

Disaggregation Of Class-Level Seismic Fragilities Using Reduced- Order Models

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A meus Pais

“Learn from yesterday, live for today, hope for tomorrow. The important thing is not to stop questioning”

Albert Einstein

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RESUMO

Na avaliação do risco sísmico a nível regional ou de portfólio, é bastante comum que os investigadores empreguem fragilidades que representem amplas classes estruturais. Estas são tipicamente caracterizadas pela data de construção (ou seja, nível do código sísmico), material (ou sistema de resistência à carga lateral), altura (número de pisos), etc. Assim, as classes de fragilidade têm sido publicadas ao longo dos anos para várias tipologias para edifícios industriais, residenciais ou outros tipos de estruturas, como, por exemplo, o Modelo Europeu de Risco Sísmico lançado em 2020 (ESRM20). Tais curvas de fragilidade podem substituir o dispendioso processo, tanto em termos de tempo como de financiamento, de desenvolvimento de fragilidades específicas de edifícios no âmbito de um projeto de avaliação de risco sísmico, enquanto características salientes, como assimetrias planas ou verticais, a presença de recuos e propriedades semelhantes individualmente importantes são suavizadas para se enquadrarem numa classe ampla.

Procurando um caminho intermédio entre a fragilidade específica do edifício e a fragilidade genérica de classes, propõe-se uma metodologia para desagregar as fragilidades sísmicas ao nível da classe utilizando sistemas de múltiplos graus de liberdade (MDOF) "equivalentes" para determinar a diferença relativa entre as curvas de fragilidade causadas por diferentes macrocaracterísticas e calcular os fatores de correção a aplicar. Além disso, quando o modelo original, tipicamente modelos de um grau de liberdade (SDOF), está disponível, a incorporação de sismos específicos ao local também é permitida, oferecendo correções de tendências para o local de interesse.

A título de exemplo, aplica-se a desagregação considerando a forma da planta de edifícios escolares típicos com base em tipologias de betão armado descritas no ESRM20. Foram utilizados dados de inquéritos a escolas em Boeotia, Grécia. Cada curva de fragilidade que representa a classe é assim substituída por três curvas de fragilidade desagregadas, dada a forma de planta (especificamente quadrada, retangular e em forma de L) do edifício. Estas são capazes de reconstruir perfeitamente a fragilidade da classe original, enquanto individualmente mostram diferenças de até 28% nos valores da mediana com reduções na dispersão. Estes resultados apresentam uma abordagem prática para reduzir o enviesamento e a variância dos edifícios, explorando e combinando bases de dados de fragilidade de elevada qualidade já estabelecidas. Por sua vez, proporcionam flexibilidade, refinamento e maior resolução espacial para avaliações de risco regionais ou de portfólio, levando a avaliações geolocalizadas mais precisas e melhores estimativas da vulnerabilidade estrutural.

PALAVRAS-CHAVE: curvas de fragilidade, ESRM20, desagregação, planta, avaliação do risco

ABSTRACT

In seismic risk assessment at the regional or portfolio level, it is quite common for researchers to employ “ensemble” fragilities that represent broad classes of structures. These are typically characterized by age of construction (i.e., seismic code level), material (or lateral-load resisting system), height (number of stories) etc. Accordingly, class fragilities have been published throughout the years for various class-typologies for industrial, residential buildings or other types of structures as well, e.g. the European Seismic Risk Model released in 2020 (ESRM20). Such fragilities can replace the costly process, both in terms of time and funding, of developing building-specific fragilities within a seismic risk assessment project; at the same time, salient characteristics, such as plan or vertical asymmetries, the presence of setbacks, and similar individually important properties are smoothed over to fit within the broad class.

Seeking a middle path between the building-specific and class-generic fragility, a methodology is proposed to disaggregate class-level seismic fragilities by using “equivalent” multi-degree-of-freedom (MDOF) models to ascertain the relative difference between the fragility curves caused by different macro-characteristics and to calculate the correction factors to be applied. In addition, where the original, typically single-degree-of-freedom (SDOF), model is available, the incorporation of site-specific ground motions is also enabled, offering bias-correction for the site of interest.

As an example, disaggregation is applied to consider the plan-shape of typical school buildings based on reinforced concrete typologies described in ESRM20. Survey data of schools from Boeotia Greece were employed. Each class-typology fragility is thus replaced by three disaggregated fragility curves, given the plan-shape (specifically square, rectangle, and L-shape) of the building. These are able to perfectly reconstruct the original class fragility, while individually showing differences up to 28% in the median values with reduced dispersions. These results present a practicable approach to reducing bias and variance for buildings by exploiting and matching already established high-quality fragility databases. In turn, they provide flexibility, refinement, and higher spatial resolution for regional or portfolio risk assessments, leading to more precise and better geolocated evaluations of structural vulnerability.

KEYWORDS: fragility curves, ESRM20, disaggregation, plan-shape, risk assessment

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SYMBOLS

Sa – Spectral acceleration
Sd – Spectral displacement
Sdy – Yielding displacement
Sdult – Ultimate displacement
T1 – Fundamental period of vibration
M – Diagonal mass matrix
U – Displacement vector
 $\dot{U}$ – Derivative of displacement vector
C – Damping matrix
R – Vector of internal forces
 $\ddot{x}_g$ – Ground acceleration
 ϕ – Displacement shape
 ϕ^T – Transpose displacement shape
P – Externally applied loads
 η – Displacement vector in modal coordinates
 η^* - Displacement of the SDOF
 η^{**} - Acceleration of the SDOF
 m^* - Equivalent mass of the SDOF
 c^* - Equivalent viscous damping constant of the SDOF
 F^* - Force of the SDOF
 Γ – Modal participation factor
 V_b – Base shear
 δ_R – Roof displacement
 ϕ – Normalized Gaussian probability density function
 μ – Mean of the normal distribution
 σ – Standard deviation
 Θ – Logarithmic mean
 β – Logarithmic standard deviation
 λ – Correction factor imposed by Eurocode 8
q – Behaviour factor
 a_g – Design ground acceleration

S – Soil factor

g – Gravitational force

C_t – Correction factor regarding the material of the building imposed by Eurocode 8

H – Height of the building

K – Stiffness

W – Angular frequency

a_{S2S} – Plan-shape modifier to square buildings

a_{S2R} – Plan-shape modifier to Rectangles buildings

a_{S2L} – Plan-shape modifier to L-plan shape buildings

ACRONYMS AND ABBREVIATIONS

ADRS – Acceleration Displacement Response Spectrum
ASCE – American Society Civil Engineering
CA – Cloud Analysis
CDF – Cumulative distribution function
CP – Collapse Prevention
DL – Damage Limitation
DS – Damage State
EAK2000 – Greek Seismic Regulation 2000
EC8 – Eurocode 8
EDP – Engineering Demand Parameter
EPPO – Earthquake Planning and Protection Organization of Greece
ESRM20 – European Seismic Risk Model 2020
FEMA – Federal Emergency Management Agency
GEM – Global Earthquake Model
GMPE – Ground Motion Prediction Equation
IDA – Incremental Dynamic Analysis
IM – Intensity Measure
IO – Immediate Occupancy
LLRS – Lateral Load Resisting System
LS – Life Safety
MDOF -Multi Degree of Freedom
MDR – Mean Damage Ratio
MLE – Maximum Likelihood Method
MSA – Multiple Stripe Analysis
NC – Near Collapse
OL – Operational Level
PDF – Probability density function
PGA – Peak Ground Acceleration
PMF – Probability mass function
SCWB – Strong Column Weak Beam

SD – Significant Damage

SDOF – Single Degree of Freedom

WCSB – Weak Column Strong Beam

1

INTRODUCTION

1.1. BACKGROUND TO THE THESIS AND OBJECTIVES

Fragility curves are fundamental tools in seismic risk assessment, providing a probabilistic measure of a building's likelihood to reach or exceed a certain damage state under varying seismic intensities. In many instances, studies related to the fragility and vulnerability of structures either rely on conducting fragility curves through modelling the structure and performing analyses, or they exploit the fragility curves of various literature sources. Literature fragility curves are typically categorized into class-typologies for easier and quicker selection for the various structures being evaluated. However, these classes are defined using generalized properties of the structures and might not be addressing more detailed characteristics, that usually are contained within each one the defined classes.

The European Seismic Risk Model 2020 (ESRM20), developed by Crowley et al. [1] was designed to present fragility and vulnerability curves for the building stock across Europe, thereby providing a faster and more straightforward seismic risk analysis for regional of portfolio risk assessment. Following such an approach it is noted that completely different buildings in different seismic regions of Europe, but falling within the same class-typology, are assigned the same fragility curve. The class-typologies cannot account for the unique structural and non-structural characteristics of individual buildings, which can differ in several aspects such as the presence of soft-stories, ductile or brittle elements (typically accounted for only indirectly via the code level / age of construction), the plan-shape representing plan irregularities, torsional effects resulting from mass eccentricity, conservation levels, etc. This indicates that there are still significant differences among buildings, and adopting the same fragility curves for all, may not be the most detailed approach as buildings will respond differently to seismic actions. In other words, the standard class-level fragility approach may be accurate at the ensemble level, e.g., for aggregated losses over an entire portfolio of assets, but cannot distinguish impact at higher resolutions.

Therefore, this work aims to find a way to obtain fragility and vulnerability curves derived from literature-based fragility curves that can better describe the characteristics of more well-defined buildings, taking into account those differentiations they might have (e.g., ESRM20). Three methodologies for disaggregating these fragility curves are proposed, based on a case study of buildings observed in Greece. In this work, the disaggregation focuses on the plan-shape, but the method could be employed for other structural characteristics. These methodologies use probabilistic laws, such as the law of total expectation and the law of total variance, in their formulation to disaggregate the curves into three new ones using a corresponding factor that consider the building's plan-shape, differentiating between square-shape, rectangle-shape, and L-shape. Since the study focuses on a specific area and to effectively assess the future reliability of those buildings, hazard consistent ground motions were

employed, while Incremental Dynamic Analysis (Vamvatsikos and Cornell [2]) were performed to assess the building's response to the selected earthquakes.

1.2. OUTLINE OF THE THESIS

The present thesis has been divided in seven different chapters. The first, and current, Chapter provides a general introduction to several aspects addressed in the thesis and the main scope.

The second Chapter commences with a description of capacity curves for the European building stock derived from the European Seismic Risk Model 2020 (ESRM20). It begins by introducing the taxonomy employed for categorizing buildings in the case study. This taxonomy is briefly outlined to facilitate the reader's comprehension of the scope of this work. Furthermore, the chapter elaborates on the development of capacity curves, not only within the ESRM20 project but also within the broader context. It elucidates the methodology behind obtaining capacity curves for residential masonry and concrete building classes utilized in ESRM20.

In the third Chapter, an introduction to probability distributions is provided to acquaint the reader with the terminologies utilized throughout the thesis. The distinction between normal and lognormal distributions is explained, alongside the rationale for the prevalent use of the lognormal distribution in earthquake engineering. Additionally, the concept of fragility curves and their calculation methods are expounded upon. The chapter also delves into Intensity Measure (IM), Engineering Demand Parameter (EDP), and the various damage states, providing detailed descriptions. Concluding this chapter, the levels of damage states utilized in ESRM20 are delineated, along with an overview of how Cloud Analysis was employed to develop fragility curves.

Chapter four details the building survey data that laid the groundwork for contemplating the disaggregation process and determining the class-typologies suitable for disaggregation. Real data from building surveys facilitated a clearer understanding of how to characterize class-typologies for buildings in Greece. Additionally, the period of vibration for Single-Degree-Of-Freedom (SDOF) models used in ESRM20 was inferred based on the observed class-typologies from building surveys. Furthermore, the fragility curves derived from the European Seismic Risk Model 2020 for four intensity measures and the damage state levels outlined in the preceding chapter are presented.

The fifth Chapter elucidates the methodology behind the disaggregation process, outlining all the steps involved to enhance understanding. Three approaches to disaggregating curves derived from Incremental Dynamic Analysis (IDA) results are presented, in contrast to Cloud Analysis utilized in ESRM20. Results for all the analysed approaches are described in detail.

In Chapter six, a vulnerability assessment is provided to the class-typologies that were disaggregated in this study. It is shown the damage-to-loss models that were chosen to be the most appropriate to apply to Greek buildings and then the fragility curves obtained by the disaggregation methodology were converted into vulnerability curves, where it is spotted the differences in losses among the different plan-shapes.

Seventh chapter and the final one, final remarks are made regarding the main conclusions to be taken from this work. Also, recommendations are made for future developments.

2

CAPACITY CURVES FOR EUROPEAN BUILDING STOCK

2.1. TAXONOMY

2.1.1. BUILDING CLASSES

A common classification scheme (i.e. taxonomy [3]) as the one applied in ESRM20 [1] was utilized to assess the buildings evaluated in this study, employing a singular classification scheme, so then it is possible to ensure that fragility models obtained in this work are compatible with the ones calculated and used by ESRM20¹

The building taxonomy derives from a revised international standard (the GEM Building Taxonomy, initially developed by Brzev et al. [4] and subsequently updated by Silva et al. [5]). This taxonomy enables the classification of buildings according to various structural attributes. The key characteristics chosen to ensure a uniform definition of building classes across Europe are the following:

- The main construction material (reinforced concrete, unreinforced masonry, reinforced/confined masonry, adobe, concrete block masonry, clay brick masonry, dressed stone masonry, rubble stone masonry, steel, and wood/timber).
- Lateral load resisting system, LLRS (dual frame-wall system, infilled frame, load bearing wall, moment frame, and braced frame).
- Code level or Ductility (Absence of seismic design, low code level, moderate code level, high code level, non-ductile, low ductility, moderate ductility, and high ductility).
- Lateral force coefficient (For low, medium, and high code levels, the value of the lateral force coefficient, i.e. the partition of weight that was specified as the design level force in the seismic design, expressed in %, but this is only applied to reinforced concrete moment and infilled frames only).
- Height (number of storeys).

The values of each attribute of this taxonomy currently used to describe the European Residential Building are presented in Table 2.1, whereas a preliminary list of 512 distinct structural systems used in European residential buildings, described using the GEM Building Taxonomy, is presented in Annex A.

¹ <https://vulnecurves.eu-risk.eucentre.it/>

Table 2.1. Values of attributes of GEM Building Taxonomy currently used to describe European residential buildings

Attribute	Element Code	Meaning
Materials (Level 1)	CR	Reinforced Concrete
	MR	Reinforced Masonry
	MCF	Confined Masonry
	MUR	Unreinforced Masonry
	S	Steel
	W	Wood/Timber
Materials (Level 2)	ADO	Adobe
	CB99	Concrete block masonry
	CL99	Clay brick masonry
	STDRE	Dressed stone masonry
	STRUB	Rubble stone masonry
Lateral Load Resisting Systems (LLRS)	LDUAL	Dual frame-wall system
	LFINF	Infilled frame
	LWAL	Load bearing wall
	LFM	Moment frame
	LFBR	Braced frame
Code Level	CDN	Absence of seismic design
	CDL	Low code level
	CDM	Medium code level
	CDH	High code level
Ductility Level	DNO	Non-ductile
	DUL	Low ductility
	DUM	Medium ductility
	DUH	High ductility
Height	H	Number of storeys above ground

A brief description, from GEM, for each typology is presented in Table 2.2, Table 2.3, and Table 2.4 representing the masonry buildings, concrete buildings, and steel & timber, respectively.

Table 2.2. Description of the masonry typology buildings

Masonry Buildings	Description
	Confined Masonry (MCF) This building employs a masonry construction technique where the process begins with the laying of masonry walls. Subsequently, horizontal and vertical reinforced concrete elements are cast around these walls. This integration results in a strong bond between the concrete and masonry.
	Reinforced Masonry (MR) In this method of construction, masonry walls are reinforced in a way that harnesses the combined strength of two materials to withstand forces. The reinforcement is designed to resist tension, while the masonry itself is adept at handling compression.
	Unreinforced Masonry Rubble Stone (MUR-STRUB) Rubble stone masonry comprises unshaped, rounded field stones or roughly cut stones utilized in construction, with or without mortar. Typically, it consists of two wall panels enclosing a core, often filled with a mixture of stone rubble and mud.
	Unreinforced Masonry with Dressed Stone (MUR-STDRE) Dressed stone masonry is crafted from precisely shaped and finished stones, typically of uniform size. These stones are meticulously cut and smoothed to fit together seamlessly, often with mortar binding them in place.
	Unreinforced Masonry with CL99 (MUR-CL99) CL99 brick construction involves the meticulous assembly of precisely manufactured bricks to create buildings of exceptional quality and uniformity. These bricks are carefully laid in courses, with mortar used to secure them firmly in place.
	Unreinforced Masonry with CB99 (MUR-CB99) Concrete blocks, typically molded into rectangular prisms, form the Foundation of masonry construction. Masons use mortar to secure the blocks together, ensuring structural integrity and stability.
	Unreinforced Masonry with Adobe (MUR-ADO) In un-dried (or air-dried), unfired mud (clay) masonry, the building process involves casting clay into blocks which are then laid to construct walls.

Table 2.3. Description of the concrete typology buildings

Concrete Buildings	Description
 Reinforced Concrete with Moment Frames (CR_LFM)	This frame comprise beams and columns, featuring robust connections between beams and columns. It encompasses frames with thin walls unable to withstand a substantial earthquake load. These frames may be singular or comprise multiple bays arranged horizontal and/or vertically.
 Reinforced Concrete with Infills Walls (CR_LFINF)	This framework consists of beams and columns, with certain bays filled with masonry walls. These wall, whether mechanically connected to the frame or not, possess significant stiffness and strength within their planes.
 Reinforced Concrete with Shear Walls (CR_LWALL)	The “wall” refers to a lateral load-resisting structure comprised of both walls and frames. These walls, owing to their considerable lengths, predominantly withstand the lateral load. Typically, reinforced concrete walls are monolithic in nature.
 Reinforced Concrete with Dual System (CR_LDUAL)	The lateral load-resisting structure integrates moment frames and shear walls, working collaboratively in a unified direction. This configuration is primarily composed by reinforced concrete. The extreme thin walls within a dual frame-wall system may or may not have been engineered to withstand potential earthquake forces.

Table 2.4. Description of the steel and timber typology buildings

Steel and Timber Buildings	Description
	Steel Braced Frame (S_LFBR) A framework of beams and columns inclined, often diagonal, to provide strength and rigidity. If diagonal members are stocky, they resist both tension and compression forces. Braced frames may or may not be infilled.
	Steel Frame with Infill Walls (S_LFINF) This framework comprises beams and columns, with certain bays filled with masonry walls that may or may not be linked mechanically to the frame. These infill walls possess substantial stiffness and strength within their planes, preventing beams and columns from bending under horizontal loading.
	Steel Moment Frame (S_LFM) This frame comprises beams and columns featuring robust beam-to-column connections. It can be a single frame or have multiple bays repeated horizontally and/or vertically. Importantly, these frames are not infilled with materials such as masonry or concrete.
	Timber Moment Frame (W_LFM) This frame comprises beams and columns with robust beam-to-column connections. These frames may be singular or consist of multiple bays arranged horizontally and/or vertically. They are not filled with masonry or concrete.

2.1.2. DESIGN CODE TYPE AND LATERAL FORCE COEFFICIENT

In the field of seismic design for reinforced concrete building classes, additional attributes are typically assigned based on the year of construction. A study led by EUCENTRE had the goal of identifying, for each country, the type of seismic code (CDN, CDL, CDM, and CDH) and lateral load coefficients (i.e. percentage of weight that should be resisted laterally), so a comprehensive examination was done to explore the temporal and spatial evolution of seismic design practice across Europe [3]. The classification of these classes can be extended by considering the year of construction, with further distinctions based on the degree of ductility. This degree of ductility is associated with both the design code in effect during the construction year and the prevailing level of seismic hazard. Throughout Europe and across different periods, there has been notable diversity in seismic design codes, their enforcement timelines, and the comprehensiveness of the associated regulations.

In the case of Greece, 1959, 1985, and 1995 are some of the benchmark years of the introduction of successive earthquake codes (Doudoumis [6]), and because of that, the seismic code depends on these benchmark years.

- CDN, before 1959
- CDL, between 1959 and 1984
- CDM, between 1985 and 1994
- CDH, 1995 to present.

The lateral load coefficients were calculated with a process that involved a combination of factors, such as a seismic coefficient (depending on seismic zonation and possibility soil category), a structural coefficient (reflecting the ductility and damping characteristics of the lateral load resisting system), an importance coefficient (categorizing buildings based on occupancy and post-earthquake importance), and a dynamic coefficient (influenced by the period of vibration of the structure).

2.2.3. DUCTILITY LEVEL

The gradations in ductility extend beyond seismic regulations alone. In regions with minimal or seismic hazard, structures will inherently exhibit low ductility. The following framework, Crowley et al. [7], which takes into account the construction year and hazard level, delineates the likely approach for assigning buildings to non-ductile, low, medium, and high ductility levels:

- For structures under pre/low code, non-ductile levels apply to unreinforced masonry (URM) and adobe construction, while low ductility is assigned to reinforced concrete, confined masonry, reinforced masonry, wood, and steel.
- Under a moderate code:
 - Low ductility is assigned if the Peak Ground Acceleration (PGA) for the 475-year return period hazard is $<0.1g$.
 - Moderate ductility is assigned if the PGA for the 475-year return period hazard is $>0.1g$.
- Under a high code:
 - Low ductility is assigned if the PGA for the 475-year return period hazard is $<0.1g$.
 - Moderate ductility is assigned if the PGA for the 475-year return period hazard is $>0.1g$ but $<0.3g$.
 - High ductility is assigned if the PGA for the 475-year return period hazard is $>0.3g$.

2.2. CAPACITY CURVES

2.2.1. PUSHOVER ANALYSIS

A pushover curve is a graphical representation illustrating a building's lateral load resistance concerning a characteristic lateral displacement (Kappos et al. [8]). This representation is typically derived from inelastic static analysis, often presented as a base shear versus top displacement curve as it can be seen in Figure 2.1. To enable a direct comparison with spectral demand, the base shear is transformed into spectral acceleration, and the roof displacement is converted to spectral displacement. The primary limitation of pushover analysis lies in its inability to adequately capture localized responses, such as the formation of soft stories, rebar pullout, buckling, and the influence of higher mode effects (Bencturk et al. [9]).

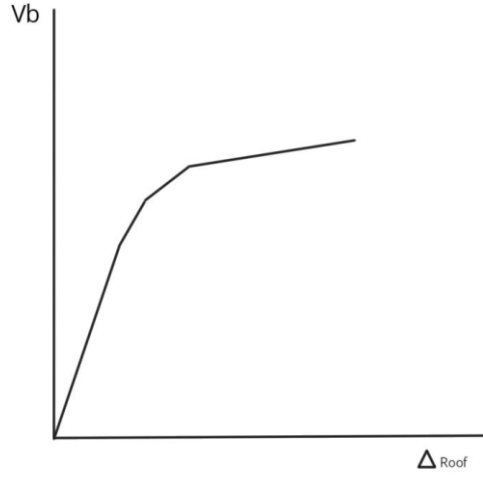


Figure 2.1. Pushover Curve

2.2.2. CONVERSION FROM PUSHOVER CURVE INTO CAPACITY CURVE

When transforming pushover curves into capacity diagrams, the multi-degree-of-freedom (MDOF) system is depicted as an equivalent single-degree-of-freedom (SDOF) system (Gencturk et al. [9]). The equation of motion governing the MDOF system is expressed as:

$$M\ddot{U} + C\dot{U} + R = -M1\ddot{x}_g \quad (2.1)$$

where,

M is the diagonal mass matrix,

U is the displacement vector,

C is the damping matrix,

R is the vector of internal forces,

$\ddot{x}_g$ is the ground acceleration.

Given the assumption that the displacement shape (Φ) remains constant throughout the structural response to ground motion and recognizing that the internal forces (R) equal the externally applied loads (P), the displacement vector is defined as per Equation (2.2). Consequently, Eq.(2.3) is obtained replacing the values U in Eq.(2.1) by the term presented in Eq.(2.2) and multiplying both sides by Φ^T .

$$U = \Phi\eta \quad (2.2)$$

$$\Phi^T M \Phi \ddot{\eta} + \Phi^T C \Phi \dot{\eta} + \Phi^T M \Phi p = -\Phi^T M 1 \ddot{x}_g \quad (2.3)$$

If the left part of Equation (2.3) be multiplied and divided on both sides by $\Phi^T M 1 \ddot{x}_g$ yields to:

$$m^* \ddot{\eta}^* + c^* \dot{\eta}^* + F^* = -m^* \ddot{x}_g \quad (2.4)$$

Where m^* and c^* is the equivalent mass and equivalent viscous damping constant of the SDOF systems, respectively. These quantities are achieved by:

$$m^* = \Phi^T M \mathbf{1} = \sum m_i \Phi_i \quad (2.5)$$

$$c^* = \Gamma \Phi^T C \Phi \quad (2.6)$$

The displacement and the force of the SDOF are described by Equations (2.7) and (2.8), respectively.

$$\eta^* = \frac{\eta}{\Gamma} \quad (2.7)$$

$$F^* = \frac{V_b}{\Gamma} \quad (2.8)$$

Where, η is the displacement vector in modal coordinates, V_b is the base shear force, and Γ is the modal participation factor and used to convert the MDOF system to SDOF system and vice-versa. The expression to calculate this last value is:

$$\Gamma = \frac{\Phi^T M \mathbf{1}}{\Phi^T M \Phi} = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2} \quad (2.9)$$

Having now the transformation of the base shear into the force of the SDOF, and the displacement of the roof into the displacement of the SDOF it can be possible to transform the pushover curve to a capacity curve, which is described in Figure 2.2.

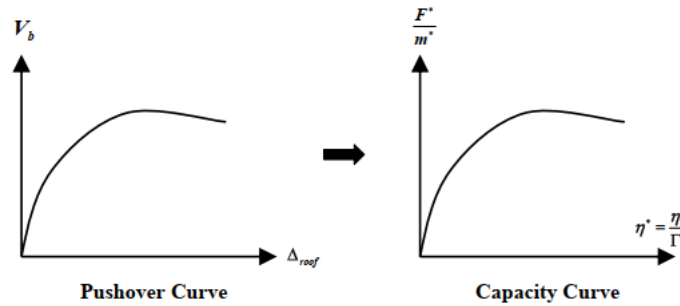


Figure 2.2. Conversion of a pushover curve into a capacity curve (Extracted from Gencturk et al. [9])

2.2.3. DEVELOPMENT OF CAPACITY CURVE

Capacity curves offer insights into the lateral strength and deformation capacity of buildings or specific building classes and are frequently converted into the Acceleration Displacement Response Spectrum (ADRS) format in order to facilitate the comparison with spectral demand for the development of fragility functions (Kappos et al. [10]).

Each single-degree-of-freedom (SDOF) oscillator, representing different building classes, exhibited hysteresis behaviour determined by its corresponding capacity curve, depicting spectral acceleration (S_a) in relation to spectral displacement (S_d).

In the ADRS format, the mean capacity is determined by estimating the roof displacement (δ_R) of the multi-degree-of-freedom structure. The values of the spectral displacement and acceleration are determined using the following expressions.

$$S_d = \frac{\delta_R}{\Gamma} \quad (2.10)$$

$$S_a = S_d \cdot \left(\frac{2 \cdot \pi}{T} \right)^2 \quad (2.11)$$

In this study, the capacity curves are the same ones that were used to develop the fragility models for ESRM20². These capacity curves (Martins and Silva [11]) were released by the GEM Foundation and used to represent the European CR_LDUAL, CR_LWAL, MCF, MR, MUR, S and W typologies with different heights and ductility, while capacity curves for residential concrete building classes CR_LFINF and CR_LFM were developed by Romão et al [12]. The median capacity curves in Romão et al. [12] were read using the VMTK's 'nlth_on_sdof.py' script to produce the Single-Degree-Of-Freedom (SDOF) models using the pinching4 hysteresis curve of OpenSees [13]. The Pinching4 is a type of material characterized by a distinctive “pinched” load-deformation behaviour and is prone to degradation when subjected to cyclic loading. Alongside the capacity curves outlining its response envelope, SDOF models simulate various material types, incorporate the hysteretic properties outlined in Table 2.5.

Table 2.5. Parameter of the Pinching4 hysteresis model used in ESRM20 (simpleSDOF4.tcl provided by Vamvatsikos, 2011)

Parameters of Pinching4	Degradation = True	Degradation = False
rdispP/N, fForceP/N, uForceP/N	0.5, 0.25, 0.05	0.5, 0.25, 0.05
gK1 gK2 gK3 gK4 gKlim	0, 0.1, 0, 0, 0.2	0, 0, 0, 0, 0
gD1 gD2 gD3 gD4 gDlim	0, 0.1, 0, 0, 0.2	0, 0, 0, 0, 0
gF1 gF2 gF3 gF4 gFlim	0, 0.4, 0, 0.4, 0.9	0, 0, 0, 0, 0
gE	10	10
dmgType	energy	energy

The cyclic degradation of the strength and stiffness occurs in three ways: unloading stiffness degradation, reloading stiffness degradation and strength degradation. To follow the same that was done in ESRM20, for the typologies: CR (CDL, CDM, and CDH), MUR, MCF (DUL) and MUR (DUL) it

² https://gitlab.seismo.ethz.ch/efehr/esrm20_vulnerability/-/tree/master/ESRM20_capacity_curves

was assumed “True” for the cyclic degradation. The mass proportional to damping was also assumed using different ratios regarding the material of the typology, thereby implementing 10% damping for masonry, 7.5% for confined masonry, for reinforced concrete and wood was 5%, and 3% for steel.

2.2.3.1. Capacity curves in residential masonry building classes, ESRM20.

Non-linear time-history analyses were conducted on equivalent Single Degree of Freedom (SDOF) oscillators, each characterized by the mean capacity curve. The expected period of vibration for each building class was determined from simplified period-height relationships commonly found in the literature and widely used in large-scale risk assessment studies (Martins [11]) and these relationships are presented in Table 2.6, and Table 2.7. In these capacity curves, there are cases where no significant reduction in base shear capacity was anticipated due to damage accumulation (e.g., steel frames, timber, and composite structures), so it was presumed to adhere to a trilinear elastoplastic model, as illustrated in Figure 2.3. These curves were established using yielding (S_{dy}) and ultimate (S_{du}) displacements, along with elastic (T_1) and yielding (T_y) periods.

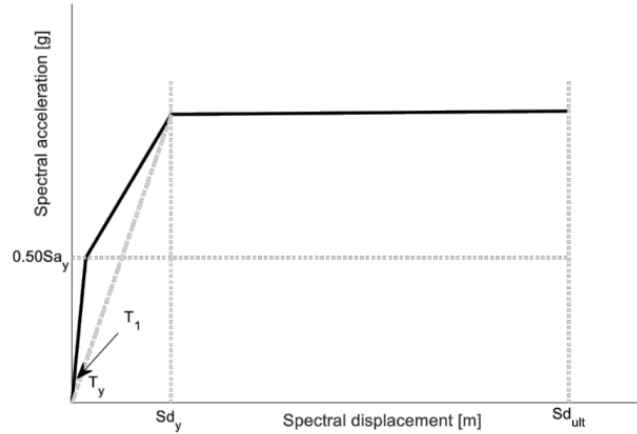


Figure 2.3. Generic capacity curve format used for Masonry Buildings (Extracted from Martins and Silva [11])

Table 2.6 lists the parameters used to define the capacity curves of each masonry building class (MUR: unreinforced masonry, MUR+STRUB: unreinforced masonry with rubble stone, MUR+STDRE: unreinforced masonry with dressed stone, MUR+CL99: unreinforced masonry with clay bricks, MUR+CB99: unreinforced masonry with clay bricks, MUR+ADO: unreinforced masonry with adobe bricks, MR: reinforced masonry, and MCF: confined masonry). It resulted in a compilation of data coming from different research studies and experimental campaigns (Snoj and Dolšek [14]; Riahi et al. [15]; Tomažević and Klemenc [16]; García and Degrande [17]; Alcoocer et al. [18]; Lovon et al. [19]; Loftly et al. [20]; Hamzeh et al. [21]; Preciado et al. [22]; Karanikoloudis et al. [23]). Note that unreinforced masonry buildings were assumed to have no seismic design (CDN) whereas reinforced and confined masonry can either be CDN or CDL.

Table 2.6. Parameters used to compute masonry capacity curves (Martins and Silva [11])

Building Class	T_y [S]	N° of storeys	Yield drift [%]	Ultimate drift [%]
MUR/LWAL-DNO	$T_y = 0.062H^{0.9}$	1-5	0.14	0.60
MUR+STRUB/LWAL-DNO	$T_y = 0.062H^{0.9}$	1-5	0.12	0.51
MUR+STDRE/LWAL-DNO	$T_y = 0.062H^{0.9}$	1-5	0.13	0.54
MUR+CL99/LWAL-DNO	$T_y = 0.065H^{0.9}$	1-5	0.13	0.57
MUR+CB99/LWAL-DNO	$T_y = 0.065H^{0.9}$	1-5	0.16	0.69
MUR+ADO/LWAL-DNO	$T_y = 0.066H^{0.9}$	1-3	0.12	0.55
MR/LWAL-DUL	$T_y = 0.060H^{0.9}$	1-5	0.17	0.80
MR/LWAL-DUM	$T_y = 0.058H^{0.9}$	1-5	0.20	0.90
MR/LWAL-DUH	$T_y = 0.056H^{0.9}$	1-5	0.22	1.00
MCF/LWAL-DUL	$T_y = 0.044H$	1-6	0.19	0.90
MCF/LWAL-DUM	$T_y = 0.042H$	1-6	0.24	1.00
MCF/LWAL-DUH	$T_y = 0.040H$	1-6	0.28	1.10

In the case of building classes that anticipate a substantial decrease in base shear capacity due to damage accumulation in masonry panels (Dolšek, et al. [24]) a modelling approach was employed featuring a quadrilinear capacity curve, illustrated in Figure 2.4.

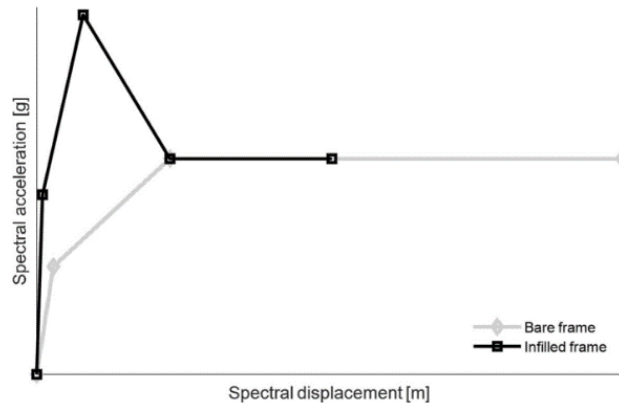


Figure 2.4. Example of capacity curves of bare frame vs infilled frames considering reduction on initial stiffness (Extracted from Martins and Silva [11])

2.2.3.2. Capacity Curves in residential concrete building classes, ESMR20

For residential concrete building classes CR_LDUAL and CR_LWAL as it was mentioned before the capacity curves were released by GEM Foundation and the respective parameters can be seen in Table 2.7.

Table 2.7. Parameters used to compute RC capacity curves based on simplified models (Martins and Silva [11])

Building Class	T_y [S]	N° of storeys	Yield drift [%]	Ultimate drift [%]
CR/LDUAL/DUL/H	$T_y = 0.036H$	1-12	0.17	1.16
CR/LDUAL/DUM/H	$T_y = 0.034H$	1-12	0.21	1.28
CR/LDUAL//DUH/H	$T_y = 0.033H$	1-12	0.26	1.40
CR/LWAL/DUL/H	$T_y = 0.088N$	1-12	0.17	1.10
CR/LWAL//DUM/H	$T_y = 0.083N$	1-12	0.22	1.30
CR/LWAL//DUH/H	$T_y = 0.078N$	1-12	0.28	1.16

As part of the European SERA project, a detailed set of capacity curves for European reinforced concrete infilled frames (CR_LFINF) and moment frames (CR_LFM) were developed by Romão et al. 2020³. Precisely 79200 capacity curves were developed for buildings fitting the typology of reinforced concrete or with infills and with moment frames. Figure 2.5 shows an example of a typology of reinforced concrete with infill frames. This typology of reinforced concrete with infills was done considering three different types of infills, following the work developed by Hak et al. [25].

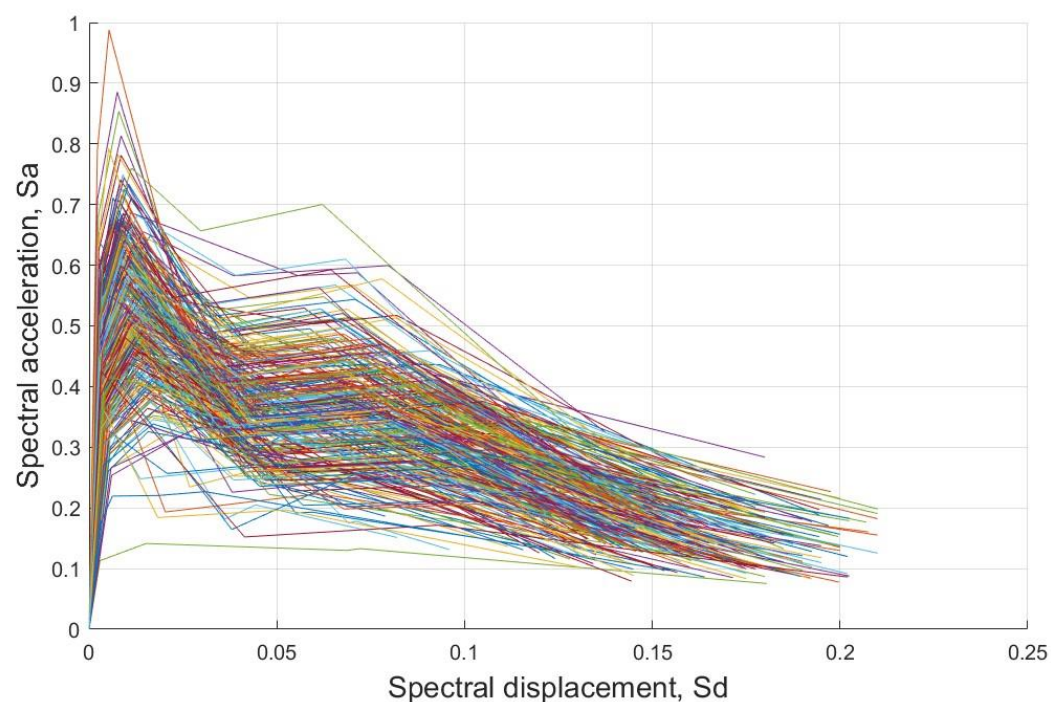


Figure 2.5. Capacity curves for CR_LFINF_CDM-15_H1 developed by Romão et al. [12]

³ https://gitlab.seismo.ethz.ch/efehr/esrm20_vulnerability

3

Development of Fragility Curves

3.1. PROBABILITY DISTRIBUTIONS

3.1.1. INTRODUCTION

To understand the concept of fragility, it is important to dive into the meaning of uncertain quantities and probabilities. The term “uncertain” here encompasses a broader spectrum than merely “random”. For example, the building’s ability to withstand an earthquake may exhibit relatively stable characteristics over time, however until the moment of collapse the exact capacity of the building to resist remains uncertain (Porter [26]).

