rd];
            end
        end
    end
end

clear aCheck z

% used to compatibilize reinforcement patterns allong height
if flag == "yes"
    %for the given areas give the reinforcements
    %all the reinforcement possibilities
    auxVals = primeColumns(:, [1,2,6]);
    auxRefs = [];
    for j = 1 : noStories
        switch j
            case 1
                auxRefs(:, [1:3], j) = auxVals;
                width = auxVals(1,2);
                reinfArea = auxVals(1,3);
                [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea);
                auxRefs(1, [4:6], j) = [phiRebar, noRebar, areaRebar];
                givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
                for k = 2 : noStories
                    width = auxVals(k,2);
                    reinfArea = auxVals(k,3);
                    [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea, givenLong);
                    auxRefs(k, [4:6], j) = [phiRebar, noRebar,
areaRebar];
                    givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
                end
            case noStories
                auxRefs(:, [1:3], j) = auxVals;
                width = auxVals(noStories,2);
                reinfArea = auxVals(noStories,3);
                [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea);
                auxRefs(noStories, [4:6], j) = [phiRebar, noRebar,
areaRebar];
                givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
                for k = (noStories - 1) : -1 : 1
                    width = auxVals(k,2);
                    reinfArea = auxVals(k,3);

```

```
        [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea, givenLong);
        auxRefs(k, [4:6], j) = [phiRebar, noRebar,
areaRebar];
        givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
        end

    otherwise
        auxRefs(:, [1:3], j) = auxVals;
        width = auxVals(j,2);
        reinfArea = auxVals(j,3);
        [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea);
        auxRefs(j, [4:6], j) = [phiRebar, noRebar, areaRebar];
        givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');

        for k = (j+1) : noStories
            width = auxVals(k,2);
            reinfArea = auxVals(k,3);
            [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea, givenLong);
            auxRefs(k, [4:6], j) = [phiRebar, noRebar,
areaRebar];
            givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
        end

        width = auxVals(j,2);
        reinfArea = auxVals(j,3);
        [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea);
        auxRefs(j, [4:6], j) = [phiRebar, noRebar, areaRebar];
        givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');

        for k = (j-1) : -1 : 1
            width = auxVals(k,2);
            reinfArea = auxVals(k,3);
            [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea, givenLong);
            auxRefs(k, [4:6], j) = [phiRebar, noRebar,
areaRebar];
            givenLong = columnComp(noRebar, phiRebar, 'EC8',
'yes');
        end
    end
end

clear areaRebar auxVals j k noRebar phiRebar reinfArea width
end

%delete the impossible ones
[~, delIdx] = find(auxRefs(:,6,:) == 0);
if length(unique(delIdx)) == noStories
    for k = 1 : size(auxRefs,3)
        for j = 1 : size(auxRefs,1)
            if auxRefs(j,4,k) == 0
                width = auxRefs(j,2,k);
                reinfArea = auxRefs(j,3,k);
```

```

        [phiRebar, noRebar, areaRebar] = columnRebar(width,
cover, reinfArea);
        auxRefs(j, [4:6], k) = [phiRebar, noRebar,
areaRebar];
    end
end
end
delIdx = [];
else
    auxRefs(:, :, delIdx) = [];
end

clear delIdx

%calculate the overshooting and the best in average
auxRefs(:,7,:) = auxRefs(:,6,:) ./ auxRefs(:,3,:);
means = mean(auxRefs(:,7,:),1);
[~, bestIdx] = min(means);
auxRefs = auxRefs(:, :, bestIdx);

%update columns
for j = 1 : noStories
    primeColumns(j, [4:6]) = auxRefs(j, [4:6]);
    areaRebar = primeColumns(j, 6);
    sec_b = primeColumns(j,2); sec_h = sec_b;
    primeColumns(j, 7) = areaRebar * fyk / (sec_b * sec_h * fck);
    N_axial = max(DataDesignMax(DataDesignMax(:,1,1) ==
primeColumns(j,1), 2, seismicCasesIdx));
    primeColumns(j, 8) = MrdColumn(fck, fyk, sec_b, sec_h,
areaRebar, N_axial);

    longReinfN = primeColumns(j, 5);
    longReinfPh = primeColumns(j, 4);
    Ved = columns(columns(:,1) == primeColumns(j, 1), 15);
    [~, ~, ~, ~, ~, ~, ~, ~, shearReinfPhi, shearReinfSpac,
shearReinfLoops, shearReinfArea, Vrd, sCondition] = DC3columnDesign(fck,
fyk, cover, 10, 10, 10, sec_b, [longReinfPh, longReinfN, areaRebar], [],
sec_b, longReinfN, longReinfPh);
    primeColumns(j, [9:15]) = [shearReinfPhi, shearReinfSpac,
shearReinfLoops, shearReinfArea, Vrd, sCondition, Ved];
end

clear auxRefs areaRebar bestIdx givenLong longReinfN longReinfPh
means ...
    N_axial sCondition sec_b sec_h shearReinfArea shearReinfLoops
...
    shearReinfPhi shearReinfSpac Ved Vrd
end

columns13 = [columns13; primeColumns];

loading = waitbar(i / (size(barsOfColumns,1)), loading, 'Column re-
reinforcement progress', 'Name', 'DC3Slab: Step 6 of 9');
clear primeColumns
newColumns = [];
end
save([folder '\DC3columnsIt2rule13.mat'], 'columns13');

```

```
clear noStories flag columns increNeed newColumns

%% STEP 7 - SHEAR CAPACITY DESIGN BEAMS
close(loading); loading = waitbar(0, 'Initializing Beam seismic
equilibrium', 'Name', 'DC3Slab: Step 7 of 9'); pause(1);
seismicBeams = []; seismicBeamsMidShear = [];
for i = 1 : length(beamDesiOrd)
    %bar info
    res = element(element(:,1) == beamDesiOrd(i), [2 3]);
    node1 = res(1); node2 = res(2);
    auxNode = [node1, node2];
    cLength = DataDesignMax(DataDesignMax(:,1,1) == beamDesiOrd(i) , 7, 1);
    direction = element(element(:,1) == beamDesiOrd(i), 4);

    for k = 1 : 2
        %obtain max sum of bending moment on top each node of the beams,
        each direction
        [row, ~] = find(element(:, [2 3]) == auxNode(k) & element(:,4) ==
direction); barsBeam = element(row,1);
        bendRdNodeAux = 0;
        for j = 1 : length(barsBeam)
            M_Rd = beams(beams(:,1) == barsBeam(j), 6);
            bendRdNodeAux = bendRdNodeAux + M_Rd;
        end
        bendRdBeam(k) = bendRdNodeAux;
    end

    % for each combination
    Vseismo = [];
    for p = seismicCasesIdx
        for k = 1 : 2
            % for each node get the bars that connect there
            [row, ~] = find(element(:, [2 3]) == auxNode(k) & element(:,4)
== 3); barsZ = element(row,1);

            % get the sumMrd for each node
            bendRdZAux = 0;
            for j = 1 : length(barsZ)
                [barRow, ~] = find(columns13(:,1) == barsZ(j));
                b = columns13(barRow, 3);
                h = columns13(barRow, 2);
                areaRebar = columns13(barRow, 6);
                N_Axial = DataDesignMax(DataDesignMax(:,1) == barsZ(j) , 2,
p);
                Mrc = MrdColumn(fck, fyk, b, h, areaRebar, N_Axial);
                bendRdZAux = bendRdZAux + Mrc;
            end
            bendRdZ(k) = bendRdZAux;

            if bendRdBeam(k) < bendRdZ(k)
                Mid(k) = G_RD * bendRdBeam(k);
            else
                Mid(k) = G_RD * bendRdBeam(k) * (bendRdZ(k) /
bendRdBeam(k));
            end
        end

        VofMom = sum(Mid)/cLength;
    end
end
```

```

        V1 = DataDesignMax(DataDesignMax(:,1,1) == beamDesiOrd(i), 4,
seismicVerticalLoadCaseIdx);
        V2 = DataDesignMin(DataDesignMin(:,1,1) == beamDesiOrd(i), 4,
seismicVerticalLoadCaseIdx);
        Vseismo = [Vseismo; V1 + VofMom];
        Vseismo = [Vseismo; V2 - VofMom];
    end

    Vseismo = abs(Vseismo);
    Vshear = max(max(Vseismo));

    % shear reinforcement
    barIndex = find(beams(:,1) == beamDesiOrd(i));
    given_b = beams(barIndex, 3);
    given_h = beams(barIndex, 2);
    longReinfN = beams(barIndex, 4);
    longReinfPh = beams(barIndex, 5);
    [~, ~, ~, ~, ~, ~, ~, shearReinfPhi, shearReinfSpac, shearReinfLoops,
V_Rd, sCondition] = DC3beamDesign(fck, fyk, cover, 10, Vshear, given_b,
given_h, longReinfN, longReinfPh);
    seismicBeams = [seismicBeams; [beams(barIndex,[1:7]),shearReinfPhi,
shearReinfSpac, shearReinfLoops, V_Rd, sCondition, Vshear]];

    %mid shear
    [shearReinfPhi, shearReinfSpac, shearReinfLoops, V_Rd] =
DC3beamDesignMidShear(fck, fyk, cover, Vshear, given_b, given_h,
longReinfN, longReinfPh);
    seismicBeamsMidShear = [seismicBeamsMidShear;
[beams(barIndex,[1:7]),shearReinfPhi, shearReinfSpac, shearReinfLoops,
V_Rd, 0, Vshear]];

    clear barIndex cLength given_b given_h longReinfN longReinfPh Mid
sCondition ...
        shearReinfLoops shearReinfPhi shearReinfSpac V_Rd Vseismic Vshear

    loading = waitbar(i / (length(beamDesiOrd)),loading,'Beam seismic
equilibrium','Name', 'DC3Slab: Step 7 of 9');
end
save([folder '\DC3beamsIt2SeismicEq.mat'],'seismicBeams');
save([folder '\DC3beamsIt2SeismicEqMid.mat'],'seismicBeamsMidShear');
clear seismicBeamsMidShear

%% STEP 8 - SHEAR CAPACITY DESIGN COLUMNS
close(loading); loading = waitbar(0,'Initializing column seismic
equilibrium','Name', 'DC3Slab: Step 8 of 9'); pause(1);
pilars = setxor(beamDesiOrd, element(:,1));
seismicColumns = []; seismicColumnsMidShear = [];
for i = 1 : length(pilars)
    %bar info
    barRow = find(columns13(:,1) == pilars(i));
    res = element(element(:,1) == pilars(i), [2 3]);
    node1 = res(1); node2 = res(2);
    auxNode = [node1, node2];
    cLength = DataDesignMax(DataDesignMax(:,1,1) == pilars(i) , 7, 1);

    for k = 1 : 2

```

```
%obtain max sum of bending moment on top each node of the beams,
each direction
[ row, ~ ] = find(element(:,[2 3]) == auxNode(k) & element(:,4) ==
1); barsX = element(row,1);
bendRdXaux = 0;
for j = 1 : length(barsX)
    M_Rd = beams(beams(:,1) == barsX(j), 6);
    bendRdXaux = bendRdXaux + M_Rd;
end
bendRdX(k) = bendRdXaux;

[ row, ~ ] = find(element(:,[2 3]) == auxNode(k) & element(:,4) ==
2); barsY = element(row,1);
bendRdYaux = 0;
for j = 1 : length(barsY)
    M_Rd = beams(beams(:,1) == barsY(j), 6);
    bendRdYaux = bendRdYaux + M_Rd;
end
bendRdY(k) = bendRdYaux;
end

% for each combination
Vseismo = [];
for p = seismicCasesIdx
    %Mrc of the given bar
    b = columns13(barRow, 3);
    h = columns13(barRow, 2);
    areaRebar = columns13(barRow, 6);
    N_Axial = DataDesignMax(DataDesignMax(:,1) == pilars(i) , 2, p);
    Mrc = MrdColumn(fck, fyk, b, h, areaRebar, N_Axial);

    %obtain max sum of bending moment on top each node of the column
    for k = 1 : 2
        [ row, ~ ] = find(element(:,[2 3]) == auxNode(k) & element(:,4)
== 3); barsZ = element(row,1);
        bendRdZaux = 0;
        for j = 1 : length(barsZ)
            b = columns13(columns13(:,1) == barsZ(j), 3);
            h = columns13(columns13(:,1) == barsZ(j), 2);
            areaRebar = columns13(columns13(:,1) == barsZ(j), 6);
            N_Axial = DataDesignMax(DataDesignMax(:,1,1) == barsZ(j) ,
2, p);

            M_Rd = MrdColumn(fck, fyk, b, h, areaRebar, N_Axial);
            bendRdZaux = bendRdZaux + M_Rd;
        end
        bendRdZ(k) = bendRdZaux;
    end

    for k = 1 : 2
        if bendRdX(k) > bendRdZ(k)
            MidX(k) = G_RD * Mrc;
        else
            MidX(k) = G_RD * Mrc * (bendRdX(k) / bendRdZ(k));
        end

        if bendRdY(k) > bendRdZ(k)
            MidY(k) = G_RD * Mrc;
        else
```

```

        MidY(k) = G_RD * Mrc * (bendRdY(k) / bendRdZ(k));
    end
end
Vseismo = [Vseismo; sum(MidX)/cLength];
Vseismo = [Vseismo; sum(MidY)/cLength];
end

Vseismo = abs(Vseismo);
Vshear = max(max(Vseismo));

given_h = columns13(barRow, 2);
longReinfN = columns13(barRow, 4);
longReinfPh = columns13(barRow, 5);
[~, ~, ~, ~, ~, ~, ~, shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition] = DC3columnDesign(fck, fyk, cover, 10,
10, 10, given_h, [longReinfPh, longReinfN, areaRebar], Vshear, given_h,
longReinfN, longReinfPh);
seismicColumns = [seismicColumns;
[colums13(barRow, [1:8]), shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, V_Rd, sCondition, Vshear]];