Random variables are described using probability distributions. There are three kinds of probability distributions: probability density functions (PDFs), probability mass functions (PMFs), and cumulative distribution functions (CDFs). The probability density function for a continuous scalar random variable can be plotted on an x-y axis measures the possible value the variable can take on and the y-axis measures the probability per unit of x that the variable takes on that particular x value. The area under the probability density function between any two values a and b gives the probability that X will take on a value between those two bounds.

$$P[a < X \leq b] = \int_a^b f_X(x)dx \quad (3.1)$$

When the probability density function of the random variable X is integrated from negative infinity to a specific value x, the resulting integral, known as the cumulative distribution function, represents the probability that X will assume a value less than or equal to x.

$$P[X \leq x] = \int_{z=-\infty}^x f_X(z)dz \quad (3.2)$$

Some variables can only take on discrete values. For that, the probability mass function is used to express the probability that X takes on any given value x:

$$P[X = x] = p_x(x) \quad (3.3)$$

The cumulative distribution function of a discrete random variable can be expressed as:

$$P[X \leq x] = \sum_{z=-\infty}^x p_x(z) \quad (3.4)$$

3.1.2. NORMAL AND LOGNORMAL DISTRIBUTIONS

The Gaussian probability density function is expressed in Eq. (3.5) (a) and graphically is represented by Figure 3.1. Sometimes is expressed in the normalized form shown like the one represented in the second line of the equation with the lower-case Greek letter φ .

$$\begin{aligned} f_X(x) &= \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \\ &= \varphi\left(\frac{x-\mu}{\sigma}\right) \end{aligned} \quad (3.5)$$

The cumulative distribution function (CDF) can be expressed as follows:

$$\begin{aligned} P[X \leq x] &= F_X(x) \\ &= \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \\ &= \Phi\left(\frac{x-\mu}{\sigma}\right) \end{aligned} \quad (3.6)$$

where, Φ denotes the standard normal cumulative distribution function; σ represents the standard deviation, and μ in the mean of the distribution. This function is represented, also, in Figure 3.1 (b).

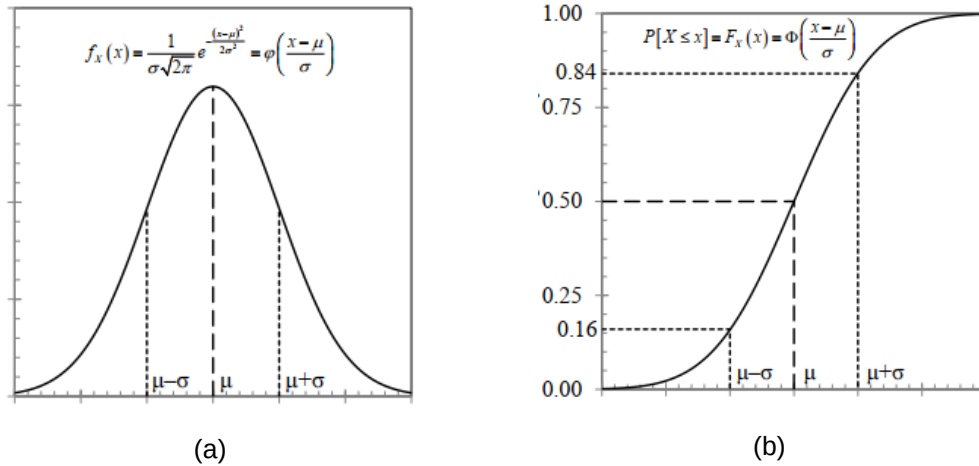


Figure 3.1. (a) Gaussian probability density function; (b) Gaussian cumulative distribution function; (adopted from Porter [26])

The Lognormal distribution is the one more used by the earthquake engineer. It is ubiquitous in Probabilistic Seismic Hazard Analysis (PSHA) and Probabilistic Seismic Risk Analysis (PSRA). The lognormal cumulative distribution is widely used for fragility compared to the Gaussian distribution, and four reasons described by Porter [26] justify that:

- **Simplicity:** This distribution offers a straightforward parametric representation for approximating positive-valued uncertainty. It relies solely on estimates of central value and uncertainty, making it a simple and practical tool for such approximations.
- **Established Usage:** With a history spanning several decades in earthquake engineering, this distribution enjoys widespread acceptance and application. Its longstanding presence in the field underscores its reliability and utility.
- **Information-Theoretic Basis:** Rooted in information theory principles, this distribution embodies the concept of maximum entropy. Specifically, when only knowledge of the variable's positive values, along with specified median and logarithmic standard deviation, is available, this distribution assumes the least amount of additional information.
- **Often fits data:** Notably, this distribution frequently aligns well with empirical data. It tends to offer reasonable fits to observed distributions across various domains of interest, such as ground motion relative to magnitude and distance, structural collapse capacity, and the distribution of losses in relation to seismic activity.

The probability density function is given by Eq. (3.7), and represented graphically in Figure 3.2.

$$f_X(x) = \frac{1}{x\beta\sqrt{2\pi}} e^{-\frac{(\ln(x/\theta))^2}{2\beta^2}}$$

$$= \varphi\left(\frac{\ln(x/\theta)}{\beta}\right) \quad (3.7)$$

The lognormal CDF can be expressed as follows:

$$P[X \leq x] = F_X(x)$$

$$= \Phi\left(\frac{\ln x - \ln \theta}{\beta}\right)$$

$$= \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \quad (3.8)$$

$$= \Phi\left(\frac{\ln x - \mu_{\ln X}}{\sigma_{\ln X}}\right)$$

where, θ and β represent the median and the logarithmic standard deviation, respectively. $\mu_{\ln X}$ is the mean of the natural logarithm of the uncertain variable X and $\sigma_{\ln X}$ denotes the standard deviation of the natural logarithm of X . The mean value of $\ln X$, represented by $\mu_{\ln X}$, is equivalent to the natural logarithm of θ . Similarly, the standard deviation of $\ln X$, denoted by β or $\sigma_{\ln X}$, reflects the logarithmic variability of X .

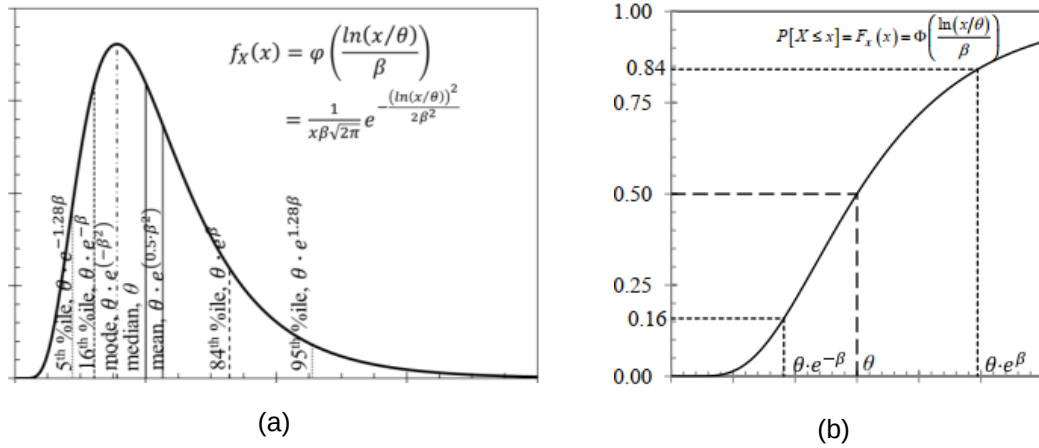


Figure 3.2. (a) Lognormal probability density function; (b) Lognormal cumulative distribution function; (adopted from Porter, [26])

3.2. FRAGILITY FUNCTION DEFINITION

The concept of a fragility function in earthquake engineering can be found as a probabilistic relationship between frequency of failure of a component of a nuclear power plant and peak ground acceleration in an earthquake, Kennedy et al. [27]. Fragility curves provide the probability of reaching or exceeding a given damage conditional on the ground shaking intensity level. These functions predict a binary outcome, F, indicating either failure or non-failure, based on ground-motion intensity. Failure is determined relative to a specific outcome of interest (i.e., the failure criteria can be material yielding, cracking windows, reduction of x% capacity of an element, structural collapse, breakage in a pipe, soil liquefaction, etc). Frequently encountered form of a seismic fragility function, although not universally applicable or always optimal, is the lognormal cumulative distribution function (CDF), Eq. (3.9):

$$F_d(x) = P[D \geq d | X = x] \quad d \in \{1, 2, \dots, N_D\}$$

$$= \Phi\left(\frac{\ln(x/\theta_d)}{\beta_d}\right) \quad (3.9)$$

where,

$P[A|B]$ = probability that A is true given that B is true;

D = uncertain damage state of a particular component;

d = a particular value of D , i.e., with no uncertainty;

n_D = number of possible damage states;

X = uncertain excitation intensity;

x = particular value of X , i.e., with no uncertainty;

$F_d(x)$ = a fragility function for damage state d evaluated at x ;

$\Phi(s)$ = standard normal cumulative distribution function evaluated at s ;

$\ln(s)$ = natural logarithm of s ;

θ_d = the median capacity of the asset to withstand damage state “ d ”.

β_d = the standard deviation of the natural logarithm of the asset's capacity to resist damage state “d”.

An example fragility function, depicted using the Eq. (3.9), is presented in Figure 3.3. Setting $\Theta = 0.5$ ensures that the fragility function yields a probability of 0.5 when the $IM = 0.5$. Additionally, $\beta = 0.4$ dictated the slope of the function, where larger β values result in a gentler slope of the fragility function at its median.

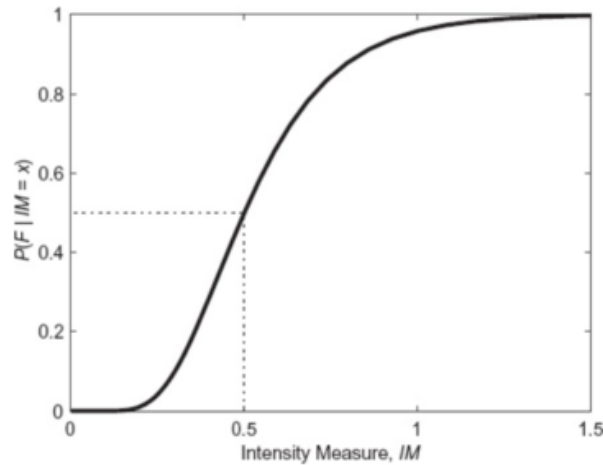


Figure 3.3. Fragility function providing the probability of failure as a function of the ground-motion intensity, IM , using $\Theta=0.5$ and $\beta=0.4$. (extracted from Baker, Bradley, and Stafford [28])

3.3. IMs AND EDPs PARAMETERS

Fragility functions, as discussed by Bakalis and Vamvatsikos [29], serve as crucial tools in seismic risk assessment, providing a probabilistic characterization of vulnerability at both component and structural levels. These functions express the likelihood of failure as a function of a ground motion intensity measure (IM), allowing engineers and researchers to better understand and quantify the potential impact of earthquake on various types of structures.

An intensity measure, or IM , serves as an indicative of the severity of ground motions at a specific location and can be characterized by several metrics such as, peak ground acceleration (PGA), first-mode spectral acceleration ($S_a(T_1)$), peak ground velocity, modified Mercalli intensity, etc, each providing unique insights into the seismic environment.

In structural analysis, each dynamic analysis yields a pair of IM and associated engineering demand parameters (EDP). These pairs are crucial for adequately characterizing the EDP - IM space and estimating seismic demand. However, the choice of IM can significantly impact the results, where for example, the effectiveness of $S_a(T_1)$ can vary depending on the specific characteristics of the structure and the seismic hazard being considered. Thus, careful consideration must be given to the selection of IM s to ensure accurate and reliable fragility assessments (Kazantzi and Vamvatsikos [30]).

Elastic spectral acceleration at the first mode vibration period of the structure, $S_a(T_1)$, has emerged as a commonly used IM in fragility assessment, supported by studies such as those discussed by Suzuki and Iervolino [31]. It is favoured over PGA in many cases due to its efficiency and sufficiency for assessing first-mode dominated structures subjected to far-field earthquakes.

In practical applications, fragility may need to be translated into a target IM for various reasons. This could include situations where hazard data is only available in terms of a particular IM (e.g. *PGA* [32]) or when it is desirable to compare the loss estimation among structures using different IMs (Kohrangi et al. [33])

In essence, fragility functions, guided by appropriate IMs, play a vital role in enhancing the understanding of seismic risk and informing decision-making processes aimed at mitigating the impact of earthquakes on built environments, as discussed by the aforementioned researchers.

For the development of the ESRM20 fragility curves the methodology used was the cloud analysis and required the definition of the best-fit curve between an intensity measure (IM) and an engineering demand parameter (EDP) in the logarithmic space. Four intensity measures, *PGA*, $S_a(0.3s)$, $S_a(0.6s)$ and $S_a(1.0s)$ were considered here.

3.4. STRUCTURAL DAMAGE STATES

3.4.1. DEFINITION OF DAMAGE STATE

Fragility curves are developed for specific damage states, meaning that the selection of these damage states must be clear and unambiguous. The different levels of structural damage are related with different levels of the structure performance, allowing to predict other consequences of the earthquake. Therefore, damage states levels are often associated with pushover curves, as can be seen in Figure 3.4, or consequently related to the capacity curves. Damage states could also be obtained as a progression of damage through several elements (e.g. beams, slabs, columns, connections, etc) at different levels of performance in the entire building or structural system.

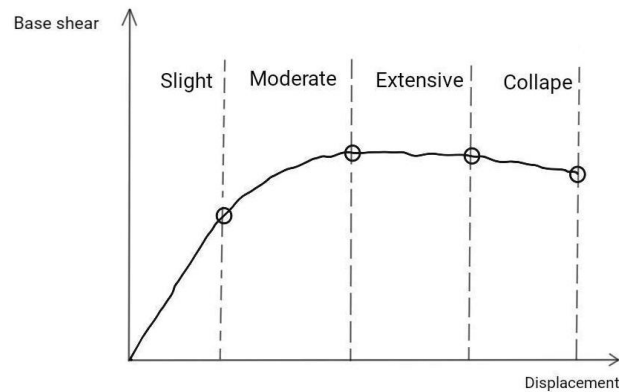


Figure 3.4. Damage states in a pushover curve

3.4.2. DIFFERENT APPROACHES TO DAMAGE STATES

Damage states in different codes are defined in different ways. As it mentioned in Figure 3.5 on the left side is presented the FEMA-58 / FEMA 356 where they define the damage states in four levels, operational level, Immediate Occupancy, Life Safety, and Collapse Prevention. The Life Safety state damage makes sure that a 475-year return period earthquake for the 50-year life span of a building is usually sufficient to provide life safety for the building (i.e. if an earthquake with that intensity happens people inside the building will be safe and they could be able to safely get out of the building without

proper issue). On the right side is the Eurocode – EN 1998, which is divided into 3 damage states, Damage Limitation, Significant Damage, and Near Collapse.

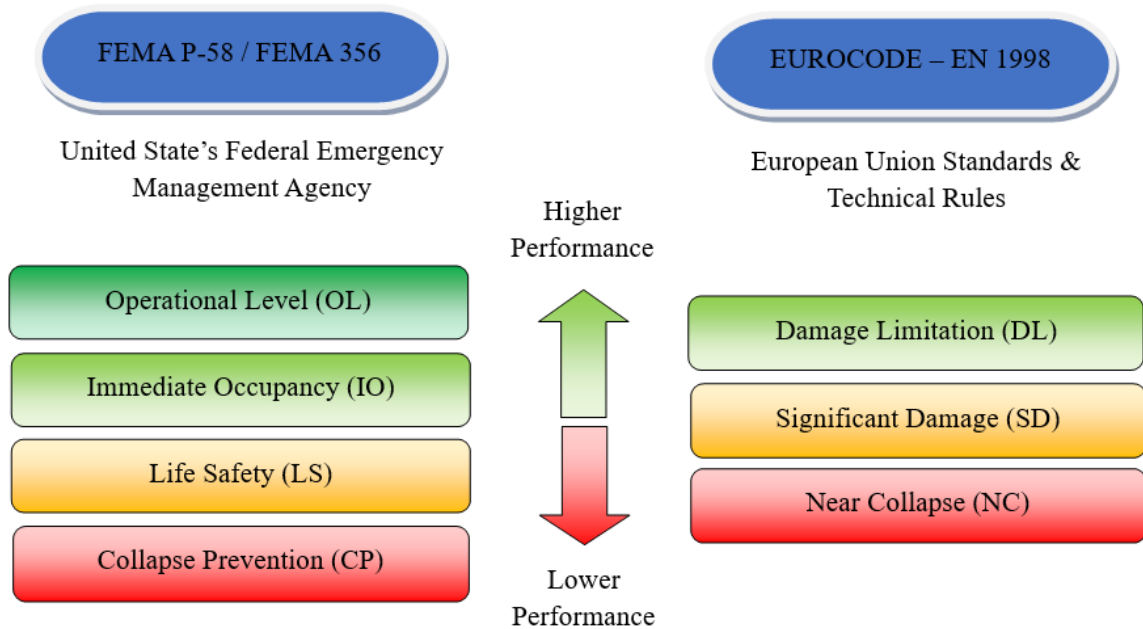


Figure 3.5. Damage states in two different codes

3.4.2.1. FEMA P-58 / 356

In the context of FEMA P-58 /FEMA 356 [34], the assessment of building damage involves delineating specific damage states, each characterized by distinct conditions and requirements. The first damage state, Operational Level (OL), signifies that the building remains essentially functional, and only negligible repairs are necessary. In this state, the overall structure and non-structural integrity allow for continued operation without compromising safety.

Moving to the second damage state, Immediate Occupancy (IO), the expectation is that the building's spaces and systems are still reasonably usable and operational. While some minor repairs may be warranted, the structural and non-structural components remain largely intact, enabling occupants to resume normal activities promptly.

Transitioning further, the third damage state, Life Safety (LS), is associated with an extremely low probability of threat to life safety. This applies to both structural and non-structural elements. Although there is minimal risk to occupants, the condition requires comprehensive inspections and repairs, ranging from minor to major, to ensure long-term safety and structural integrity.

The final damage state is Collapse Prevention (CP), denoting a state where the building is anticipated to be stable under vertical loads. At this point, the primary focus is on repairs and inspections to prevent any potential collapse. The building is structurally sound, and attention is directed towards maintaining this stability through necessary interventions.

American Society of Civil Engineers (ASCE) [35] has several descriptions of different structural performance levels of various Elements, such as concrete frames, steel moment frames, braced steel

frames, concrete walls, unreinforced masonry infills walls, reinforced masonry walls, etc. Table 3.1 describes the three damage states related to concrete frames and unreinforced masonry infill walls.

Table 3.1. Structural Performance Level and Requirements for Concrete Frames and Unreinforced Masonry Walls (Excerpted from Table C1-3 ASCE/SEI 31-06)

Performance Level		Immediate Occupancy (IO)	Life Safety (LS)	Collapse Prevention (CP)
Concrete Frames	Primary	Minor hairline cracking; limited yielding possible at a few locations; no crushing (strains below 0.003).	Extensive damage to beams. Spalling of cover and shear cracking ($<1/8$ -in. width) for ductile columns. Minor spalling in nonductile columns.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.
	Secondary	Minor spalling in a few places in ductile columns and beams; flexural cracking in beams and columns; shear cracking in joints $<1/16''$ width.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive Spaling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.
Unreinforced Masonry Infill Walls	Primary	Minor ($< 1/8$ -in. width) cracking of masonry infills and veneers. Minor infills and veneers. Minor spalling in veneers at a few corner openings. Same as primary.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings. Same as primary.	Extensive cracking and crushing; portions of face course shed.
	Secondary	Same as primary	Same as primary	Extensive crushing and shattering; some walls dislodge.

3.4.2.2. Eurocode EN-1998

The Eurocode [36] is more general, and it is not divided based on typology. It has only three damage states, starting with the Damage Limitation (DL) which has a mean return period of 225 years (i.e. the probability of exceedance in 50 years is 20%), Significant Damage (SD) with a mean return period of 475 years (i.e. the probability of exceedance in 50 years is 10%) and Near Collapse with the mean return period of 2475 years (i.e. the probability of exceedance in 50 years is 2%). The description of these performance levels of each damage state is presented in Table 3.2.

Table 3.2. Description of the damage states in Eurocode EN-1998-3

Eurocode EN-1998-3	Performance Level	Damage Limitation (DL)	Significant Damage (SD)	Near Collapse (NC)
	Observed Damage	Building is considered as slightly damaged. Sustain minimal or no damage to their structural elements and only minor damage to their non-structural components.	Building is considered as significantly damaged. Extensive damage to structural and non-structural components.	Building is considered as heavily damage. Experience a significant hazard to life safety resulting from failure of non-structural components

3.4.3. Damage states used in ESRM20.

In the context of the Earthquake Seismic Risk Model 2020 (ESRM20), the characterization of damage states involves a nuanced approach. The quantification of damage to structural elements is intricately linked to the level of displacement, where the initiation of damage occurs at the yielding point. Before reaching this critical juncture, the building is presumed to undergo elastic deformation, exhibiting resilience with no significant damage anticipated.

The formulation of damage states in ESRM20 is designed to capture the spectrum of structural responses. Specifically, the thresholds delineating moderate and extensive damage are strategically positioned between the onset of damage (S_{dy}) and the ultimate displacement capacity (S_{du}). This thoughtful placement allows for a comprehensive assessment of structural conditions at varying stages of displacement.

By initiating damage at the yielding point, ESRM20 acknowledges the onset of inelastic behaviour, marking the transition from elastic deformation to the plastic response. This ensures that the damage states not only reflect the severity of structural impairment but also account for the structural system's capacity to withstand displacement before incurring damage.

One prevalent method for characterizing damage involves adopting a single damage model for each building class. This approach was employed by GEM in the 2018 release of its global risk model and is extensively elucidated in the work of Martins and Silva [11].

For the European fragility curves four distinct damage states were established, each assigned specific thresholds (Figure 3.6): DS1 (slight), DS2 (moderate), DS3 (extensive), and DS4 (complete). The vulnerability models for Europe that were developed by these fragility curves consider a fundamental assumption that the damage state encapsulates the collective condition of damage affecting both structural and non-structural elements, as well as contents.

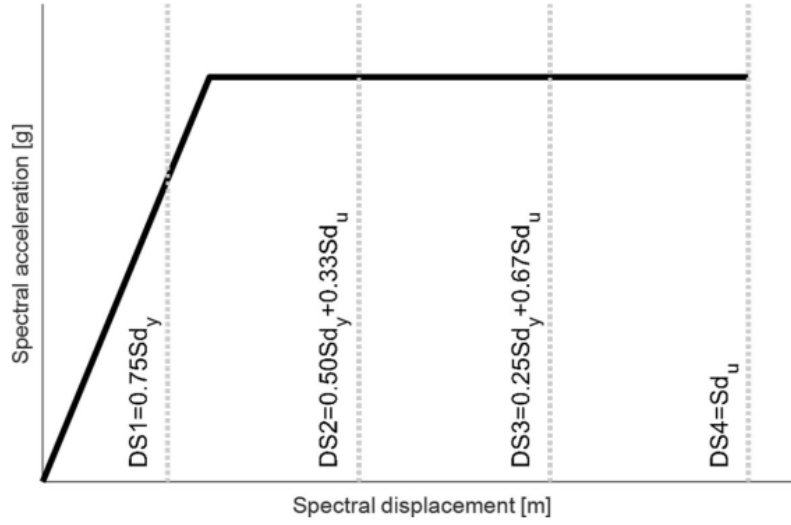


Figure 3.6. Damage thresholds used to develop the European fragility functions (Martins and Silva [11])

The definition of the damage states presented by ESRM20 follows closely a proposal by Lagomarsino and Giovinazzi [37]. As it can be seen in Figure 3.6 to account for damage initiation in non-structural elements (e.g. infill walls), slight damage was assumed to begin in 75% of the yielding displacement, which followed a similar approach done in the work of Villar-Vega et al [38]. The intermediate damage states (i.e. moderate-DS2 and extensive-DS3) were considered to be equalled spaced from the first and the last damage state. Complete damage was considered to reach the ultimate displacement capacity of the structure.

3.5. CLOUD ANALYSIS

Cloud Analysis (CA) is based on simple regression in the logarithmic space of structural response versus intensity for a set of registered records (Jalayer et al. [39]). Similar to the work developed by Crowley et al. [40], Cloud Analysis was employed as the method used to develop the parameters for generating fragility curves within the ESRM20 framework. This methodology requires the definition of a best-fit curve between an intensity measure (IM) and an engineering demand parameter (EDP) in the logarithmic space (Martins and Silva, [11]).

The expected EDP given an IM ($E[\ln(EDP_i)]$) and the respective uncertainty due to the record-to-record variability (σ_{R2R}) can be computed from Eq. (3.10).

$$\left\{ \begin{array}{l} E[\ln EDP] = \ln a + b \ln IM \\ \sigma_{R2R} = \sqrt{\frac{\sum_{i=1}^n (\ln EDP_i - E[\ln EDP])^2}{n - 2}} \end{array} \right. \quad (3.10)$$

In numerical analysis, an intriguing phenomenon can arise where convergence is achieved at levels of deformation so extreme that the structure integrity is severely compromised, bordering irreparable damage. Consequently, integrating these data points directly into the estimation of the best fit curve

would introduce a notable bias into fragility models. For this reason, it was considered a censored regression method (Schnedler [41]) for the calculation of the parameters of the best-fit curve (a and b defined in Eq. (3.10)). These parameters can be found using the Maximum Likelihood Method (MLE), as represented in Eq. (3.11). This method aims to estimate the parameters of the best-fit curve (a and b) that maximize the probability of occurrence of the observed data where Φ is the probability density function of the standard Gaussian distribution.

$$Likelihood = \prod_{i=1}^n \left[\Phi \left(\frac{EDP_i - a - bIM_i}{\sigma} \right) \right] \quad (3.11)$$

Because of the previously mentioned phenomenon that can arise leading to serious levels of deformation, in ESRM20 it was assumed a threshold of 1.5 times the EDP for complete damage. As illustrated in Figure 3.7. Within the investigation of Martins and Silva [11] it becomes evident that the linear regression model did not offer the optimal fit for the entirety of the presented data. Particularly with the introduction of the threshold at 1.5 times the Engineering Demand Parameter (EDP_c) it becomes apparent that there exists a more viable and practical avenue for identifying a curve that better aligns with the observed data points.

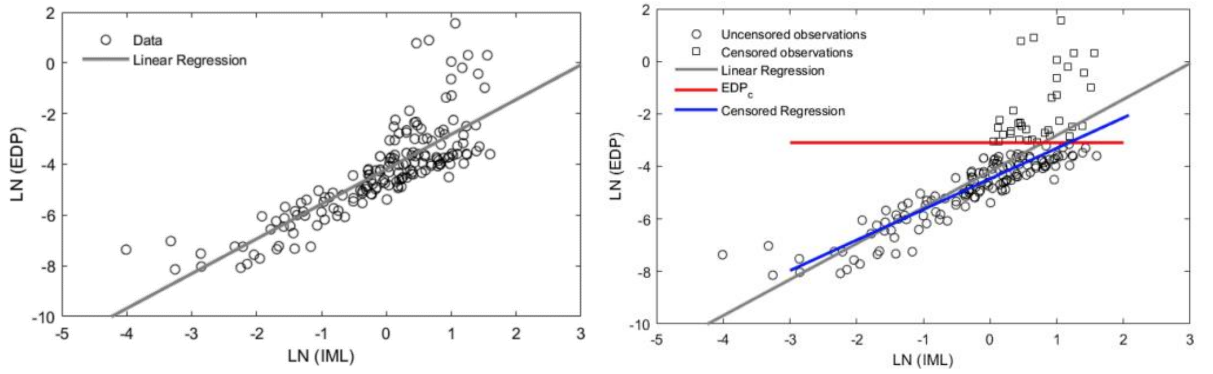


Figure 3.7. Scatter of $\ln(EDP)$ and $\ln(IM)$, with the associated best-fit uncensored and censored regressions (adapted from Martins and Silva [11])

So, in Figure 3.7 can be observed that the uncensored (n_o) observations are used within the MLE method as defined in Eq. (3.11), while for the censored (n_c) observations it was necessary to minimize the negative log-likelihood function given by:

$$\ln(Likelihood) = \sum_i^n \ln \Phi \left(\frac{EDP_i - a - bIM_i}{\sigma} \right) \quad (3.12)$$

With the presence of the censored variables the likelihood function becomes:

$$Likelihood = \prod_j^{n_c} \left[1 - \Phi \left(\frac{EDP_i - a - bIM_i}{\sigma} \right) \right] + \prod_i^{n_o} \left[\Phi \left(\frac{EDP_i - a - bIM_i}{\sigma} \right) \right] \quad (3.13)$$

Taking the logarithm of this expression, as before, gives:

$$\ln(\text{Likelihood}) = \sum_{j=1}^{n_c} \left[\ln \left(1 - \Phi \left(\frac{EDP_c - a - b IM_j}{\sigma} \right) \right) \right] + \sum_{i=1}^{n_o} \left[\ln \Phi \left(\frac{EDP_i - a - b IM_i}{\sigma} \right) \right] \quad (3.14)$$

Where EDP_c is the censored engineering demand parameter threshold. In practical terms, the best-fit curve will be closer to the peak of the probability density function of the uncensored observations, and below the probability density function of the censored observations.

An example of these censored regressions for a specific typology analysed in ESRM20 for different IMs, like PGA , $S_a(0.3s)$, $S_a(0.6s)$ and $S_a(1.0s)$ is illustrated in Figure 3.8. The minimum threshold was established by considering 10% of the yield displacement from the respective capacity curve, while the restricted threshold is set at 1.5 times the ultimate displacement of the corresponding capacity curve for the typology under investigation.

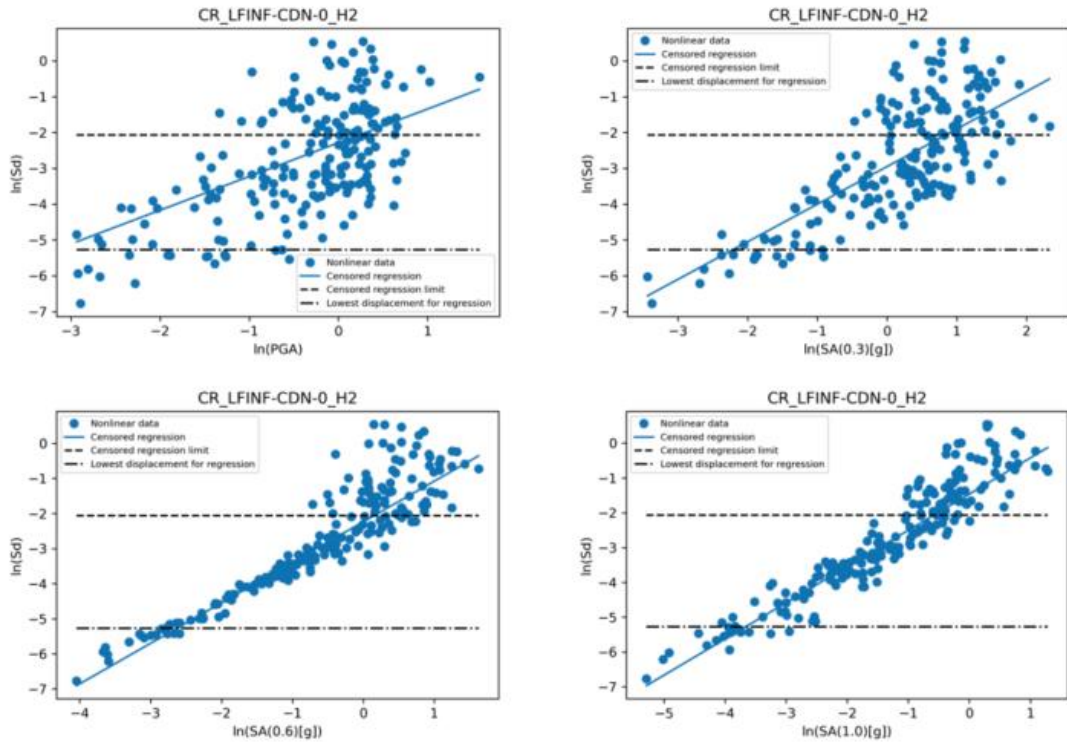


Figure 3.8. Regression plots of the response of an SDOF, and the censored linear regression. (extracted from ESRM20 [1])

This method, illustrated in Figure 3.8, accounts for the record-to-record variability (σ_{R2R}), and does not account for an important variability in fragility analysis of building classes, the building-to-building variability (σ_{B2B}). In ESRM20 the introduction of this variability in the fragility derivation framework

followed the same approach described in the FEMA P-58 guidelines, which was to increase the standard deviation directly using the Eq. (3.15), (square-root-sum-of-squares) with a value of 0.3.

$$\sigma_{total} = \sqrt{(\sigma_{R2R})^2 + (\sigma_{B2B})^2} \quad (3.15)$$

So, the probability that a given damage state will be reached or exceeded given an intensity measure ($P[DS \geq ds_i | IM]$) can be computed as indicated in Eq. (3.16):

$$P[DS \geq ds_i | IM] = \Phi \left(\frac{\ln EDP_{ds_i,j} - E[\ln EDP | \ln IM]}{\sigma_{total}} \right) \quad (3.16)$$

Consider the scenario where $EDP_{ds_i,j}$ denotes an incremental value with the potential spectrum of the engineering demand parameters (EDPs) for damage state ds_i , and $P[EDP_{ds_i,j}]$ signifies the likelihood of encountering $EDP_{ds_i,j}$. The determination of the probability of exceeding a moderate damage is elucidated in Figure 3.9 (a), while the ensuing likelihood of moderate damage per intensity Measure (IM) is depicted in Figure 3.9 (b).

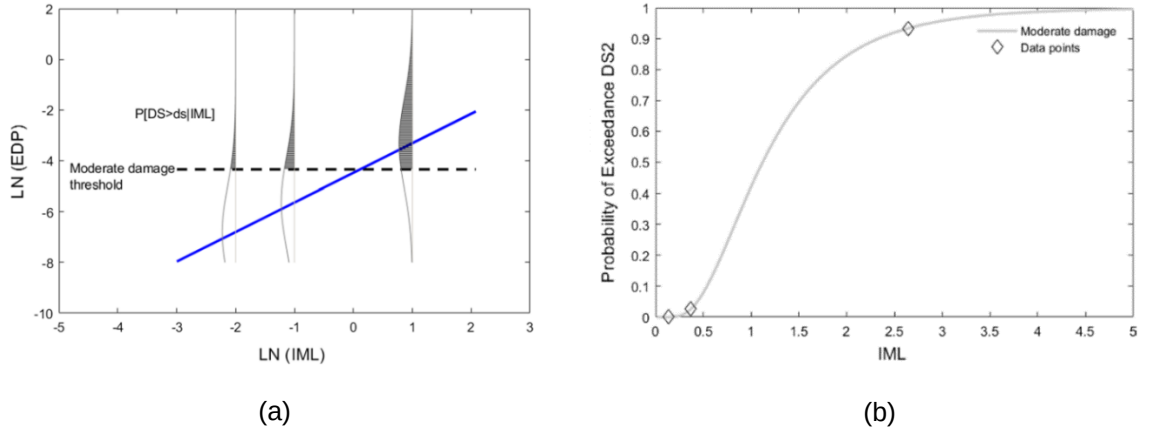


Figure 3.9. Example of construction of a fragility function following the cloud analysis approach (adapted from L. Martins and V. Silva [11])

4

Survey Data

4.1. INTRODUCTION TO SURVEY DATA

In pursuit of the overarching objective of this thesis, it became imperative to meticulously examine a representative sample of buildings, discerning their nuanced distinctions in order to effectively proceed with the disaggregation of the class-level fragilities developed within the ESRM20 project framework. Consequently, a comprehensive analysis was undertaken, focusing on a selection of 25 buildings situated in Greece. The primary aim was to delineate the diverse characteristics exhibited by these structures, thereby facilitating an endeavour to categorize them within a typological framework, as previously delineated in a preceding chapter. By studying these buildings closely, a better understanding of their details and complexities was reached. This helped to build a strong base for further analysis and conclusions in the thesis.

During the process of surveys collection, various observations were made, including:

- State of the building (abandoned or not);
- The number of stories;
- The average height of the first storey;
- The average height of the other storeys;
- Floor system diaphragm;
- Date of construction;
- Conservation level of the building;
- If the building is irregular in elevation;
- Plan-shape;

Not all the gathered information was deemed essential for categorizing the surveyed buildings into the class-typologies utilized in the ESRM20 framework. However, Table 4.1 provides a comprehensive overview of all the buildings analysed, along with their respective specifications, encompassing the various factors previously mentioned.

Table 4.1. Parameters evaluated in the surveys

Building	Material	N° of stories	Average height of the first story	Average height of the other storeys	Floor system diaphragm	Date of construction	Is the building irregular in elevation?	Plan shape
1	MUR - stone	1	3.5 to 5.0	-	Concrete slab	Before 1959	No	Regular
2	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1959-1984	Yes	Irregular
3	MUR - stone	1	3.5 to 5.0	-	Wooden	Before 1959	No	Regular
4	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1959-1984	No	Regular
5	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1959-1984	No	Regular
6	RC - frame	1	2.5 to 3.5	-	Concrete slab	1985-1994	No	O-shape
7	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1985-1994	No	Regular
8	RC - frame	1	2.5 to 3.5	-	Wooden	1985-1994	No	Regular
9	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1985-1994	Yes	Regular
10	RC - frame	1	2.5 to 3.5	-	Concrete slab	1985-1994	No	H-shape
11	MUR - stone	2	3.5 to 5.0	2.5 to 3.5	Wooden	Before 1959	No	Regular
12	RC - frame	1	3.5 to 5.0	-	Concrete slab	Before 1959	No	Regular
13	RC - frame	1	2.5 to 3.5	-	Concrete slab	1995-present	No	L-shape
14	RC - frame	1	2.5 to 3.5	-	Concrete slab	1995-present	No	Regular
15	RC - frame	1	2.5 to 3.5	-	Concrete slab	1959-1984	No	T-shape
16	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1959-1984	No	Regular
17	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1995-present	No	Regular
18	RC - frame	1	2.5 to 3.5	-	Concrete slab	1995-present	Yes	Regular
19	MUR - stone	1	3.5 to 5.0	-	Wooden	Before 1959	No	Regular
20	RC - frame	1	2.5 to 3.5	-	Concrete slab	Before 1959	No	Regular
21	MUR - stone	1	3.5 to 5.0	-	Wooden	Before 1959	No	Regular
22	RC - frame	1	2.5 to 3.5	-	Concrete slab	1985-1994	No	Regular
23	MUR - stone	1	3.5 to 5.0	-	Wooden	Before 1959	No	Regular
24	RC - frame	1	2.5 to 3.5	-	Concrete slab	1985-1994	No	Regular
25	RC - frame	2	2.5 to 3.5	2.5 to 3.5	Concrete slab	1985-1994	No	Regular

4.2. CONSTRUCTION DATE

The determination of construction dates holds significant relevance within this context, as it facilitates the precise categorization of buildings and the assignment of the most appropriate code level to each structure or ductility level in case of buildings that are not built with reinforced concrete with infills (CR_LFINF) or with concrete frames only (CR_LFM). As elaborated upon in subchapter 2.2.1, particularly concerning Greece, pivotal years such as 1959, 1985, and 1995 serve as benchmarks delineating the evolution of building code standards. Prior to 1959, for instance, no standardized building code was in effect, denoted as CDN (Code Not Defined). Subsequently, between 1959 and 1984, constructions were classified under a low code level, designated as CDL (Low Code). Following this, the period spanning from 1985 to 1994 corresponds to the medium design code, denoted as CDM (Code Medium), while structures erected post-1995 adhere to high code standards. A detailed description of these buildings is provided in Table 4.2.

Table 4.2. Code level for each building on the survey

Building	Construction Date	Code Level	Ductility Level
1	Before 1959	-	DNO*
2	1959-1984	CDL	-
3	Before 1959	-	DNO
4	1959-1984	CDL	-
5	1959-1984	CDL	-
6	1985-1994	CDM	-
7	1985-1994	CDM	-
8	1985-1994	CDM	-
9	1985-1994	CDM	-
10	1985-1994	CDM	-
11	Before 1959	-	DNO
12	Before 1959	CDN	-
13	1995-present	CDH	-
14	1995-present	CDH	-
15	1959-1984	CDL	-
16	1959-1984	CDL	-
17	1995-present	CDH	-
18	1995-present	CDH	-
19	Before 1959	-	DNO
20	Before 1959	CDN	-
21	Before 1959	-	DNO
22	1985-1994	CDM	-
23	Before 1959	-	DNO
24	1985-1994	CDM	-
25	1995-present	CDH	-

* Non-ductile

4.3. PERCENTAGE OF LATERAL FORCE

As seen in Chapter 2.1.1, the taxonomy used includes a parameter related to the percentage of lateral force applied to the building, i.e., the percentage of the building's weight imposed during the pushover analysis. To determine and align specific values with those used in the ESRM20 project 0%, 5%, 10%, 15%, 20%, 25%, and 30% it was necessary to understand which values are most appropriate for each class-typology. The percentage of lateral force is associated with the period of construction of the buildings, meaning it correlated with the benchmark years mentioned in the previous subchapter.

Before the year 1959, no data were found regarding the application of lateral loads to measure building behaviour. Therefore, the adopted value for buildings constructed before this year is 0%. For the benchmark years 1959-1984, as illustrated in

Figure 4.1 extracted from the Government Gazette 1959 [42], a factor corresponding to the percentage of vertical loads was used to represent the lateral loads applied in Greece after 1959. In

Figure 4.1, the Roman numerals (I, II, and III) represent seismic zones, and the Greek letters (α , β , and γ) represent soil materials. Considering the specific location of the buildings evaluated in the Table 4.1, they were in zone II with soil characteristics falling between α and β resulting in a percentage between 6% and 8%. Due to limited information collected regarding construction quality, 5% is the adopted value for the percentage of lateral force representing buildings constructed during these benchmark years.

Π Ι Ν Α Ξ Ι Ι			
Σεισμικής περιοχής	Έπι κινδυνότητας έδαφών		
	(α)	(β)	(γ)
I	0,04	0,06	0,08
II	0,06	0,08	0,12
III	0,03	0,12	0,16

Figure 4.1. Percentage of the lateral load dependent of the seismic zone and the soil material. First column is the seismic zone, subsequent ones are the soil type (a through c) (Exported from Government Gazette [42])

From 1985 to 1994, the approach remained the same, but the percentage derived from

Figure 4.1 was adjusted to account for the importance factor illustrated in

Figure 4.2. This factor depends on the seismic zone, and for buildings in seismic zone II, with and importance factor of 1.5 (Schools), the resulting percentage is between 10% and 15%. So, the value of 15% was assumed to be the one that characterize the class-typology between these benchmark years.

ΠΙΝΑΚΑΣ III.

Σεισμικότητα	Σπουδαιότητα κτιρίου	
	Μεγάλη (Μ)	Συνήθης (Σ)
I και II	1,5	1,0
III	1,2	1,0

Figure 4.2. Importance factor dependent to the seismic zone applied after 1984. First column is the seismic zone, subsequent ones are the importance factors (Μ and Σ for higher and lower importance, respectively) (Exported from Government Gazette [43])

After 1995, the EAK2000 and Eurocode 8 were implemented. The method applied in this case is different compared to the other norms applied in the other benchmark years. With EC8, seismic design is performed using spectrum. Therefore, for this particular location in Greece, applying Eq. (4.1) enables the calculation of the base shear in a building. To calculate $S_d(T_1)$, it is necessary to refer to the design spectrum and identify the position of the period intended for design. In this case, for simplification, regarding to the fact that the buildings have low period, it is assumed that the period lies within the plateau. Consequently, Eq. (4.2) facilitates the calculation of S_d periods within the plateau. With the Greek version of EC8, it became possible to obtain values for a_g and S . The parameter a_g is described in relation to the zone where the site is located, as illustrated in Figure 4.3, and must be multiplied by the importance factor of the building, which in this case, because of being schools, this factor is 1.2. For each of the three zones, a specific value of ground acceleration is provided based on gravitational force, as outlined in Table 4.3. On the other hand, following EC8, S is contingent upon the soil category, which in this case was assumed to be soil C, assigned a value of 1.15. The behaviour coefficient, q , for this building type is approximately 3. Utilizing Eq. (4.2) with the aforementioned values yields a result of $S_d(T_1) = 0.276g$. Replacing this value into Eq. (4.1) with a corresponding λ value according to EC8, the lateral load accounts for approximately 25% of the building's mass.

$$F = S_d(T_1) \cdot m \cdot \lambda \quad (4.1)$$

$$T_B \leq T \leq T_C : S_d(T_1) = a_g \cdot S \cdot \frac{2.5}{q} \quad (4.2)$$

Summarizing the entire analysis conducted on the percentage of lateral load applied to the buildings, it was possible to reach the conclusion regarding the most suitable percentages for the different periods of code application in Greece. In Table 4.4, these previously mentioned percentages can be seen.

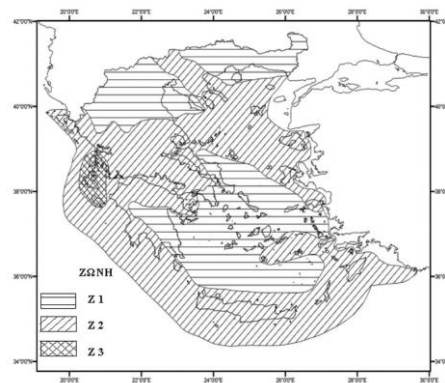


Figure 4.3. Earthquake hazard zones for Greece for use with EN1998-1

Table 4.3. Ground acceleration for each seismic zone in Greece for use with EN1998-1

Zones	a_g/g
Z1	0.16
Z2	0.24
Z3	0.36

Table 4.4. Percentage of lateral force for each benchmark years

Benchmark Years	Percentage of Lateral Force
Before 1959	0%
1959 – 1984	5%
1985 – 1994	15%
1995 – To present	25%

4.4. CONSTRUCTION TYPE

An essential aspect in categorizing buildings into various types lies in the identification of the materials utilized in their construction. During the surveys, it was noted that buildings predominantly employed two distinct materials: unreinforced masonry with rubble stone (MUR-STRUB) and reinforced concrete with infills (CR_LFINF). These typologies were discerned based on the materials employed. For a visual representation, it is outlined in Table 4.5 the construction materials alongside their corresponding typology designation.

Table 4.5. Building Materials and Corresponding Typologies

Building	Material	Material typology
1	MUR-stone	MUR-STRUB
2	RC-frame	CR_LFINF
3	MUR-stone	MUR-STRUB
4	RC-frame	CR_LFINF
5	RC-frame	CR_LFINF
6	RC-frame	CR_LFINF
7	RC-frame	CR_LFINF
8	RC-frame	CR_LFINF
9	RC-frame	CR_LFINF
10	RC-frame	CR_LFINF
11	MUR-stone	MUR-STRUB
12	RC-frame	CR_LFINF
13	RC-frame	CR_LFINF
14	RC-frame	CR_LFINF
15	RC-frame	CR_LFINF
16	RC-frame	CR_LFINF
17	RC-frame	CR_LFINF
18	RC-frame	CR_LFINF
19	MUR-stone	MUR-STRUB
20	RC-frame	CR_LFINF
21	MUR-stone	MUR-STRUB
22	RC-frame	CR_LFINF
23	MUR-stone	MUR-STRUB
24	RC-frame	CR_LFINF
25	RC-frame	CR_LFINF

A representation of what represent the unreinforced masonry with rubble stone building and the reinforced concrete with infills building is represented in the Figure 4.4(a) and Figure 4.4(b), respectively.



(a)



(b)

Figure 4.4. (a) Unreinforced masonry with rubble stone building; (b) Reinforced concrete with infilled frames building

4.5. TAXONOMY

After the comprehensive analysis of each building's specifications conducted in the surveys, a concerted effort was made to categorize these structures into one of the several class-typologies outlined in the ESRM20. Various parameters were scrutinized, including the construction materials utilized, the number of stories, and the levels of ductility such as DNO, DUL, DUM, and DUH for the masonry buildings, as well as the corresponding code levels (CDN, CDL, CDM, and CDH) applicable to buildings constructed with reinforced concrete with infills. Additionally, significant attention was paid to the percentage of lateral force, denoted as a percentage ranging from 0% to 30%, applied to reinforced concrete moment and infills frames. Table 4.6 presents the proposed class-typologies of each building, aligning with those found in ESRM20.

Table 4.6. Typology for each building

Building	Typology
1	MUR-STRUB_LWAL-DNO_H1
2	CR_LFINF-CDL-5_H2
3	MUR-STRUB_LWAL-DNO_H1
4	CR_LFINF-CDL-5_H2
5	CR_LFINF-CDL-5_H2
6	CR_LFINF-CDM-15_H1
7	CR_LFINF-CDM-15_H2
8	CR_LFINF-CDM-15_H1
9	CR_LFINF-CDM-15_H2
10	CR_LFINF-CDM-15_H1
11	MUR-STRUB_LWAL-DNO_H2
12	CR_LFINF-CDN-0_H1
13	CR_LINF-CDH-25_H1
14	CR_LINF-CDH-25_H1
15	CR_LFINF-CDL-5_H1
16	CR_LFINF-CDL-5_H2
17	CR_LFINF-CDH-25_H2
18	CR_LINF-CDH-25_H1
19	MUR-STRUB_LWAL-DNO_H1
20	CR_LFINF-CDN-0_H1
21	MUR-STRUB_LWAL-DNO_H1
22	CR_LFINF-CDM-15_H1
23	MUR-STRUB_LWAL-DNO_H1
24	CR_LFINF-CDM-15-H1
25	CR_LFINF-CDM-15-H2

As can be seen in the Table 4.6, all the 25 buildings observed in the surveys only 9 types of typologies were identified. The number of buildings for each typology is described in Table 4.7.

Table 4.7. Number of buildings for each typology

Typology	N° of buildings
MUR-STRUB_LWAL-DNO_H1	5
MUR-STRUB_LWAL-DNO_H2	1
CR_LFINF-CDN-0_H1	2
CR_LFINF-CDL-5_H1	1
CR_LFINF-CDL-5_H2	4
CR_LFINF-CDM-15_H1	5
CR_LFINF-CDM-15_H2	3
CR_LFINF-CDH-25-H1	3
CR_LFINF-CDH-25-H2	1

4.5. PERIOD OF VIBRATION OF THE BUILDING

Initially, the approach to get the fundamental period of vibration of the buildings, followed the methodology described in the ESRM20. Therefore, for unreinforced masonry buildings analysed in the surveys adhered to the guidelines in Table 2.6 to get the period of vibration directly related with the height of the building. Due to the lack of information about the period of vibration of the buildings constructed with reinforced concrete with infills, initially, it was followed the approach outlined in EN-1998-1, [44]. This approach is characterized by Eq. (4.3), wherein T_1 is calculated as a function of C_t multiplied by H raised to the power of $3/4$. The coefficient C_t varies depending on the structural type: 0.085 for steel frames, 0.075 for moment-resistant concrete frames and eccentrically braced steel frames, and 0.050 for all other structures and H is the height of the building.

$$T_1 = C_t \cdot H^{3/4} \quad (4.3)$$

Table 4.8 provides a comprehensive summary detailing the heights of the buildings under examination alongside their corresponding vibration periods.

After analysing the code developed to run nonlinear time-history analysis in single degree of freedom (SDOF) models for ESRM20, the author understood that the period of the typologies were only related with the capacity curve used and not with the formulas described in Table 2.6. To calculate the fundamental period of vibration of the SDOFs used in ESRM20 was assumed the mass, M , equals to 1, the stiffness, K , was calculated by the ratio of the force and displacement derived from the first point of the capacity curve, and using the Eq. (4.4) it was possible to deduced the angular frequency, and consequently with Eq. (4.5) the period of vibration of the respective SDOF.

$$W = \sqrt{\frac{K}{M}} \quad (4.4)$$

$$T = \frac{2\pi}{W} \quad (4.5)$$

The methodology employed in the European Seismic Risk Model 20 ensures that the height of the building analysed is irrelevant, as the Single-Degree-of-Freedom (SDOF) will consistently exhibit the same period based solely on the capacity curve of the specific typology. Therefore, Table 4.9 presents

the periods of all SDOF models corresponding to the typologies outlined in Table 4.7 for the surveyed buildings.

Table 4.8. Fundamental period of vibration

Building	Material typology	Number of Stories	Total height	T1
1	MUR-STRUB	1	4.05	0.22
2	CR_LFINF	2	7.39	0.34
3	MUR-STRUB	1	3.19	0.18
4	CR_LFINF	2	6.26	0.30
5	CR_LFINF	2	6.26	0.30
6	CR_LFINF	1	3.03	0.17
7	CR_LFINF	2	5.90	0.28
8	CR_LFINF	1	2.92	0.17
9	CR_LFINF	2	6.19	0.29
10	CR_LFINF	1	2.92	0.17
11	MUR-STRUB	2	7.51	0.38
12	CR_LFINF	1	4.35	0.23
13	CR_LFINF	1	3.25	0.18
14	CR_LFINF	1	3.23	0.18
15	CR_LFINF	1	2.99	0.17
16	CR_LFINF	2	6.26	0.30
17	CR_LFINF	2	6.20	0.29
18	CR_LFINF	1	3.50	0.19
19	MUR-STRUB	1	4.44	0.24
20	CR_LFINF	1	3.15	0.18
21	MUR-STRUB	1	4.00	0.22
22	CR_LFINF	1	3.20	0.18
23	MUR-STRUB	1	4.10	0.23
24	CR_LFINF	1	3.10	0.18
25	CR_LFINF	2	6.55	0.31

Table 4.9. Period of vibration of the SDOFs used in ESRM20

Typology	SDOF periods of vibration [s]
MUR-STRUB_LWAL-DNO_H1	0.08
MUR-STRUB_LWAL-DNO_H2	0.15
CR_LFINF-CDN-0_H1	0.32
CR_LFINF-CDL-5_H1	0.30
CR_LFINF-CDL-5_H2	0.53
CR_LFINF-CDM-15_H1	0.29
CR_LFINF-CDM-15_H2	0.50
CR_LFINF-CDH-25-H1	0.17
CR_LFINF-CDH-25-H2	0.33

4.7. FRAGILITY CURVES

The fragility curves obtained in ESRM20 for these class-typologies are presented, graphically in Annex B. These fragility curves were developed using Cloud Analysis, where it was possible from the linear regression to get the values for the medians (μ) for every damage state level adopted and for each intensity measure assumed (PGA , $S_a(0.3s)$, $S_a(0.6s)$ and $S_a(1.0s)$). The dispersion (β) it was assumed to be the same regardless the damage state that was evaluated. These parameters are described in Table 4.10 to Table 4.13.

Table 4.10. Fragility curves parameters with PGA as IM (ESRM20 [1])

Building class	IM	DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H1	PGA	0.24	0.47	0.49	0.47	0.66	0.47	0.80	0.47
MUR-STRUB_LWAL-DNO_H2	PGA	0.20	0.63	0.45	0.63	0.65	0.63	0.82	0.63
CR_LFINF-CDN-0_H1	PGA	0.60	0.89	0.74	0.89	0.89	0.89	1.03	0.89
CR_LFINF-CDL-5_H1	PGA	0.57	0.84	0.74	0.84	0.91	0.84	1.07	0.84
CR_LFINF-CDL-5_H2	PGA	0.35	1.21	0.48	1.21	0.62	1.21	0.75	1.21
CR_LFINF-CDM-15_H1	PGA	0.59	0.72	0.88	0.72	1.16	0.72	1.42	0.72
CR_LFINF-CDM-15_H2	PGA	0.40	1.11	0.58	1.11	0.76	1.11	0.93	1.11
CR_LFINF-CDH-25-H1	PGA	1.37	0.50	3.32	0.50	5.23	0.50	7.02	0.50
CR_LFINF-CDH-25-H2	PGA	0.76	0.75	1.82	0.75	2.89	0.75	3.90	0.75

Table 4.11. Fragility curves parameters with $S_a(0.3s)$ as IM (ESRM20 [1])

Building class	IM	DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H1	$S_a(0.3s)$	0.37	0.60	0.80	0.60	1.15	0.60	1.46	0.60
MUR-STRUB_LWAL-DNO_H2	$S_a(0.3s)$	0.30	0.61	0.78	0.62	1.21	0.61	1.59	0.61
CR_LFINF-CDN-0_H1	$S_a(0.3s)$	1.19	0.72	1.45	0.72	1.72	0.72	1.96	0.72
CR_LFINF-CDL-5_H1	$S_a(0.3s)$	1.113	0.71	1.44	0.71	1.76	0.71	2.06	0.71
CR_LFINF-CDL-5_H2	$S_a(0.3s)$	0.73	0.95	0.97	0.95	1.23	0.95	1.46	0.95
CR_LFINF-CDM-15_H1	$S_a(0.3s)$	1.13	0.63	1.69	0.63	2.25	0.63	2.76	0.63
CR_LFINF-CDM-15_H2	$S_a(0.3s)$	0.79	0.90	1.12	0.90	1.45	0.90	1.77	0.90
CR_LFINF-CDH-25-H1	$S_a(0.3s)$	3.34	0.54	5.95	0.54	8.77	0.54	11.54	0.54
CR_LFINF-CDH-25-H2	$S_a(0.3s)$	2.17	0.83	4.15	0.83	6.30	0.83	8.42	0.83

Table 4.12. Fragility curves parameters with $S_a(0.6s)$ as IM (ESRM20 [1])

Building class	IM	DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H1	$S_a(0.6s)$	0.10	0.92	0.27	0.92	0.43	0.92	0.57	0.92
MUR-STRUB_LWAL-DNO_H2	$S_a(0.6s)$	0.11	0.65	0.31	0.65	0.50	0.65	0.67	0.65
CR_LFINF-CDN-0_H1	$S_a(0.6s)$	0.57	0.51	0.70	0.51	0.85	0.51	0.98	0.51
CR_LFINF-CDL-5_H1	$S_a(0.6s)$	0.52	0.54	0.69	0.54	0.86	0.54	1.02	0.54
CR_LFINF-CDL-5_H2	$S_a(0.6s)$	0.41	0.42	0.52	0.42	0.65	0.42	0.77	0.42
CR_LFINF-CDM-15_H1	$S_a(0.6s)$	0.49	0.64	0.80	0.64	1.12	0.64	1.43	0.64
CR_LFINF-CDM-15_H2	$S_a(0.6s)$	0.42	0.42	0.58	0.42	0.75	0.42	0.91	0.42
CR_LFINF-CDH-25-H1	$S_a(0.6s)$	2.87	1.93	46.12	1.93	191.65	1.93	480.57	1.93
CR_LFINF-CDH-25-H2	$S_a(0.6s)$	0.68	0.65	1.86	0.65	3.15	0.65	4.44	0.65

Table 4.13. Fragility curves parameters with $S_a(1.0s)$ as IM. (ESRM20 [1])

Building class	IM	DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H1	$S_a(1.0s)$	0.03	1.19	0.11	1.19	0.19	1.19	0.26	1.19
MUR-STRUB_LWAL-DNO_H2	$S_a(1.0s)$	0.04	0.82	0.13	0.82	0.22	0.82	0.31	0.82
CR_LFINF-CDN-0_H1	$S_a(1.0s)$	0.25	0.66	0.32	0.66	0.40	0.66	0.47	0.66
CR_LFINF-CDL-5_H1	$S_a(1.0s)$	0.23	0.71	0.31	0.71	0.41	0.71	0.50	0.71
CR_LFINF-CDL-5_H2	$S_a(1.0s)$	0.17	0.50	0.23	0.50	0.30	0.50	0.35	0.50
CR_LFINF-CDM-15_H1	$S_a(1.0s)$	0.22	0.82	0.39	0.82	0.57	0.82	0.77	0.82
CR_LFINF-CDM-15_H2	$S_a(1.0s)$	0.18	0.53	0.26	0.53	0.34	0.53	0.42	0.53
CR_LFINF-CDH-25-H1	$S_a(1.0s)$	2.44	2.84	125.99	2.84	954.27	2.84	3525.06	2.84
CR_LFINF-CDH-25-H2	$S_a(1.0s)$	0.32	0.85	1.10	0.85	2.10	0.85	3.19	0.85

5

DISAGGREGATION OF FRAGILITY CURVES**5.1. INTRODUCTION**

In this chapter, the methodology behind the disaggregation of the fragility curves obtained is explained. The class-typologies described in the Earthquake Seismic Risk Model 2020 (ESRM20) offer a comprehensive framework for categorizing buildings based on various attributes. However, upon closer examination, it becomes evident that these class-level fragilities may not adequately capture the full spectrum of structural diversity. Important factors such as the presence of strong versus weak column designs (e.g., SCWB vs WCSB), the composition of ductile or brittle elements, plan-shape intricacies, torsion effects resulting from mass eccentricity, and conservation levels are either not explicitly addressed or are insufficiently differentiated within the existing typological framework. Recognizing these limitations, this study adopts a novel approach to disaggregate buildings based on their plan-shape. The decision to prioritize plan-shape as the primary distinguishing factor stems from two key considerations: the ease of using macroscale visualization to illustrate the different geometric natures of Greek buildings, particularly schools like those analyzed in the surveyed dataset, and the feasibility of modeling plan-shapes with relative ease compared to subjective assessments of conservation levels.

To ensure almost 100% accuracy of this process, it would be necessary to: conduct proper surveys of millions of buildings; obtain Single Degree of Freedom (SDOF) results for each specific building under hazard-consistent motions per site; and have an intensity Measure (IM) that is suitable for all of them. Due to the impracticality of achieving such a high level of accuracy within a limited timeframe, an alternative approach was adopted, which will be detailed in the following subchapters.

To achieve this objective, the following procedure was followed, as also represented, to better understand, in Figure 5.1:

- 1 Identification of the class-typology that matches better the buildings surveyed.
- 2 Performing IDA on SDOFs that represent those typology classes for hazard-consistent records.
- 3 Fitting empirical fragilities curves produced from the IDA results for specific intensity measures.
- 4 Disaggregation of the fragility curves calculated, following three different approaches taking into account a weight participation factor, w_i , and a factor that accounts for the plan-shape, a_i .
- 5 Calculation and comparison of the disaggregated fragility curves.

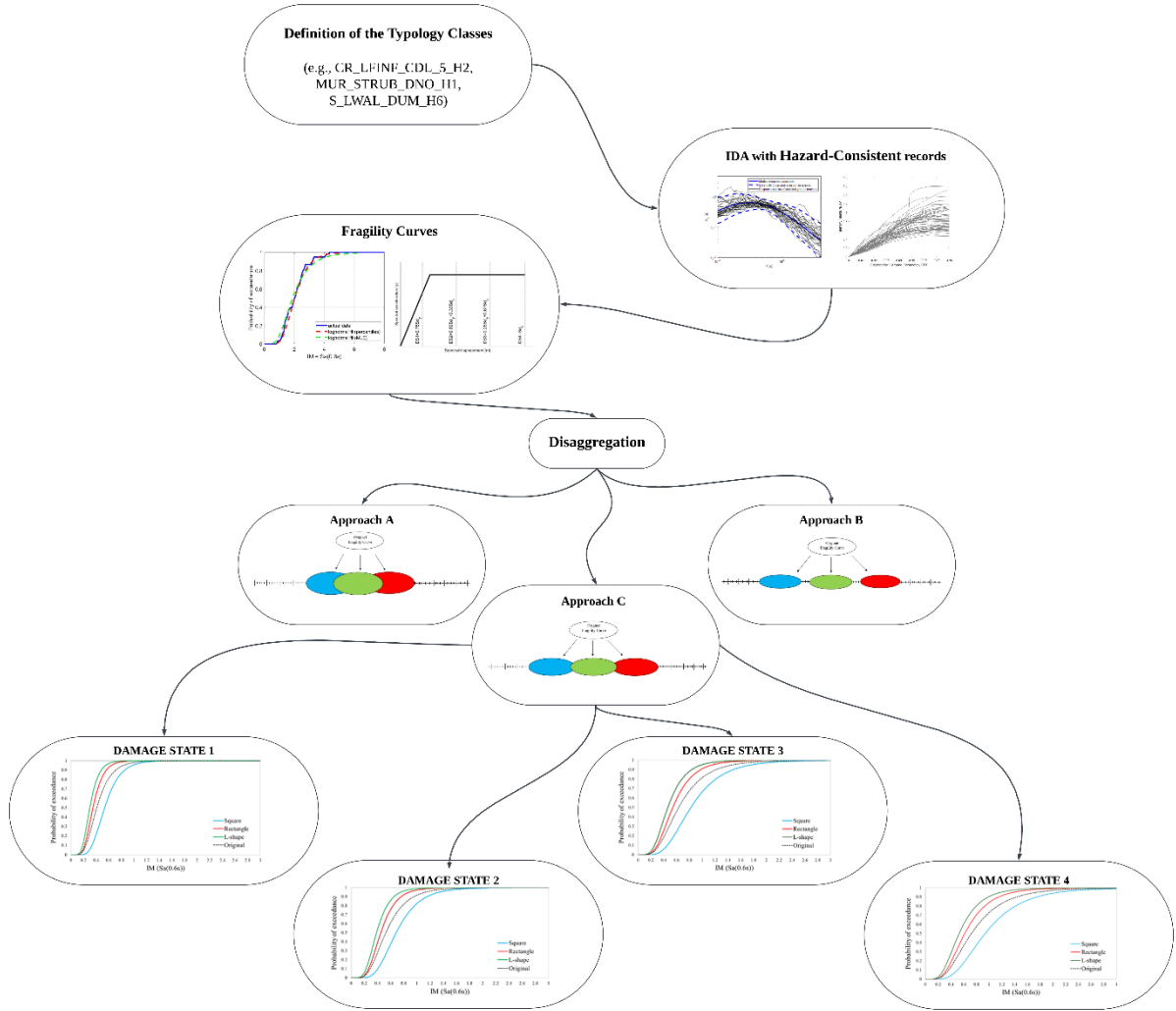


Figure 5.1. Flowchart of the procedure to disaggregate Class-Level Seismic Fragilities

This methodology aims to provide a more nuanced understanding of fragility across different plan-shapes, allowing for a more targeted and accurate assessment of vulnerability in diverse building typologies in Greece.

5.2. NONLINEAR TIME-HISTORY ANALYSIS IN CLASS-TYPOLOGIES'S SDOFs

5.2.1. HAZARD-CONSISTENT RECORDS

To adopt the most effective approach, rather than relying on fragility curves derived from generalized data across Europe, the decision was made to rectify hazard inconsistencies specific to the site of interest by analyzing buildings in Greece. Assuming hazard consistency of ground motion inputs is really important to effectively assess the future reliability of a structure in a given location. A careful selection of records ensures that the distributions of response spectra at all periods are nominally comparable, and therefore the distribution of resulting structural response should be also comparable (Lin et al. [45]). For a ground motion selection was necessary first to perform a seismic hazard analysis for the site and

the range of periods in interest, represented in Figure 5.2. In the site of interest to align closely with the periods evaluate it was used the Average Spectral acceleration with a range of period between 0.1s and 1.0s.

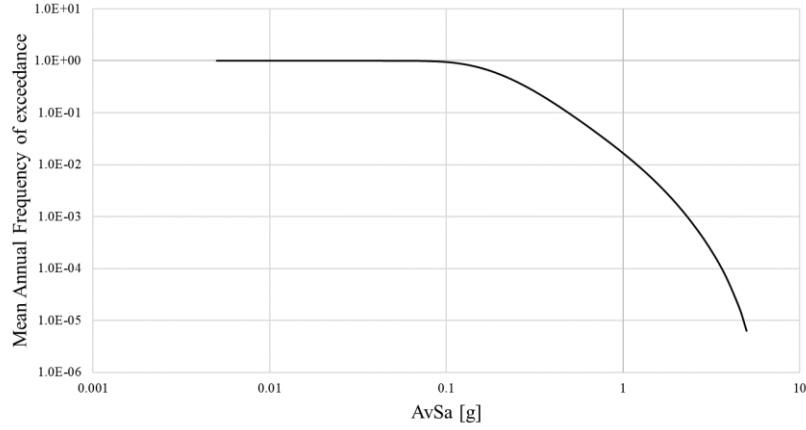


Figure 5.2. Hazard curve for the site under interest

The disaggregation at 10% in 50-year probability of exceedance was performed at also to identify the ground motion characteristics that contributed to the occurrence of a specified ground motion intensity level. OpenQuake was employed for this purpose, utilizing disaggregation to refine the hazard assessment for the particular area and source, and using the motion prediction equation (GMPE) by Boore and Atkinson, 2008.

The area source depicted in Figure 5.3, utilized for the analysis, was scaled down compared to the entire European Area Source Model due to constraints in computational power and time. Consequently, it was restricted to the specific area of interest to manage these limitations effectively.

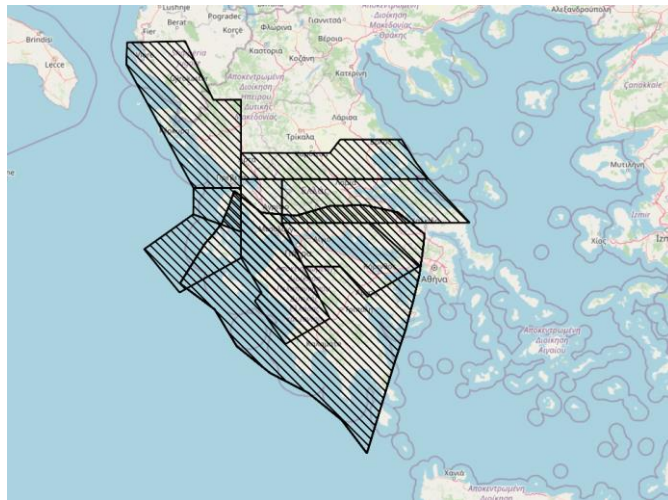


Figure 5.3. Area Source

After ground motions were selected, it was possible for the intensity level and the range of periods under interest to compute the response spectra and compare with the target seismic hazard curves to check the hazard consistency as illustrated in Figure 5.4.

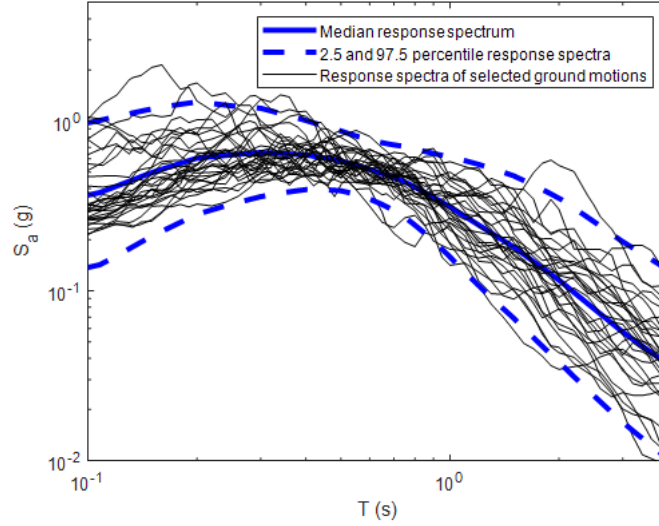


Figure 5.4. Conditional spectrum AvgSa-based record selection for the site of interest, considering the 475-year return period

5.2.2. CLOUD ANALYSIS USING HAZARD-CONSISTENT RECORDS

It was decided, at the beginning, to use the same code that developed the ESRM20 fragility curves, to get new fragility curves, but now with hazard-consistent records obtained in the previous subchapter. Using the ESRM20 fragility curves to be disaggregated it ended to be not an option regarding the reasons that will be presented in the next chapter. The values obtained for the fragility curves' parameters were not coherent with the values resulting by the project ESRM20. The linear regressions obtained using Cloud Analysis with hazard-consistent records for the intensity measures that provided the lowest dispersions are illustrated in Figure 5.5. The quality of the simple linear regression can be affected by the choice and the quantity of the ground motion records selected (Jalayer [46]). Therefore, using only 60 ground motions might have been the reason behind the poor fit of these linear regressions.

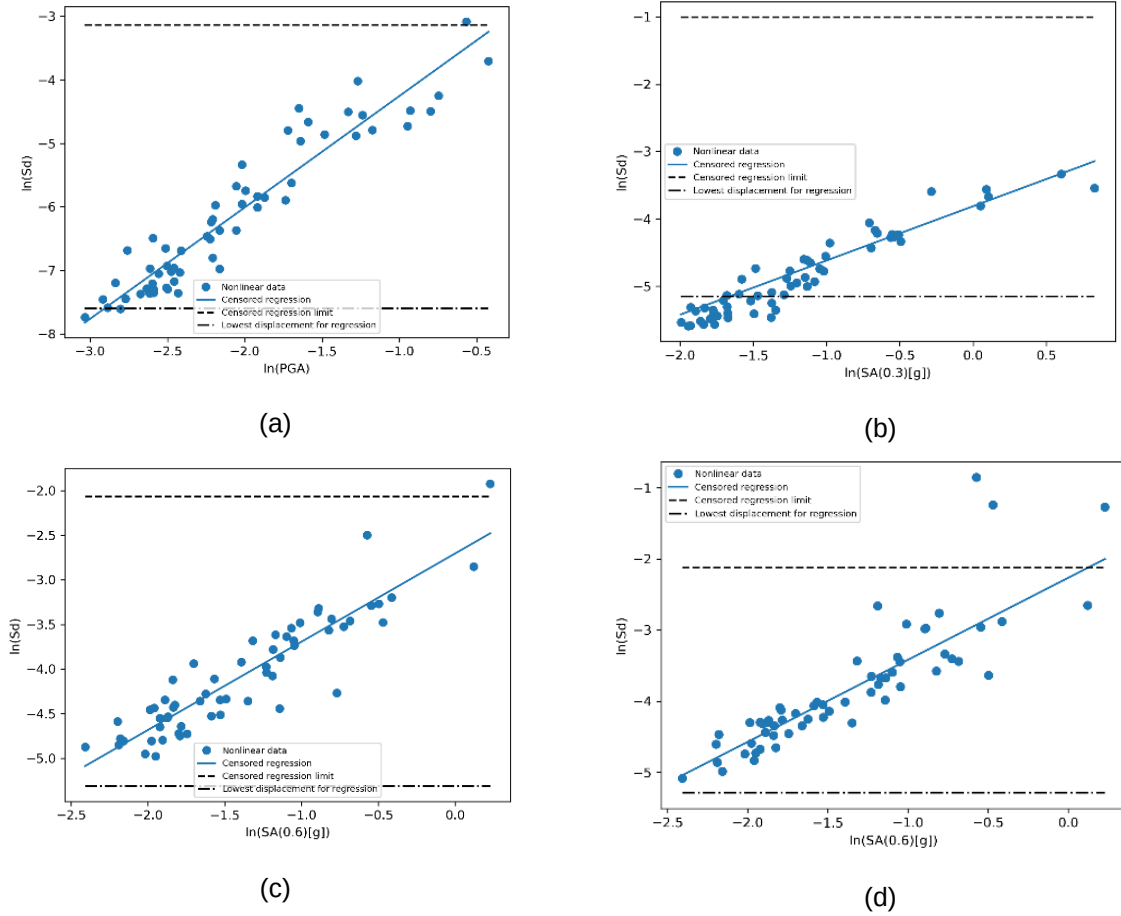


Figure 5.5. Linear regressions using Cloud Analysis. (a) Class-typology MUR-STRUB_LWAL-DNO_H2; (b) Class-typology CR_LFINF-CDH-25_H2; (c) Class-typology CR_LFINF-CDM-15_H2; (d) Class-typology CR_LFINF-CDL-5_H2

5.2.3. INCREMENTAL DYNAMIC ANALYSIS (IDA) IN SDOFs

With the hazard-consistent records already selected, it was possible to apply nonlinear time-history analysis using the same single-degree-of-freedom models employed in ESRM20. In this study, Incremental Dynamic Analysis (Vamvatsikos and Cornell [2]), was utilized instead of cloud Analysis. In IDA, a nonlinear structural model undergoes testing with a series of ground motion records that are scaled, each tailored to various levels of continuous and scalable intensity measure, such as S_a and PGA , for example. These ground motion records, originally recorded without scaling, are then adjusted using a non-negative scaling factor to produce scaled accelerograms. Through this scaling process, the amplitudes of the signal are modified while preserving their original frequency content. The decision to use Incremental Dynamic Analysis was based on several factors. Firstly, the Cloud Analysis method used in ESRM20 assumed the same variability for different damage states, as illustrated in Table 4.10 to Table 4.13. Conversely, as depicted in Figure 5.6, the dispersion in IDA results demonstrates an increase in variability as the damage state level increases, which appears to be more realistic. Additionally, using Cloud Analysis assumes a simple linear regression, yielding unreasonable results for medians of lognormal distributions of fragility curves at higher damage state levels, as evidenced in Table 4.10 to Table 4.13 for the typology CR_LFINF-CDH-25_H2.

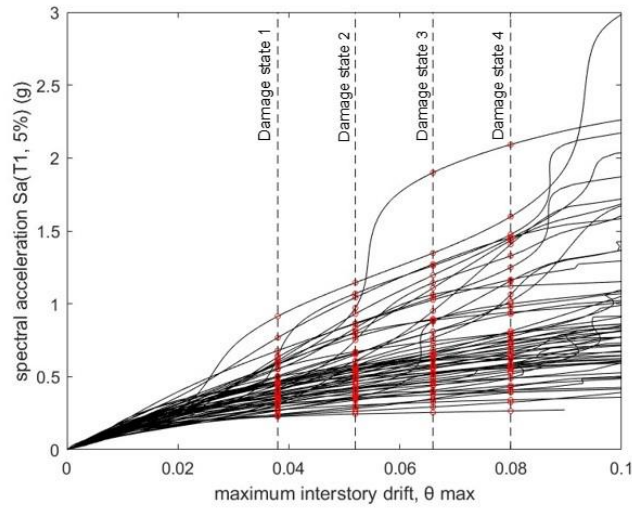


Figure 5.6. IDA curves for CR_LFINF-CDL-H2 typology

For these reasons, IDA was implemented for the building class-typologies outlined in Table 4.7, specifically focusing on those with two stories. To maintain consistency with the methodology used to develop fragility curves in ESRM20, the same periods as those employed in the SDOF Python script from ESRM20 (Table 4.9) were adopted. Additionally, the same damping ratios were applied for the respective materials – reinforced concrete (CR) and unreinforced masonry (MUR) – set at 5% and 10%, respectively. Special attention was given to typologies exhibiting the degradation of the pinching4 model, distinguishing between true and false cases. As a result, adjustments were made to the backbone parameters of the capacity curve for the pinching4 model. It was noted that the capacity curves described in ESRM20 underwent an adaptation, transitioning from the left side of Figure 5.7 to the right side, specifically for the class-typologies with reinforced concrete with infills. Following this adaptation, the capacity curves were maintained with four points; however the first point was removed, and in the new curve (on the right side of the Figure 5.7), the third point represents an average between the second and last points.