%seismic beams mid shear
[shearReinfPhi, shearReinfSpac, shearReinfLoops, shearReinfArea] =
DC3columnDesignMidShear(fck, fyk, cover, given_h, longReinfN,
longReinfPh);
seismicColumnsMidShear = [seismicColumnsMidShear;
[colums13(barRow, [1:8]), shearReinfPhi, shearReinfSpac, shearReinfLoops,
shearReinfArea, 0, 0, 0]];

clear areaRebar b barRow bendMomsaux bendMomsBeam bendRdX bendRdY ...
    bendRdZ cLength h j k longReinfN longReinfPh M_RdS mAuxS maxIndex
Mid ...
    Mrc N_AxialS p shearReinfArea shearReinfLoops shearReinfPhi
shearReinfSpac ...
    V_Rd Vshear

loading = waitbar(i / (length(pilars)), loading, 'Column seismic
equilibrium progress', 'Name', 'DC3Slab: Step 8 of 9');
end
save([folder '\DC3columnsIt3SeismicEq.mat'], 'seismicColumns');
save([folder '\DC3columnsIt3SeismicEqMid.mat'], 'seismicColumnsMidShear');

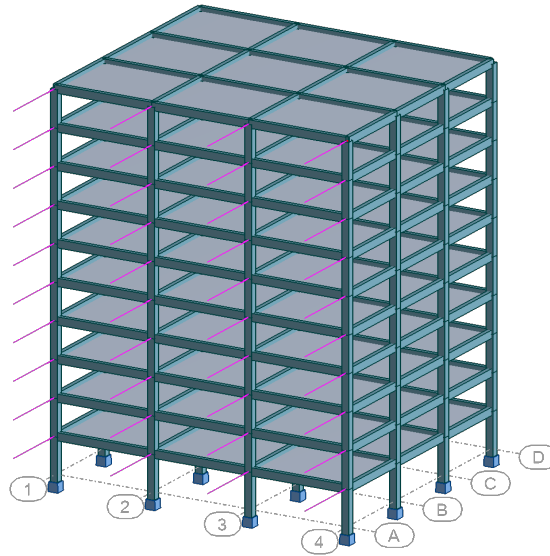
%% STEP 9 - EXPORT RESULTS TO CSV FOR SEISMOSTRUCT
close(loading); loading = waitbar(0, 'Updating and writing to
Seismo', 'Name', 'DC3Slab: Step 9 of 9'); pause(1);
toSeismo(seismicColumns(:, [1:5, 9:11]), seismicBeams(:, [1:5, 8:10]), nodes,
element, stories, DataDesignMin, DataDesignMax, seismicVerticalLoadCaseIdx,
fck, fyk, cover, folder);
loading = waitbar(1, loading, 'Updating and writing to Seismo', 'Name',
'DC3Slab: Step 9 of 9');