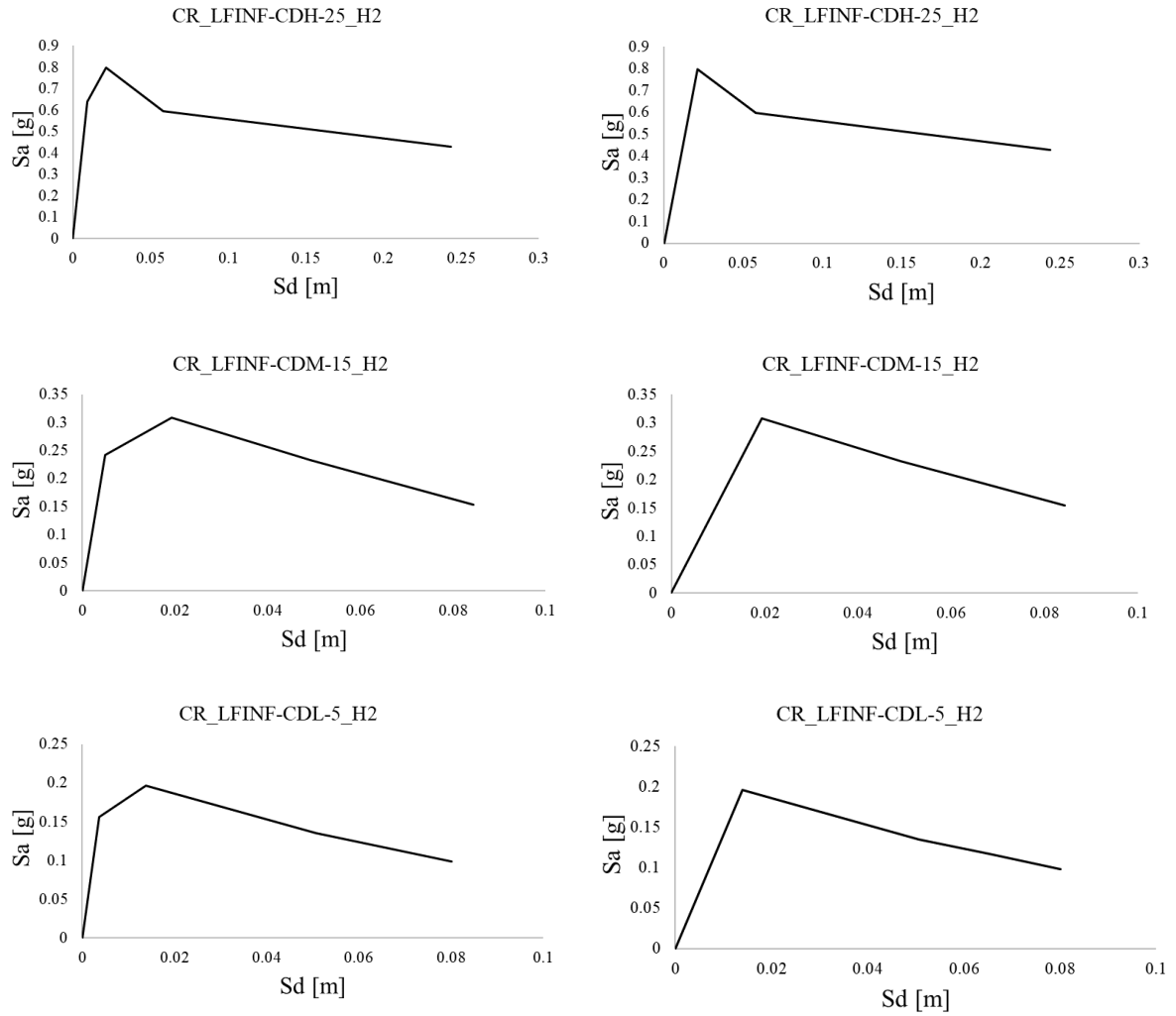


Figure 5.7. Adaptation of the Capacity curves for the pinching4 model

5.3 FRAGILITY CURVES FROM IDA RESULTS

The fragility assessment was made considering different approaches from the results of the IDA analysis. These approaches are the most common approaches to parameter estimation in the lognormal distribution, which are the Empirical cumulative distribution function (CDF), Method of Moments, 16%, 50%, 84% IM – percentiles, and Maximum Likelihood Estimation method. These methods are widely employed in estimating parameters of statistical distributions, providing distinct yet complementary approaches to inferring the parameters of the lognormal distribution from observed data (Flenga et al. [47]).

5.3.1. EMPIRICAL CUMULATIVE DISTRIBUTIVE FUNCTION (CDF)

The utilization of IDA approach yields EDPs outcomes aligned with each seismic excitation at a specific IM value. Consequently, stripes of distinct EDP values emerge as IM levels escalate. Estimating the likelihood of an EDP surpassing a designated performance threshold at a given IM value entails aggregating occurrences where the EDP exceeds capacity A across all seismic excitations and dividing by the total number of such excitations:

$$P[EDP \geq A] = \frac{m}{n} \quad (5.1)$$

Where “m” signifies the count of instances surpassing a predefined limit state (A) at the IM level, represented in Figure 5.8 with the red dots, and “n” represents the total count of seismic excitations. Figure 5.8 showcases the IDA curves for “n” ground motions alongside the fragility curve derived from the empirical CDF method.

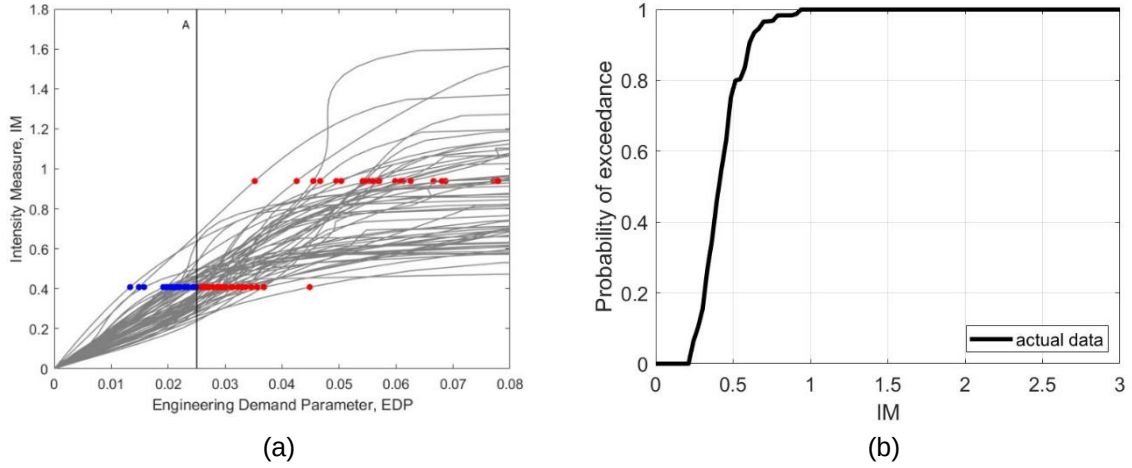


Figure 5.8. (a) IDA curves and stripes of EDP at discrete values of IM; (b) Fragility curve based on empirical cumulative distribution function (CDF)

5.3.2. METHOD OF MOMENTS

The Method of Moments estimation is a widely used technique that involves aligning sample moments with unobservable population moments, where it is possible to derive solutions for the parameters that require estimation (Ginos [48]). The median (μ) and the dispersion (β) in continuous representation are determined through the employment of the moment method. This approach, also known as “Method A” by Porter et al. [49], draws from the outcome of the IDA. While the method is straightforward to apply, its accuracy is notably limited. Notably, estimating theoretical variance using its sample equivalent often yields imprecise results (Diana [50]). It involves utilizing a specific limit state under a lognormal distribution to calculate the median (μ) and dispersion (β) values of the Intensity Measure (IM) at the point of exceedance initiation. In Figure 5.9 can be seen that for each seismic motion the value of IM for the specific level A.

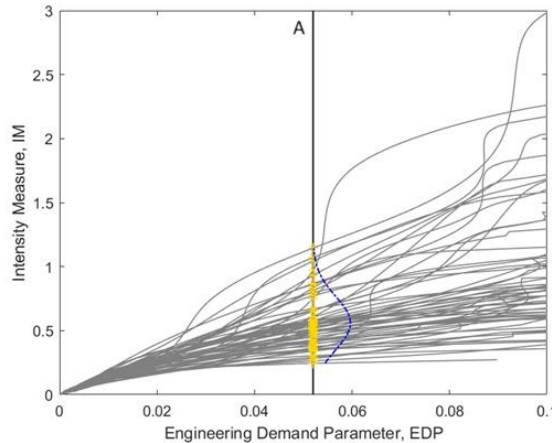


Figure 5.9. IDAs curves and corresponding values of IM for the specific level of capacity A

The parameters of the fragility curve can be estimated using:

$$\mu = \frac{1}{n} \sum_{i=1}^n \ln(IM_i) \quad (5.2)$$

$$\beta = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (\ln(IM_i) - \mu)^2} \quad (5.3)$$

where, n denotes the count of seismic excitations, while IM_i represents the intensity measure value linked with the onset of exceedance a particular performance level.

5.3.3. 16%, 50%, 84% IM – PERCENTILES

This method is very simple to obtain the median (μ) and the dispersion (β). Each data point within the condensed intensity measure (IM) and engineering demand parameter (EDP) pairs. These thresholds indicate the upper limit of EDP where 16%, 50%, and 84% of the data samples exhibit demands lower than or equal to this value at a given IM level. Consequently, within the framework of fragility analysis, the 16%, 50%, and 84% percentile curves are delineated in Figure 5.10 (a). The median (μ), often termed as the point of intersection of the 50% condensed IM-EDP curve with the defined capacity level ($\mu = IM_A, 50\%$), serves as a pivotal reference. Meanwhile, the dispersion parameter (β) is calculated by Eq.(5.4) (Freddi et al. [51]).

$$\beta = \frac{1}{2} \ln \left(\frac{IM_{84\%,A}}{IM_{16\%,A}} \right) \quad (5.4)$$

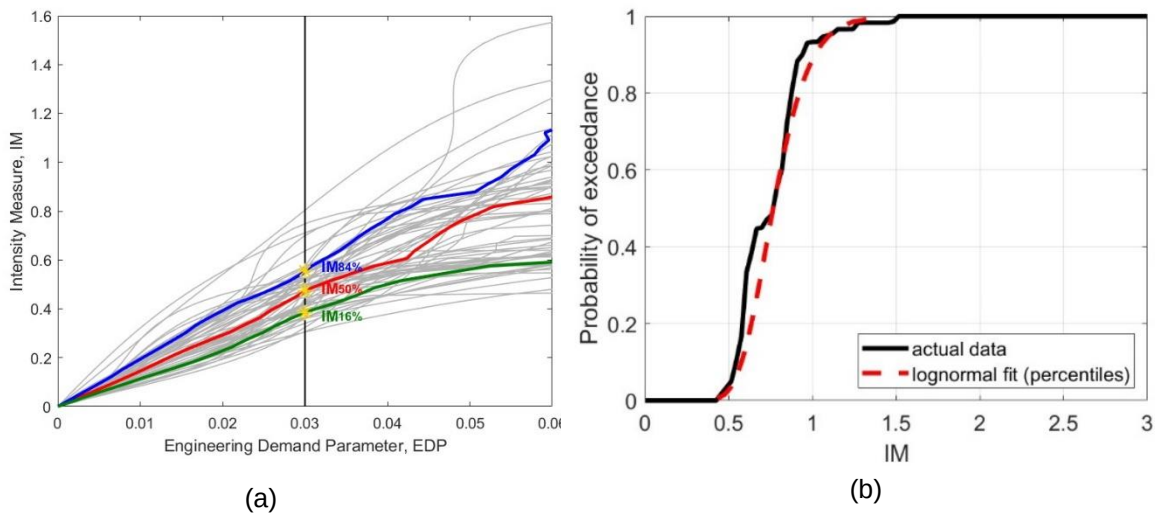


Figure 5.10. (a) 16%, 50%, 84% percentiles summarized IDAs curves for a level of capacity A; (b) Fragility curve based on the method of percentiles

5.3.4. MAXIMUM LIKELIHOOD ESTIMATION METHOD, MLE

Maximum Likelihood Estimation method emerges as a favoured technique for estimating parameters across various distributions due to its ability to select parameter values that render observed data most probable compared to any other feasible parameter values (Ginos [48]). Baker [52] introduced the MLE method to estimate the parameters of the distribution that maximize the probability of occurrence of the observed data. The likelihood function can be calculated as:

$$Likelihood = \prod_{i=1}^m \binom{n_i}{z_i} \Phi\left(\frac{\ln(IM_i)}{\beta}\right)^{z_i} \left(1 - \Phi\left(\frac{\ln(IM_i) - \mu}{\beta}\right)\right)^{n_i - z_i} \quad (5.5)$$

Where, m accounts for the number of IM levels, z_i counts how many ground motions exceed a performance level out of n_i seismic activities, and Π is a symbol for the product of all the IM levels considered. By maximizing this likelihood function of Eq.(5.5), it is possible to determine the median (μ) and the dispersion (β), described in Eq.(5.6).

$$\{\mu, \beta\} = \max_{\mu, \beta} \sum_{i=1}^m \left\{ \ln \binom{n_i}{z_i} + z_i \ln \Phi\left(\frac{\ln(IM_i)}{\beta}\right) + (n_i - z_i) \ln \left(1 - \Phi\left(\frac{\ln(IM_i) - \mu}{\beta}\right)\right) \right\} \quad (5.6)$$

5.3.5. FRAGILITY CURVES' RESULTS

From the methods described previously, only the method of percentiles and the MLE method were implemented to fit the best way possible the real data (i.e., the empirical cumulative distribution function). The fragility curves for the class-typologies were done considering the damage state levels following the same approach as used in ESRM20, illustrated in Figure 3.6. The values correspondent to those damage state levels are described in Table 5.1.

Table 5.1. Values of damage state levels for the typologies with two stories

Typology Class	Damage State Level			
	DS1	DS2	DS3	DS4
MUR-STRUB_LWAL-DNO_H2	0.004	0.012	0.021	0.029
CR_LFINF-CDH-25_H2	0.043	0.109	0.178	0.243
CR_LFINF-CDM-15_H2	0.037	0.052	0.069	0.084
CR_LFINF-CDL-5_H2	0.038	0.052	0.066	0.080

An example of the fittings for those four damage states levels for typology CR_LFINF-CDM-15_H2 is illustrated in Figure 5.11, and Table 5.2 represents the fragility curves parameters gotten from the method that best fit the real data. The choice of the IM (PGA , $S_a(0.3s)$, $S_a(0.6s)$ and $S_a(1.0s)$) relied on the

intensity measure that provided the lowest dispersion. The fragility curves developed for the other class-typologies, resulted from IDA, are presented in Annex C.1.

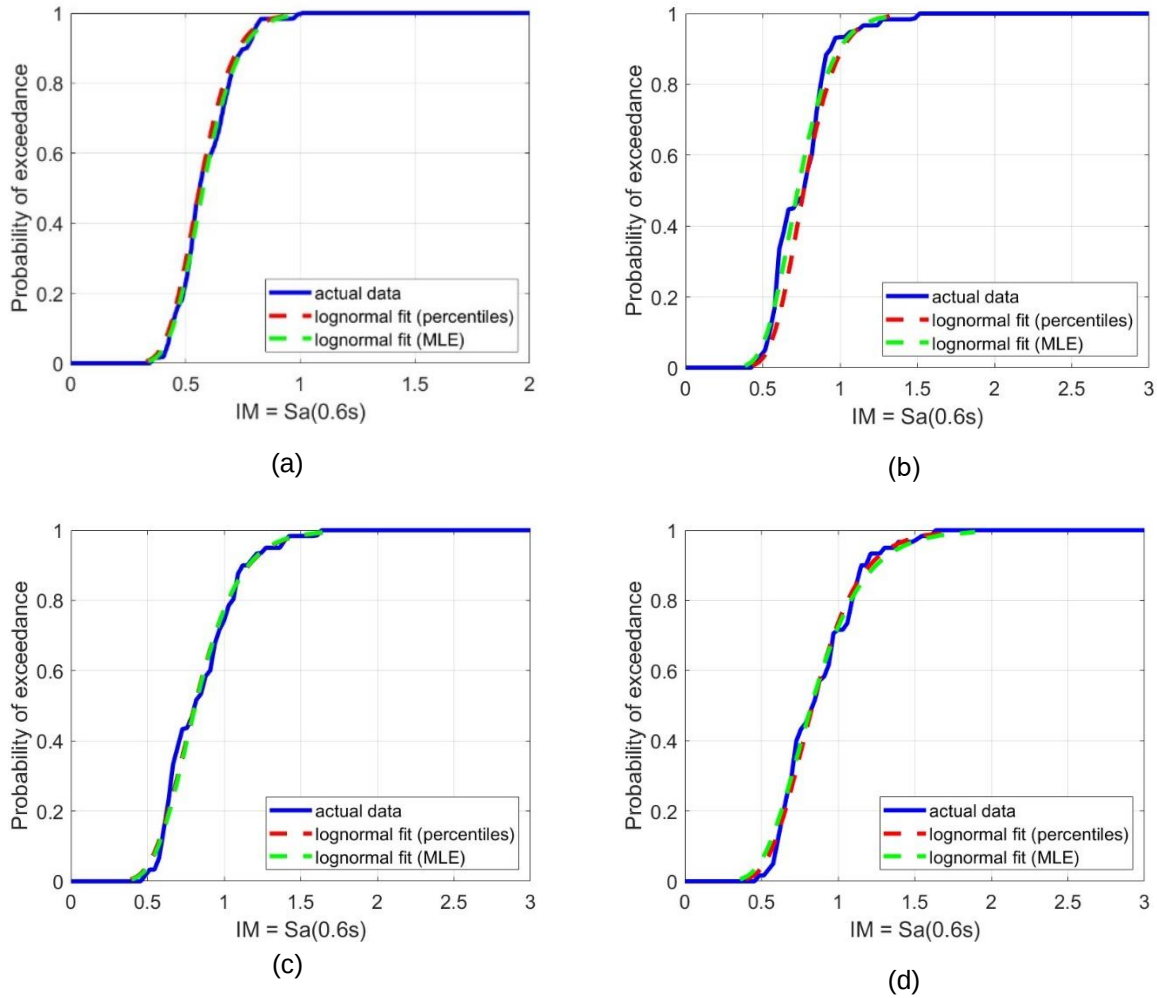


Figure 5.11. Fragility Curves derived from IDA for typology CR_LFINF_CDM_15_H2; (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

Table 5.2. Fragility curves' parameters for the class-typologies with two stories from IDA results.

Typology Class	IM	Damage State Level							
		DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H2	PGA	0.152	0.174	0.267	0.251	0.367	0.312	0.450	0.338
CR_LFINF-CDH-25_H2	$S_a(0.3s)$	1.546	0.252	2.282	0.386	2.403	0.469	2.456	0.470
CR_LFINF-CDM-15_H2	$S_a(0.6s)$	0.577	0.206	0.726	0.247	0.812	0.277	0.833	0.289
CR_LFINF-CDL-5_H2	$S_a(0.6s)$	0.410	0.304	0.525	0.357	0.633	0.413	0.720	0.442

5.4. DECOMPOSITION OF THE FRAGILITY CURVES

To disaggregate the original fragility curves resulting from Incremental Dynamic Analysis (IDA) analysis three different approaches were developed, obtaining three new curves with different medians and dispersions. Similar to the work developed by Kazantzi and Vamvatsikos [53], for these approaches to be applied it was necessary to resort to two statistical principles, enabling the decomposition of an original fragility curve. These principles delineated as the law of total expectation [54], (represented by Eq.(5.7)) and the law of total variance [55], (denoted by Eq.(5.8)), where X and Y are random variables on the same probability space, $E(X)$ is the expected value of X and the variance of X is finite.

$$E(X) = E(E(X|Y)) \quad (5.7)$$

$$Var(X) = E[Var(X|Y)] + Var(E[X|Y]) \quad (5.8)$$

In probability theory, the expected value is a generalization of the weighted average which essentially represents the average outcome of a random variable, considering each possible value it can take, and weighting them by their respective probabilities. Using these laws and adapting them to the problem provides the expressions that will be used to disaggregate the fragility curves, specifically Eq.(5.9) and Eq.(5.10).

$$\ln \mu_c = \sum_{i=1}^n w_i \ln(\mu_i a_i) \quad (5.9)$$

$$\beta_c^2 = \sum_{i=1}^n w_i \beta_i^2 + \sum_{i=1}^n w_i [\ln \mu_c - \ln(\mu_i a_i)]^2 \quad (5.10)$$

Where the $\ln \mu_c$ and β_c^2 are, respectively the mean and the variance of the combination of all the single means, $\ln \mu_i$, and dispersions β_i . The values represented by w_i , are the weights that each specific distribution has depending to the plan-shape. The values of a_i are the correction factors regarding the plan-shape calculated using a MDOF model (plan-shape modifiers).

5.4.1. METHODOLOGIES TO DISAGGREGATE CLASS-LEVELS SEISMIC FRAGILITIES

Three methodologies were developed to disaggregate class-level seismic fragilities obtained using Incremental Dynamic Analysis (IDA) as the nonlinear time-history analysis, instead of Cloud Analysis (CA) or Multiple-Stripe Analysis (MSA). The flowchart in **Error! Reference source not found.** illustrates the procedure for disaggregating fragilities. In this figure, three approaches are presented using an analogy with balloons. To clarify, three balloons depicted in Figure 5.12 represent the disaggregated fragility curves for the three plan-shape class-typologies. The size of the balloons represents the variability, with Approach A exhibiting the greatest variability, followed by Approach C

and finally Approach B. The position of the ballons indicates the difference in median values among these three plan-shape fragility curves. Approach A results in three disaggregated curves that are relatively similar to each other, while Approach B yields highly disparate outcomes, and Approach C presents median values averaging between Approaches A and B.

5.4.1.1. Approach A

The schematic representation of Approach A, as depicted in Figure 5.12, requires the original fragility curve to be divided into three new curves, taking into account the effect of the plan-shapes. As previously noted, this approach yields three disaggregated fragility curves with greater variability compared to other approaches, yet the median values of these curves do not differ significantly from each other.

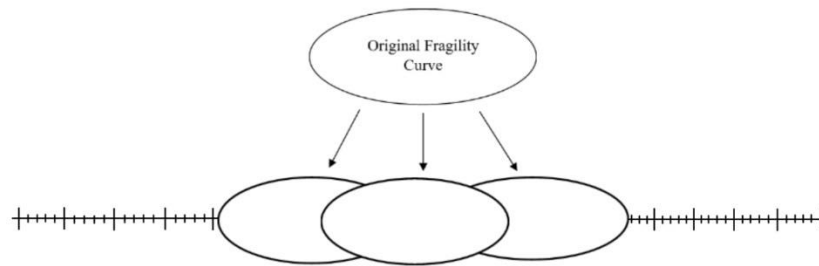


Figure 5.12. Approach A

To be as closer as possible with methodology followed by ESRM20, it was admitted an increment of 0.3 of variability, using the rule of square-root-sum-of-squares, to these original fragility curves obtained, getting the results represented in Table 5.3.

Table 5.3. Parameters of the fragility curves that will be disaggregated

Typology Class	IM	Damage State Level							
		DS1		DS2		DS3		DS4	
		μ	β	μ	β	μ	β	μ	β
MUR-STRUB_LWAL-DNO_H2	PGA	0.152	0.347	0.267	0.391	0.367	0.432	0.450	0.452
CR_LFINF-CDH-25_H2	$S_a(0.3s)$	1.546	0.391	2.282	0.489	2.403	0.557	2.456	0.556
CR_LFINF-CDM-15_H2	$S_a(0.6s)$	0.577	0.364	0.726	0.389	0.812	0.408	0.833	0.416
CR_LFINF-CDL-5_H2	$S_a(0.6s)$	0.410	0.427	0.525	0.466	0.633	0.511	0.720	0.535

In this approach only the plan-shape modifiers were employed to get a fragility curve described as “Combination”, in Figure 5.13, using the law of total expectation, Eq.(5.11), and the law of total variance, Eq.(5.12).

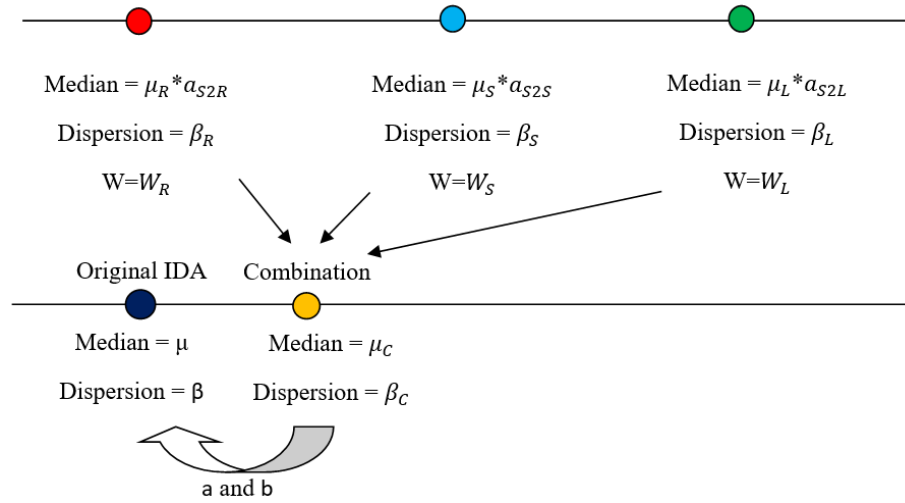


Figure 5.13. Methodology for disaggregation following Approach A

$$\mu_C = \exp \left[\sum_{i=1}^n w_i \ln(\mu_i a_i) \right] \quad (5.11)$$

$$= \exp[w_S \ln(\mu_S a_{S2S}) + w_L \ln(\mu_L a_{S2L}) + w_R \ln(\mu_R a_{S2R})]$$

$$\beta_C = \sqrt{\sum_{i=1}^n w_i \beta_i^2 + \sum_{i=1}^n w_i [\ln \mu_C - \ln(\mu_i a_i)]^2} \quad (5.12)$$

$$= \sqrt{(w_S \beta_S^2 + w_L \beta_L^2 + w_R \beta_R^2) + (w_S [\ln \mu_C - \ln(\mu_S a_{S2S})]^2 + w_L [\ln \mu_C - \ln(\mu_L a_{S2L})]^2 + w_R [\ln \mu_C - \ln(\mu_R a_{S2R})]^2)}$$

Where, a_{S2S} , a_{S2L} , and a_{S2R} are the plan-shape modifiers for square, L-shape, and rectangle, respectively. The μ_S , μ_L , μ_R , β_S , β_L , and β_R are the parameters of the fragility curves of the different plan-shape classes fragilities, which are the same at the beginning. This final result (Combination fragility curve) is, still, not equal to the fragilities previously described in Table 5.3. Therefore, it was necessary to use two correction factors, ‘a’ and ‘b’, to match the pretended fragility curve, as it was done in the work of Karaferis and Vamvatsikos [56]. The correction factor, ‘a’, is meant to adjust the median to get the same obtained in the original (Figure 5.14 (b)), and the correction factor, ‘b’, is an additional dispersion added to the dispersion obtained in the final disaggregated fragility curves using the square-root-sum-of-squares (Figure 5.14 (c)).

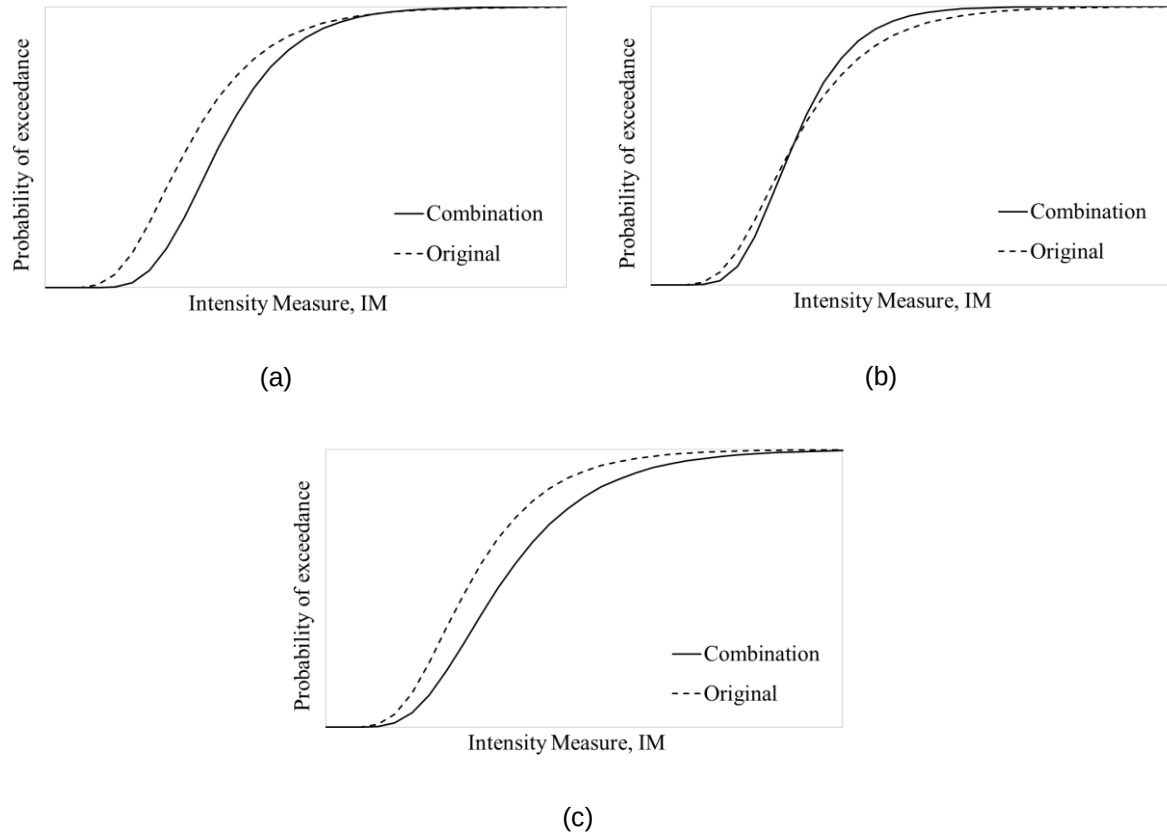


Figure 5.14. Application of the correction factors. (a) Set of fragility curves; (b) Application of correction factor, a, to match the median; (c) Application of correction factor, b, to match the dispersion

These correction factors described before, when applied to the disaggregated fragility curves, will ensure that in the end, when they are aggregated again using the law of total expectation and the law of total variance, they will result in the original curve. This final procedure is presented in Figure 5.15.

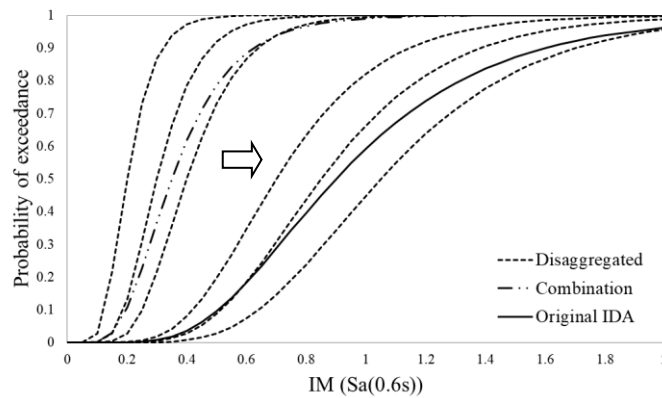


Figure 5.15. Application of correction factor in disaggregated fragility curves

Applying Approach A, these new disaggregated class-level seismic fragilities exhibit similar median values as illustrated in Figure 5.16.

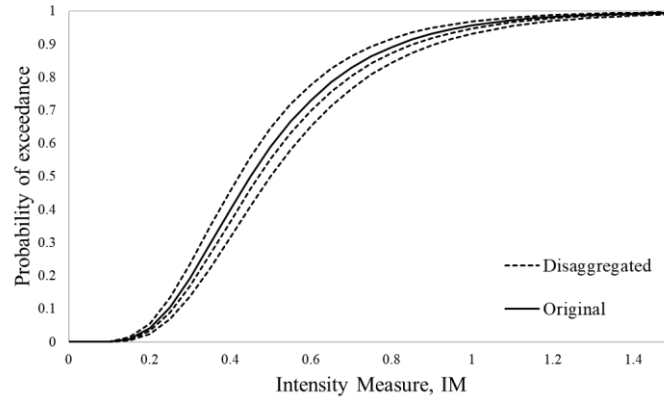


Figure 5.16. Disaggregated fragility curves following Approach A

5.4.1.2. Approach B

The approach depicted as Approach B in Figure 5.17 necessitates the original fragility curve to be subdivided into three new curves, taking into account the plan-shape modifiers. Within the same figure, this method yields three disaggregated fragility curves characterized by the least variability compared to other approaches, while the median values of these fragility curves exhibit the most significant disparities among the approaches.

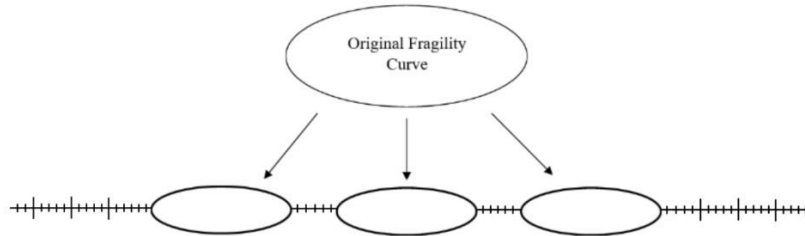


Figure 5.17. Approach B

The difference between this approach and the one described before is that this one has a correction value, C that will interfere with the rectangle and L-shape modifiers. The objective behind this value C is to allow the medians to diverge even further than the values obtained in Approach A, while simultaneously the fragility curve obtained through the law of total expectation and the law of total variance has the same dispersion as the one pretended to match, which in this case, are the ones described in Table 5.3. For that, if the plan-shape modifiers, a_{S2R} , or a_{S2L} were below value 1.0 the plan-shape modifier must be multiplied by the inverse of value C , but if the plan-shape modifiers were above 1.0 then the plan-shape modifier must be multiplied by the correction value C . These strategies, described in Figure 5.18, will permit in all the cases to maintain the median of square plan-shape seismic fragility in the same position and diverge even more the other two.

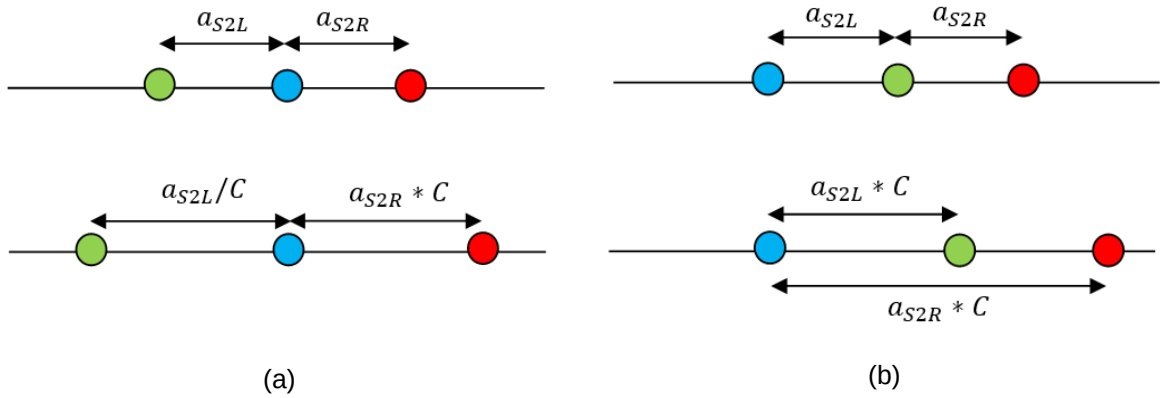


Figure 5.18. Impact of appliance factor C. (a) Case in which there is one plan-shape modifier below 1.0 and above 1.0; (b) Case in which there is both plan-shape modifiers above 1

Since the intended purpose of value C was to align with the target dispersion, the need for correction factor “b” was eliminated, resulting in only the utilization of correction factor “a” to adjust the median values. With all the C values detected for all the class-typologies and the final fragilities corrected by a correction factor “a” it was possible to calculate the disaggregated fragility curves. As it can be seen in Figure 5.19 the disaggregated fragility curves present median values that are way dispersed compared with the results obtained with Approach A in Figure 5.16.

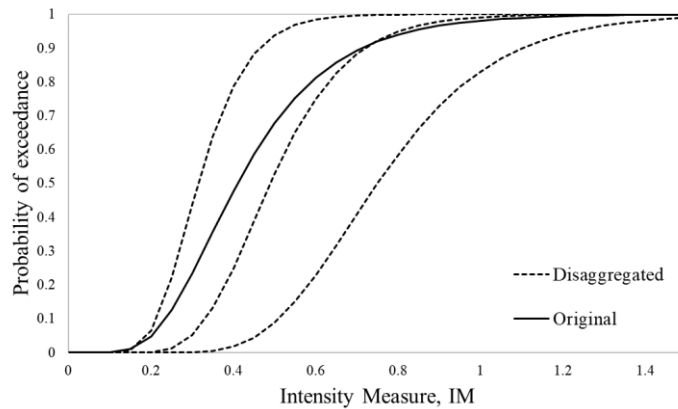


Figure 5.19. Disaggregated fragility curves following Approach B

5.4.1.3. Approach C

The final approach is Approach C depicted in Figure 5.20. Within the same figure, this method yields three disaggregated fragility curves characterized by an average variability and median values compared to other approaches. The starting values for the three plan-shape seismic fragilities, contrary to the other approaches where it was admitted the values presented in Table 5.2, this approach C admits for the SDOF which represents the square, rectangle, and L-shape an increment in variability of 0.2. using the rule of square-root-sum-of-squares, presented by Eq.(5.13). In this approach was admitted 0.2, because the variability of plan-shape of the buildings is now being consider. This value is approximately half of the increment added to the final fragility pretended to match (0.3, which is the same added in ESRM20).

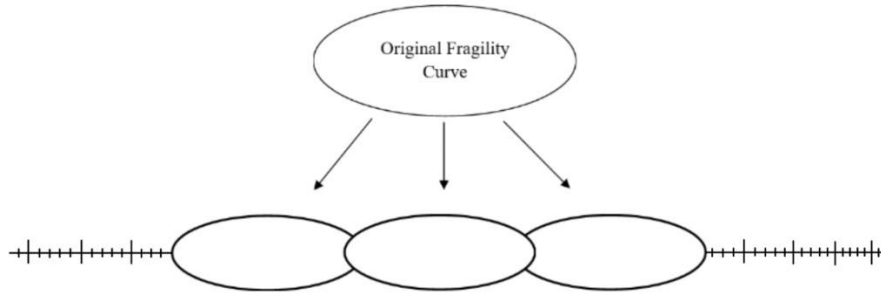


Figure 5.20. Approach C

$$\beta_{Total} = \sqrt{\beta_{R2R}^2 + 0.2^2} \quad (5.13)$$

The methodology underlying Approach C mirrors that of Approach B. Consequently, it introduces a factor, denoted as C, which is to be either multiplied or divided by the plan-shape modifier in the scenarios delineated in Figure 5.18. This is done with the aim of aligning the resultant dispersion from the combined fragility curve derived through the application of the law of total expectation (Eq.(5.9)) and the law of total variance (Eq.(5.10)) with the desired dispersion outlined in Table 5.3. The specific values of C and the correction factor “a”, which are subsequently applied to the three disaggregated fragility curves (Square, Rectangle, and L-shape) to achieve conformity with the target median. It is evident that Approach C yields to correction values, denoted as C, slightly smaller in comparison to those obtained from Approach B, resulting from the increment of additional dispersion, 0.2. Consequently, one would anticipate deriving disaggregated fragility curves from the same original curve with median values that are not as dispersed as those produced by Approach B, yet not as closely clustered as those achieved with Approach A. Additionally, there is a higher variability, attributed to the incremented value of 0.2 in dispersion. Figure 5.21 showcases the resulting disaggregated curves obtained through Approach C, juxtaposed with the original fragility curve.

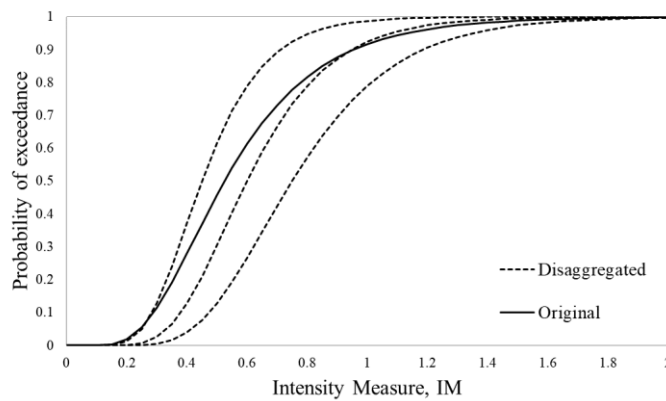


Figure 5.21. Disaggregated Fragility Curves following Approach C

5.4.2. CALCULATION OF THE WEIGHTS, W_i

Ideally, an exhaustive enumeration of every individual building in Greece, coupled with the determination of their respective plan-shapes, would represent the optimal scenario to calculate the weights. Acknowledging the challenges posed by attempting to survey every single building in Greece and determine its corresponding plan-shape, a different approach was chosen.

Nine regions of Greece were chosen, specifically Anatoloki Makedonia, Dytiki Makedonia, Kentriki Makedonia, Ipeiros, Dytiki Ellada, Sterea Ellada, Thessalia, Peloponnisos, and Attiki, illustrated in Figure 5.22.

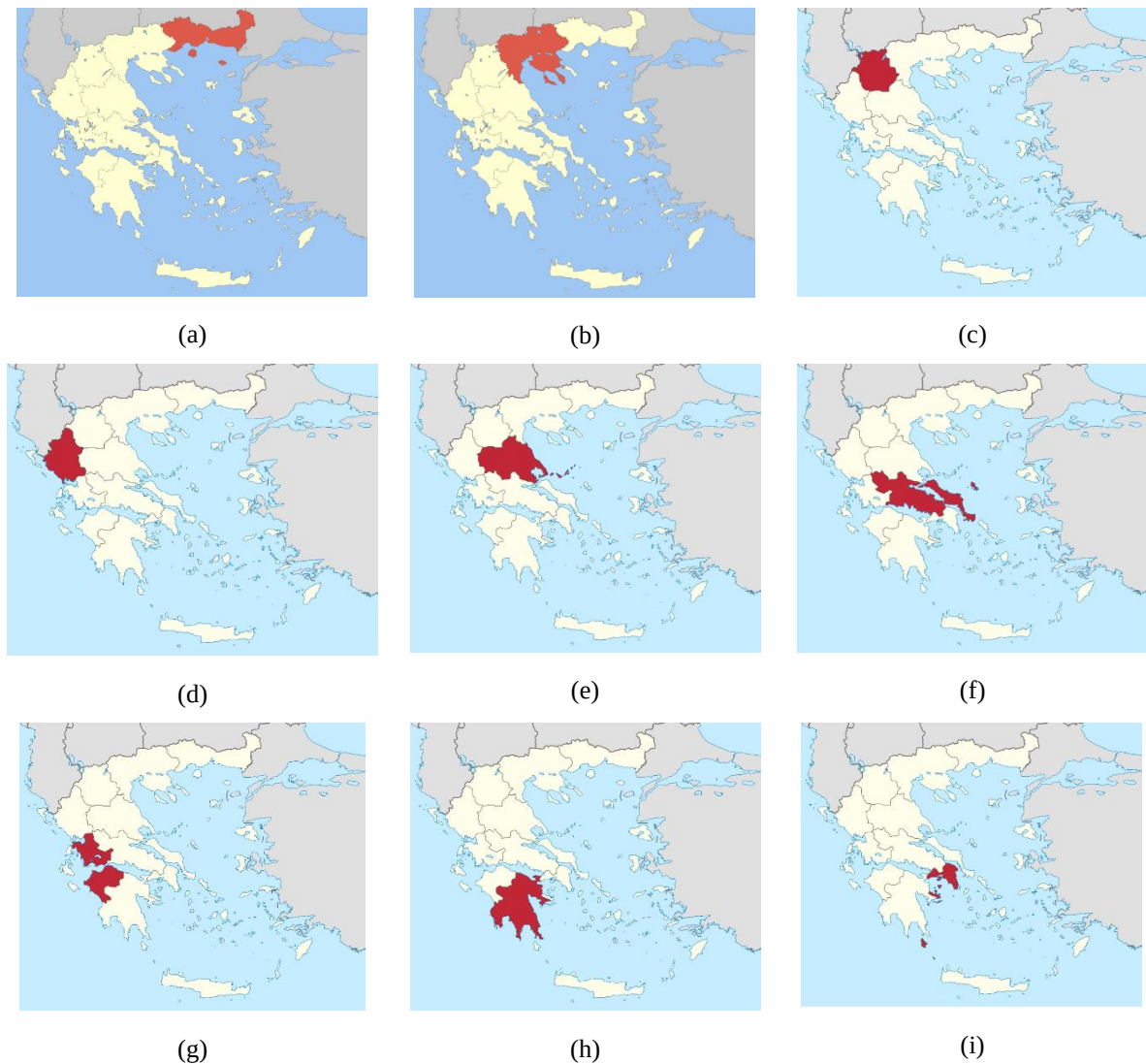


Figure 5.22. Regions in Greece: (a) Anatoloki Makedonia; (b) Kentriki Makedonia; (c) Dytiki Makedonia; (d) Ipeiros; (e) Thessalia ; (f) Sterea Ellada ; (g) Dytiki Ellada; (h) Peloponnisos; (i) Attiki (Exported from Wikipedia)

Following this, a random selection process was employed to designate three zones within each previously referenced area. These zones were then surveyed to determine the distribution of various building plan-shapes. Subsequently, the total number of buildings exhibiting specific plan-shapes was aggregated across all sampled zones. This data compilation process was repeated for a total of 27

randomly selected areas spanning those nine regions of Greece. The resulting dataset facilitated the calculation of the percentage of buildings with square, rectangle, L-shape, C-shape, and T-shape configurations. In Figure 5.23, an example from the Anatoloki Makedonia region illustrates the selection of three random zones for building count. While these zones were chosen randomly, efforts were made to avoid densely populated areas to ensure a representative sample for analyzing the distribution of different plan-shapes among single and two-story buildings.

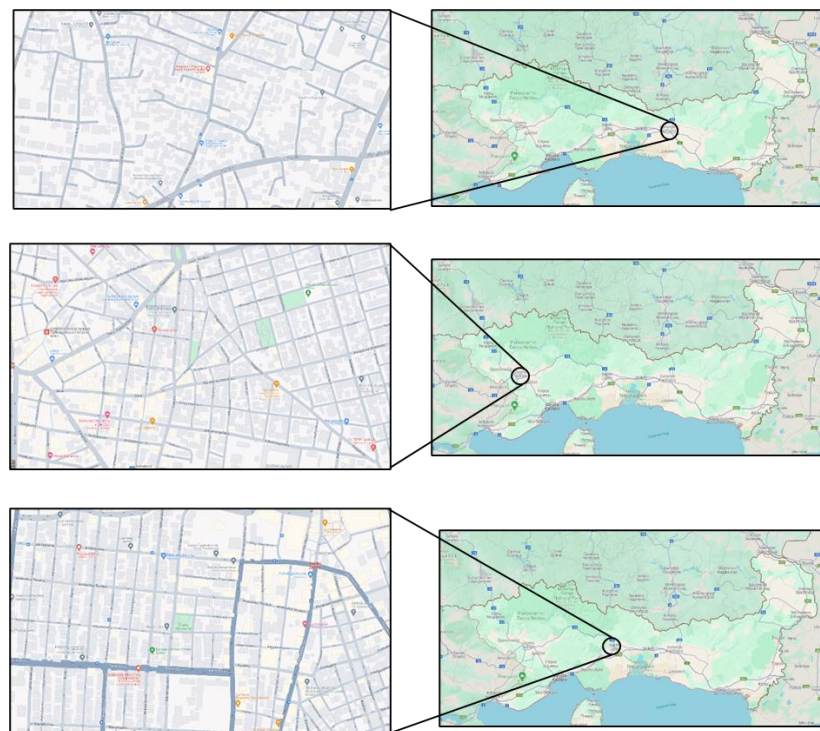


Figure 5.23. Zones chosen for one of the 9 regions evaluated, Anatoloki Makedonia

For each individual zone within the 9 regions of Greece beforementioned, three wide windows were delineated in Google Maps to encompass the areas under evaluation. Within these designated windows, buildings were visually highlighted using distinct colours, facilitating easy differentiation, and enabling the tallying of each specific plan-shape. The colour scheme corresponding to each plan-shape is detailed in Table 5.4, while an illustrative example showcasing a zone with buildings represented by different colour points is depicted in Figure 5.24.

Table 5.4. Colour for each plan-shape

Plan-shapes	Colors of points
Rectangle	Black
Square	Yellow
L-shape	Green
C-shape	Blue
T-shape	Pink

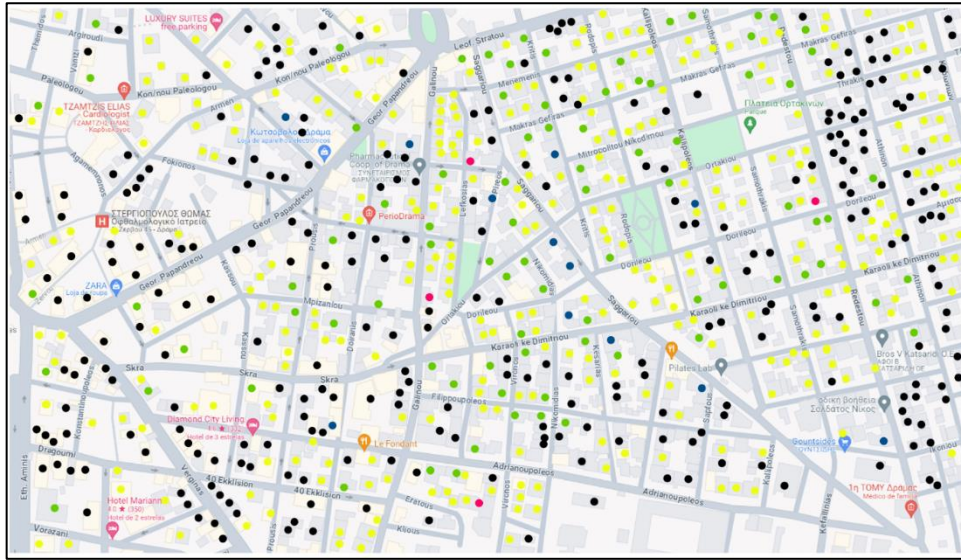


Figure 5.24. Color differentiation of plan-shapes in a zone in Anatoliki Makedonia

The plan-shape of the buildings needed to be clarified, because as can be seen in Figure 5.25, it is not clear what plan-shape those figures represent exactly. In Figure 5.25 (a) it is not clear if it is a Rectangle or an H-shape, in Figure 5.25 (b) if it is a square or an L-shape, or in Figure 5.25 (c) if it is a C-shape or a Square. Considering this, it became necessary to establish criteria to distinguish between those three examples and others. One method could involve referencing EN-1998-1 [57], specifically clause 4.2.3.2 (3), which states that buildings with setbacks wherein the area between the floor outline and a convex polygonal line encompassing the floor does not exceed 5% of the floor area can be deemed regular in plan. While this approach may be effective for analyzing a small number of buildings, it would prove time-consuming for a larger dataset. Therefore, an alternative approach was adopted, wherein the subjective opinion of the author was relied upon for a swift and simplified assessment of the buildings' plan-shapes.

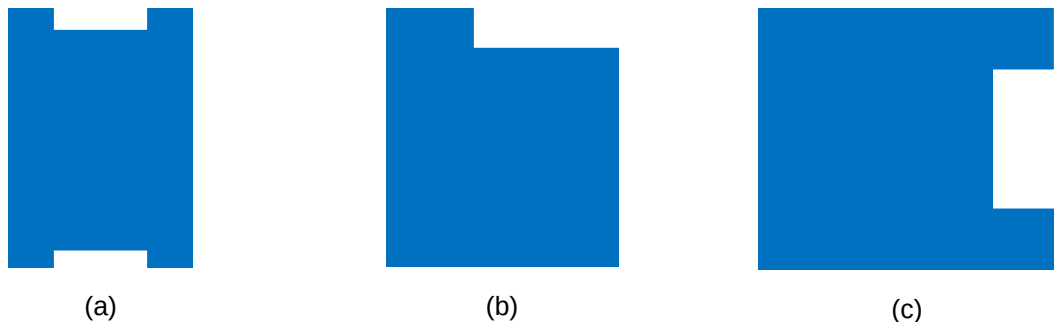


Figure 5.25. Different plan-shapes of buildings in Greece

Following an analysis of the plan-shapes of buildings across the 27 zones within the 9 continental regions of Greece, the final percentages of buildings with square, rectangle, L, C, T, O, and H shapes could be determined. Table 5.5 presents the percentages for each of these aforementioned plan-shapes.

Table 5.5. Weights of plan-shapes

Plan-shape	N° of Buildings	%
Square	2594	40.63
Rectangle	3115	48.79
L-shape	626	9.80
C-shape	29	0.45
T-shape	20	0.31
O-shape	-	0.00
H-shape	1	0.02
Total	6385	100

Despite the fact of not being as precise, this alternative method that was used to calculate the weights proved to be considered a viable and worthwhile strategy for addressing this task at hand.

5.4.3. CALCULATE THE SHAPE MODIFIERS, A_i

The major goal was to disaggregate the fragility curves into new ones that take into consideration the plane-shape of the buildings. The single-degree-of-freedom (SDOF) used to model the class-typologies do not consider this factor, so a multi-degree-of-freedom (MDOF) model developed by Vamvatsikos in 2008 was used to evaluate the effect of changing this parameter. Figure 5.26 illustrates the different plan-shapes that was possible to modify in the MDOF model.

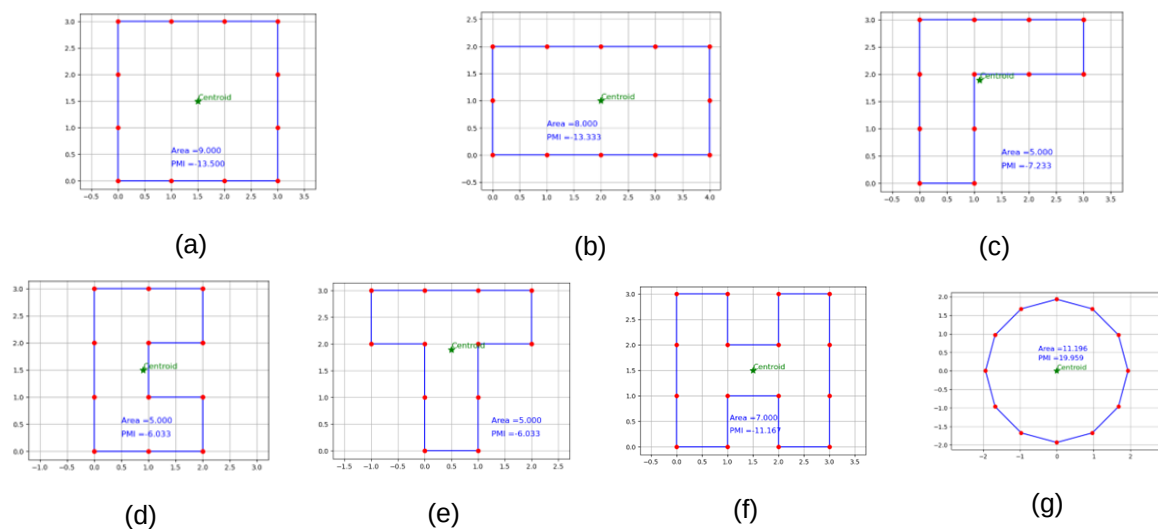


Figure 5.26. Plan-shapes evaluated by the MDOF models

Because the percentages (w_i) of buildings with C, T, H, and O shapes were so much lower compared to the other plan-shapes, the disaggregation process of the fragility curves focused on the other three (square, rectangle, and L-shape).

The plan-shape modifiers were calculated as the ratio between the median values of the fragility curve of the reference shape (Square) and the other shapes. The square was chosen as the reference shape because the ground motions applied to obtain the capacity curves in ESRM20 (e.g., CR_LFINF) were uniformly distributed in both directions, resulting in a median capacity curve. This approach enabled the assumption of a square shape for SDOFs without differentiation between y and x directions. Eq.(5.14) to Eq.(5.16) outline the formulas used to compute the plan-shape modifiers (a_i) for the various shapes analysed.

$$a_{S2S} = \frac{\mu_{square}}{\mu_{square}} \quad (5.14)$$

$$a_{S2R} = \frac{\mu_{rectangle}}{\mu_{square}} \quad (5.15)$$

$$a_{S2L} = \frac{\mu_{L-shape}}{\mu_{square}} \quad (5.16)$$

For the MDOF models, the capacity curves differed significantly from those obtained for the SDOFs shown in Figure 5.7. Interstorey drift ratio was assumed as the Engineering Demand Parameter because compared to roof drift can account better for local failure mechanisms and not for the average behaviour of the structure. That difference is evident from the pushover curves depicting interstorey drift for the three distinct plan-shapes (square, rectangle, and L) as shown in Figure 5.27 for both directions. For the square and L plan-shaped, the capacity curves were identical in both directions due to their symmetrical design. However, the rectangular plan showed a marked difference in capacity, with a notable decrease in the y-direction.

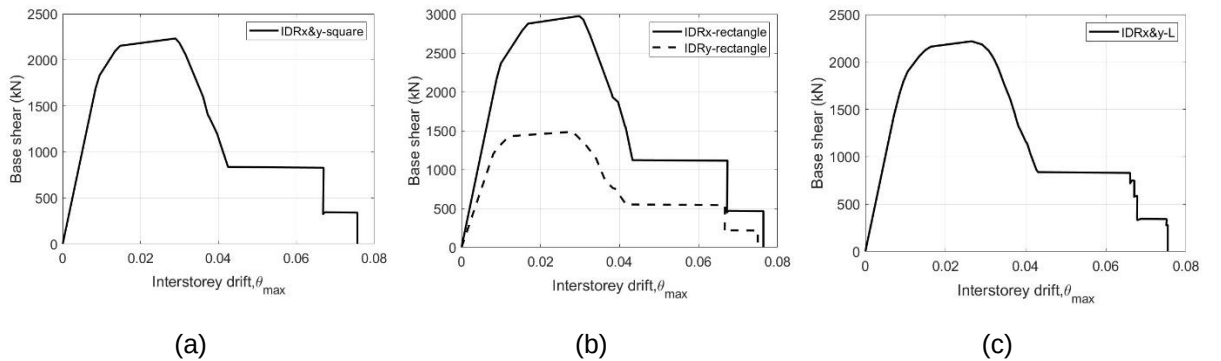


Figure 5.27. Pushover curves of the MDOF models. (a) Square-shape; (b) Rectangle-shape; (d) L-shape

The median values from the fragility curves were obtaining using the same procedure described previously to obtain the SDOF results. The values of the Damage States (DS) followed the same approach used in ESRM20 represented by Eq.(5.17).

$$Damage\ State = \begin{cases} 0.75S_{dy} & \text{for DS1} \\ 0.50S_{dy} + 0.33S_{du} & \text{for DS2} \\ 0.25S_{dy} + 0.67S_{du} & \text{for DS3} \\ S_{du} & \text{for DS4} \end{cases} \quad (5.17)$$

Due to the difference of these pushover curves, it was necessary to decide where the yielding and ultimate displacements were. In this case, S_{dy} denotes the yielding point (i.e., the point where the initial line intersects the line on the top plateau) depicted in Figure 5.28 with the number 1. Conversely, S_{du} representing the ultimate displacement, is identified as the final point along the lower plateau just after the shear force begins to decline, represented with the number 2. For simplification, and because of the similarity of the MDOF pushover yielding (1%) and ultimate (6.5%) points it was admitted the same values for the damage states, denoted in Table 5.6.

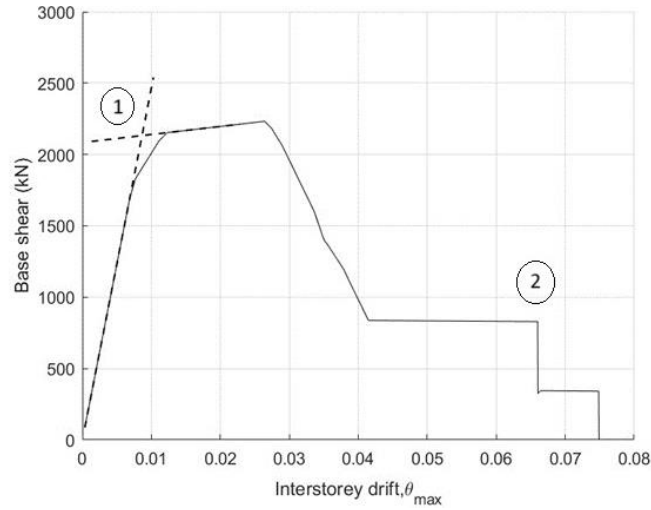


Figure 5.28. Yielding and ultimate displacement in the Capacity Curve of the MDOFs

Table 5.6. Damage state values for the MDOF models

Damage State levels	Interstorey drift
DS1	0.8%
DS2	2.6%
DS3	4.6%
DS4	6.5%

The MDOF models represented by different plan-shapes (square, rectangle, and L-shape) were developed by Vamvatsikos in 2008 to present the same mass, stiffness, and consequently different periods. Those periods are different from the SDOFs that represent each class-typology that are pretended to be disaggregated. Using Eq.(5.14), Eq.(5.15), and Eq.(5.16) to calculate the plan-shape

modifiers a fair judgement has been made to compare equivalent buildings that have different plan-shapes. The plan-shape modifiers, in this study, were only applied just to the class-typologies CR_LFINF-CDL-5_H2 and CR_LFINF-CDM-15_H2, because are the ones from the surveyed class-typologies that present similar periods and characteristics from the default ones of the MDOF models (Table 5.8). To make a fair comparison of buildings with three different plan-shapes, represented by these MDOF models, it was essential to evaluate them in terms of equivalent characteristics, which could be, for example, the period of vibration or the mass. For this purpose, three different approaches were followed to characterize the MDOF models, ensuring a fairer assessment for applying plan-shape modifiers to the SDOFs in the disaggregation process. These approaches are:

- Case 1: MDOFs with the same periods, different masses, same stiffnesses;
- Case 2: MDOFs with different periods, same masses, same stiffnesses;
- Case 3: MDOFs with different periods but the square plan-shaped MDOF matched the SDOF period, same masses, same stiffnesses.

5.4.3.1. Plan-shape modifiers given by Case 1

In this case, it was assumed that to get equivalent MDOF models, all of them were adapted to have the same first period of vibration used in the SDOF models that represent each class typology evaluated on the disaggregation process. The default values of the models presented different periods for the same mass, so to get the same periods of the SDOFs, the respective masses of the different plan-shapes MDOFs needed to be reduced, leading in the end to different masses. The fragility curves were developed with IDA analysis, and using the same IM that was used to describe the SDOF with the correspondent period (see Table 5.2). Analysing the results of the plan-shape modifiers obtained using MDOF models adapted to match the T_1 of the SDOFs representing the class typologies CR_LFINF-CDL-5_H2 and CR_LFINF-CDM-15_H2 denoted in Table 5.7, it is noticeable, that the rectangle-shape exhibits the best behaviour compared to the other plan-shapes.

Table 5.7. Plan-Shape modifiers of MDOFs using Case 1

T[s]	Plan-shape modifier	Damage State Levels			
		DS1	DS2	DS3	DS4
0.53	a_{S2S}	1.000	1.000	1.000	1.000
	a_{S2R}	1.132	1.126	1.114	1.089
	a_{S2L}	0.902	0.837	0.916	0.910
0.50	a_{S2S}	1.000	1.000	1.000	1.000
	a_{S2R}	1.150	1.122	1.106	1.077
	a_{S2L}	0.910	0.836	0.903	0.965

As expected, the L-shaped MDOF exhibits the poorest performance among all the plan-shapes. However, due to the reduction of mass to get the same first fundamental period of vibration, the rectangle plan-shaped MDOF was the most affected (i.e., the rectangle plan-shaped MDOF suffer the highest reduction of mass). Targeting the first period of vibration of the SDOFs representing the class-typologies

resulted, for all plan-shaped MDOFs, in a significant reduction of the second period of vibration for the rectangular plan-shape. This reduction is substantially greater compared to the other plan-shaped MDOFs, leading to a stiffer behaviour of the rectangular structure. Figure 5.29 illustrates the difference in the maximum interstorey drift between the square and rectangular shapes.

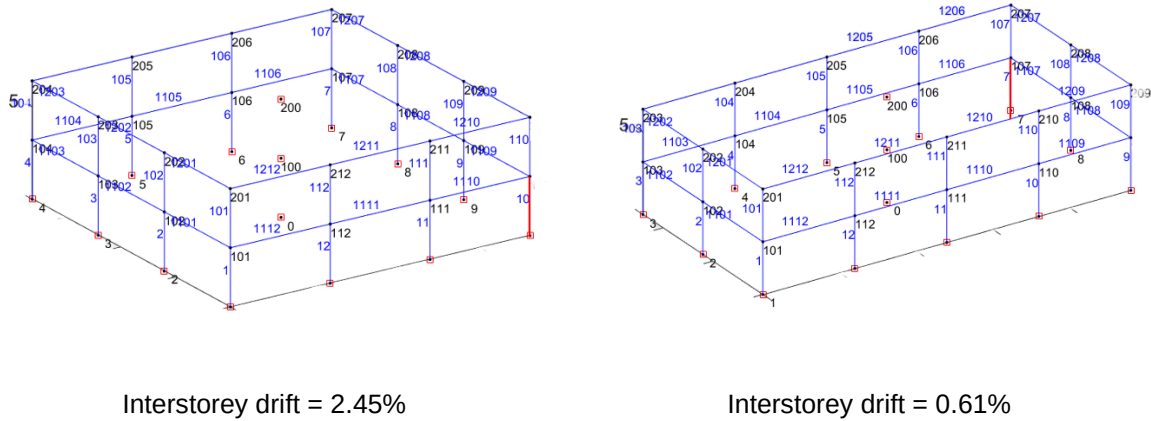


Figure 5.29. Interstorey drift comparison between square and rectangle plane-shapes using Case 1

5.4.3.2. Plan-shape modifiers given by Case 2

In this case it was assumed that to have equivalent MDOFs with different plan-shapes, the factor that dictates the equivalence was the mass. Therefore, for the buildings to have the same mass, and the same stiffness, because of the plan-shape, the periods of each one will differ. The periods of the MDOF models for this Case 2, were the default ones, and are presented in Table 5.8.

Table 5.8. The first two periods of the MDOF models following Case 2

Plan-Shape	Periods	
	T_1 (s)	T_2 (s)
Square	0.64	0.64
Rectangle	0.75	0.57
L-shape	0.65	0.63

Assuming the same mass as the factor that dictates the equivalence of the buildings it is seen a change in the behaviour of the plan-shaped MDOFs. On contrary of what was the results of the last case, the plan-shape modifiers are applied to all the class-typologies regardless the period of the SDOFs. As illustrated in the Table 5.9, the L-shape is the one that has the worst behaviour, presenting the highest interstorey drifts for the same Intensity Measure as seen in Figure 5.30.

Table 5.9. Plan-Shape modifiers of MDOFs using Case 2

Plan-shape modifier	Damage State Levels			
	DS1	DS2	DS3	DS4
a_{S2S}	1.000	1.000	1.000	1.000
a_{S2R}	0.947	0.993	0.907	0.901
a_{S2L}	0.866	0.831	0.859	0.935

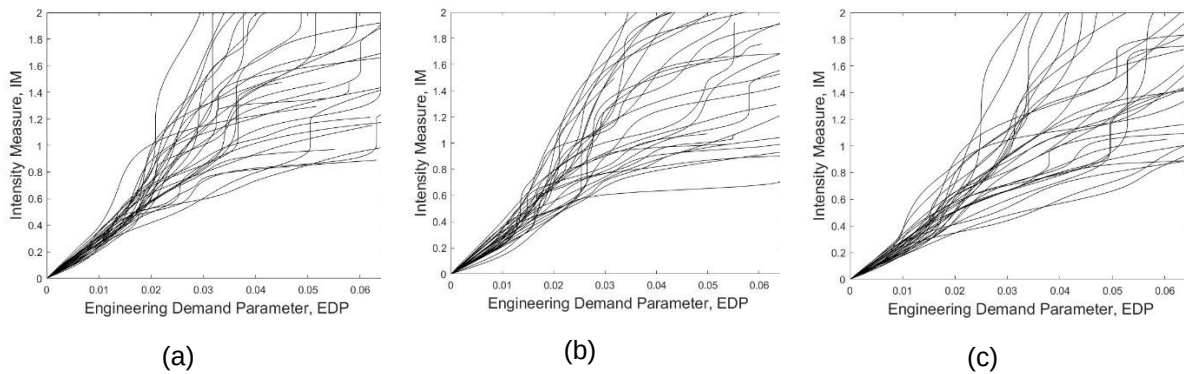


Figure 5.30. IDA results for MDOFs using Case 2. (a) Square plan-shape; (b) Rectangle-shape; (c) L-shape

5.4.3.3. Plan-shape modifiers given by Case 3

In Case 3, the goal was to equalize the mass for all plan-shaped MDOF models while also matching the fundamental period (T_1) of the square-shaped MDOF to that of the SDOFs representing each class-typology. The T_1 of the square shaped MDOF was matched because it serves as the reference shape, as explained in Subchapter 5.4.3. Although the other plan-shaped MDOFs have the same mass as the square-shaped model, their periods differ. Essentially, this approach is similar to Case 2, but with the mass of the square shaped MDOF aligned to the period of the corresponding SDOF for each class-typology. The EDP (interstorey drift) was calculated using the same previous approach (i.e., using the SRSS rule) and due to the similarity with Case 2, the sequence of the plan-shaped remains L-shape – Rectangle – Square, as evidenced by the plan-shape modifiers in Table 5.10.

Table 5.10. Plan-Shape modifiers of MDOFs using Case 3

T[s]	Plan-shape modifier	Damage State Levels			
		DS1	DS2	DS3	DS4
0.53	a_{S2S}	1.000	1.000	1.000	1.000
	a_{S2R}	0.935	0.984	0.980	0.943
	a_{S2L}	0.891	0.821	0.904	0.889
0.50	a_{S22}	1.000	1.000	1.000	1.000
	a_{S2R}	0.917	0.984	0.944	0.951
	a_{S2L}	0.909	0.815	0.881	0.915

5.4.3.7. Remarks of plan-shape modifiers

Considering all the previously presented cases, it became clear that assuming the period of vibration or the mass as the factors representing the equivalence, between the MDOFs, affects the performance sequence. Using the SRSS rule for the critical column for the three presented cases, allowed to conclude that the L-shape performs worse compared to the other plan-shapes. Figure 5.31 highlights, in red, the columns with the highest interstorey drift in the x and y direction of the L-shape.

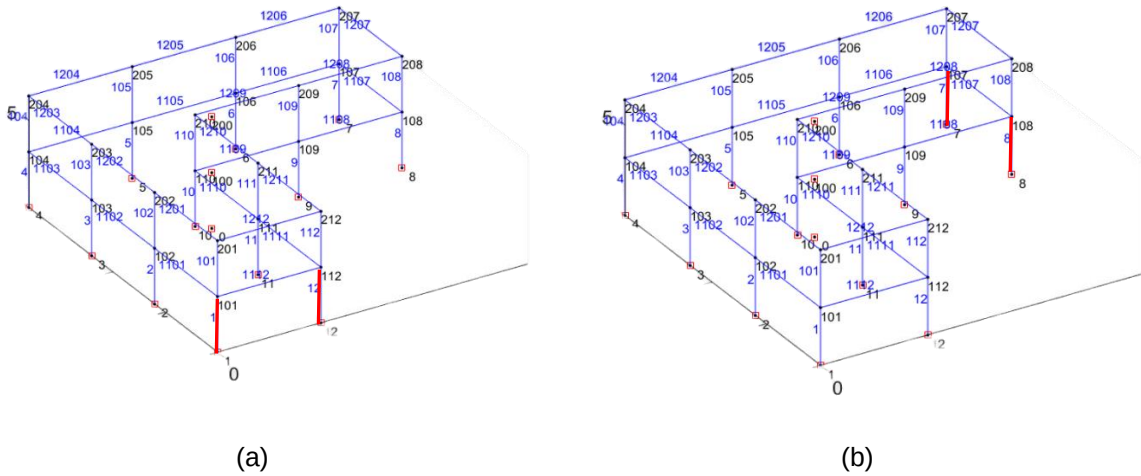


Figure 5.31. Columns with the highest drifts in L-shape. (a) x direction; (b) y direction

Therefore, the author believes that the best methodology to apply the plan-shape modifiers would be to follow something similar to case 3, where the equivalence factor is the mass, ensuring all plan-shaped MDOF models have the same mass and attempting to approximate the fundamental period of the square shaped MDOF to the respective value of the SDOF. Additionally, it is important to highlight the different issues that arose when using different plan-shape modifiers per Damage State (DS). One issue relies on the variation of the median values of the rectangle and L-shape disaggregated curves as the damage states increase. That difference can be seen, for example, in the Case 3 in Figure 5.32, where for the class-typology CR_LFINF-CDL-5_H2, the difference for the DS1, DS2, DS3, and DS4 are respectively 5%, 20%, 8% and 6%.

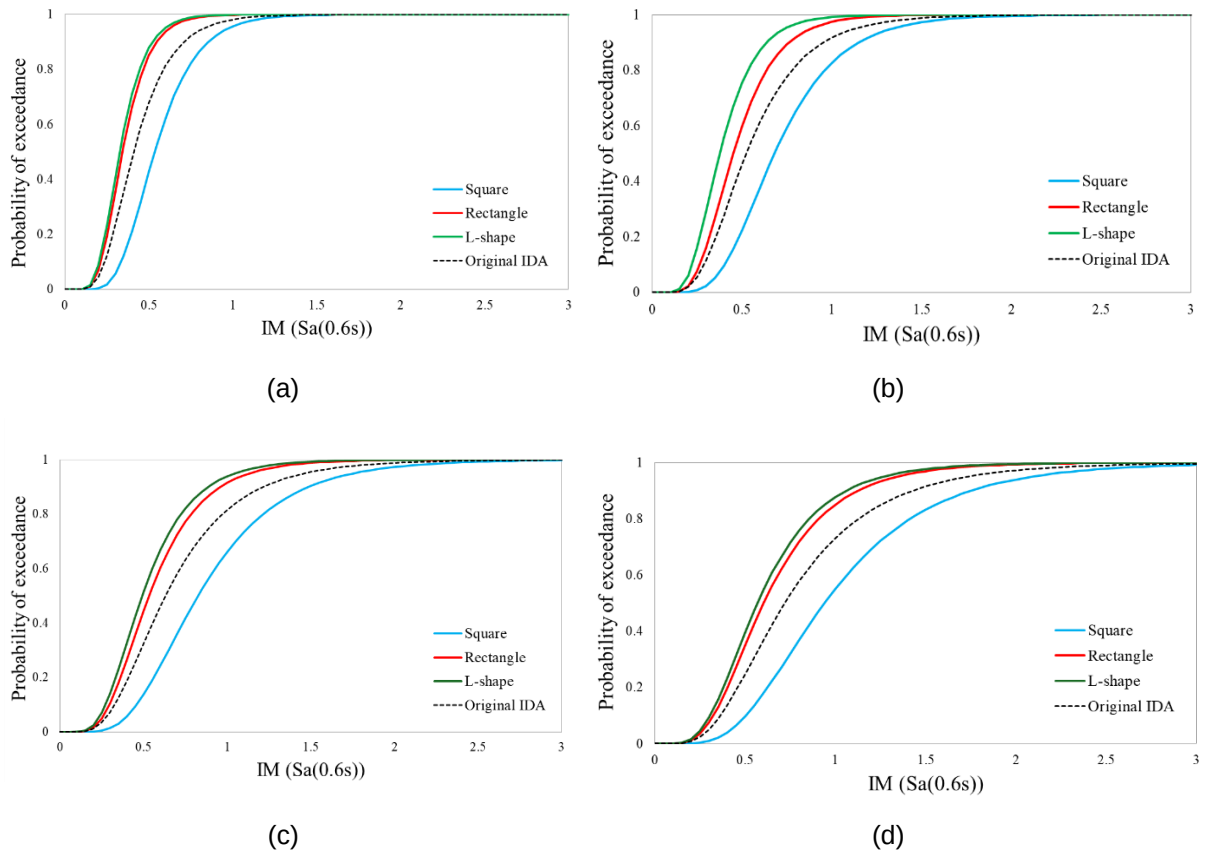


Figure 5.32. Effect of using different plan-shape modifiers per damage state for the class-typology CR_LFINF-CDL-5_H2 using Case 3. (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

In case 2, with the class-typology CR_LFINF-CDM-15_H2, another issue is noticeable regarding the use of different plan-shape modifiers per DS. Using Approach C, with the correction factors C and “a”, the median value for the rectangular disaggregated curve for DS3 is larger compared to that for DS4, which poses a problem. This issue arises from the proximity of the median values of the SDOFs for both damage states and the difference in the values of C and “a”, which affect the final median values representing the disaggregated fragility curves. Table 5.11 shows the values of the steps used in Approach C for the plan-shapes in case 2 for DS3 and DS4.

Table 5.11. The problem of using different plan-shape modifiers in the class-typology CR_LFINF-CDM-15_H2 using Case 2

Damage State	Median of the SDOFs, μ_R	Corrected Median, $\mu_R * a_{S2R}/C$	Final Median
DS3	0.812	$\frac{0.812 \cdot 0.907}{1.414} = 0.521$	0.681
DS4	0.833	$\frac{0.833 \cdot 0.901}{1.441} = 0.521$	0.678

Due to the inconsistency of the values of the plan-shape modifiers, and following the approach presented by case 3, it was used only plan-shape modifiers that take into account the elastic behaviour of the structure marked in red in Figure 5.33. For this, a DS of 1% is suggested, which can be used for comparison of the different plan-shaped MDOF models.

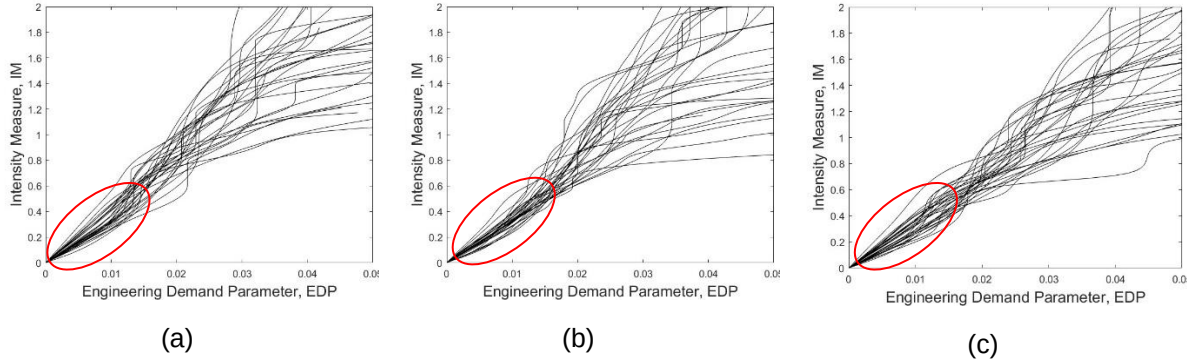


Figure 5.33. Elastic behaviour detected using IDA. (a) Square plan-shape; (b) Rectangle-shape; (c) L-shape

Adopting this solution of using one plan-shape modifier per DS made it possible to compare the plan-shaped MDOF models in a way that mitigates the errors previously discussed in the evaluated cases. The plan-shape modifiers calculated for the class-typologies CR_LFINF-CDL-5_H2 and CR_LFINF-CDM-15_H2 are presented in Table 5.12, with the period of vibration 0.53s and 0.50s, respectively.