%% FINISH
time = toc; save([folder '\time.mat'], 'time');
minutes = floor(time/60);
seconds = floor(time - minutes*60);
disp(['Finished in ' num2str(minutes) ' minutes and ' num2str(seconds) '
seconds.']);

```

close (loading) ;

Annex C PUSHOVER ANALYSIS – AXIAL FORCE PROGRESSION



The vertical red line represents the displacement for which the design total base shear was reached.

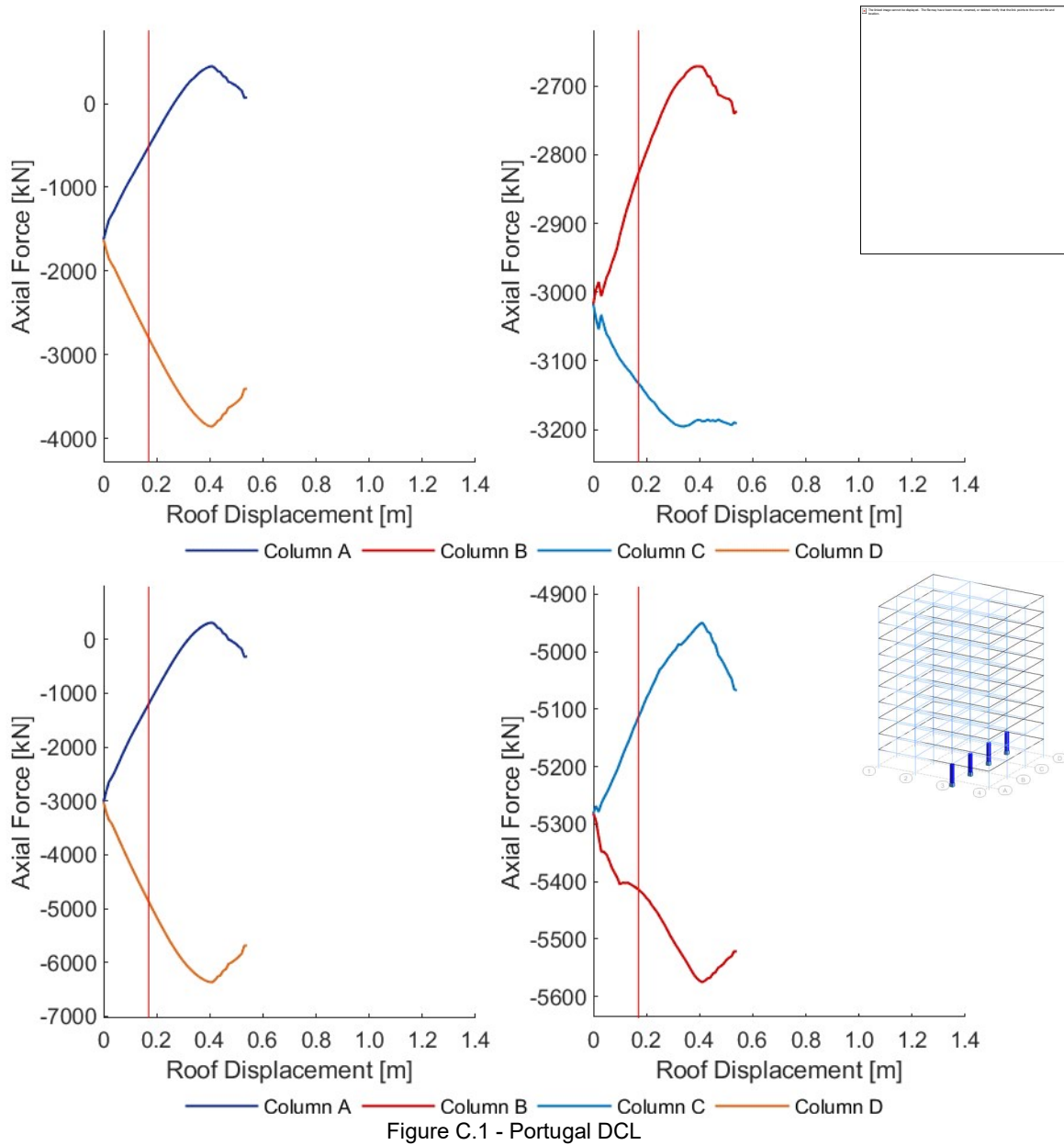
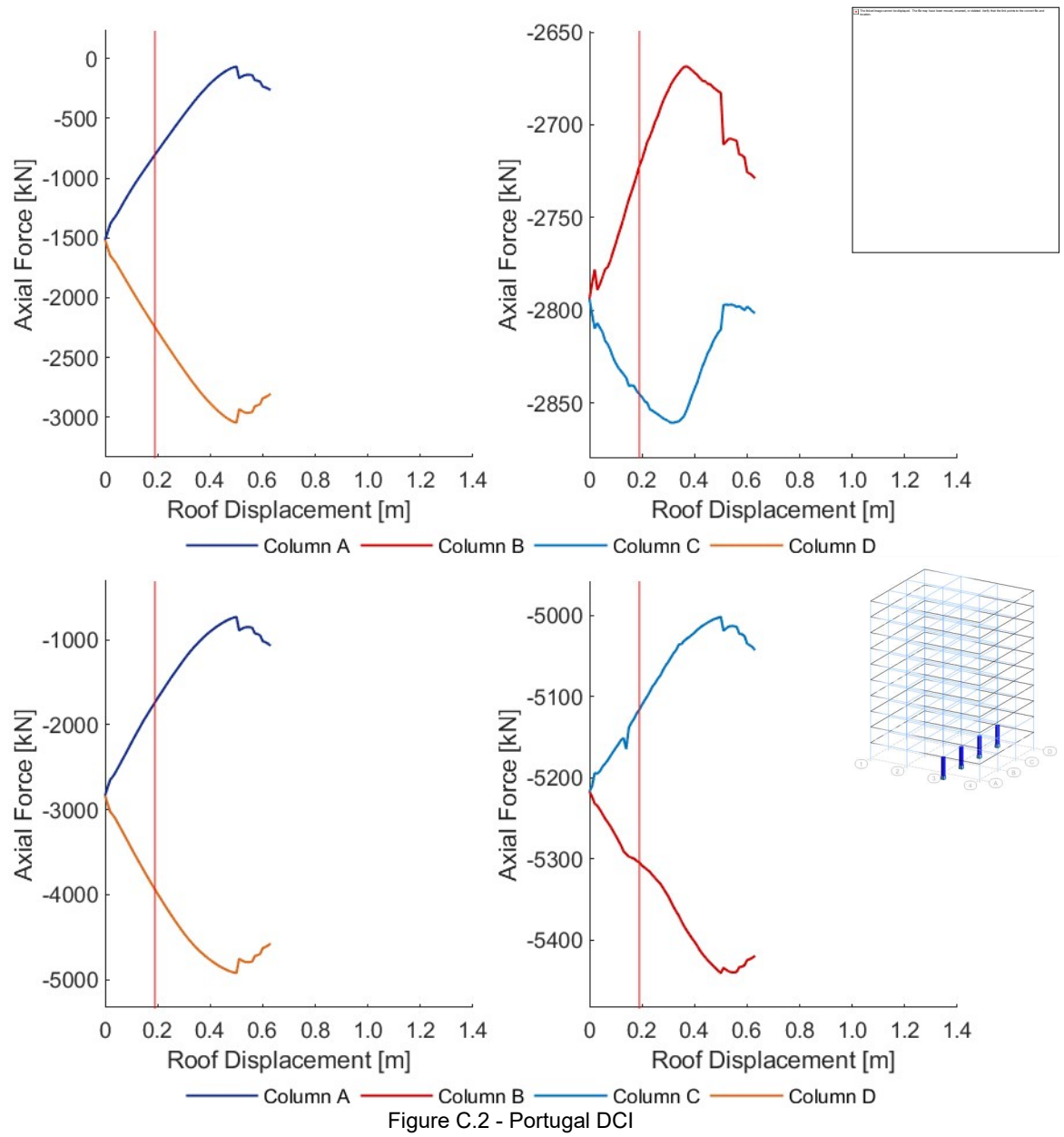


Figure C.1 - Portugal DCL



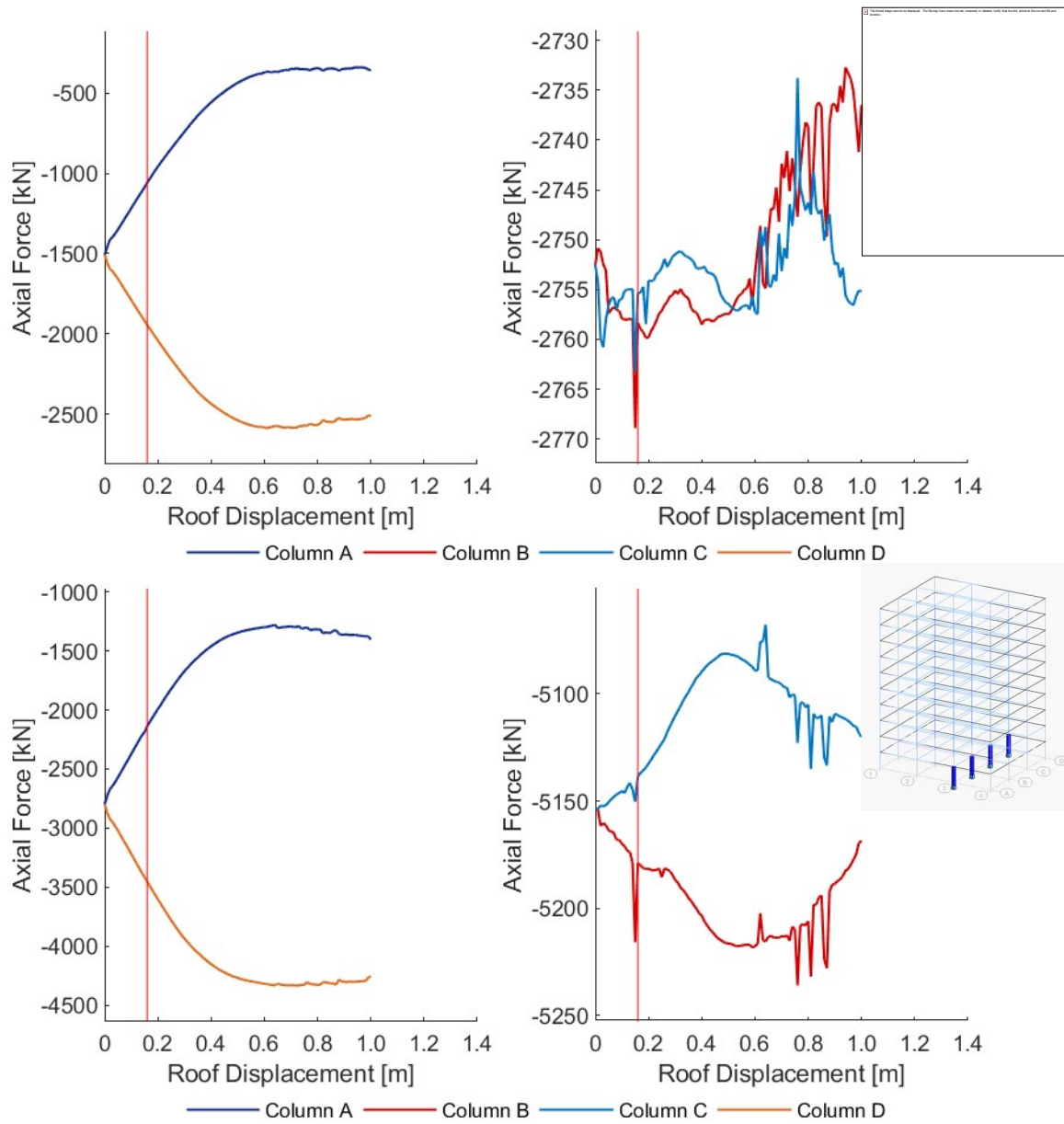


Figure C.3 - Portugal DCM

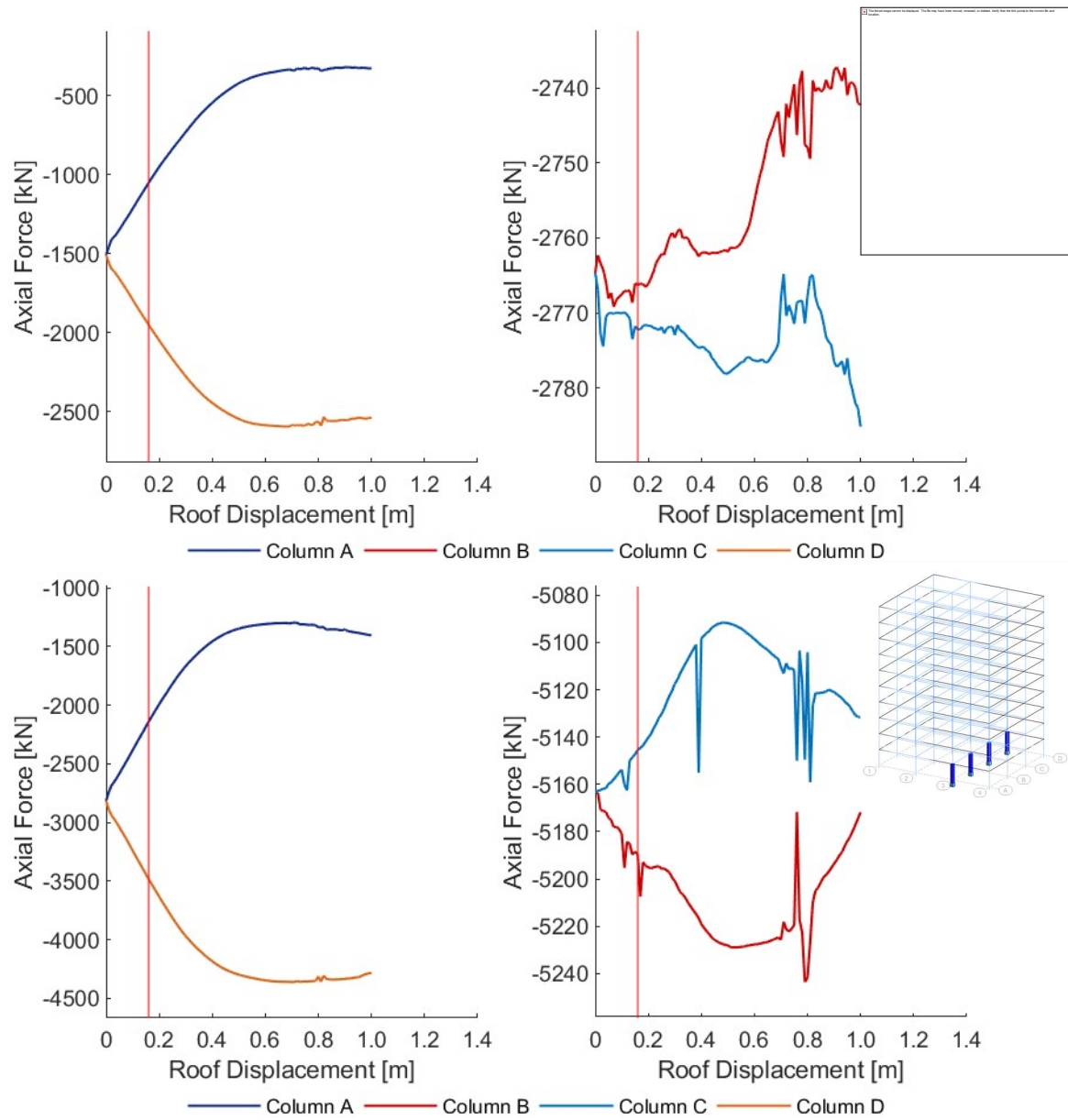


Figure C.4 - Portugal DCMslab

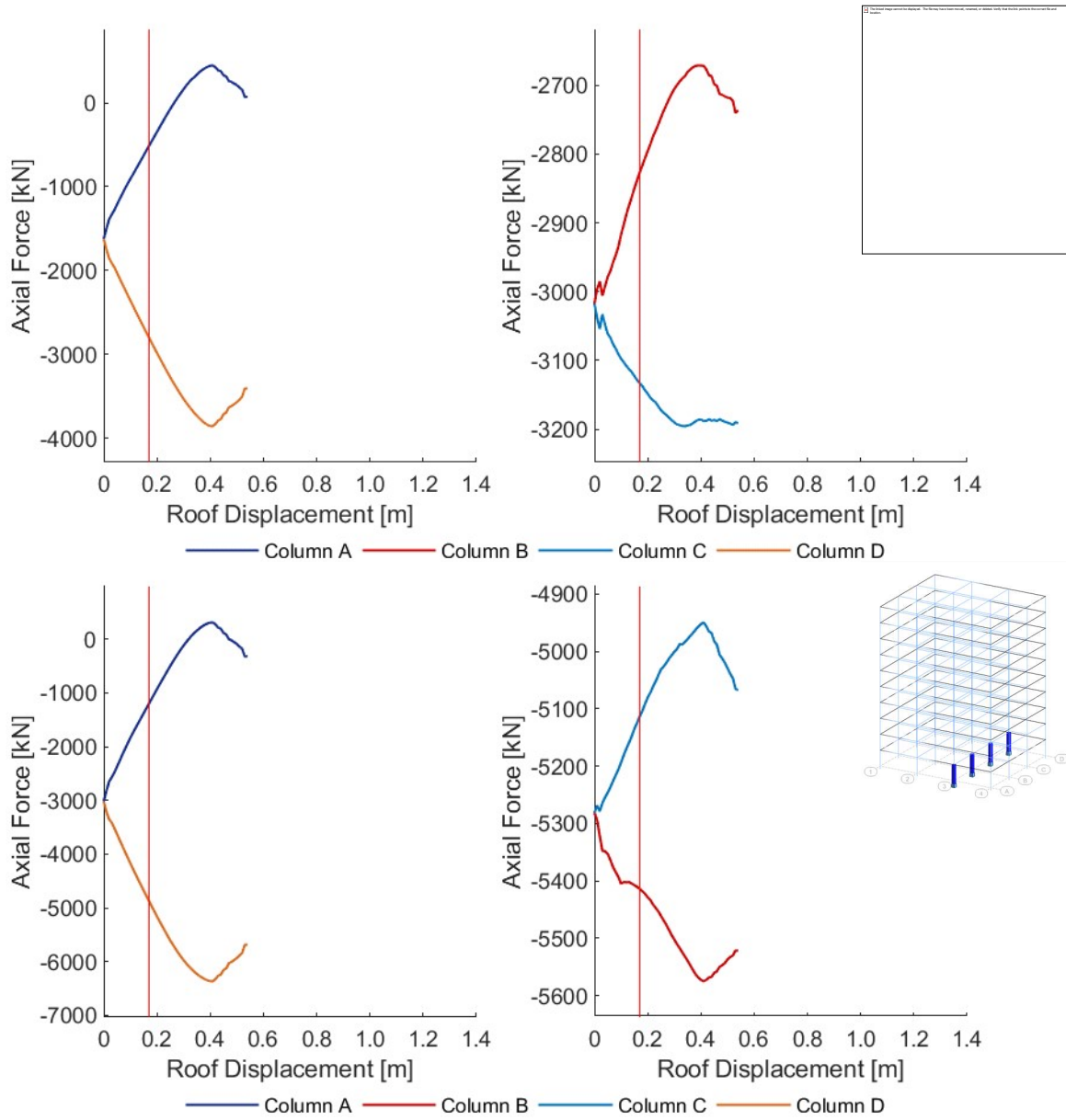
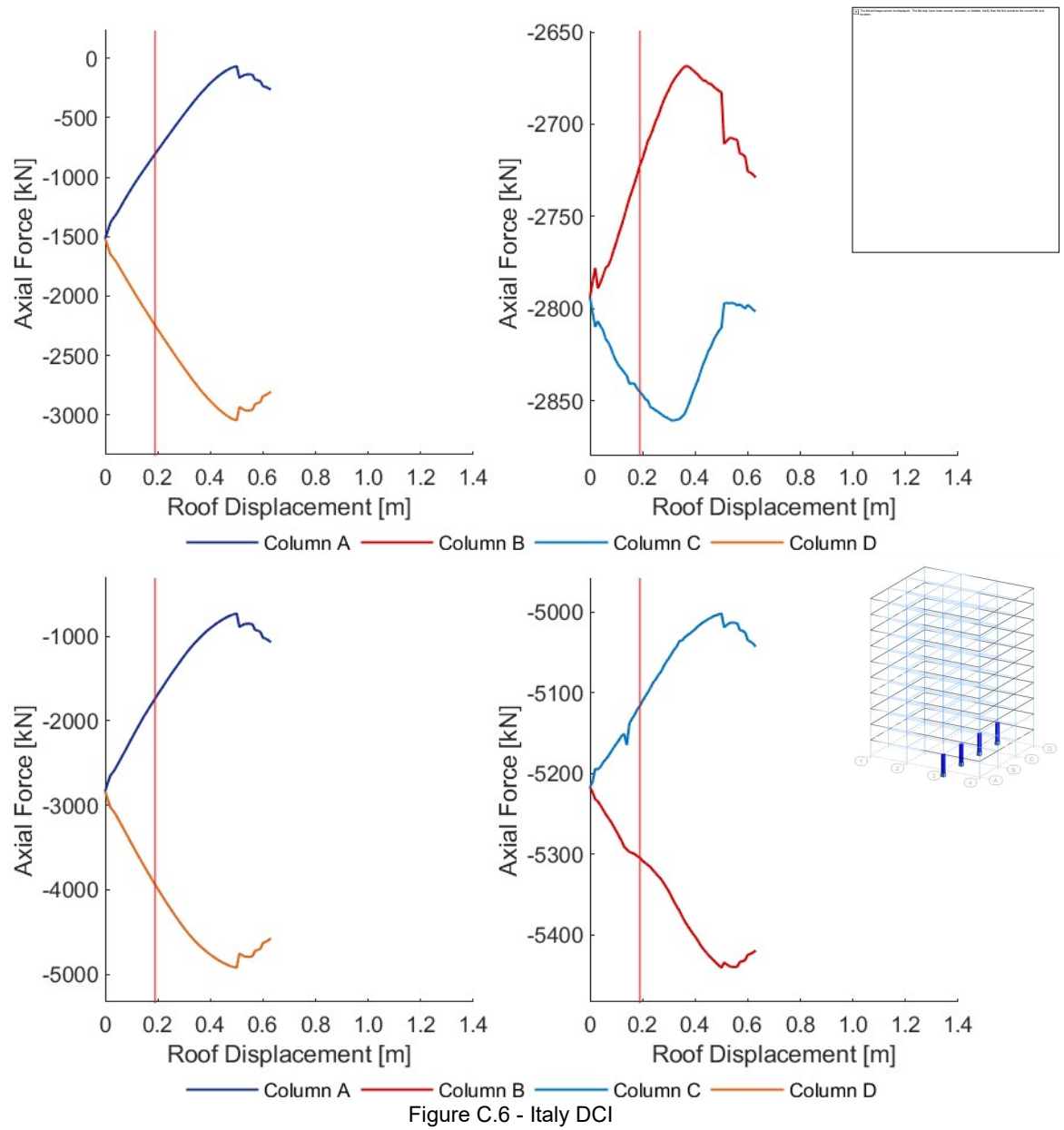