Table 5.12. Final plan-shape modifiers

T[s]	Plan-shape modifier	Damage State
		DS (1%)
0.53	a_{S2S}	1.000
	a_{S2R}	0.944
	a_{S2L}	0.886
0.50	a_{S2S}	1.000
	a_{S2R}	0.913
	a_{S2L}	0.862

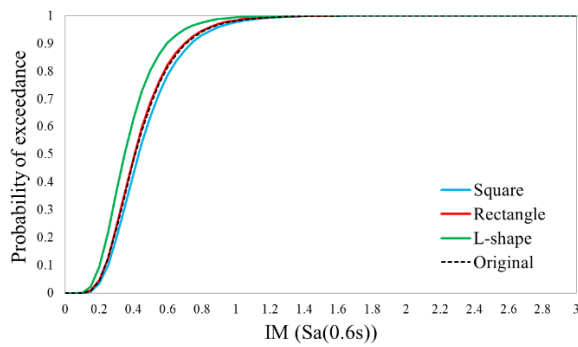
5.5. APPLICATION OF THE DISAGGREGATION PROCEDURE

To demonstrate the applicability of this method, it was chosen one class-typology from the ones discussed about the applicability of plan-shape modifiers. Using the plan-shape modifiers discussed in the previous subchapter 5.4.3.7 and applying the Approach A, B, and C, it was possible to disaggregate the fragility curve that characterized the class-typology CR_LFINF-CDL-5_H2.

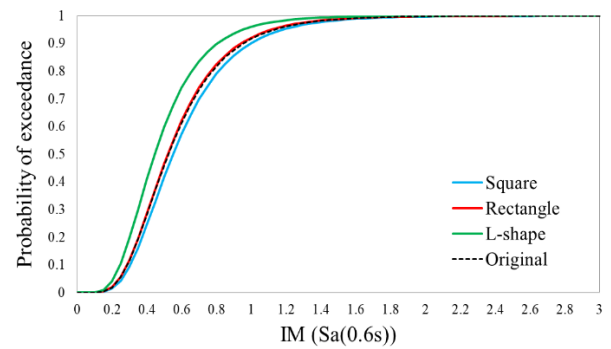
Applying Approach A, the correction factors resulted were 1.042 for the value “a” and 0.297 for the value “b”. The fragility curves were, as expected, really close to each other’s, as it can be seen by the parameters illustrated in Table 5.13 and graphically by Figure 5.34.

Table 5.13. Parameters of the class-typology CR_LFINF-CDL-5_H2 according with Approach A

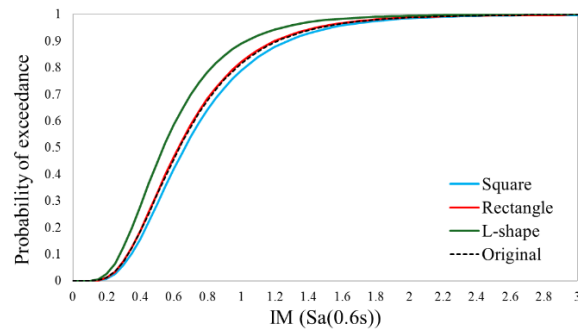
Typology Class	Damage State Level	Original		Disaggregated					
				Square		Rectangle		L-shape	
		μ	β	μ	β	μ	β	μ	β
CR_LFINF-CDL-5_H2	DS1	0.410	0.427	0.427	0.425	0.403	0.425	0.374	0.425
	DS2	0.525	0.466	0.547	0.464	0.516	0.464	0.479	0.464
	DS3	0.633	0.511	0.659	0.509	0.622	0.509	0.577	0.509
	DS4	0.720	0.535	0.750	0.533	0.708	0.533	0.656	0.533



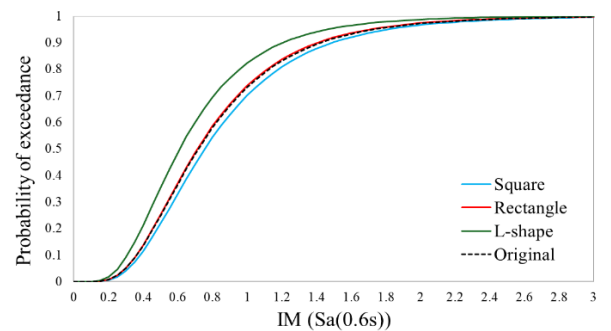
(a)



(b)



(c)



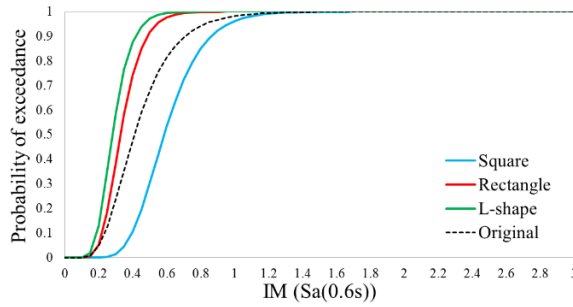
(d)

Figure 5.34. Disaggregated Fragility Curves for Class-Typology CR_LFINF-CDL-5_H2 following Approach A; (a) Damage state level 1; (b) Damage state level 2; (c) Damage state level 3; (d) Damage state level 4

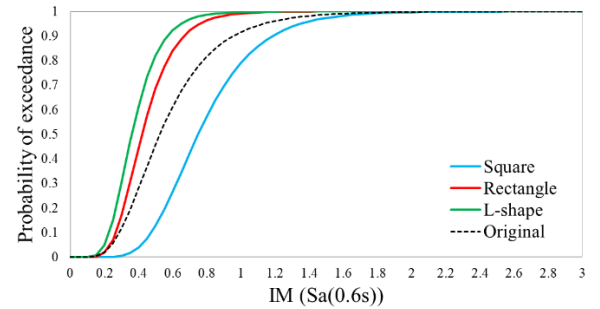
With Approach B the disaggregated fragility curves were way more spread away compared with the ones presented in Figure 5.34 using Approach A. Due to the fact that the rectangle has lower median values and the L-shape higher compared with the square plan-shape, made them to spread away with the factor of 1.713 times (i.e. the value C is 1.713). As it can be seen in the Table 5.14 the dispersion values decrease significantly, and the median values got even more spread away from the original fragility curve. This can be seen clearly in Figure 5.35.

Table 5.14. Parameters of the class-typology CR_LFINF-CDL-5_H2 according with Approach B

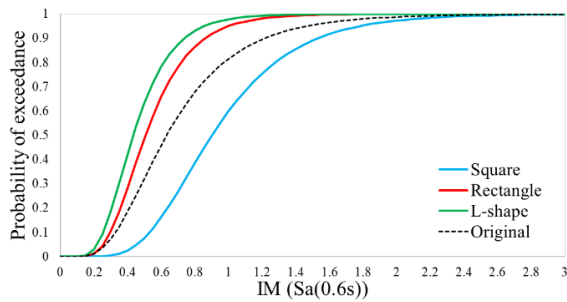
Typology Class	Damage State Level	Original		Disaggregated					
				Square		Rectangle		L-shape	
		μ	β	μ	β	μ	β	μ	β
CR_LFINF-CDL-5_H2	DS1	0.410	0.427	0.587	0.304	0.323	0.304	0.300	0.304
	DS2	0.525	0.466	0.751	0.357	0.414	0.357	0.384	0.357
	DS3	0.633	0.511	0.906	0.413	0.499	0.413	0.463	0.413
	DS4	0.720	0.535	1.030	0.442	0.568	0.442	0.526	0.442



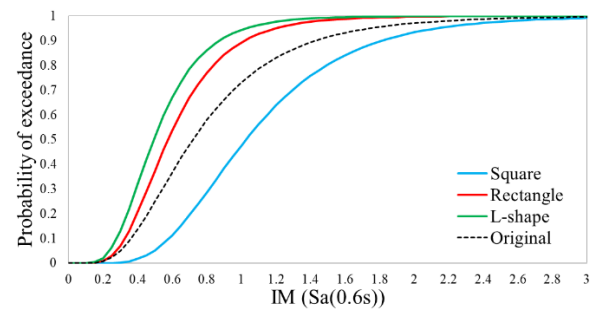
(a)



(b)



(c)



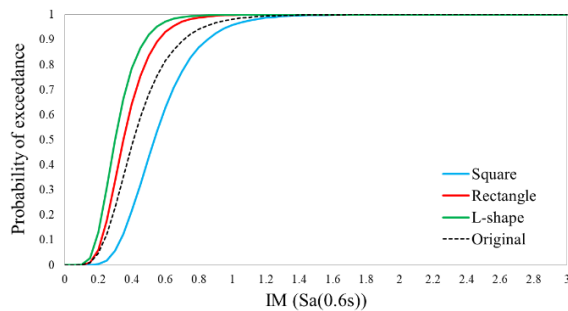
(d)

Figure 5.35. Disaggregated Fragility Curves for Class-Typology CR_LFINF-CDL-5_H2 following Approach B; (a) Damage state level 1; (b) Damage state level 2; (c) Damage state level 3; (d) Damage state level 4

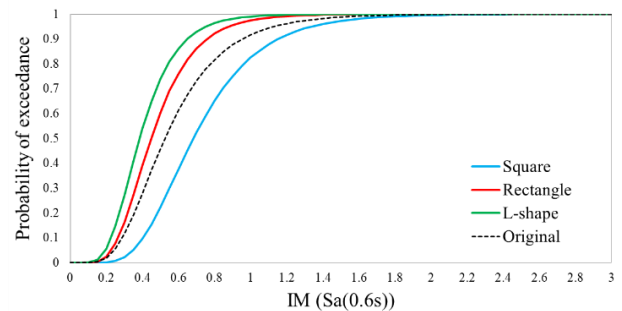
Finally, applying Approach C, the results changed. As it can be seen in Table 5.15 the dispersion values are higher compared to the last ones described with the previous approach, but there are the ones that consider other effects that not only the different plan-shapes of the buildings. Figure 5.36, shows that taking into consideration an increment of the variability for each plan-shaped fragility curve with a value of 0.2, brought the disaggregated fragility curves closer to the one pretended to match compared with Approach B, but not close as Approach A.

Table 5.15. Parameters of the class-typology CR_LFINF-CDL-5_H2 according with Approach C

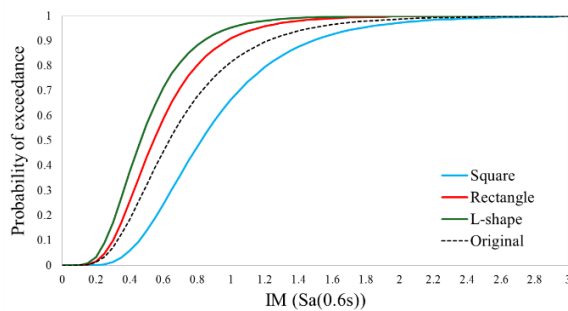
Typology Class	Damage State Level	Original		Disaggregated					
				Square		Rectangle		L-shape	
		μ	β	μ	β	μ	β	μ	B
CR_LFINF-CDL-5_H2	DS1	0.410	0.427	0.535	0.364	0.345	0.364	0.320	0.364
	DS2	0.525	0.466	0.685	0.409	0.442	0.409	0.409	0.409
	DS3	0.633	0.511	0.826	0.459	0.532	0.459	0.493	0.459
	DS4	0.720	0.535	0.940	0.486	0.605	0.486	0.561	0.486



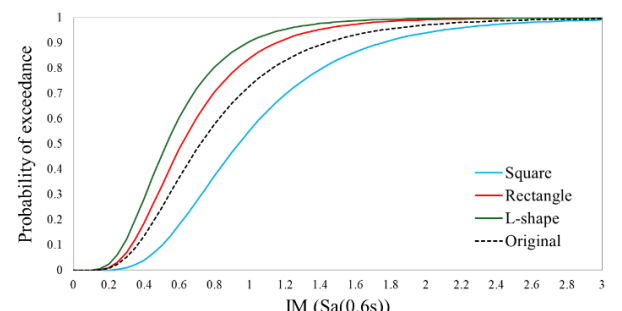
(a)



(b)



(c)



(d)

Figure 5.36. Disaggregated Fragility Curves for Class-Typology CR_LFINF-CDL-5_H2 following Approach C; (a) Damage state level 1; (b) Damage state level 2; (c) Damage state level 3; (d) Damage state level 4

6

Vulnerability Assessment

6.1. DAMAGE-TO-LOSS MODELS

Consequence Models also known as Damage-to-loss models, are used to convert the fragility functions, which describe the probability of exceeding or reaching a set of damage states, given a certain level of ground motion, to vulnerability functions, that provide the distribution of loss ratios conditional on a level of ground motion. These models estimate the average ratio of repair cost (DR) to replacement cost for each damage state, as outlined in Eq.(6.1).

$$DR = \frac{\text{Repair costs of a building}}{\text{Replacement cost of a building}} \quad (6.1)$$

The damage ratios (DR) utilized in ESRM20 framework to develop the vulnerability curves are shown in Table 6.1. were based on damage-loss models developed for different countries in Europe, such as Cosenza et al. [58], De Martino et al. [59], and Di Ludovico et al. [60] for Italy, Tyangunov et al. [61] for Germany, Akkar [60] for Turkey and Erdik et al. [60] from an insurance perspective.

Table 6.1. Damage ratios used in ESRM20

Damage State	Damage Ratio
Slight Damage (DS1)	5%
Moderate Damage (DS2)	15%
Extensive Damage (DS3)	60%
Complete Damage (DS4)	100%

Since the damage ratios used in ESRM20 were meant for all European buildings, they might not be the best fit for specific building types in Greece, evaluated in this thesis. This helps not only to estimate losses more accurately but also figure out which strategies to reduce vulnerability are worth investing in. To determine appropriate damage factor for Greek reinforced concrete buildings, data on repair costs were compiled, drawing primarily from observations following the 1999 Athens earthquake (Karabinis and Baltzopoulou [62], Kappos et al. [63], and Vlachos and Vlachos [64]). Subsequently, in 2014, a

study by Pomonis et al. [65], collected findings from these sources and adjusted them to align with the damage scale proposed by the Earthquake Planning and Protection Organization of Greece (EPPO), as detailed in Table 6.2.

Table 6.2. Adaptation of EPPO's damage scale for RC buildings [65]

Damage Scale	Damage Description
White (D0)	No damage.
Green (D1)	Fine cracks to the infill walls and ceiling mortar.
	Hairline cracks in horizontal RC structural members.
Yellow (D2)	Large patches of mortar falling off walls and ceilings.
	Cracks in structural RC members but to an extent that does not constitute danger of collapse. Slight distortion of structural elements.
Red (D3)	Heavy damage and distortion of structural elements. Large number of crushed structural elements and connections.
	Considerable dislocation of a storey and of the whole building.
Black (D4)	Partial or complete collapse (loss of 50% of the building's volume has taken place at one or more floors).

The previously mentioned studies investigated the repair costs regarding the 1999 Athens Earthquake, but focus on the two most heavily impacted municipalities, Acharnes and Ano Liosia. The damage ratios, expressed as the ratio of repair costs to the replacement cost of buildings, varied between these studies. According to Karabinis and Baltzopoulou [62], the damage ratio for “green-tag” buildings was reported at 7.5%, while for “yellow-tag” buildings, it ranged from 10% to 20%, and for “red-tag” buildings, it fell between 40% and 70%. Similarly, Kappos et al. [63], found values of 6.7% for “green-tag” buildings, 17.5% for “yellow-tag” buildings, and 68.4% for “red-tag” buildings in the same municipalities. Vlachos and Vlachos, [64], derived damage ratios of 4.4%, 15.5%, and 43.7% for “green-tag”, “yellow-tag”, and “red-tag” buildings, respectively. The “black-tag” designation corresponds to a 100% economic loss factor, indicating buildings that collapsed to such an extent that repair was deemed unfeasible. Additional studies by Kappos et al. [10] and [66], utilized a framework with six damage states, as outlined in Table 6.3, to further explore the relationship between damage states and damage ratios.

Table 6.3. Damage ratios developed by Kappos et al. 2006 for RC buildings in Greece

Damage State	Damage Ratio
None (DS0)	0.0%
Slight (DS1)	0.5%
Moderate (DS2)	5.0%
Substantial to heavy (DS3)	20%
Very heavy (DS4)	45%
Collapse (DS5)	80%

Due to the limited data available concerning damage ratios for Greek masonry buildings, values influenced by Dolce et al. [67], were adopted. In that study it was developed standard beta probability functions for each damage scale outlined in Table 6.3, with values of 0.5% for DS0, 3.5% for DS1, 14.5% for DS2, 30.5% for DS3, and 80% for DS4. An alternative method for calculating vulnerability curves in terms of spectral displacement was introduced by Kappos et al. [66] for Greek masonry buildings. This approach was based on establishing the relationship between spectral displacement and the range of loss index as detailed in Table 6.4.

Table 6.4. Damage ratios developed by Kappos et al. [66] for Masonry buildings in Greece

Damage State	Damage State Label	Spectral Displacement	Range of Loss Index
DS0	None	$< 0.7S_{dy}$	0
DS1	Slight	$0.7S_{dy} \leq S_d < S_{dy}$	0-4
DS2	Moderate	$S_{dy} \leq S_d < 2S_{dy}$	4-20
DS3	Substantial to heavy	$2S_{dy} \leq S_d < 0.7S_{du}$	20-50
DS4	Very heavy	$0.7S_{du} \leq S_d < S_{du}$	50-100
DS5	Collapse	$> S_{du}$	

Considering the synthesis of findings from various studies, the damage ratios presented in Table 6.5 for RC buildings and Table 6.6 for URM buildings emerge as pivotal in the further vulnerability analysis. These values rooted in empirical evidence constitute the basis for calculating the Mean Damage Ratio (MDR), representing the anticipated damage ratio at specific earthquake intensities.

Table 6.5. Damage ratios for RC buildings

Damage State	Damage Ratio
No Damage (DS0)	0%
Slight Damage (DS1)	5%
Moderate Damage (DS2)	20%
Extensive Damage (DS3)	50%
Complete Damage (DS4)	90%

Table 6.6. Damage ratios for URM buildings

Damage State	Damage Ratio
No Damage (DS0)	0%
Slight Damage (DS1)	3%
Moderate Damage (DS2)	15%
Extensive Damage (DS3)	35%
Complete Damage (DS4)	85%

6.2. VULNERABILITY FUNCTIONS

Vulnerability functions are fundamental components in the process of assessing seismic risk. They are continuous functions expressing the probability of exceeding a given damage state given a function of the earthquake intensity. The derivation of the vulnerability curves can be from empirical data (e.g., Rosseto et al. [68]) or from analytical data (e.g., Silva et al. [69]). The procedure adopted in this study was the analytical approach deriving vulnerability curves from fragility curves using Eq.(6.2).

$$MDR_{ds,I} = \sum_{ds=1}^i P(ds_i|IM) \cdot DR_{ds_i} \quad (6.2)$$

Where MDR is the mean damage ratio, that basically is the sum of the multiplication between the probability of having a certain damage state (ds) given a certain level of intensity (I) by the damage ratio (DR) specified for each damage state described in the previous subchapter. A graphical representation of this formulation is presented in the Figure 6.1.

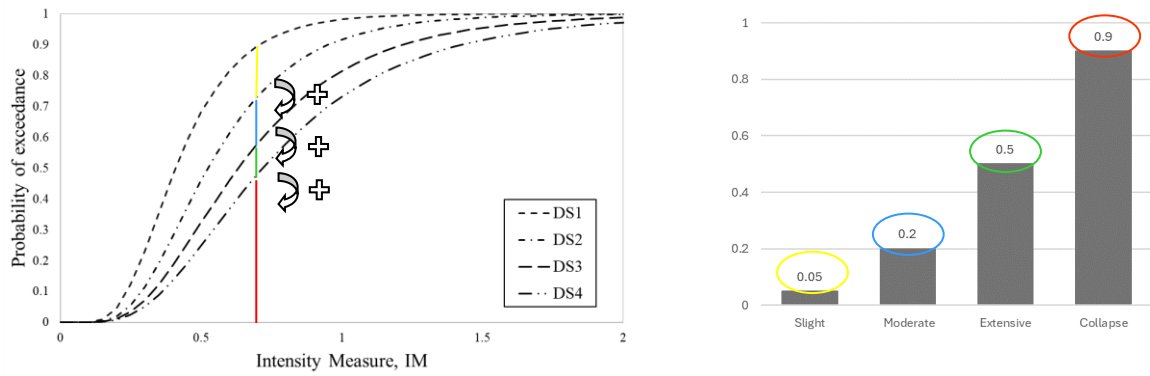


Figure 6.1. Graphical explanation of obtaining the mean damage ratio from a fragility curve and damage ratios

Utilizing the methodology outlined above and employing the disaggregated fragility curves obtained through Approach C, the study discerned the variance in losses between utilizing class-level fragilities acquired via Incremental Dynamic Analysis and the corresponding disaggregated plan-shape class-level fragilities. Figure 6.2 illustrates these disparities, wherein a building with a square plan-shape experiences equivalent economic losses at a lower earthquake intensity level. Conversely, for various class-typologies assessed, buildings with rectangular or L-shaped configurations exhibit underestimated losses when employing the original curve rather than the disaggregated ones. These findings underscore the impact that a simple plan-shape can have on loss estimation for a basic class-level typology. For instance, considering the class typology CR_LFINF-CDL-5_H2 at an earthquake intensity of 0.6g, the original curve yields 36% losses, whereas the disaggregated square building incurs 18%, and the rectangular and L-shaped buildings record 48% and 53% respectively.

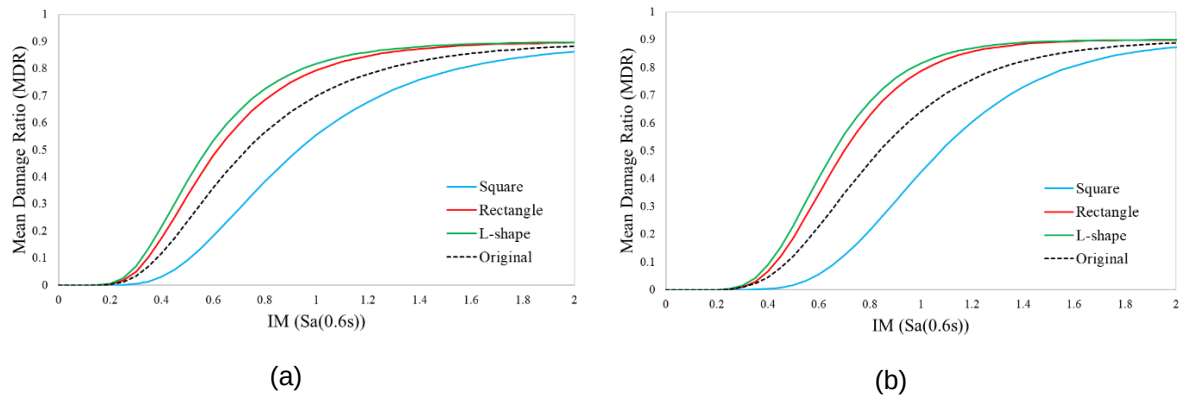


Figure 6.2. Vulnerability curves for disaggregated class-typologies. (a) CR_LFINF-CDL-5_H2; (b) CR_LFINF-CDM-15_H2

7

FINAL REMARKS

7.1. CONCLUSIONS

This study aimed to obtain more detailed fragility curves exploiting already established literature fragilities for different typology classes. The original taxonomy used in this work was the same as the one used in the ESRM20 project, where the class-typologies are divided in a general manner, not taking into account different aspects by which buildings can be distinguished, such as the presence of strong versus weak column designs (e.g., SCWB vs. WCSB), the composition of ductile or brittle elements, plan-shape intricacies, torsion effects resulting from mass eccentricity, conservation levels, etc. By using the methodology of disaggregation fragility curves, it is possible to obtain more detailed curves instead of merely using fragility curves from the literature with an additional dispersion of 0.3 to account for building-to-building vulnerability, like it was done in ESRM20. To define the most appropriate disaggregation of the curves, several surveyed buildings were considered, allowing the author to determine, for this work, which characteristic could be adopted as the primary factor for disaggregating the original class fragilities according to the ESRM20 project. These surveyed buildings characteristics were used to define the most appropriate class-typologies for the observed buildings, taking into account the codes used in Greece over the years and all the other factors described in ESRM20. Three different methodologies were developed to disaggregate the fragility curves, but Approach C can be considered the most appropriate and accurate. These methodologies use probabilistic laws, such as the law of total expectation and the law of total variance, in their formulation to disaggregate the curves into three new ones that consider the building's plan-shape factor, specifically differentiating between square-shape, rectangle-shape, and L-shape. Corrective factors are applied to aggregate the curves resulting from the probabilistic laws into the final desired curves. Very interesting results were obtained using the adopted methodologies, showing significant differences between the original and the disaggregated curves. For example, using the final proposed approach, evaluating the schools in Greece with class-typology with reinforced concrete with frame infills built after 1959 and before 1984 with two storeys height (CR_LFINF-CDL-5_H2), it was noticeable differences in median values up to 28% comparing the disaggregated fragility curves and the original one, and a significant reduction in dispersion. The disaggregation methodology has limitations when applying different correction factors that account for the plan-shape, known as plan-shape modifiers, for the four damage state levels. In some cases, due to the proximity of the median of the class-typology represented by an SDOF (Single Degree of Freedom system) and the methodology behind the disaggregation approach, the median values for the third damage state surpass those for the fourth damage state. To address this issue, a single plan-shape modifier was adopted for all damage states, considering only the elastic behaviour of the structure. For that was adopted using only one plan-shape modifier for all damage states considering only the elastic behaviour of the structure. Despite the limitation, the results obtained for the analysed and disaggregated

class-level seismic fragilities demonstrate that the adopted methodology proposes a way to obtain more appropriate fragility curves considering the numerous differences that buildings can present while being represented by the same class-typology. Vulnerability analyses also showed that this same disaggregation methodology allows for a better understanding and differentiation in losses for buildings with different plan-shapes.

7.2. RECOMMENDED FUTURE DEVELOPMENTS

The methodology proposed for disaggregating fragility curves is something with a lot to explore. This work shows, basically that there is a possibility to get fragility curves more appropriate from the ones used from the literature. Therefore, it is suggested the following developments for the future:

- A complete automation of a code capable of obtaining results for all class-typologies used in ESRM20, not just the ones used in this study;
- Development of an MDOF model that can represent variations in differences in buildings beyond just the plan-shape;
- Disaggregation of fragility curves into more properties beyond just the plan-shape of the observed buildings;
- Expansion of this methodology beyond a case study in Greece, allowing its application to all countries in Europe.

8

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ANNEXES

ANNEX A**CODENAMES FOR THE CLASS-TYPELOGIES DEVELOPED IN ESRM20**

Table A.1. Codenames for the typology of Concrete with Dual frame-wall system

CR_LDUAL_DUL_H1	CR_LDUAL_DUL_H10	CR_LDUAL_DUM_H7	CR_LDUAL_DUH_H4
CR_LDUAL_DUL_H2	CR_LDUAL_DUL_H11	CR_LDUAL_DUM_H8	CR_LDUAL_DUH_H5
CR_LDUAL_DUL_H3	CR_LDUAL_DUL_H12	CR_LDUAL_DUM_H9	CR_LDUAL_DUH_H6
CR_LDUAL_DUL_H4	CR_LDUAL_DUM_H1	CR_LDUAL_DUM_H10	CR_LDUAL_DUH_H7
CR_LDUAL_DUL_H5	CR_LDUAL_DUM_H2	CR_LDUAL_DUM_H11	CR_LDUAL_DUH_H8
CR_LDUAL_DUL_H6	CR_LDUAL_DUM_H3	CR_LDUAL_DUM_H12	CR_LDUAL_DUH_H9
CR_LDUAL_DUL_H7	CR_LDUAL_DUM_H4	CR_LDUAL_DUH_H1	CR_LDUAL_DUH_H10
CR_LDUAL_DUL_H8	CR_LDUAL_DUM_H5	CR_LDUAL_DUH_H2	CR_LDUAL_DUH_H11
CR_LDUAL_DUL_H9	CR_LDUAL_DUM_H6	CR_LDUAL_DUH_H3	CR_LDUAL_DUH_H12

Table A.2. Codenames for the typology of Concrete with Infill frames

CR_LFINF_CDL_0_H1	CR_LFINF_CDL_25_H1	CR_LFINF_CDM_15_H1	CR_LFINF_CDH_5_H1
CR_LFINF_CDL_0_H2	CR_LFINF_CDL_25_H2	CR_LFINF_CDM_15_H2	CR_LFINF_CDH_5_H2
CR_LFINF_CDL_0_H3	CR_LFINF_CDL_25_H3	CR_LFINF_CDM_15_H3	CR_LFINF_CDH_5_H3
CR_LFINF_CDL_0_H4	CR_LFINF_CDL_25_H4	CR_LFINF_CDM_15_H4	CR_LFINF_CDH_5_H4
CR_LFINF_CDL_0_H5	CR_LFINF_CDL_25_H5	CR_LFINF_CDM_15_H5	CR_LFINF_CDH_5_H5
CR_LFINF_CDL_0_H6	CR_LFINF_CDL_25_H6	CR_LFINF_CDM_15_H6	CR_LFINF_CDH_5_H6
CR_LFINF_CDL_5_H1	CR_LFINF_CDL_30_H1	CR_LFINF_CDM_20_H1	CR_LFINF_CDH_10_H1
CR_LFINF_CDL_5_H2	CR_LFINF_CDL_30_H2	CR_LFINF_CDM_20_H2	CR_LFINF_CDH_10_H2
CR_LFINF_CDL_5_H3	CR_LFINF_CDL_30_H3	CR_LFINF_CDM_20_H3	CR_LFINF_CDH_10_H3
CR_LFINF_CDL_5_H4	CR_LFINF_CDL_30_H4	CR_LFINF_CDM_20_H4	CR_LFINF_CDH_10_H4
CR_LFINF_CDL_5_H5	CR_LFINF_CDL_30_H5	CR_LFINF_CDM_20_H5	CR_LFINF_CDH_10_H5
CR_LFINF_CDL_5_H6	CR_LFINF_CDL_30_H6	CR_LFINF_CDM_20_H6	CR_LFINF_CDH_10_H6
CR_LFINF_CDL_10_H1	CR_LFINF_CDM_0_H1	CR_LFINF_CDM_25_H1	CR_LFINF_CDH_15_H1
CR_LFINF_CDL_10_H2	CR_LFINF_CDM_0_H2	CR_LFINF_CDM_25_H2	CR_LFINF_CDH_15_H2
CR_LFINF_CDL_10_H3	CR_LFINF_CDM_0_H3	CR_LFINF_CDM_25_H3	CR_LFINF_CDH_15_H3
CR_LFINF_CDL_10_H4	CR_LFINF_CDM_0_H4	CR_LFINF_CDM_25_H4	CR_LFINF_CDH_15_H4

CR_LFINF_CDL_10_H5	CR_LFINF_CDM_0_H5	CR_LFINF_CDM_25_H5	CR_LFINF_CDH_15_H5
CR_LFINF_CDL_10_H6	CR_LFINF_CDM_0_H6	CR_LFINF_CDM_25_H6	CR_LFINF_CDH_15_H6
CR_LFINF_CDL_15_H1	CR_LFINF_CDM_5_H1	CR_LFINF_CDM_30_H1	CR_LFINF_CDH_25_H1
CR_LFINF_CDL_15_H2	CR_LFINF_CDM_5_H2	CR_LFINF_CDM_30_H2	CR_LFINF_CDH_25_H2
CR_LFINF_CDL_15_H3	CR_LFINF_CDM_5_H3	CR_LFINF_CDM_30_H3	CR_LFINF_CDH_25_H3
CR_LFINF_CDL_15_H4	CR_LFINF_CDM_5_H4	CR_LFINF_CDM_30_H4	CR_LFINF_CDH_25_H4
CR_LFINF_CDL_15_H5	CR_LFINF_CDM_5_H5	CR_LFINF_CDM_30_H5	CR_LFINF_CDH_25_H5
CR_LFINF_CDL_15_H6	CR_LFINF_CDM_5_H6	CR_LFINF_CDM_30_H6	CR_LFINF_CDH_25_H6
CR_LFINF_CDL_20_H1	CR_LFINF_CDM_10_H1	CR_LFINF_CDH_0_H1	CR_LFINF_CDN_0_H1
CR_LFINF_CDL_20_H2	CR_LFINF_CDM_10_H2	CR_LFINF_CDH_0_H2	CR_LFINF_CDN_0_H2
CR_LFINF_CDL_20_H3	CR_LFINF_CDM_10_H3	CR_LFINF_CDH_0_H3	CR_LFINF_CDN_0_H3
CR_LFINF_CDL_20_H4	CR_LFINF_CDM_10_H4	CR_LFINF_CDH_0_H4	CR_LFINF_CDN_0_H4
CR_LFINF_CDL_20_H5	CR_LFINF_CDM_10_H5	CR_LFINF_CDH_0_H5	CR_LFINF_CDN_0_H5
CR_LFINF_CDL_20_H6	CR_LFINF_CDM_10_H6	CR_LFINF_CDH_0_H6	CR_LFINF_CDN_0_H6
CR_LFINF_CDN_5_H1	CR_LFINF_CDN_10_H4	CR_LFINF_CDN_20_H1	CR_LFINF_CDN_25_H4
CR_LFINF_CDN_5_H2	CR_LFINF_CDN_10_H5	CR_LFINF_CDN_20_H2	CR_LFINF_CDN_25_H5
CR_LFINF_CDN_5_H3	CR_LFINF_CDN_10_H6	CR_LFINF_CDN_20_H3	CR_LFINF_CDN_25_H6
CR_LFINF_CDN_5_H4	CR_LFINF_CDN_15_H1	CR_LFINF_CDN_20_H4	CR_LFINF_CDN_30_H1
CR_LFINF_CDN_5_H5	CR_LFINF_CDN_15_H2	CR_LFINF_CDN_20_H5	CR_LFINF_CDN_30_H2
CR_LFINF_CDN_5_H6	CR_LFINF_CDN_15_H3	CR_LFINF_CDN_20_H6	CR_LFINF_CDN_30_H3
CR_LFINF_CDN_10_H1	CR_LFINF_CDN_15_H4	CR_LFINF_CDN_25_H1	CR_LFINF_CDN_30_H4
CR_LFINF_CDN_10_H2	CR_LFINF_CDN_15_H5	CR_LFINF_CDN_25_H2	CR_LFINF_CDN_30_H5
CR_LFINF_CDN_10_H3	CR_LFINF_CDN_15_H6	CR_LFINF_CDN_25_H3	CR_LFINF_CDN_30_H6

Table A.3. Codenames for the typology of Concrete with Moment Frames

CR_LFM_CDL_0_H1	CR_LFM_CDM_0_H1	CR_LFM_CDH_0_H1	CR_LFM_CDN_0_H1
CR_LFM_CDL_0_H2	CR_LFM_CDM_0_H2	CR_LFM_CDH_0_H2	CR_LFM_CDN_0_H2
CR_LFM_CDL_0_H3	CR_LFM_CDM_0_H3	CR_LFM_CDH_0_H3	CR_LFM_CDN_0_H3
CR_LFM_CDL_0_H4	CR_LFM_CDM_0_H4	CR_LFM_CDH_0_H4	CR_LFM_CDN_0_H4
CR_LFM_CDL_0_H5	CR_LFM_CDM_0_H5	CR_LFM_CDH_0_H5	CR_LFM_CDN_0_H5
CR_LFM_CDL_0_H6	CR_LFM_CDM_0_H6	CR_LFM_CDH_0_H6	CR_LFM_CDN_0_H6

CR_LFM_CDL_5_H1	CR_LFM_CDM_5_H1	CR_LFM_CDH_5_H1	CR_LFM_CDN_5_H1
CR_LFM_CDL_5_H2	CR_LFM_CDM_5_H2	CR_LFM_CDH_5_H2	CR_LFM_CDN_5_H2
CR_LFM_CDL_5_H3	CR_LFM_CDM_5_H3	CR_LFM_CDH_5_H3	CR_LFM_CDN_5_H3
CR_LFM_CDL_5_H4	CR_LFM_CDM_5_H4	CR_LFM_CDH_5_H4	CR_LFM_CDN_5_H4
CR_LFM_CDL_5_H5	CR_LFM_CDM_5_H5	CR_LFM_CDH_5_H5	CR_LFM_CDN_5_H5
CR_LFM_CDL_5_H6	CR_LFM_CDM_5_H6	CR_LFM_CDH_5_H6	CR_LFM_CDN_5_H6
CR_LFM_CDL_10_H1	CR_LFM_CDM_10_H1	CR_LFM_CDH_10_H1	CR_LFM_CDN_10_H1
CR_LFM_CDL_10_H2	CR_LFM_CDM_10_H2	CR_LFM_CDH_10_H2	CR_LFM_CDN_10_H2
CR_LFM_CDL_10_H3	CR_LFM_CDM_10_H3	CR_LFM_CDH_10_H3	CR_LFM_CDN_10_H3
CR_LFM_CDL_10_H4	CR_LFM_CDM_10_H4	CR_LFM_CDH_10_H4	CR_LFM_CDN_10_H4
CR_LFM_CDL_10_H5	CR_LFM_CDM_10_H5	CR_LFM_CDH_10_H5	CR_LFM_CDN_10_H5
CR_LFM_CDL_10_H6	CR_LFM_CDM_10_H6	CR_LFM_CDH_10_H6	CR_LFM_CDN_10_H6
CR_LFM_CDL_15_H1	CR_LFM_CDM_15_H1	CR_LFM_CDH_15_H1	CR_LFM_CDN_15_H1
CR_LFM_CDL_15_H2	CR_LFM_CDM_15_H2	CR_LFM_CDH_15_H2	CR_LFM_CDN_15_H2
CR_LFM_CDL_15_H3	CR_LFM_CDM_15_H3	CR_LFM_CDH_15_H3	CR_LFM_CDN_15_H3
CR_LFM_CDL_15_H4	CR_LFM_CDM_15_H4	CR_LFM_CDH_15_H4	CR_LFM_CDN_15_H4
CR_LFM_CDL_15_H5	CR_LFM_CDM_15_H5	CR_LFM_CDH_15_H5	CR_LFM_CDN_15_H5
CR_LFM_CDL_15_H6	CR_LFM_CDM_15_H6	CR_LFM_CDH_15_H6	CR_LFM_CDN_15_H6
CR_LFM_CDL_20_H1	CR_LFM_CDM_20_H1	CR_LFM_CDH_20_H1	CR_LFM_CDN_20_H1
CR_LFM_CDL_20_H2	CR_LFM_CDM_20_H2	CR_LFM_CDH_20_H2	CR_LFM_CDN_20_H2
CR_LFM_CDL_20_H3	CR_LFM_CDM_20_H3	CR_LFM_CDH_20_H3	CR_LFM_CDN_20_H3
CR_LFM_CDL_20_H4	CR_LFM_CDM_20_H4	CR_LFM_CDH_20_H4	CR_LFM_CDN_20_H4
CR_LFM_CDL_20_H5	CR_LFM_CDM_20_H5	CR_LFM_CDH_20_H5	CR_LFM_CDN_20_H5
CR_LFM_CDL_20_H6	CR_LFM_CDM_20_H6	CR_LFM_CDH_20_H6	CR_LFM_CDN_20_H6
CR_LFM_CDL_25_H1	CR_LFM_CDM_25_H1	CR_LFM_CDH_25_H1	CR_LFM_CDN_25_H1
CR_LFM_CDL_25_H2	CR_LFM_CDM_25_H2	CR_LFM_CDH_25_H2	CR_LFM_CDN_25_H2
CR_LFM_CDL_25_H3	CR_LFM_CDM_25_H3	CR_LFM_CDH_25_H3	CR_LFM_CDN_25_H3
CR_LFM_CDL_25_H4	CR_LFM_CDM_25_H4	CR_LFM_CDH_25_H4	CR_LFM_CDN_25_H4
CR_LFM_CDL_25_H5	CR_LFM_CDM_25_H5	CR_LFM_CDH_25_H5	CR_LFM_CDN_25_H5
CR_LFM_CDL_25_H6	CR_LFM_CDM_25_H6	CR_LFM_CDH_25_H6	CR_LFM_CDN_25_H6
CR_LFM_CDL_30_H1	CR_LFM_CDM_30_H1	CR_LFM_CDH_30_H1	CR_LFM_CDN_30_H1
CR_LFM_CDL_30_H2	CR_LFM_CDM_30_H2	CR_LFM_CDH_30_H2	CR_LFM_CDN_30_H2