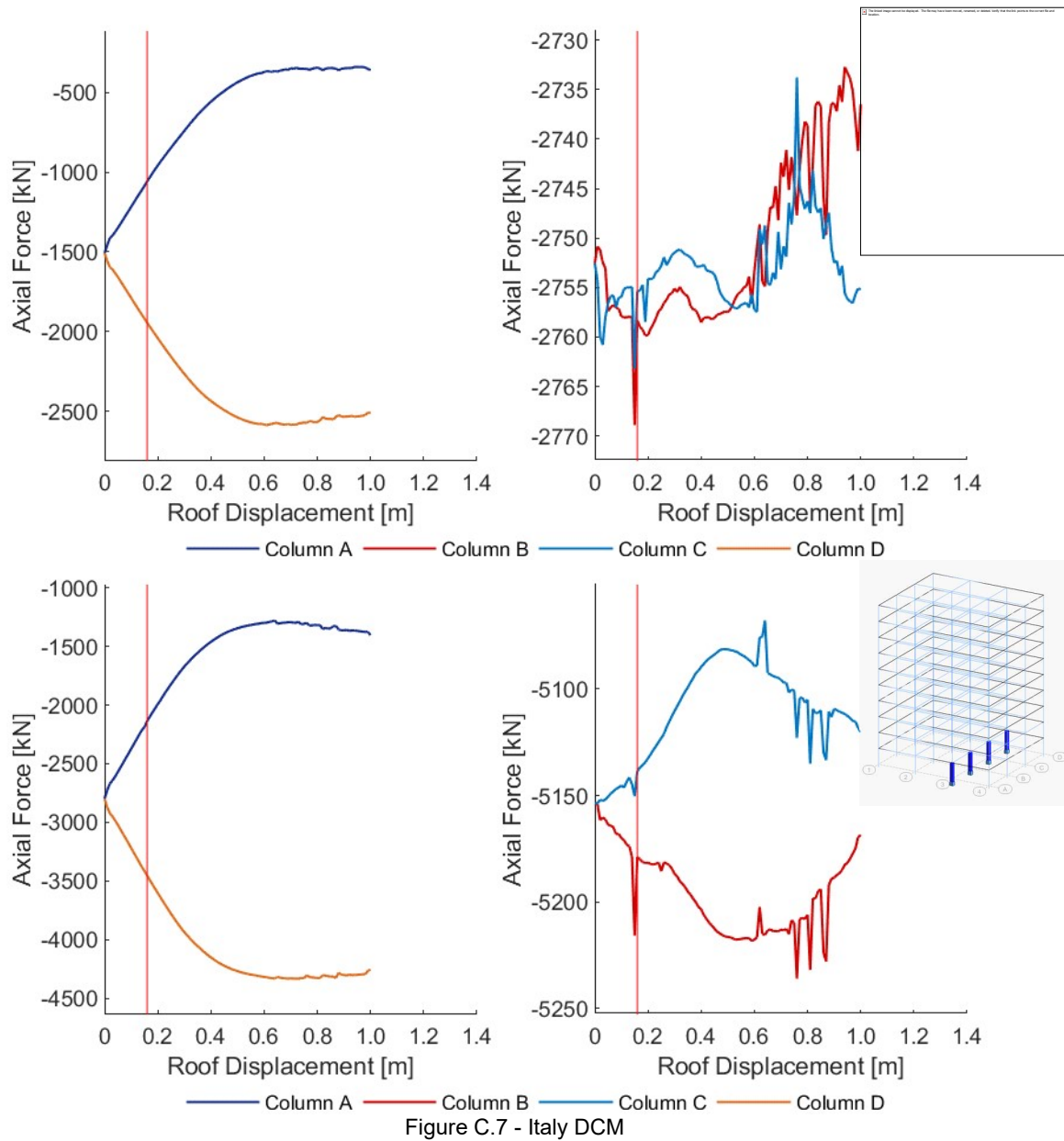
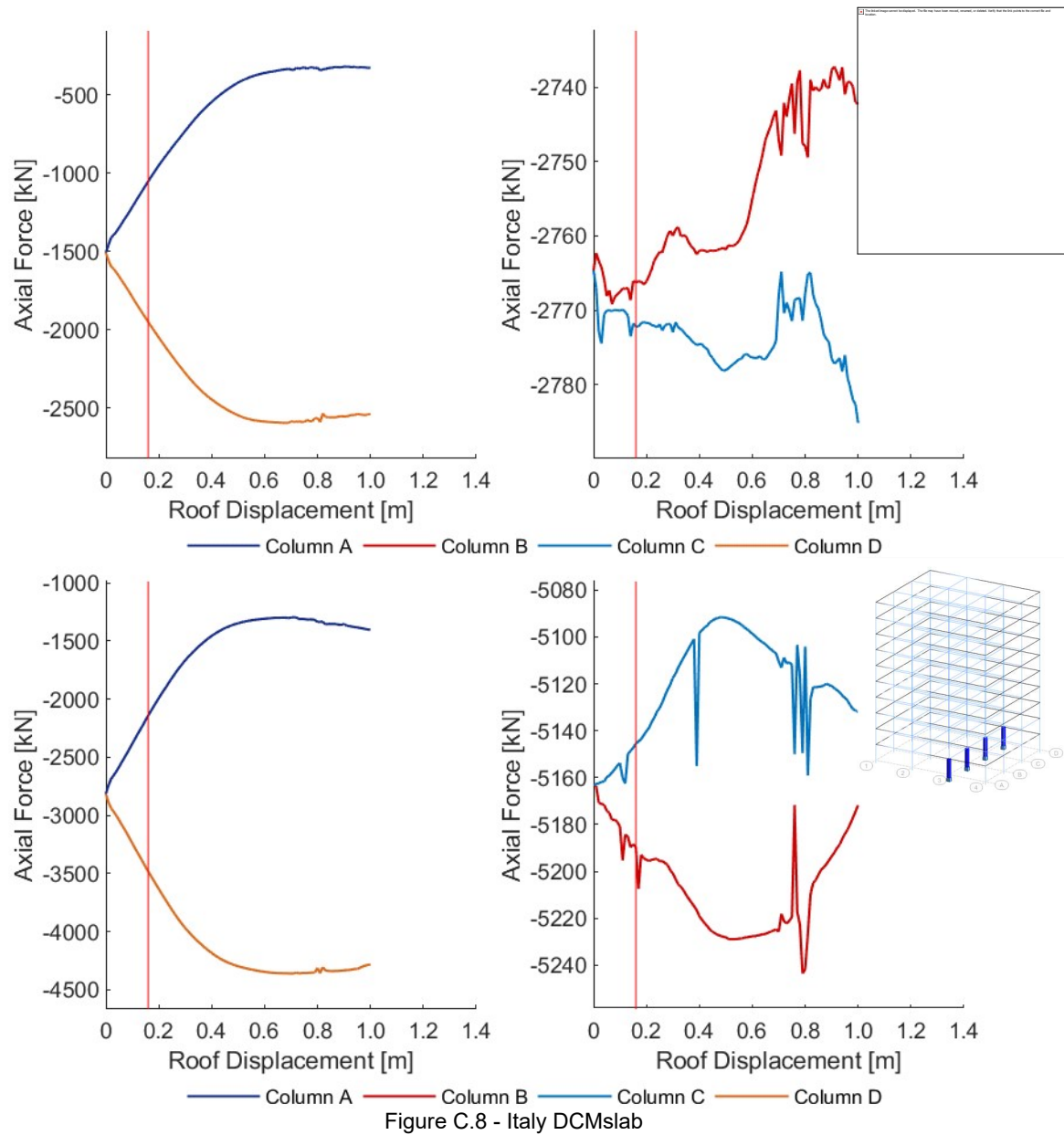


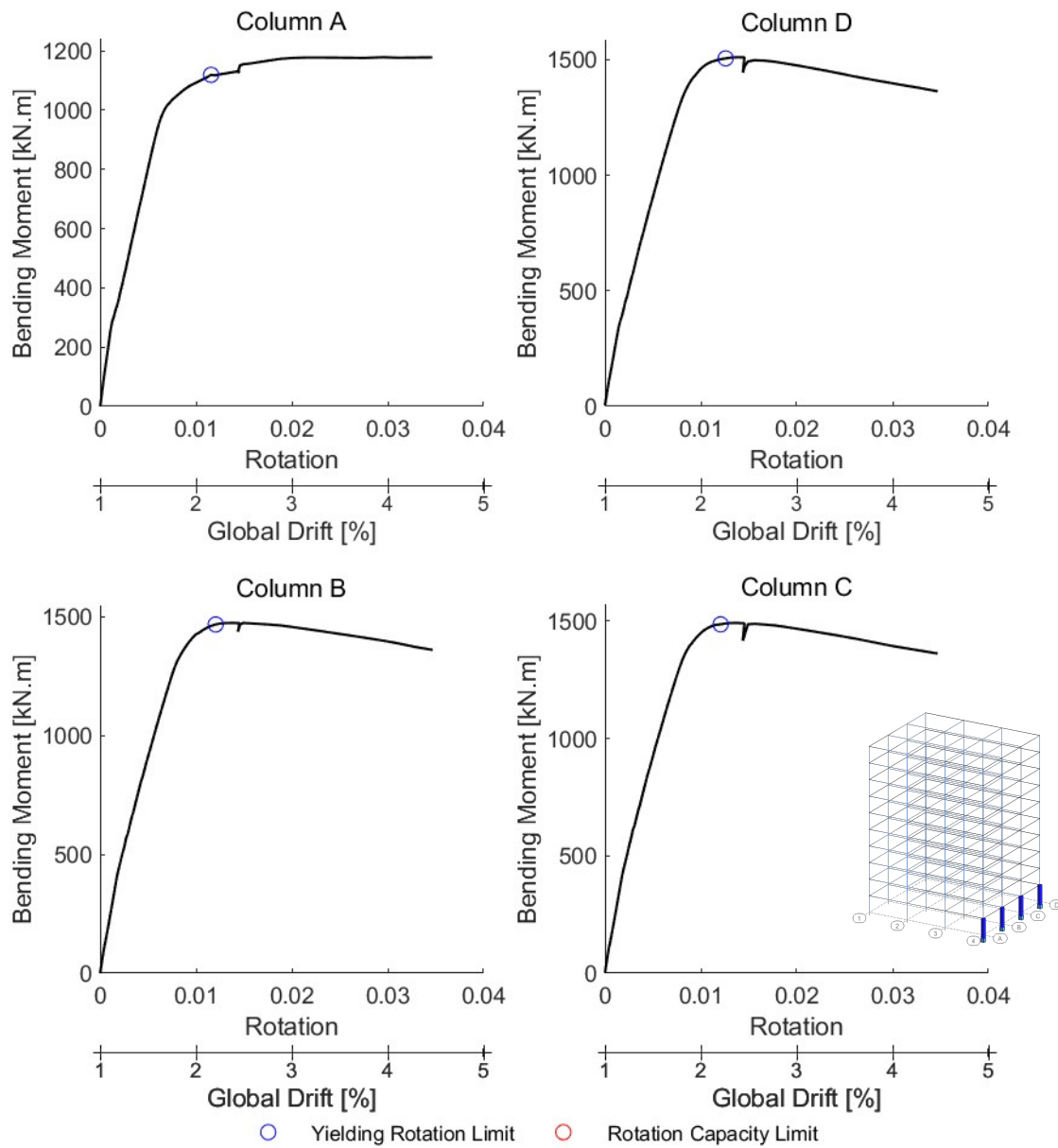
Figure C.5 - Italy DCL







Annex D PUSHOVER ANALYSIS – BENDING - ROTATION DIAGRAM



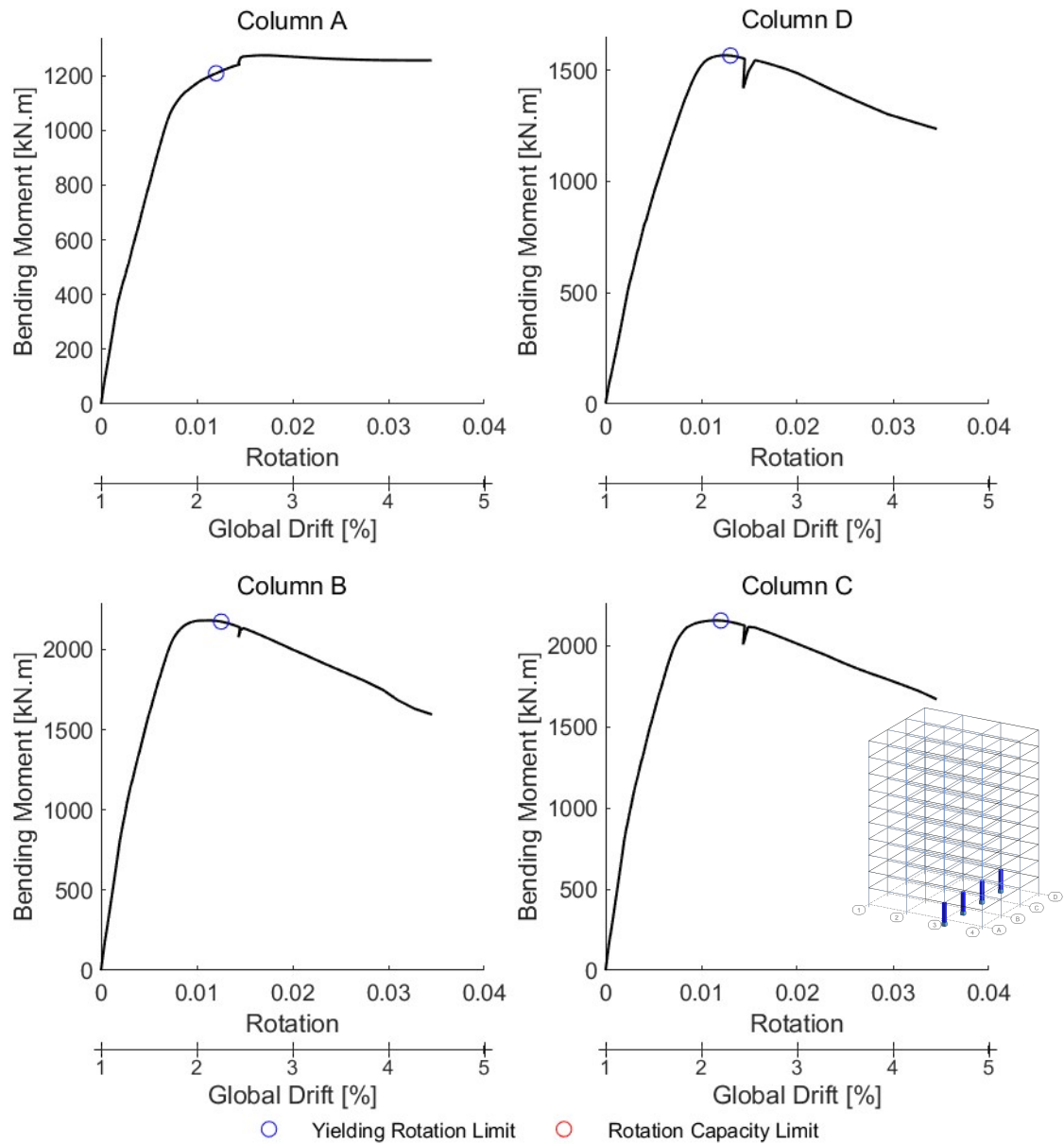


Figure D.2 - Portugal DCL interior frame

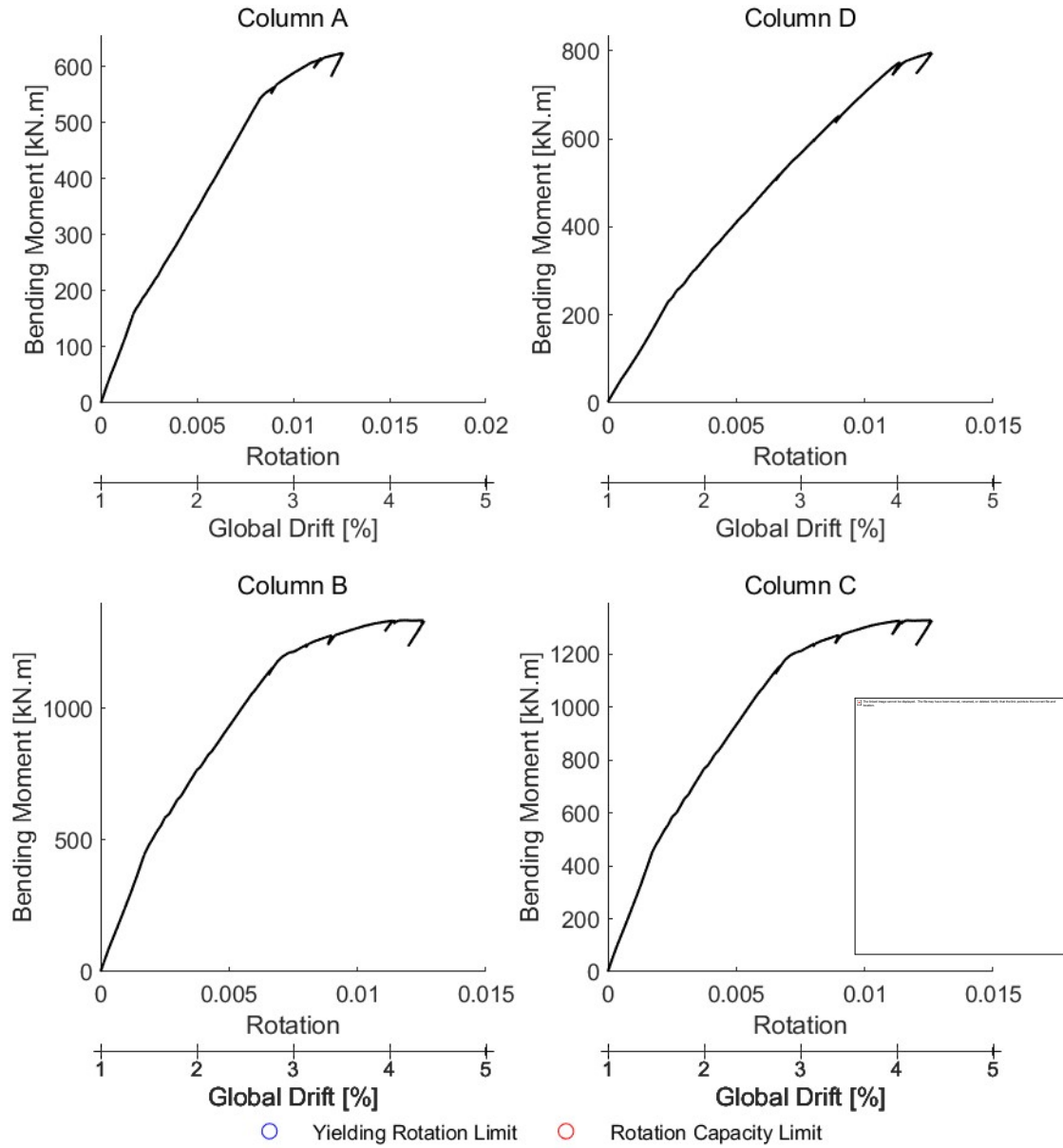