CR_LFM_CDL_30_H3	CR_LFM_CDM_30_H3	CR_LFM_CDH_30_H3	CR_LFM_CDN_30_H3
CR_LFM_CDL_30_H4	CR_LFM_CDM_30_H4	CR_LFM_CDH_30_H4	CR_LFM_CDN_30_H4
CR_LFM_CDL_30_H5	CR_LFM_CDM_30_H5	CR_LFM_CDH_30_H5	CR_LFM_CDN_30_H5
CR_LFM_CDL_30_H6	CR_LFM_CDM_30_H6	CR_LFM_CDH_30_H6	CR_LFM_CDN_30_H6

Table A.4. Codenames for the typology of Concrete with Loading bearing walls

CR_LWAL-DUL_H1	CR_LWAL-DUL_H10	CR_LWAL-DUM_H7	CR_LWAL-DUH_H4
CR_LWAL-DUL_H2	CR_LWAL-DUL_H11	CR_LWAL-DUM_H8	CR_LWAL-DUH_H5
CR_LWAL-DUL_H3	CR_LWAL-DUL_H12	CR_LWAL-DUM_H9	CR_LWAL-DUH_H6
CR_LWAL-DUL_H4	CR_LWAL-DUM_H1	CR_LWAL-DUM_H10	CR_LWAL-DUH_H7
CR_LWAL-DUL_H5	CR_LWAL-DUM_H2	CR_LWAL-DUM_H11	CR_LWAL-DUH_H8
CR_LWAL-DUL_H6	CR_LWAL-DUM_H3	CR_LWAL-DUM_H12	CR_LWAL-DUH_H9
CR_LWAL-DUL_H7	CR_LWAL-DUM_H4	CR_LWAL-DUH_H1	CR_LWAL-DUH_H10
CR_LWAL-DUL_H8	CR_LWAL-DUM_H5	CR_LWAL-DUH_H2	CR_LWAL-DUH_H11
CR_LWAL-DUL_H9	CR_LWAL-DUM_H6	CR_LWAL-DUH_H3	CR_LWAL-DUH_H12

Table A.5. Codenames for the typology of Confined masonry with Loading bearing walls

MCF_LWAL-DUL_H1	MCF_LWAL-DUM_H1	MCF_LWAL-DUH_H1
MCF_LWAL-DUL_H2	MCF_LWAL-DUM_H2	MCF_LWAL-DUH_H2
MCF_LWAL-DUL_H3	MCF_LWAL-DUM_H3	MCF_LWAL-DUH_H3
MCF_LWAL-DUL_H4	MCF_LWAL-DUM_H4	MCF_LWAL-DUH_H4
MCF_LWAL-DUL_H5	MCF_LWAL-DUM_H5	MCF_LWAL-DUH_H5
MCF_LWAL-DUL_H6	MCF_LWAL-DUM_H6	MCF_LWAL-DUH_H6

Table A.6. Codenames for the typology of Reinforced Masonry with Dual frame-wall system

MR_LWAL-DUL_H1	MR_LWAL-DUM_H1	MR_LWAL-DUH_H1
MR_LWAL-DUL_H2	MR_LWAL-DUM_H2	MR_LWAL-DUH_H2
MR_LWAL-DUL_H3	MR_LWAL-DUM_H3	MR_LWAL-DUH_H3
MR_LWAL-DUL_H4	MR_LWAL-DUM_H4	MR_LWAL-DUH_H4
MR_LWAL-DUL_H5	MR_LWAL-DUM_H5	MR_LWAL-DUH_H5

Table A.7. Codenames for the typology of Unreinforced Masonry with Adobe

MUR_ADO_LWAL_DNO_H1	MUR_ADO_LWAL_DNO_H2	MUR_ADO_LWAL_DNO_H3
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Table A.8. Codenames for the typology of Unreinforced Masonry with Concrete blocks (CB99) and clay bricks (CL99)

MUR_CB99_LWAL_DNO_H1	MUR_CL99_LWAL_DNO_H1
MUR_CB99_LWAL_DNO_H2	MUR_CL99_LWAL_DNO_H2
MUR_CB99_LWAL_DNO_H3	MUR_CL99_LWAL_DNO_H3
MUR_CB99_LWAL_DNO_H4	MUR_CL99_LWAL_DNO_H4
MUR_CB99_LWAL_DNO_H5	MUR_CL99_LWAL_DNO_H5

Table A.9. Codenames for the typology of Unreinforced Masonry with Dressed stone (STDRE) and Rubble stone (STRUB)

MUR_STDRE_LWAL_DNO_H1	MUR_STRUB_LWAL_DNO_H1
MUR_STDRE_LWAL_DNO_H2	MUR_STRUB_LWAL_DNO_H2
MUR_STDRE_LWAL_DNO_H3	MUR_STRUB_LWAL_DNO_H3
MUR_STDRE_LWAL_DNO_H4	MUR_STRUB_LWAL_DNO_H4
MUR_STDRE_LWAL_DNO_H5	MUR_STRUB_LWAL_DNO_H5

Table A.10. Codenames for the typology of Unreinforced Masonry with Dual wall-system frame

MUR_LWAL__DNO_H1
MUR_LWAL__DNO_H2
MUR_LWAL__DNO_H3
MUR_LWAL__DNO_H4
MUR_LWAL__DNO_H5

Table A.11. Codenames for the typology of Steel with Braced system (LFBR), Moment Frame (LFM), Dual wall-system frame (LWAL) and Infill frame (LFINF)

S_LFBR_DUM_H1	S_LFBR_DUH_H1	S_LFM_DUM_H1
S_LFBR_DUM_H2	S_LFBR_DUH_H2	S_LFM_DUM_H2
S_LFBR_DUM_H3	S_LFBR_DUH_H3	S_LFM_DUM_H3
S_LFBR_DUM_H4	S_LFBR_DUH_H4	S_LFM_DUM_H4
S_LFBR_DUM_H5	S_LFBR_DUH_H5	S_LFM_DUM_H5
S_LFBR_DUM_H6	S_LFBR_DUH_H6	S_LFM_DUM_H6
S_LFBR_DUM_H7	S_LFBR_DUH_H7	S_LFM_DUM_H7
S_LFBR_DUM_H8	S_LFBR_DUH_H8	S_LFM_DUM_H8
S_LFBR_DUM_H9	S_LFBR_DUH_H9	S_LFM_DUM_H9
S_LFBR_DUM_H10	S_LFBR_DUH_H10	S_LFM_DUM_H10
S_LFBR_DUM_H11	S_LFBR_DUH_H11	S_LFM_DUM_H11
S_LFBR_DUM_H12	S_LFBR_DUH_H12	S_LFM_DUM_H12
S_LFM_DUH_H1	S_LFM_DUH_H5	S_LFM_DUH_H9
S_LFM_DUH_H2	S_LFM_DUH_H6	S_LFM_DUH_H10
S_LFM_DUH_H3	S_LFM_DUH_H7	S_LFM_DUH_H11
S_LFM_DUH_H4	S_LFM_DUH_H8	S_LFM_DUH_H12
S_LWAL_DUM_H1	S_LWAL_DUM_H5	S_LWAL_DUM_H9
S_LWAL_DUM_H2	S_LWAL_DUM_H6	S_LWAL_DUM_H10
S_LWAL_DUM_H3	S_LWAL_DUM_H7	S_LWAL_DUM_H11
S_LWAL_DUM_H4	S_LWAL_DUM_H8	S_LWAL_DUM_H12
S_LWAL_DUH_H1	S_LWAL_DUH_H5	S_LWAL_DUH_H9
S_LWAL_DUH_H2	S_LWAL_DUH_H6	S_LWAL_DUH_H10
S_LWAL_DUH_H3	S_LWAL_DUH_H7	S_LWAL_DUH_H11
S_LWAL_DUH_H4	S_LWAL_DUH_H8	S_LWAL_DUH_H12
S_LFINF_DUH_H1	S_LFINF_DUH_H5	S_LFINF_DUH_H9
S_LFINF_DUH_H2	S_LFINF_DUH_H6	S_LFINF_DUH_H10
S_LFINF_DUH_H3	S_LFINF_DUH_H7	S_LFINF_DUH_H11
S_LFINF_DUH_H4	S_LFINF_DUH_H8	S_LFINF_DUH_H12

Table A.12. Codenames for the typology of Wood/Timber with Moment frames

W_LFM_DUL_H1	W_LFM_DUM_H1	W_LFM_DUH_H1
W_LFM_DUL_H2	W_LFM_DUM_H2	W_LFM_DUH_H2
W_LFM_DUL_H3	W_LFM_DUM_H3	W_LFM_DUH_H3
W_LFM_DUL_H4	W_LFM_DUM_H4	W_LFM_DUH_H4
W_LFM_DUL_H5	W_LFM_DUM_H5	W_LFM_DUH_H5
W_LFM_DUL_H6	W_LFM_DUM_H6	W_LFM_DUH_H6

ANNEX B

ESRM20 FRAGILITY CURVES

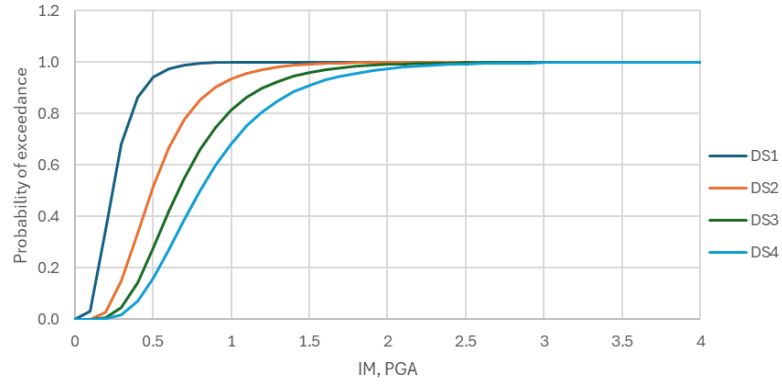


Figure B.1. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H1 with PGA as IM

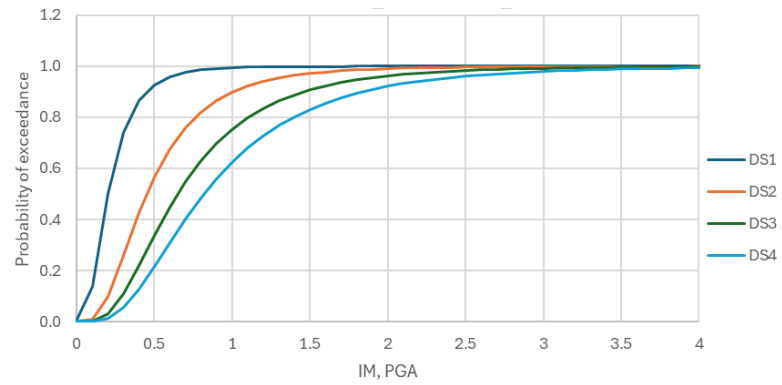


Figure B.2. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H2 with PGA as IM

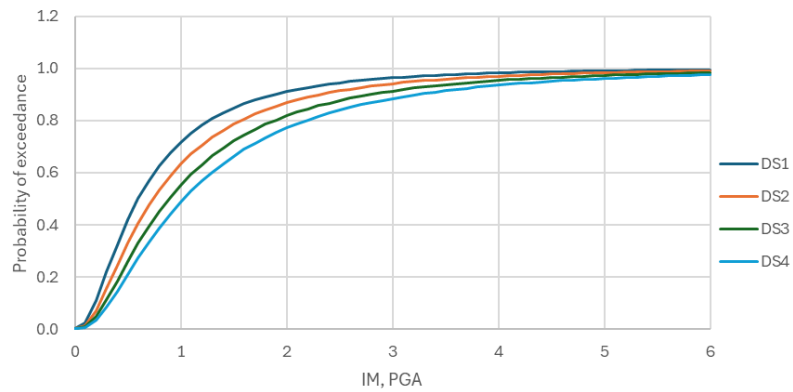


Figure B.3. Fragility curve by ESRM20 for CR_LFINF-CDN-0_H1 with PGA as IM

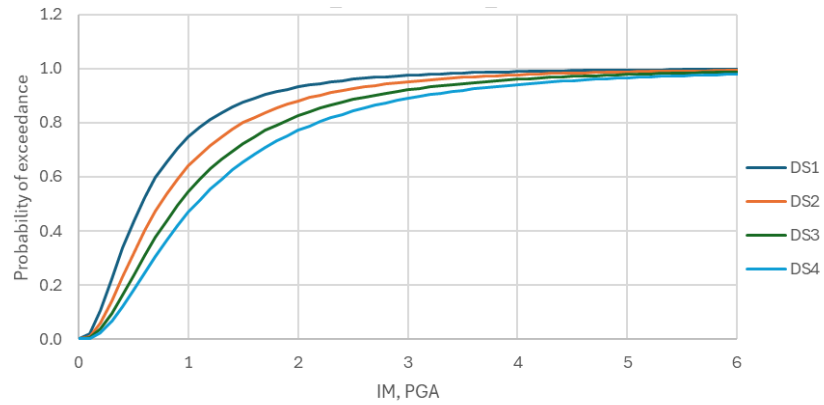


Figure B.4. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H1 with PGA as IM

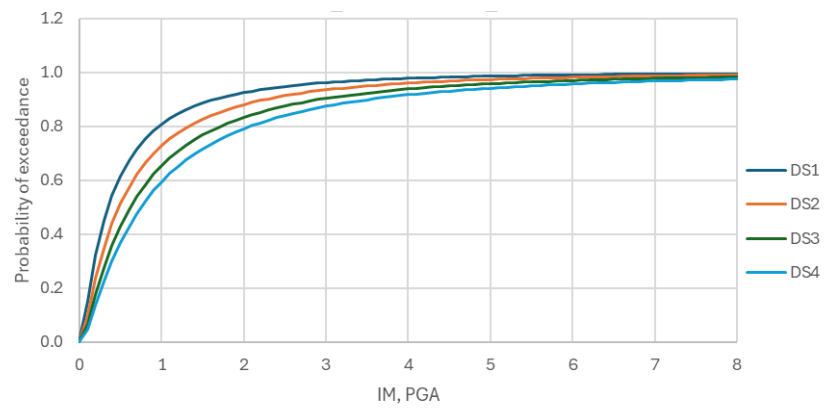


Figure B.5. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H2 with PGA as IM

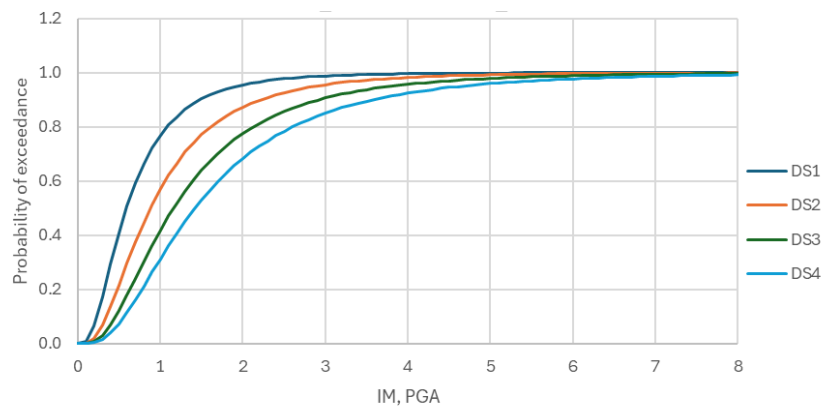


Figure B.6. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H1 with PGA as IM

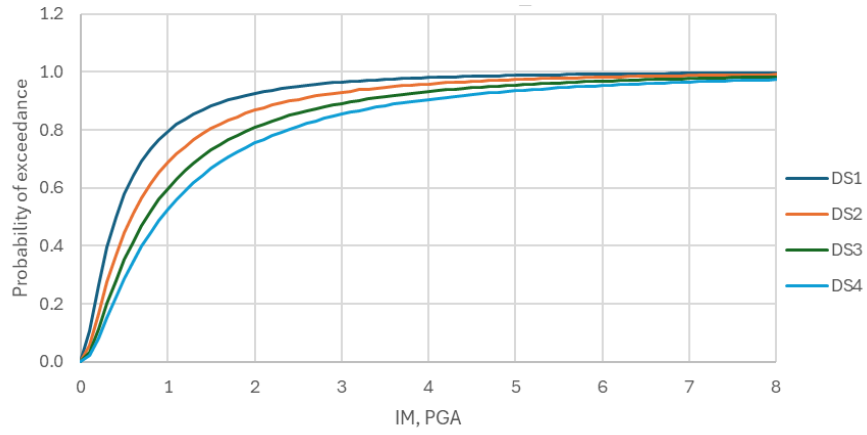


Figure B.7. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H2 with PGA as IM

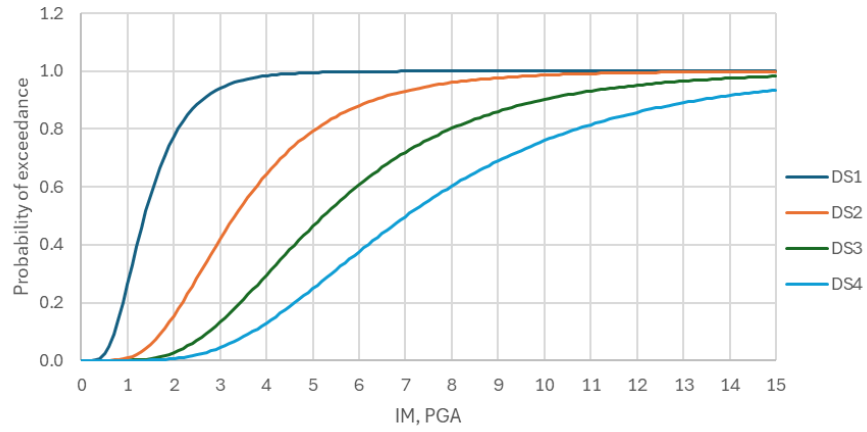


Figure B.8. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H1 with PGA as IM

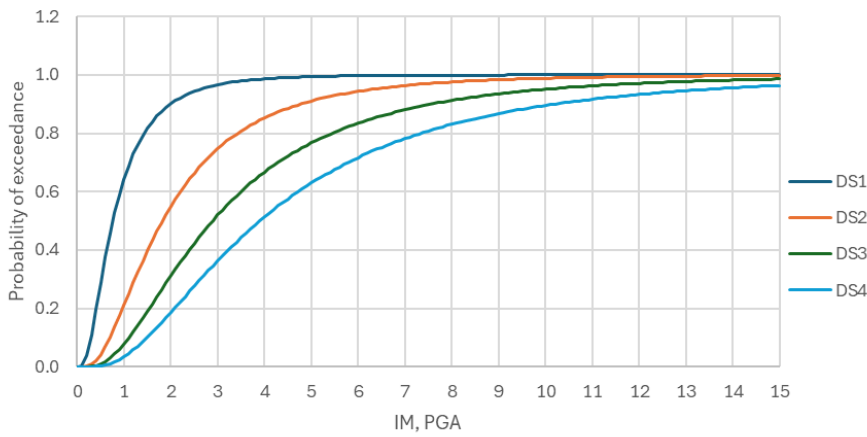


Figure B.9. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H1 with PGA as IM

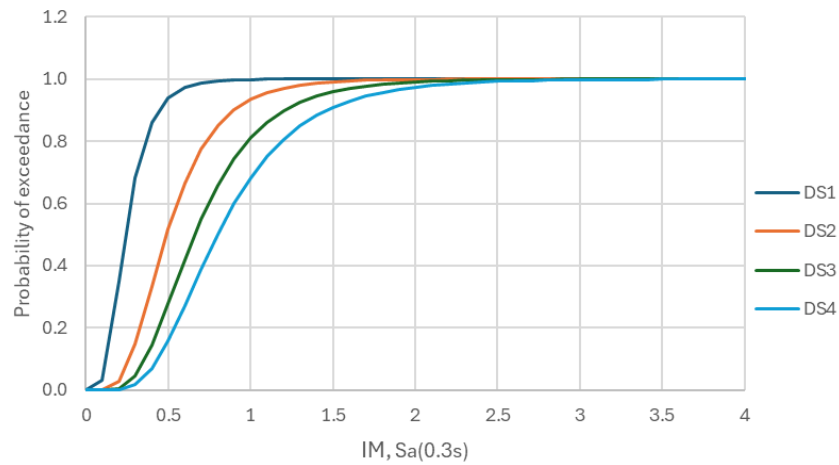


Figure B.10. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H1 with Sa(0.3s) as IM

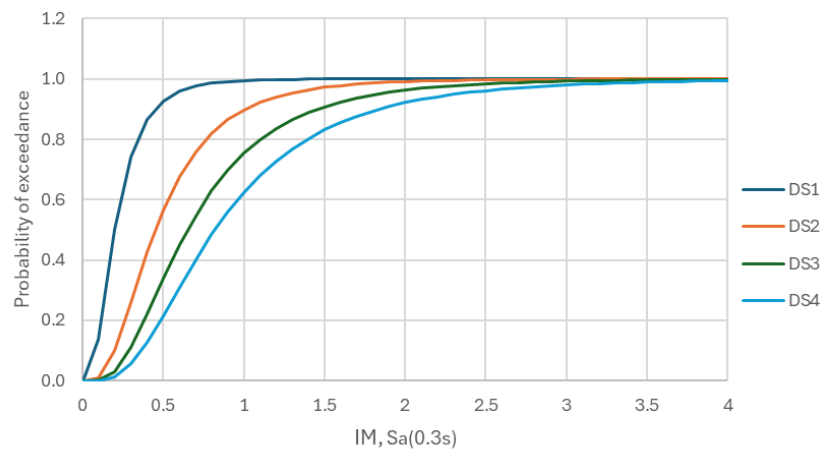


Figure B.11. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H2 with Sa(0.3s) as IM

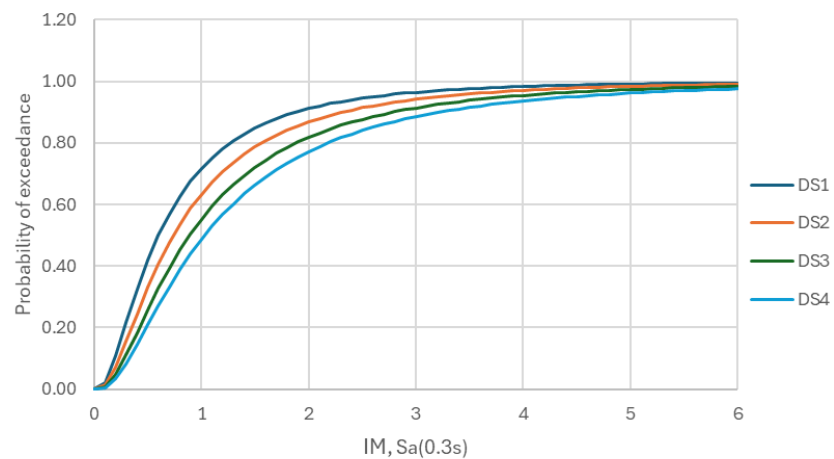


Figure B.12. Fragility curve by ESRM20 for CR_LFINF-CDN-0_H1 with Sa(0.3s) as IM

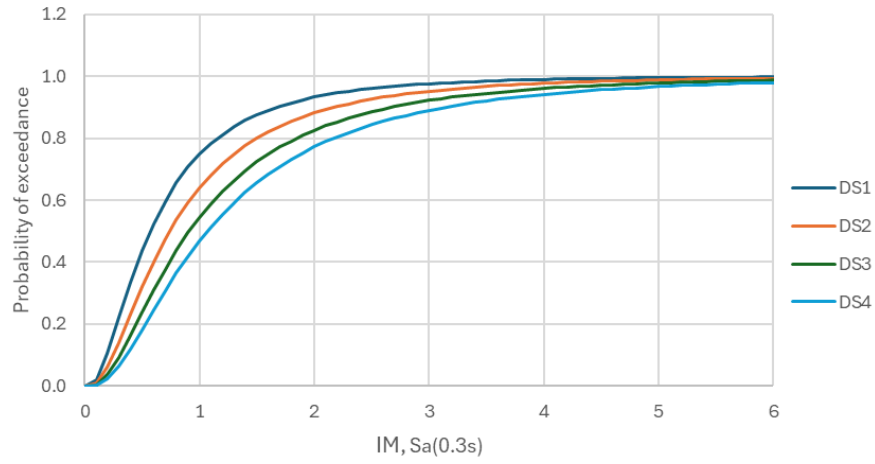


Figure B.13. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H1 with $S_a(0.3s)$ as IM

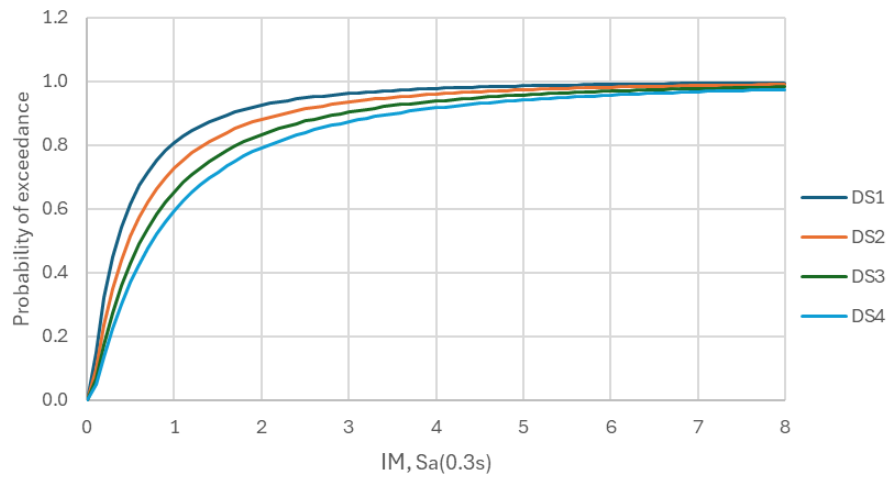


Figure B.14. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H2 with $S_a(0.3s)$ as IM

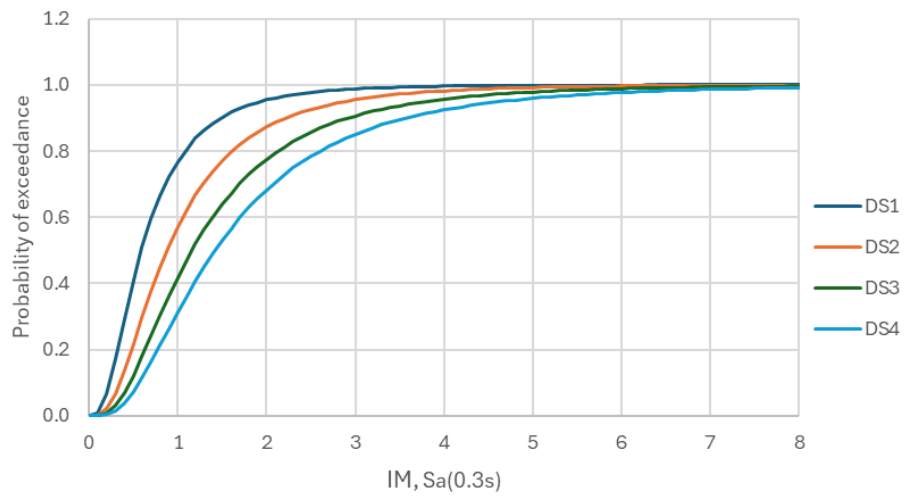


Figure B.15. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H1 with $S_a(0.3s)$ as IM

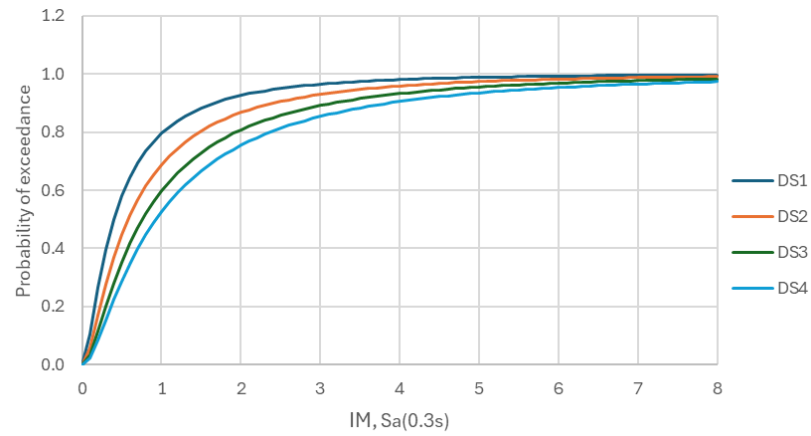


Figure B.16. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H2 with $S_a(0.3s)$ as IM

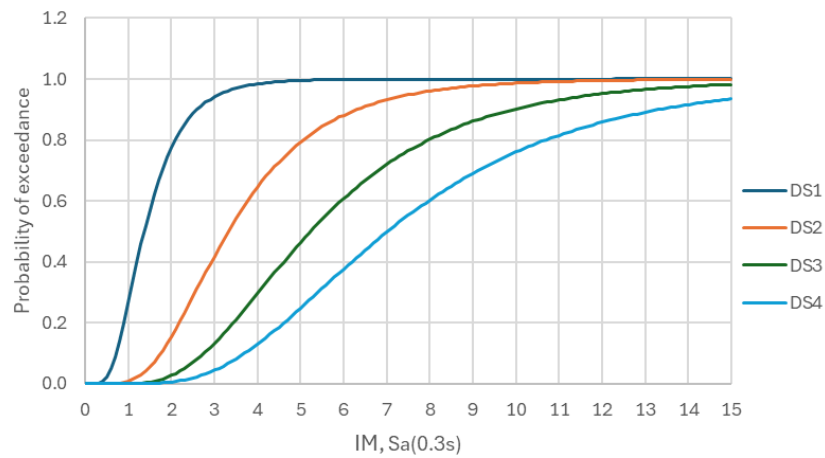


Figure B.17. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H1 with $S_a(0.3s)$ as IM

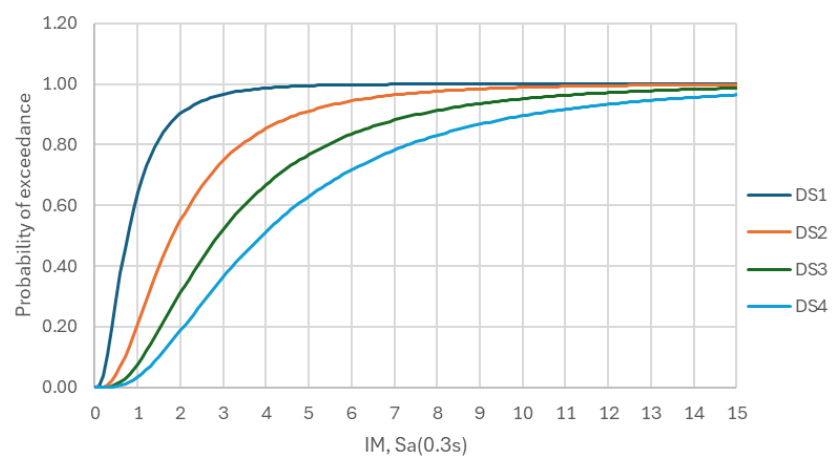


Figure B.18. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H2 with $S_a(0.3s)$ as IM

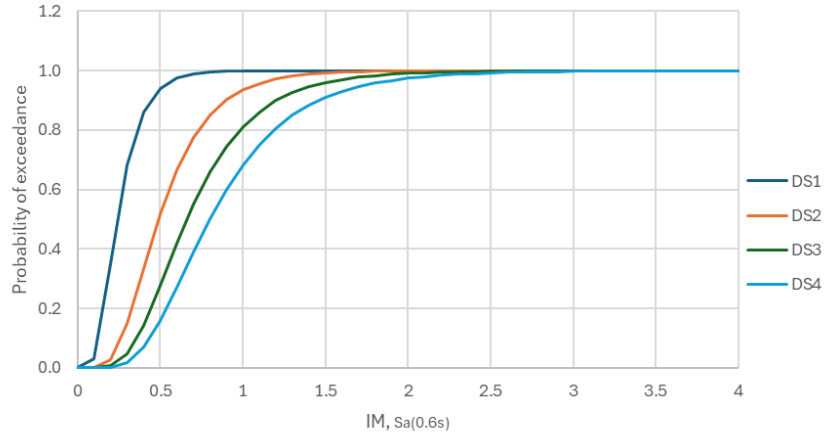


Figure B.19. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H1 with Sa(0.6s) as IM

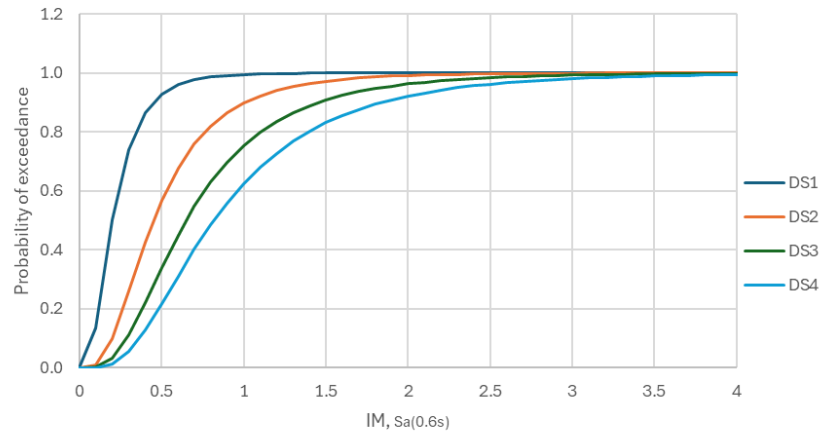


Figure B.20. Fragility curve by ESRM20 for MUR-STRUB_LWAL-DNO_H2 with Sa(0.6s) as IM

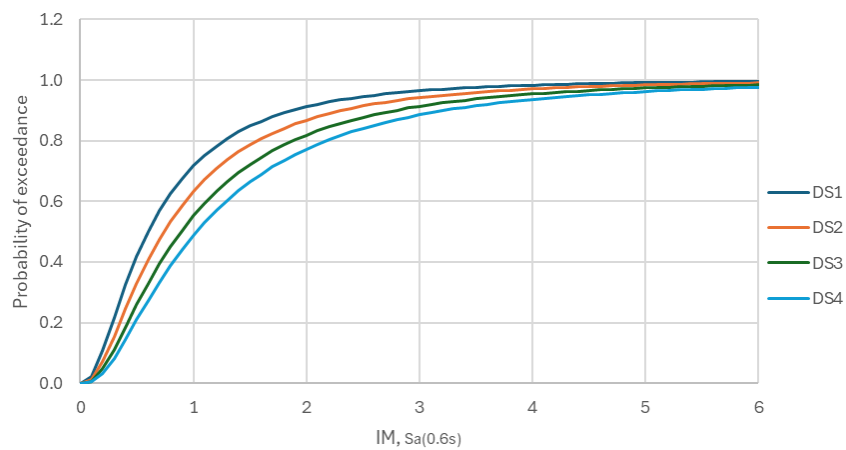


Figure B.21. Fragility curve by ESRM20 for CR_LFINF-CDN-0_H1 with Sa(0.6s) as IM

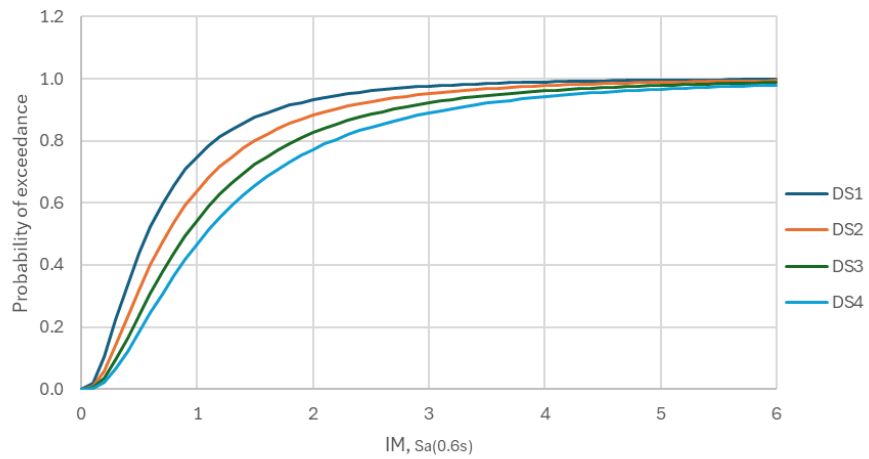


Figure B.22. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H1 with Sa(0.6s) as IM

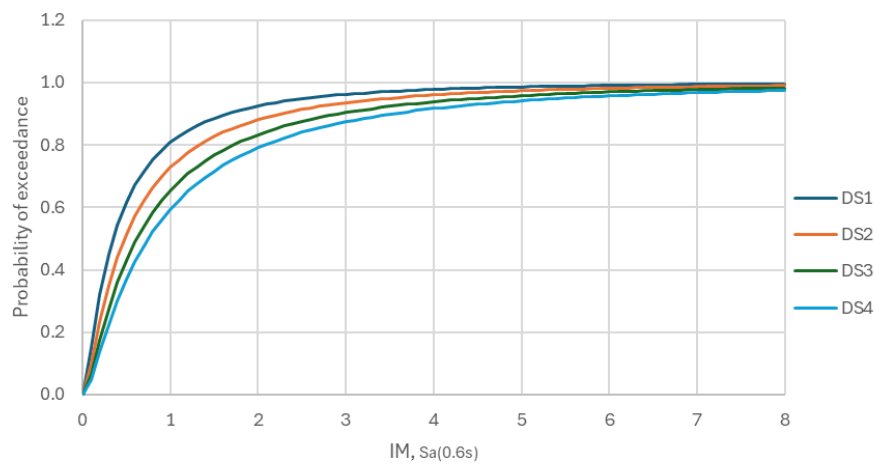


Figure B.23. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H2 with Sa(0.6s) as IM