Figure D.3 - Portugal DCI exterior frame

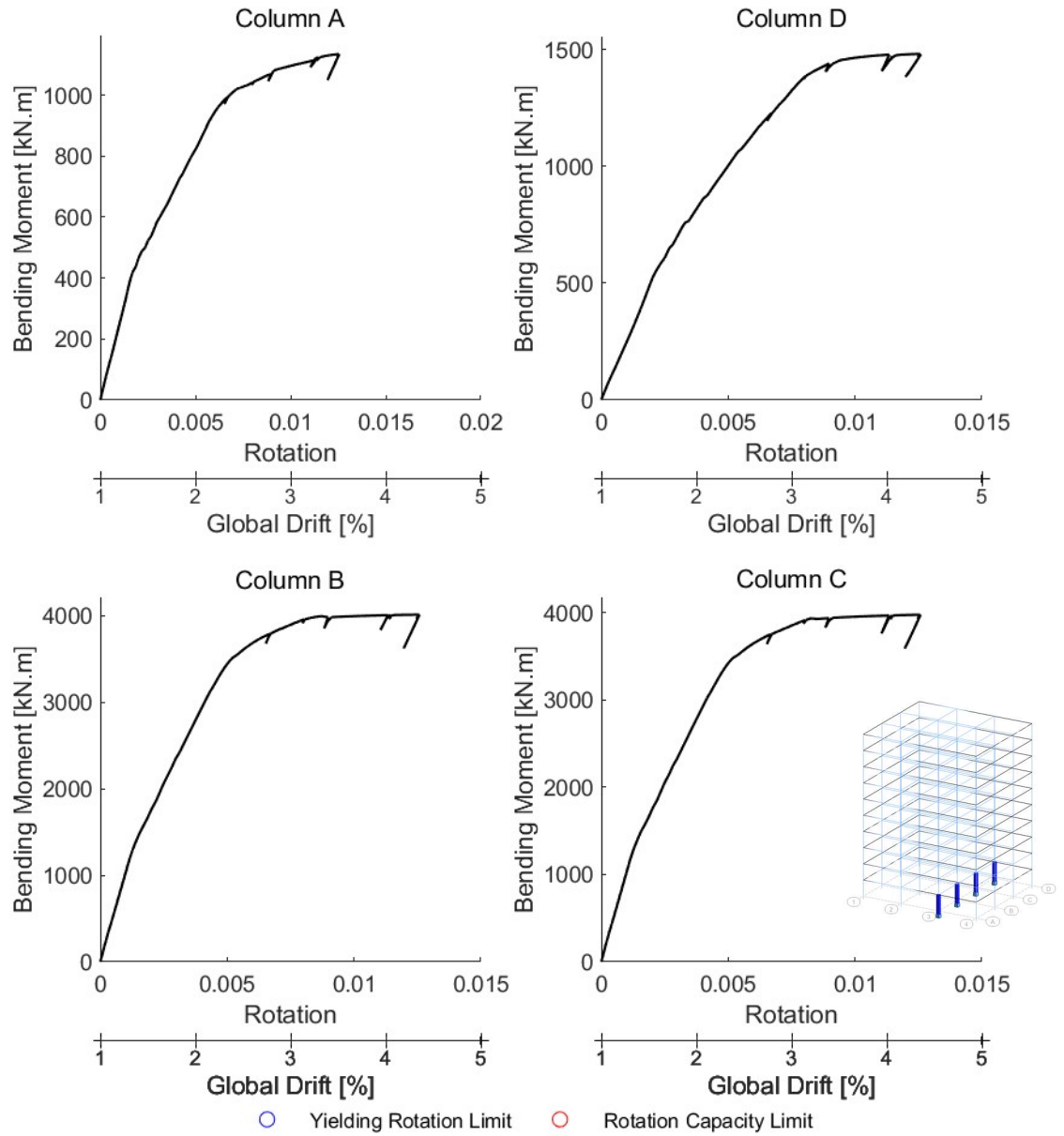


Figure D.4 - Portugal DCI interior frame

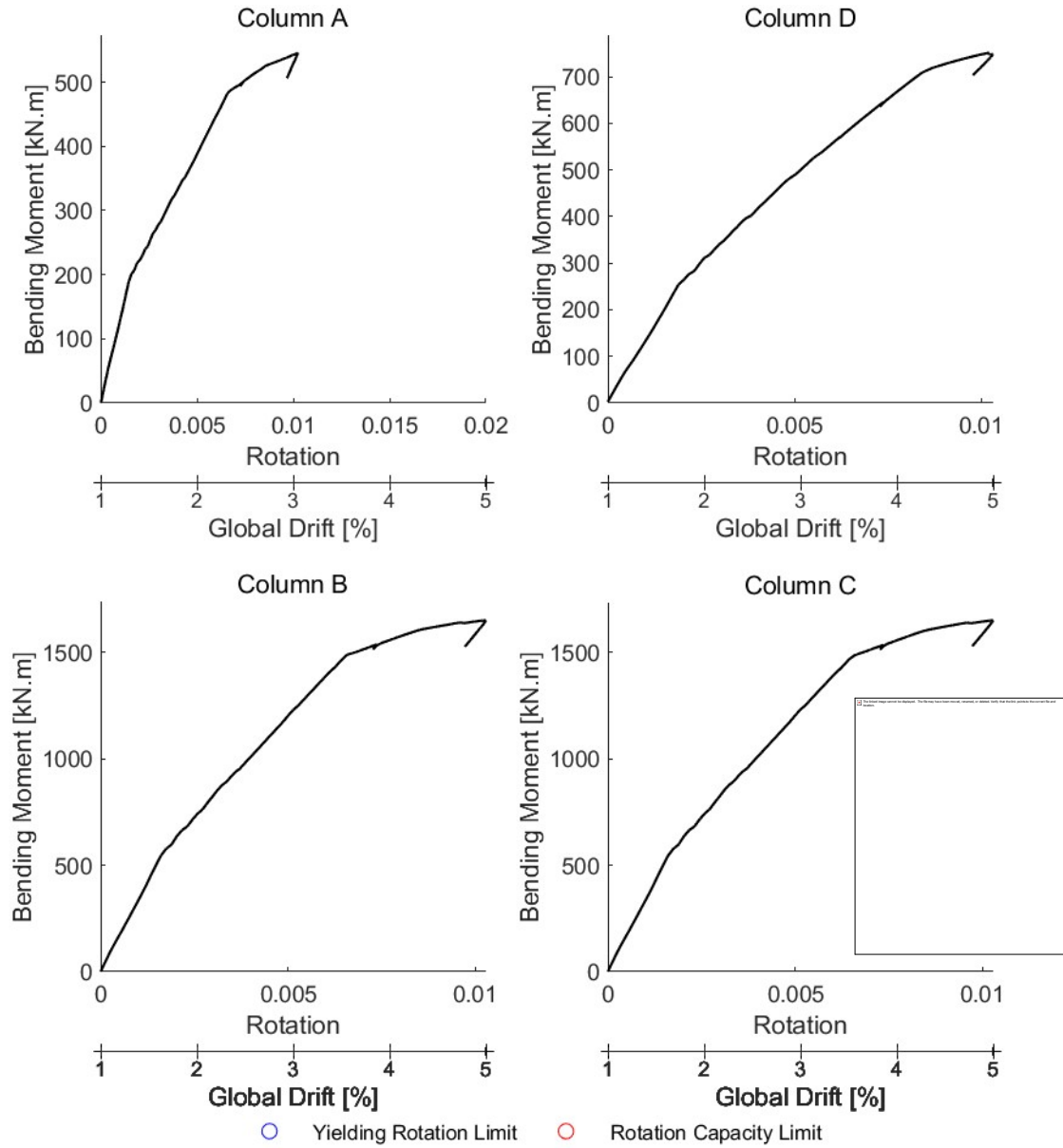


Figure D.5 - Portugal DCM exterior frame

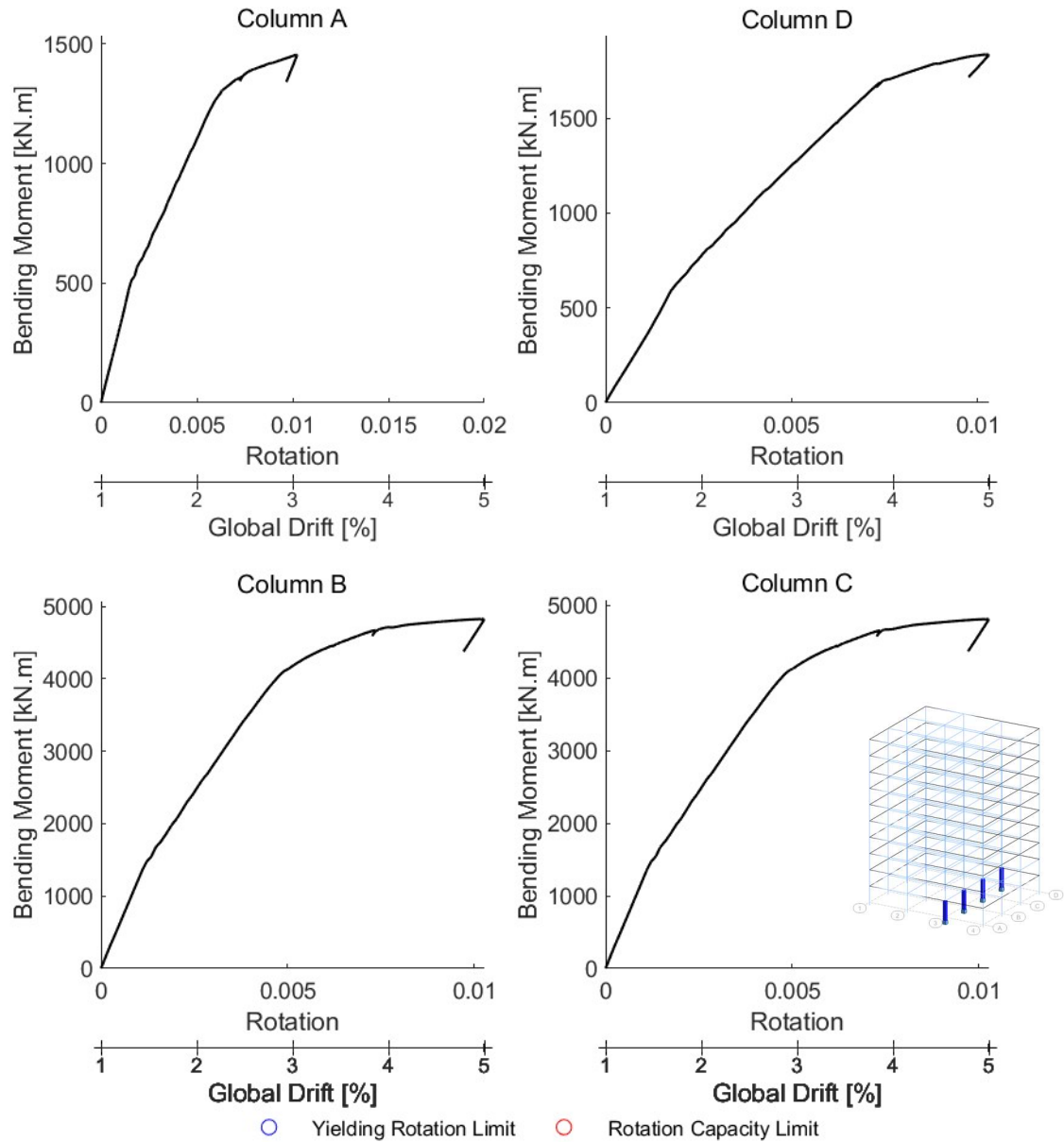


Figure D.6 - Portugal DCM interior frame

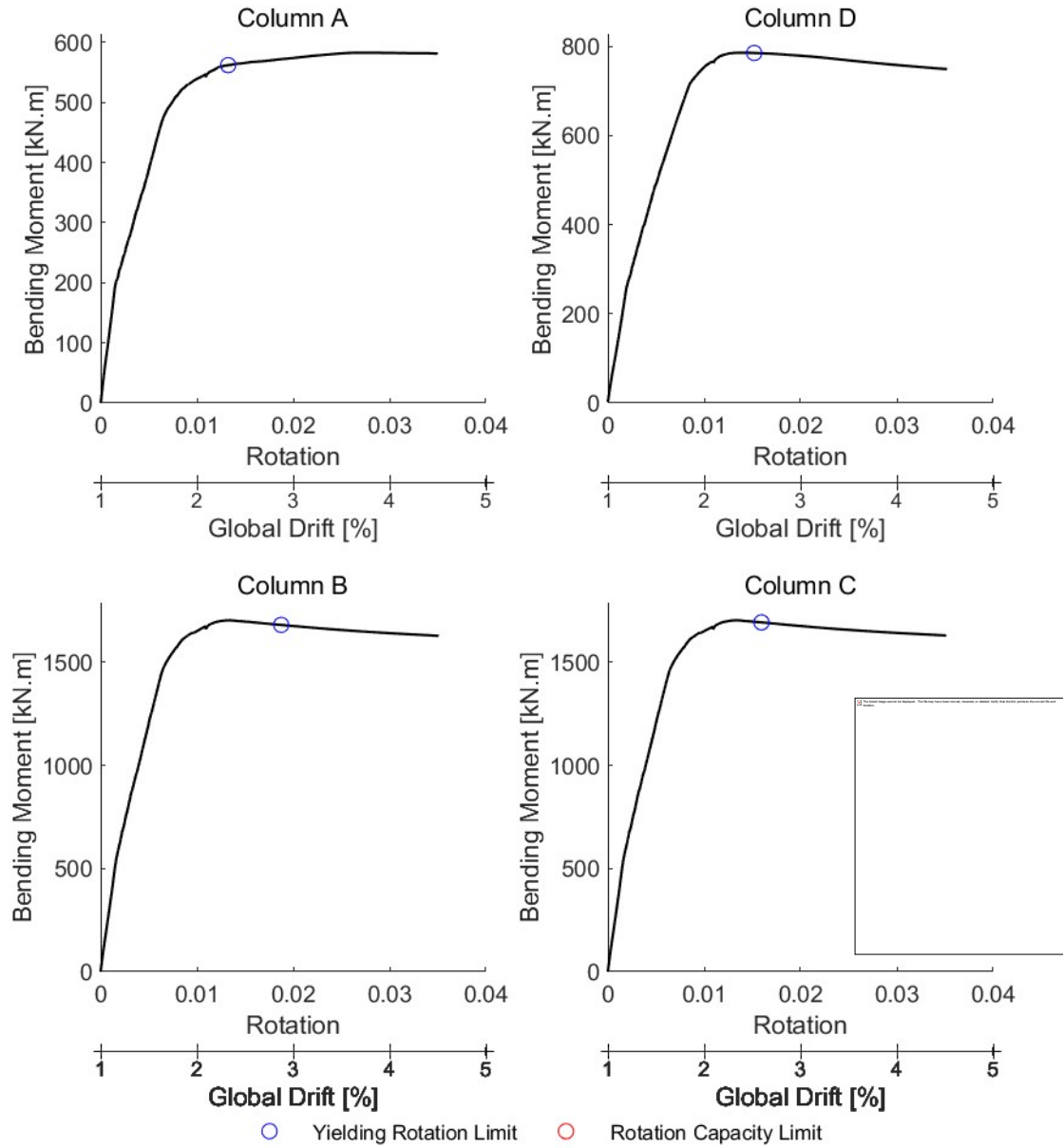


Figure D.7 - Portugal DCMSLAB exterior frame

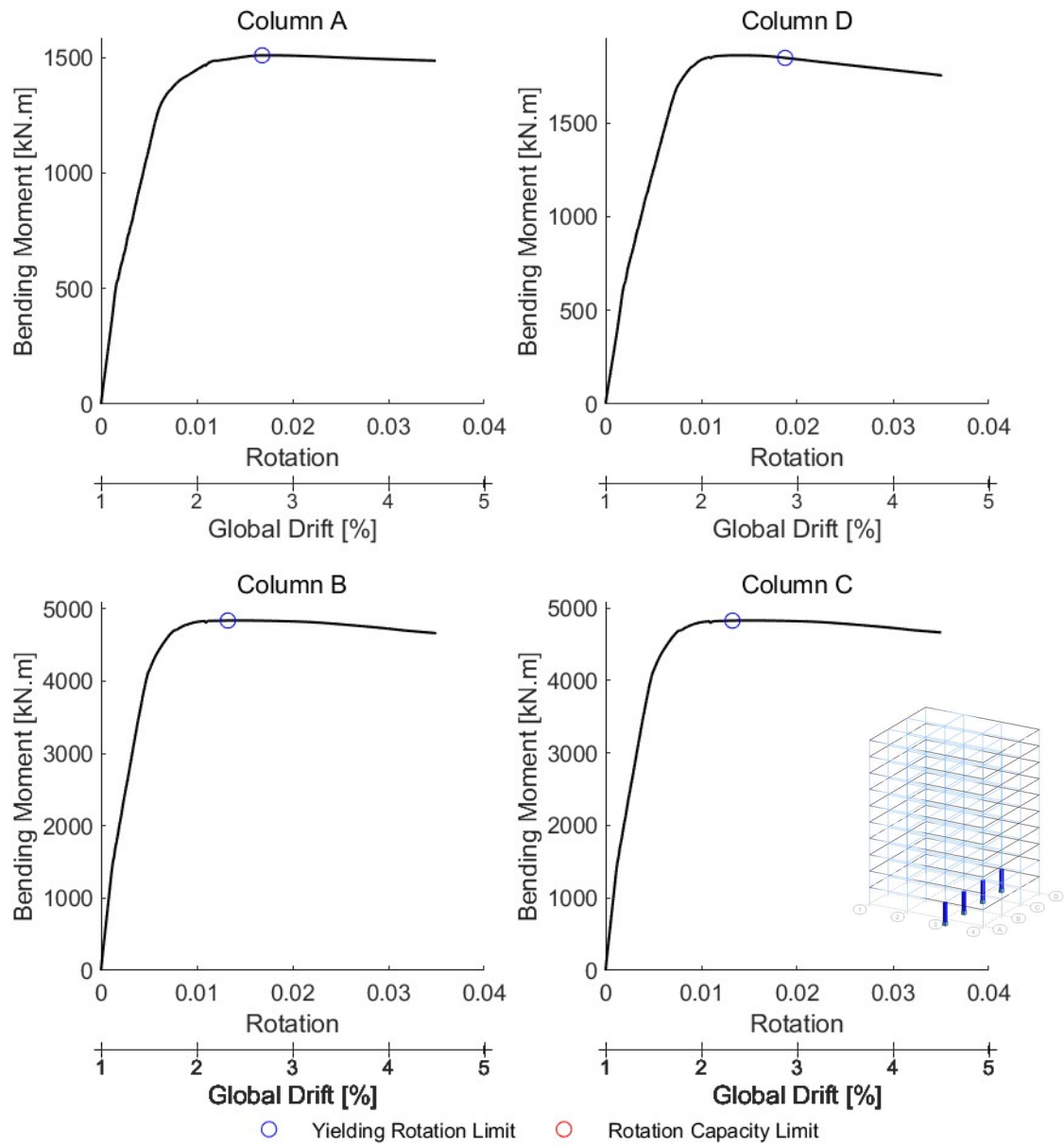


Figure D.8 - Portugal DCMslab interior frame frame

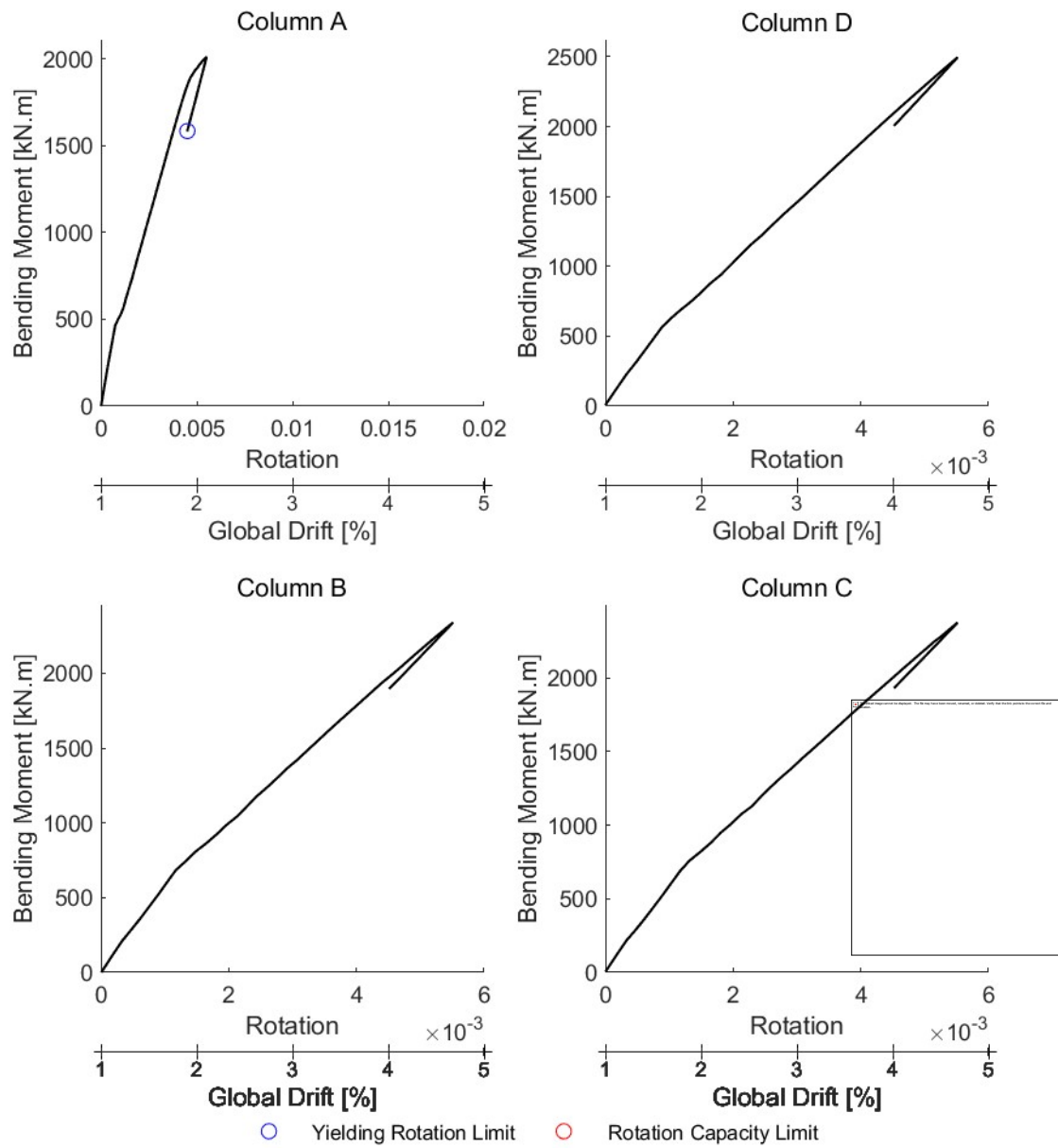


Figure D.9 - Italy DCL exterior frame

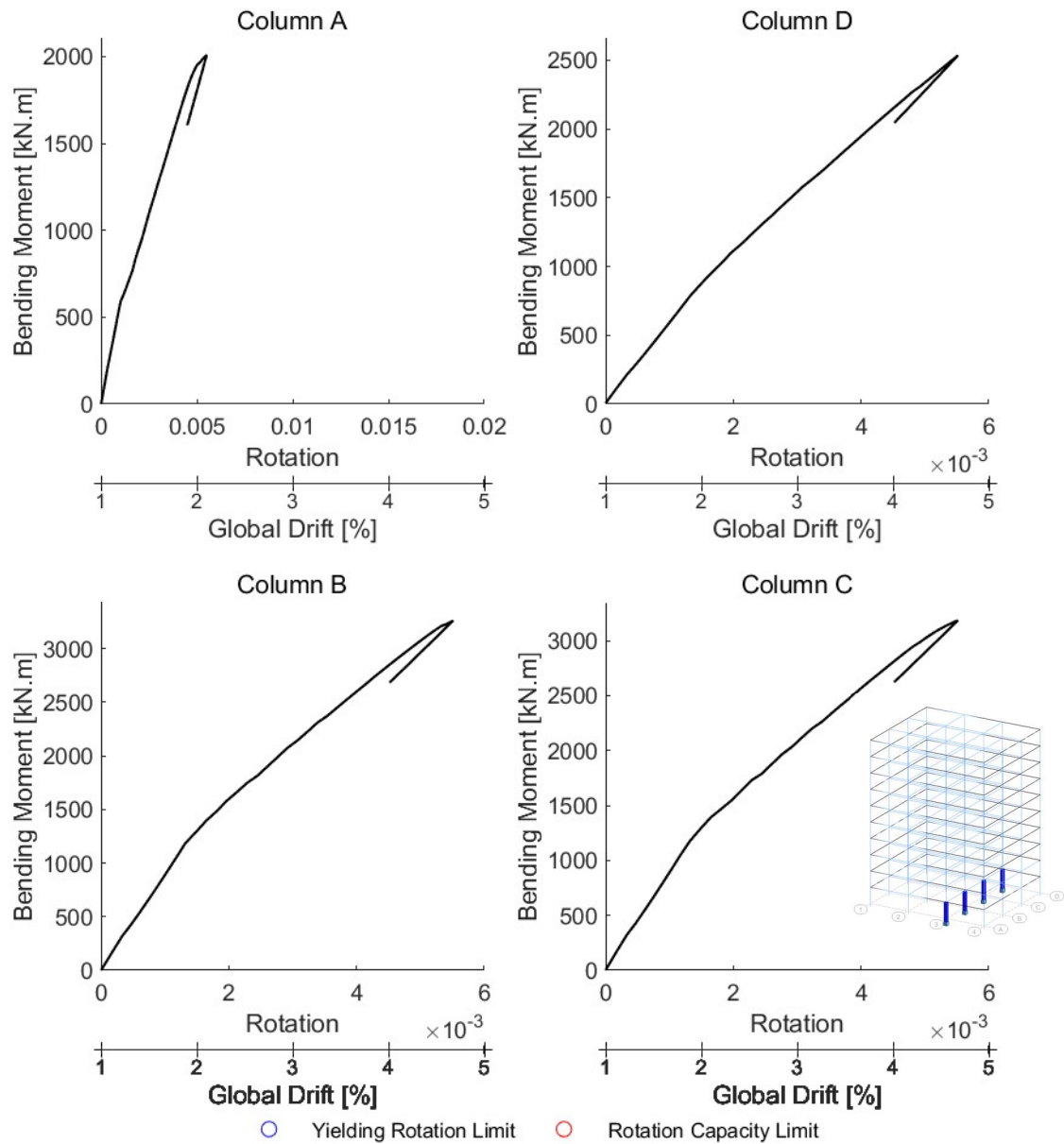


Figure D.10 - Italy DCL interior frame

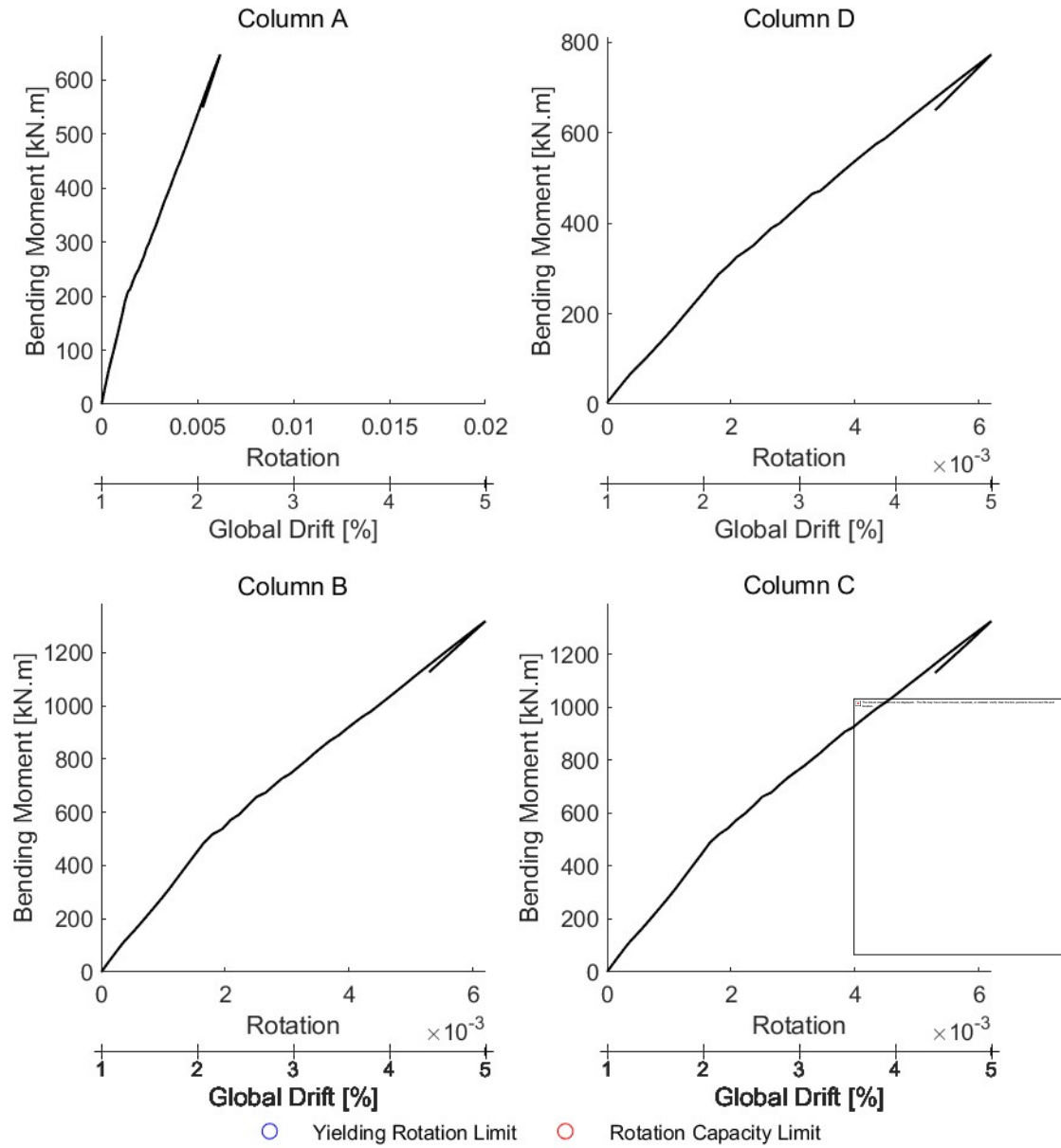


Figure D.11 - Italy DCI exterior frame

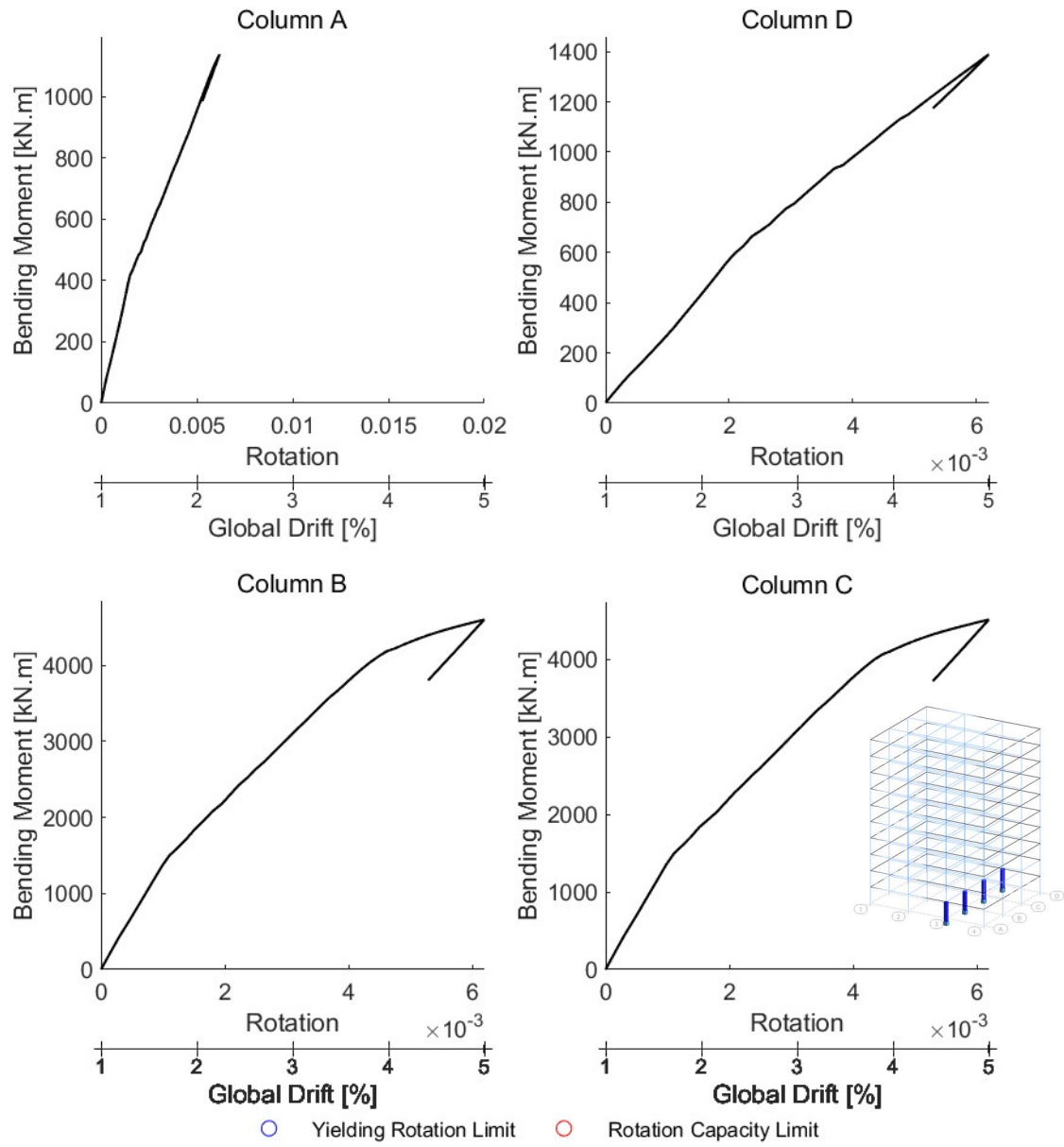


Figure D.12 - Italy DCI interior frame

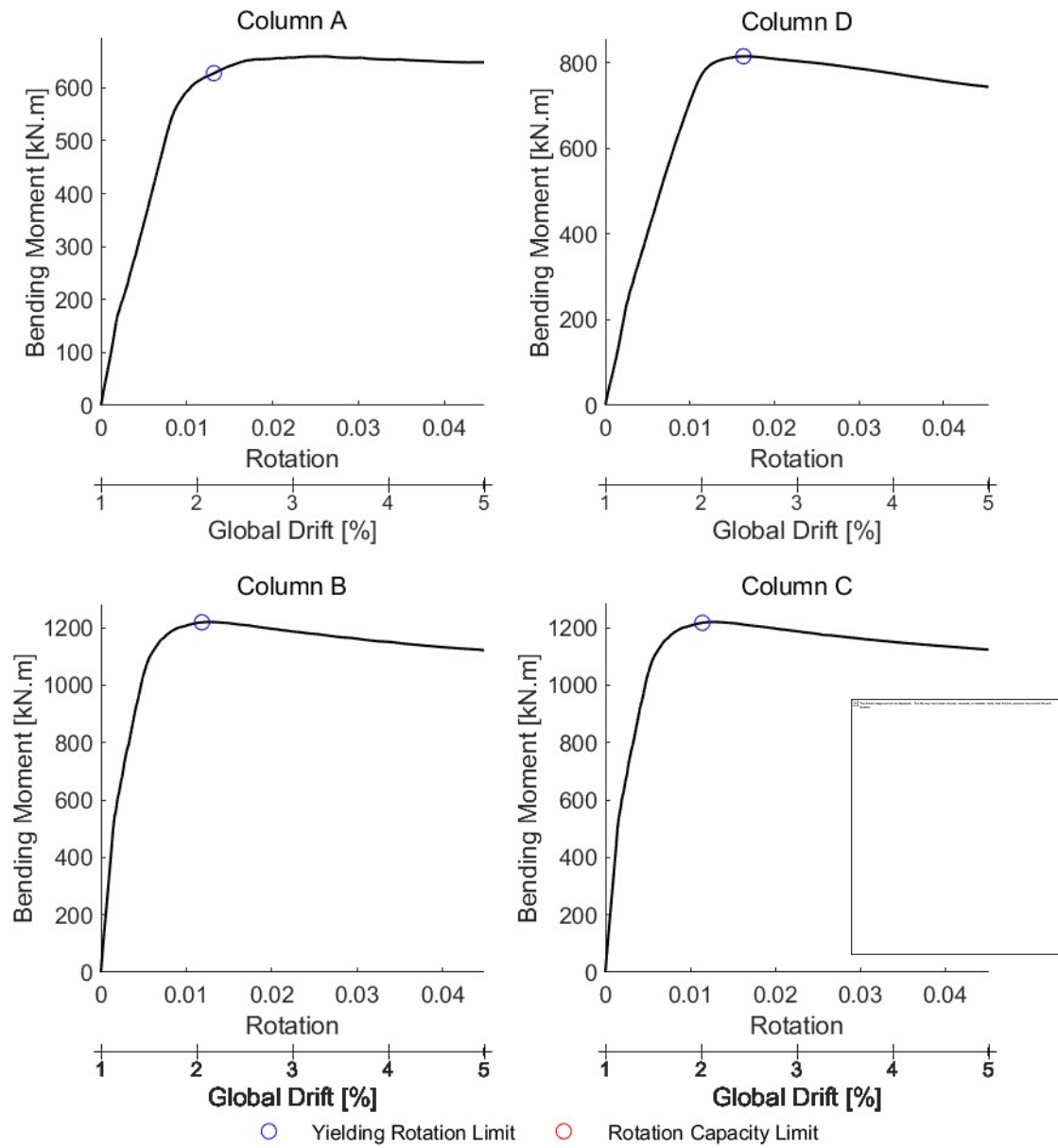
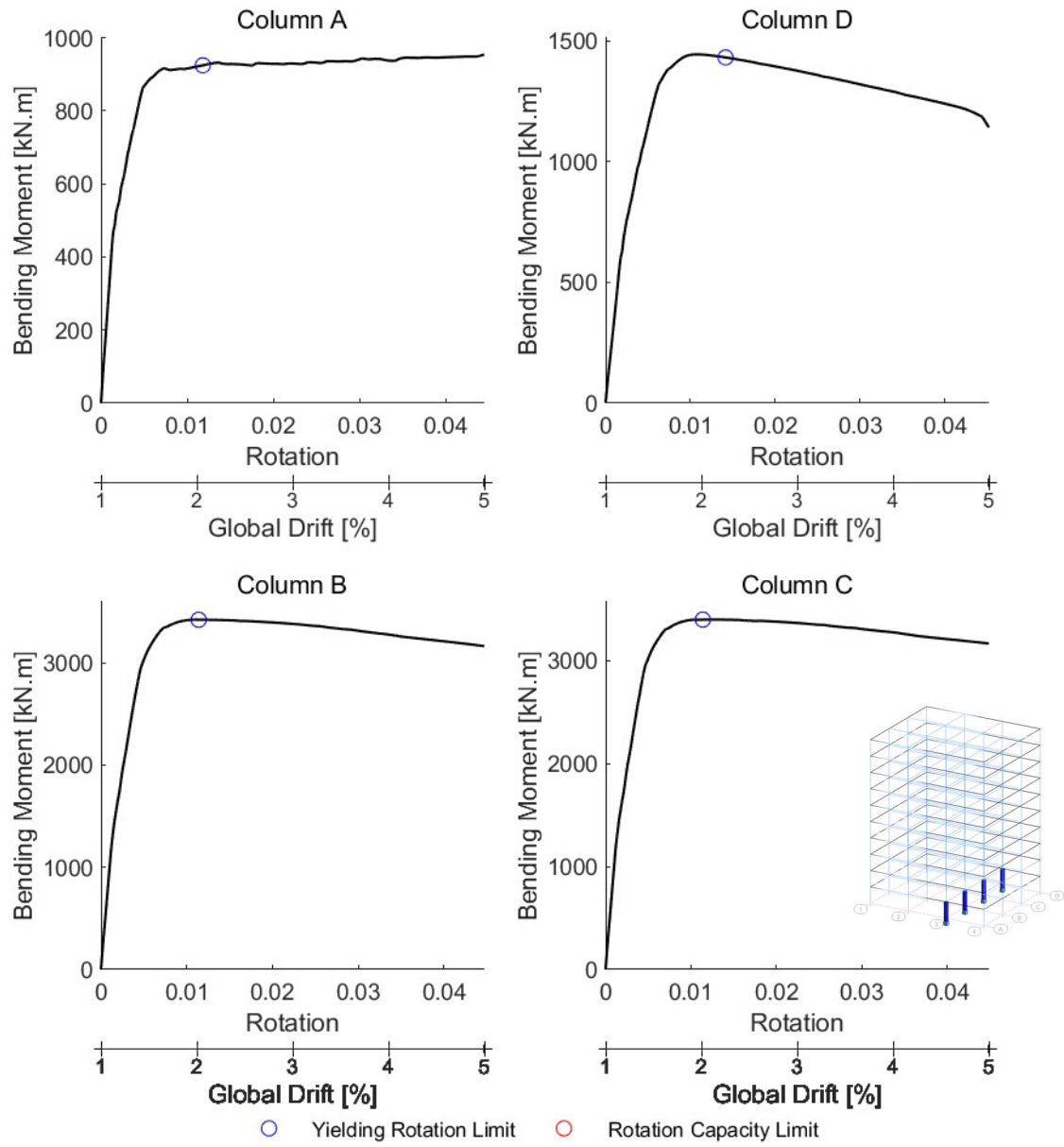


Figure D.13 - Italy DCM exterior frame



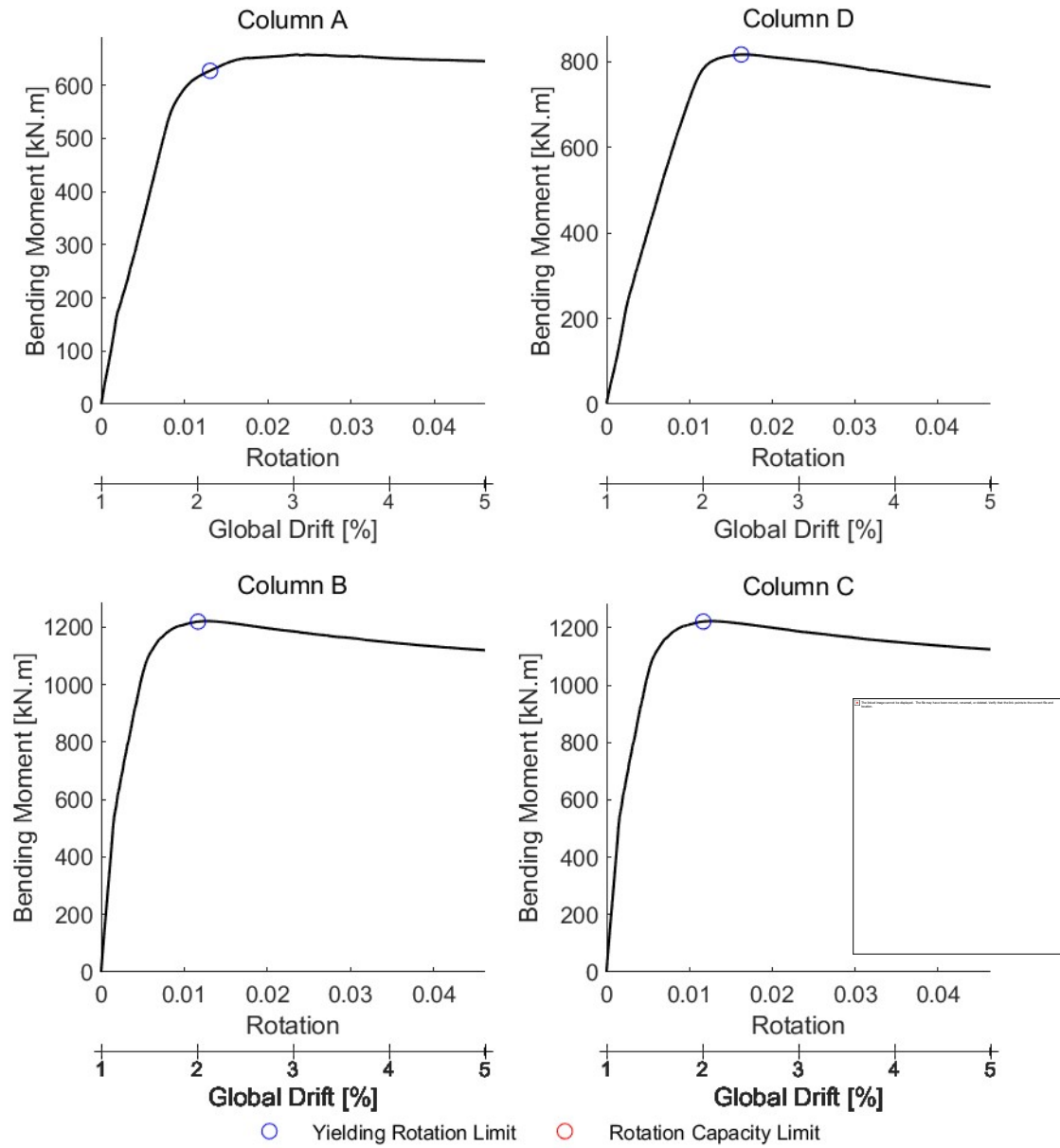


Figure D.15 - Italy DCMslab exterior frame

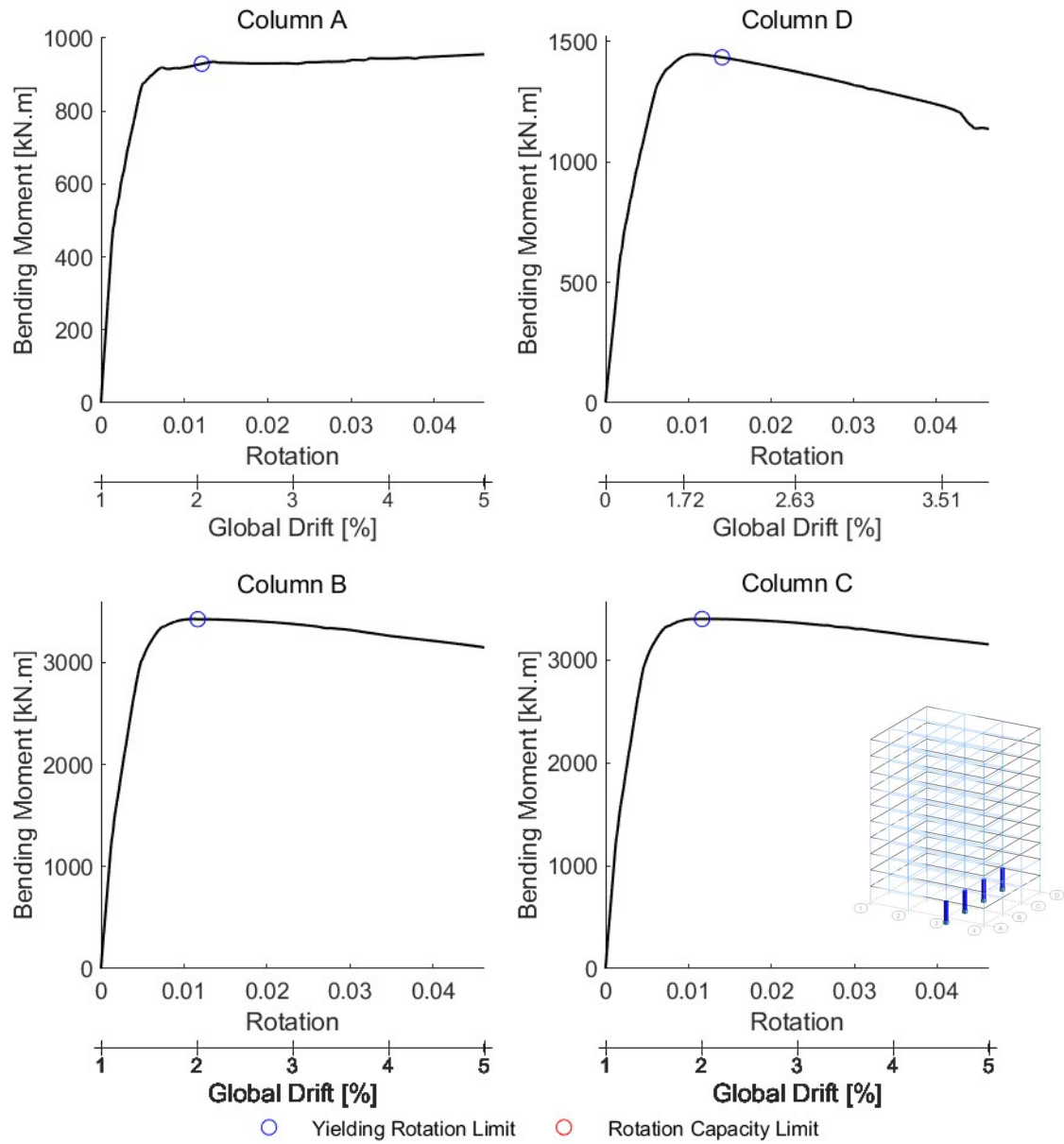


Figure D.16 - Italy DCMslab interior frame