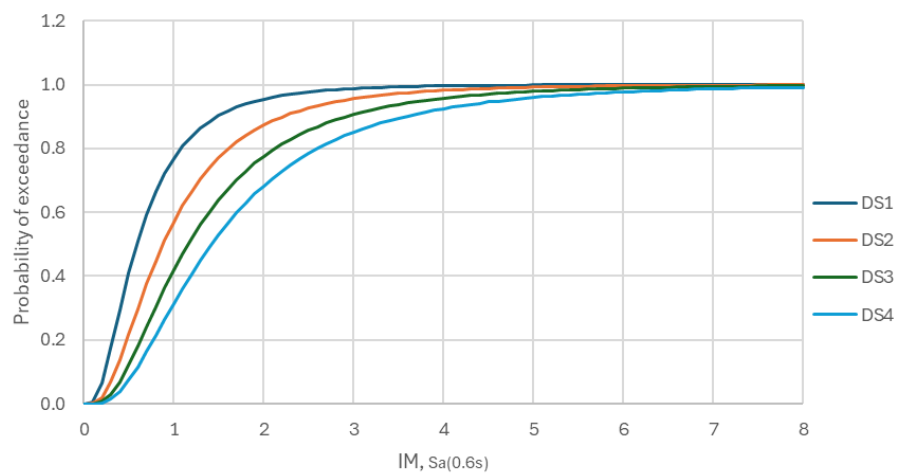


Figure B.24. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H1 with Sa(0.6s) as IM

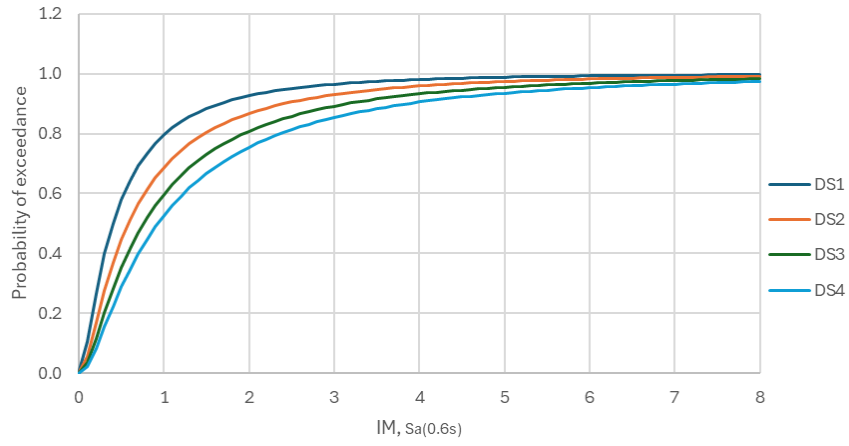


Figure B.25. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H2 with $S_a(0.6s)$ as IM

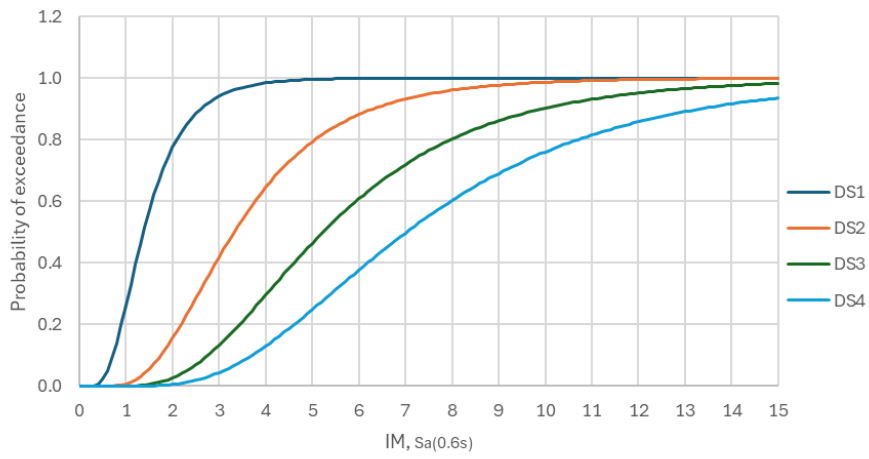


Figure B.26. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H1 with $S_a(0.6s)$ as IM

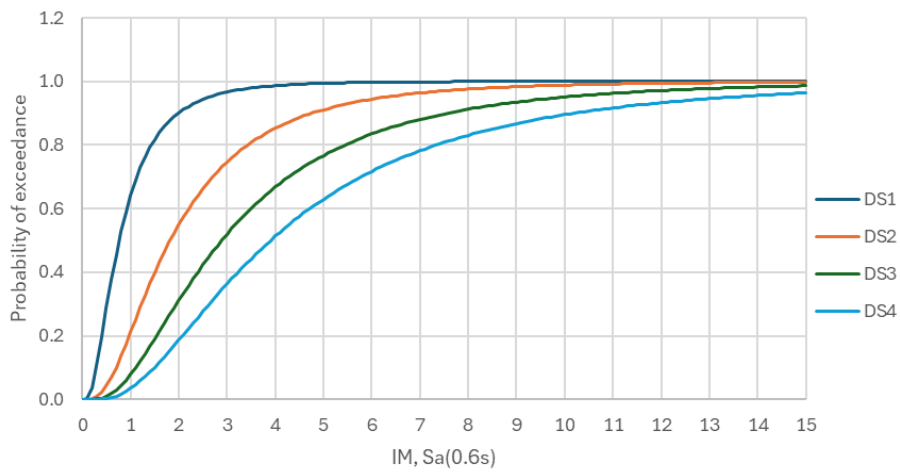


Figure B.27. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H2 with $S_a(0.6s)$ as IM

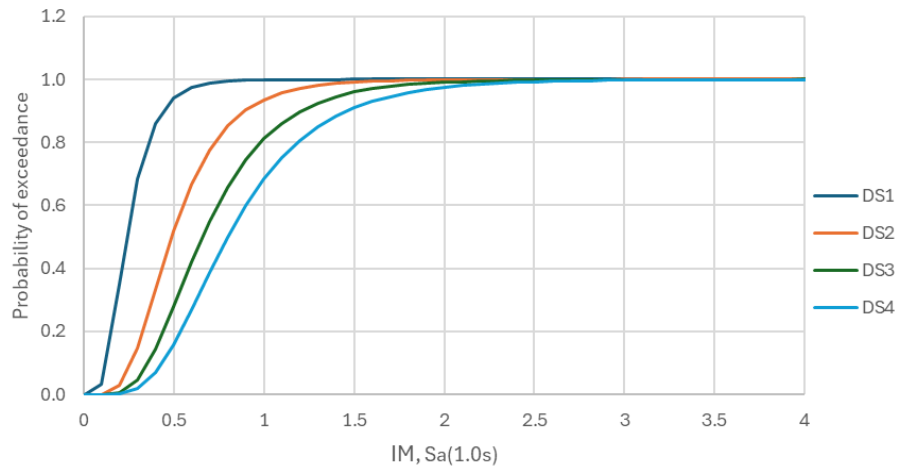


Figure B.28. Fragility curve by ESRM20 for MUR_STRUB_LWAL-DNO_H1 with Sa(1.0s) as IM

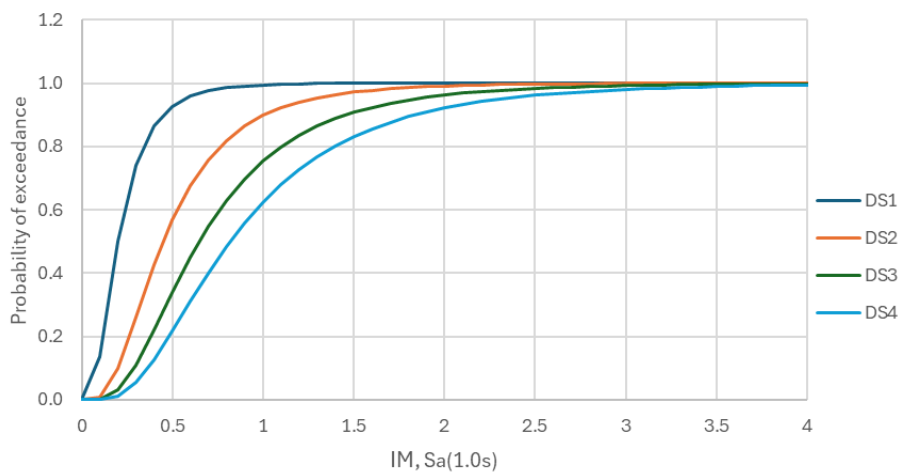


Figure B.29. Fragility curve by ESRM20 for MUR_STRUB_LWAL-DNO_H2 with Sa(1.0s) as IM.

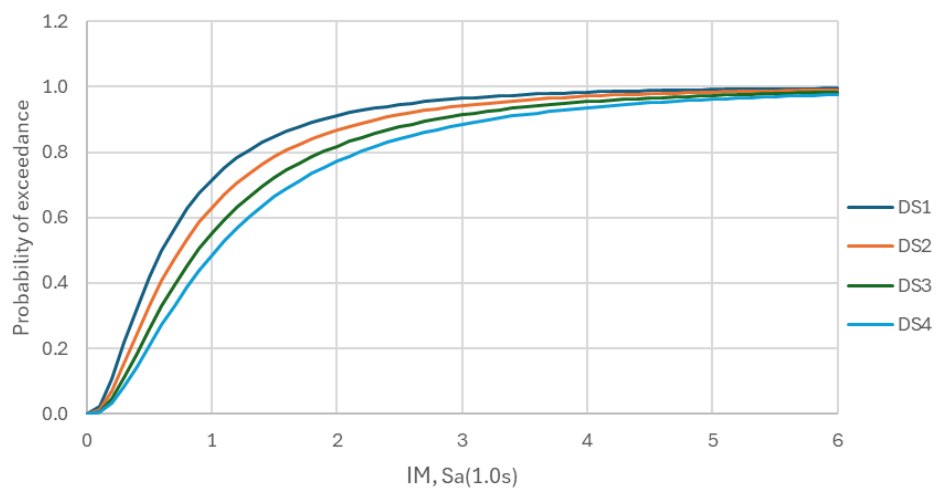


Figure B.30. Fragility curve by ESRM20 for CR_LFINF-CDN-0_H1 with Sa(1.0s) as IM

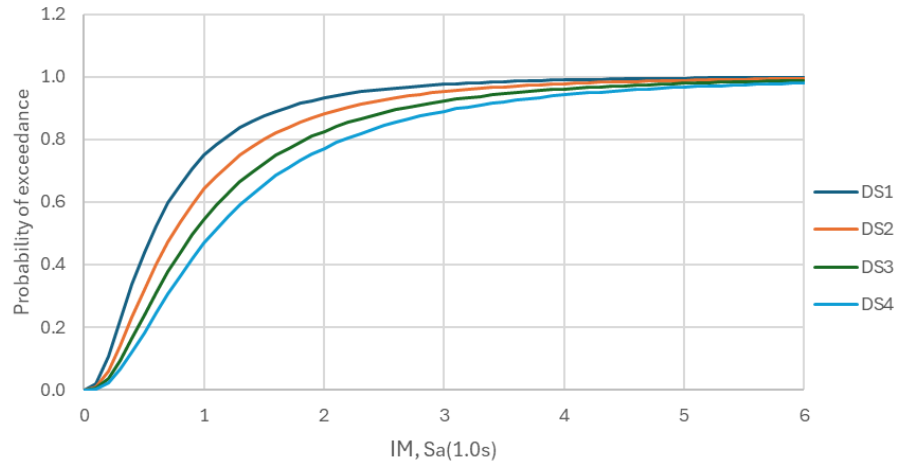


Figure B.31. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H1 with Sa(1.0s) as IM

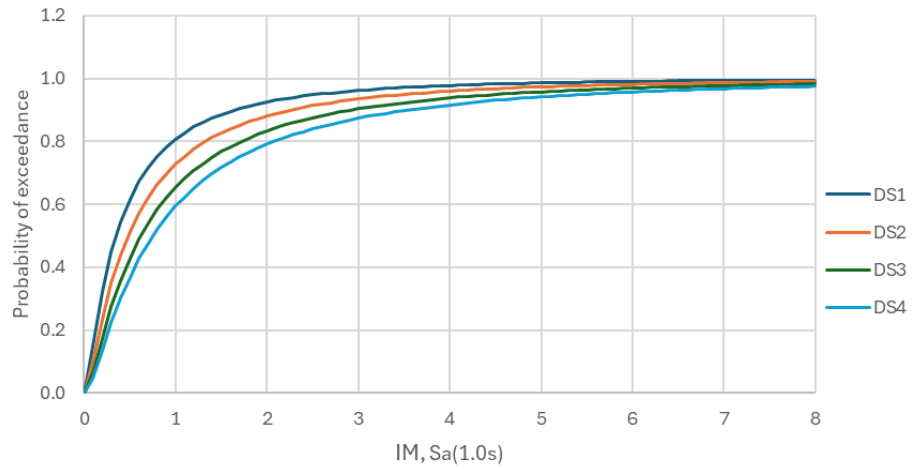


Figure B.32. Fragility curve by ESRM20 for CR_LFINF-CDL-5_H2 with Sa(1.0s) as IM

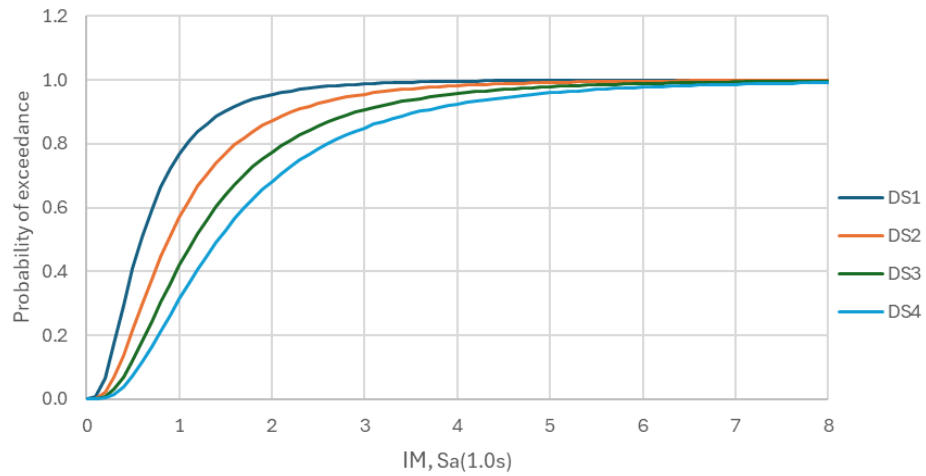


Figure B.33. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H1 with Sa(1.0s) as IM

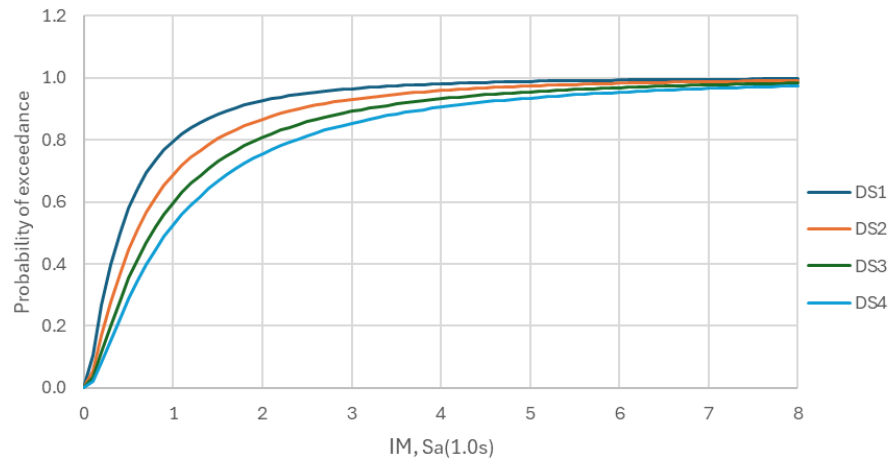


Figure B.34. Fragility curve by ESRM20 for CR_LFINF-CDM-15_H2 with Sa(1.0s) as IM

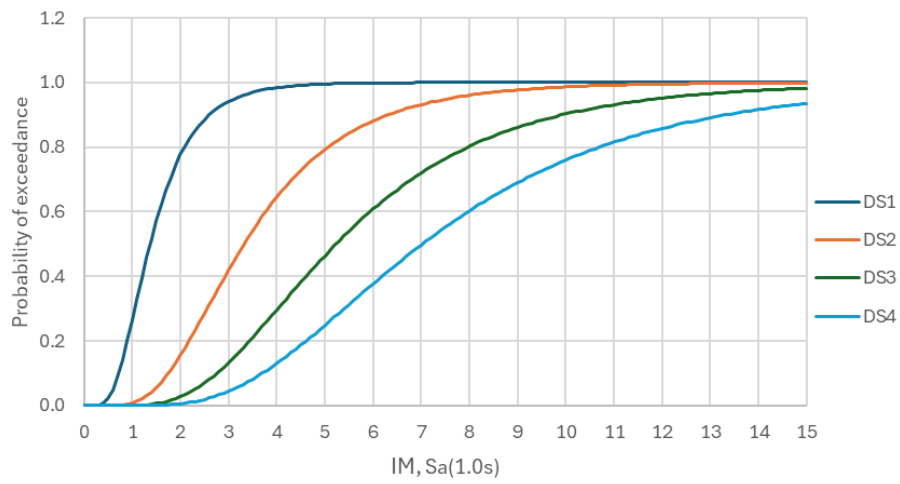


Figure B.35. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H1 with Sa(1.0s) as IM

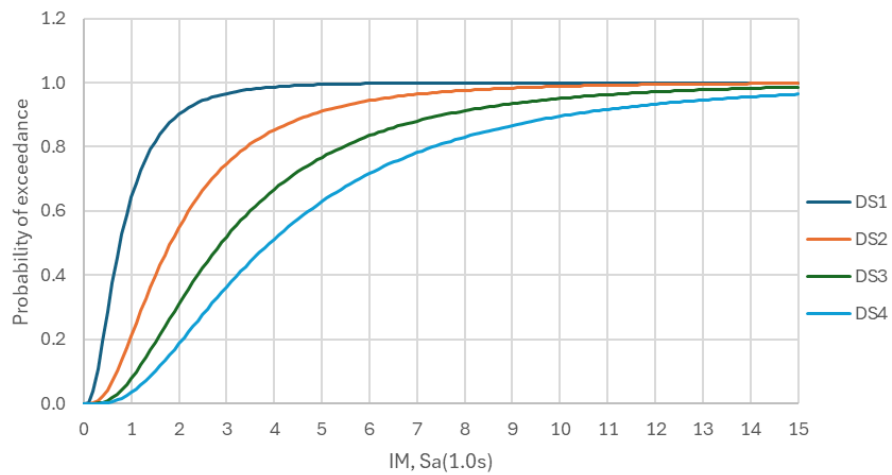


Figure B.36. Fragility curve by ESRM20 for CR_LFINF-CDH-25_H2 with Sa(1.0s) as IM

ANNEX C.1

FRAGILITY CURVES OF SDOFs RESULTED FROM IDA

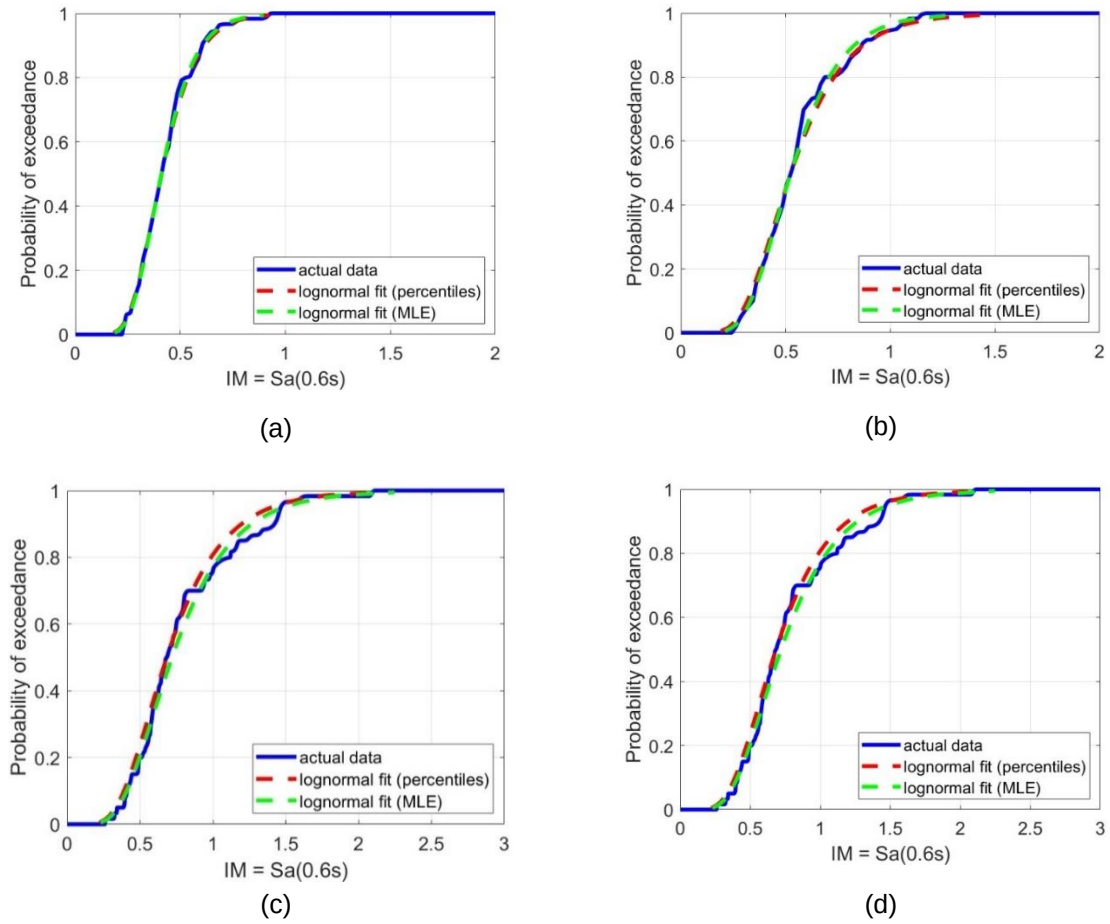
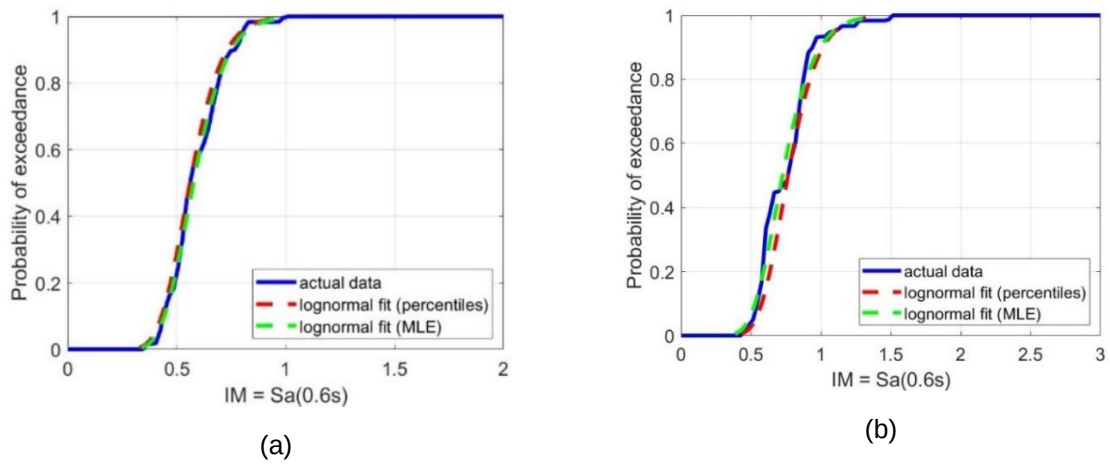


Figure C.1. Fragility Curves derived from IDA for typology CR_LFINF-CDL-5_H2; (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



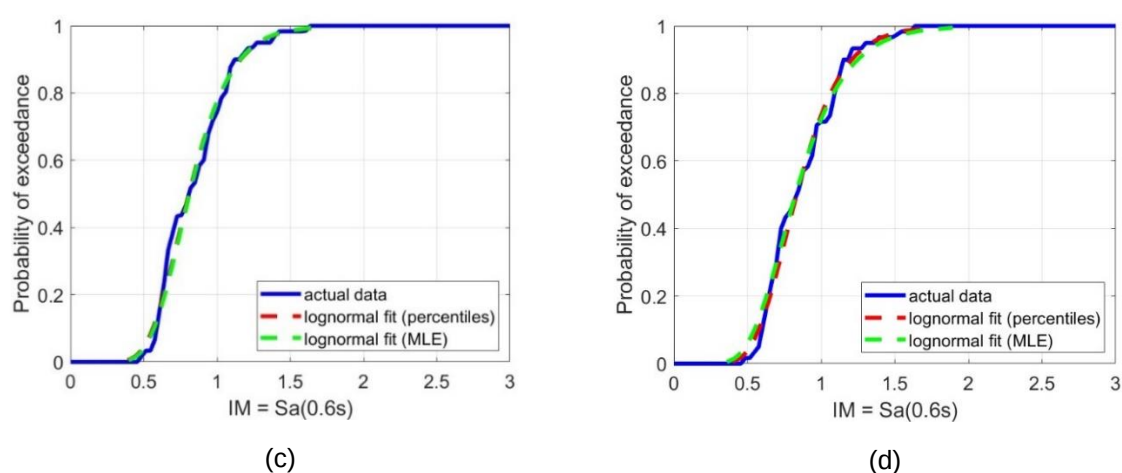


Figure C.2. Fragility Curves derived from IDA for typology CR_LFINF-CDM-15_H2; (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

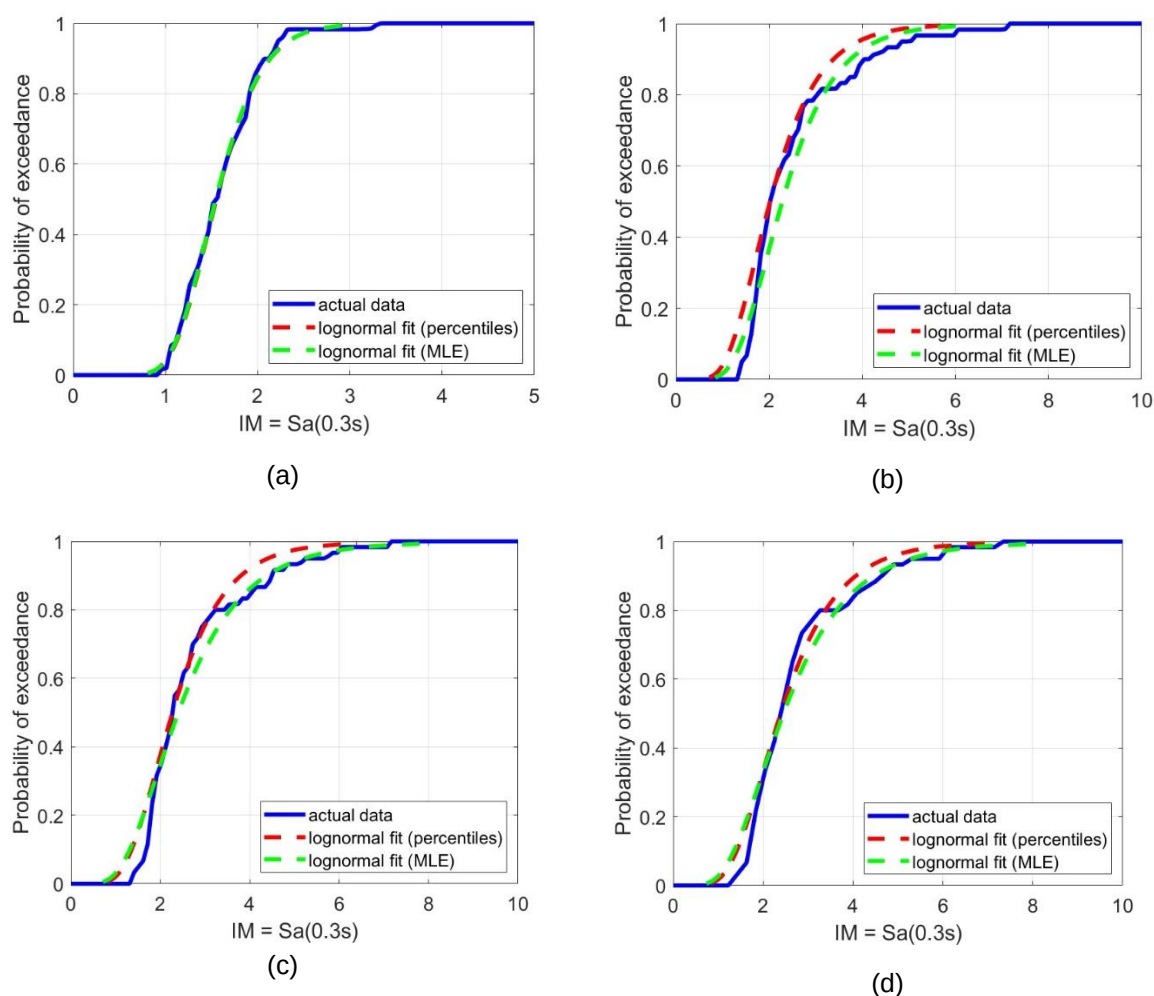


Figure C.3. Fragility Curves derived from IDA for typology CR_LFINF-CDH-25_H2; (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

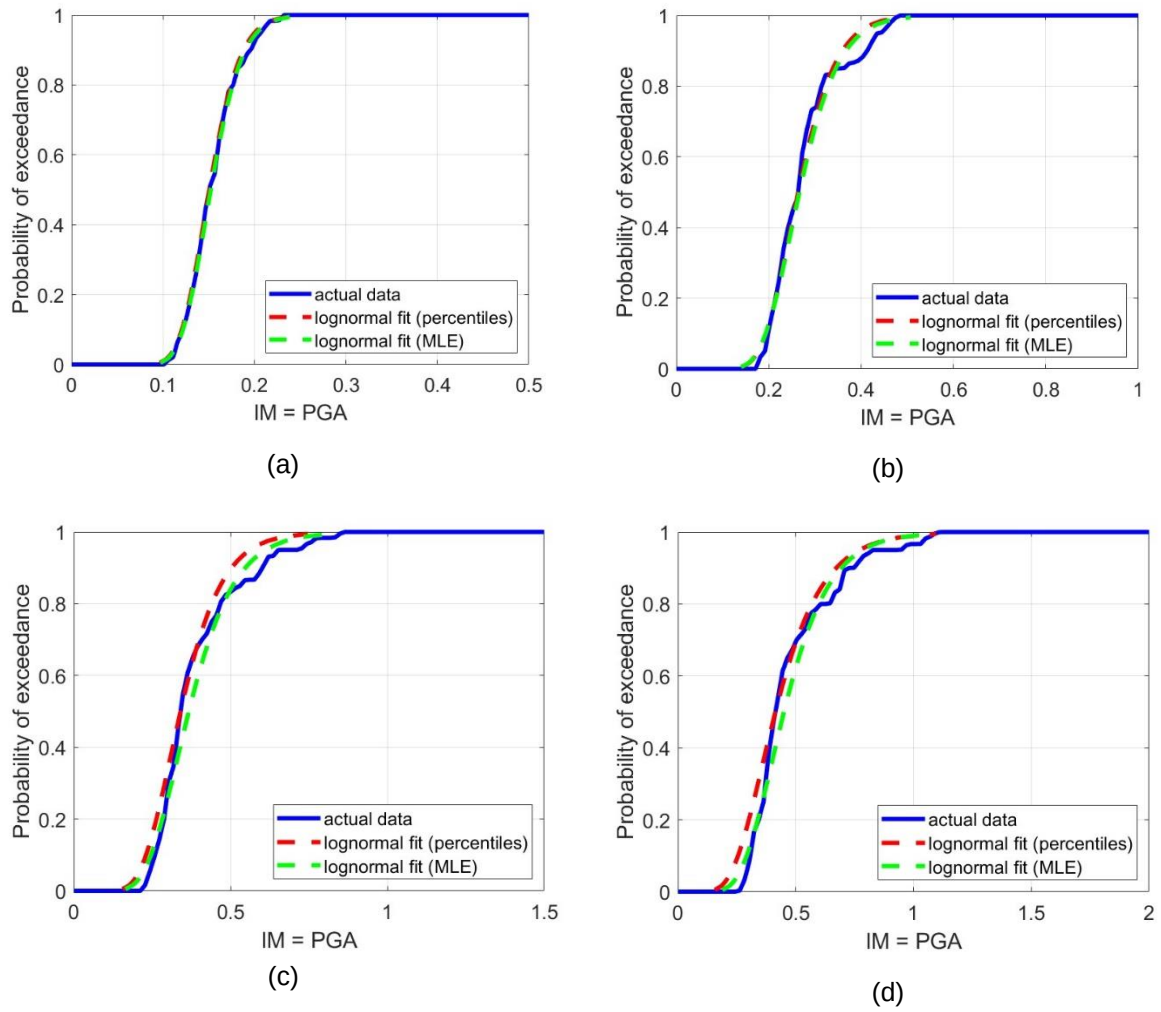


Figure C.4. Fragility Curves derived from IDA for typology MUR-STRUB_LWAL-DNO_H2 (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

ANNEX C.2

FRAGILITY CURVES OF MDOFs RESULTED FROM IDA

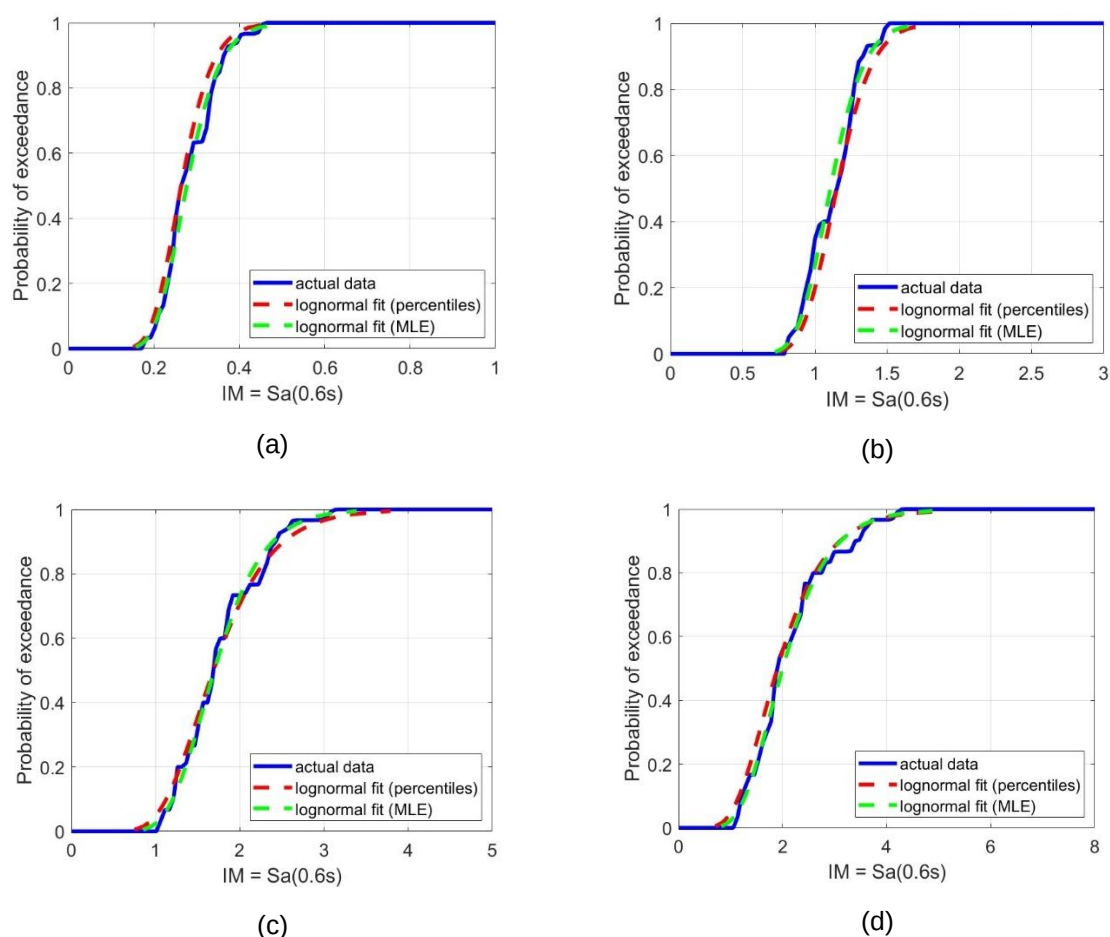
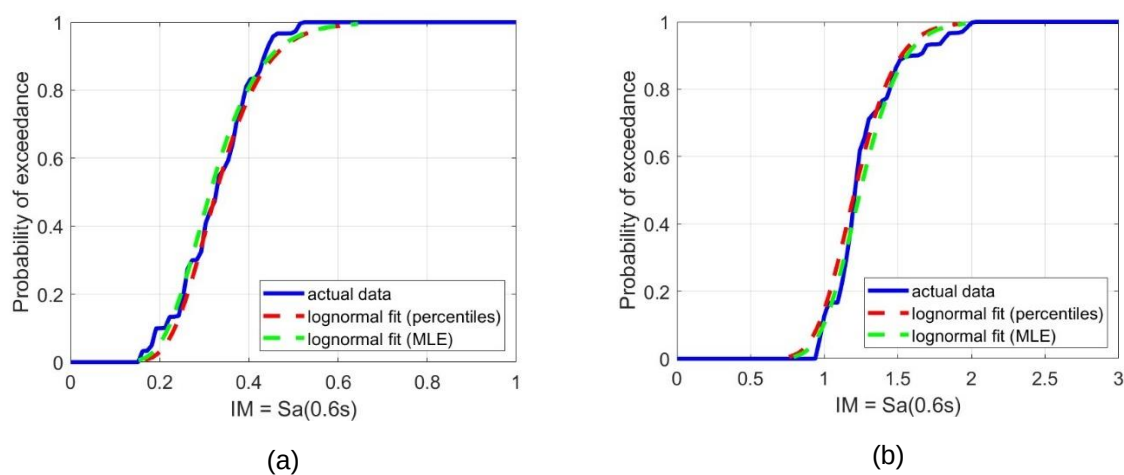


Figure C.1. Fragility Curves derived from IDA for a Square plan-shape MDOF for Case 1 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



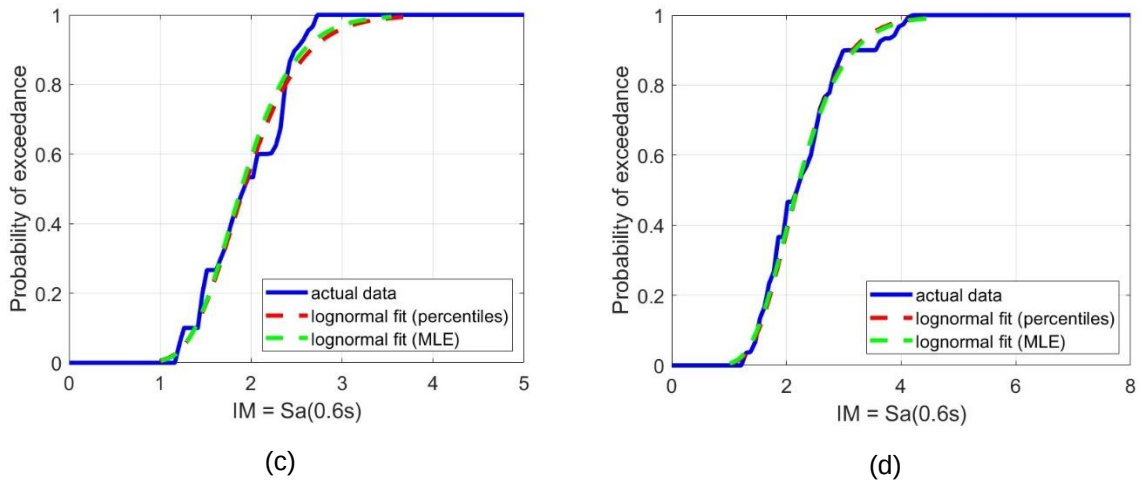


Figure C.2. Fragility Curves derived from IDA for a Rectangle plan-shape MDOF for Case 1 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

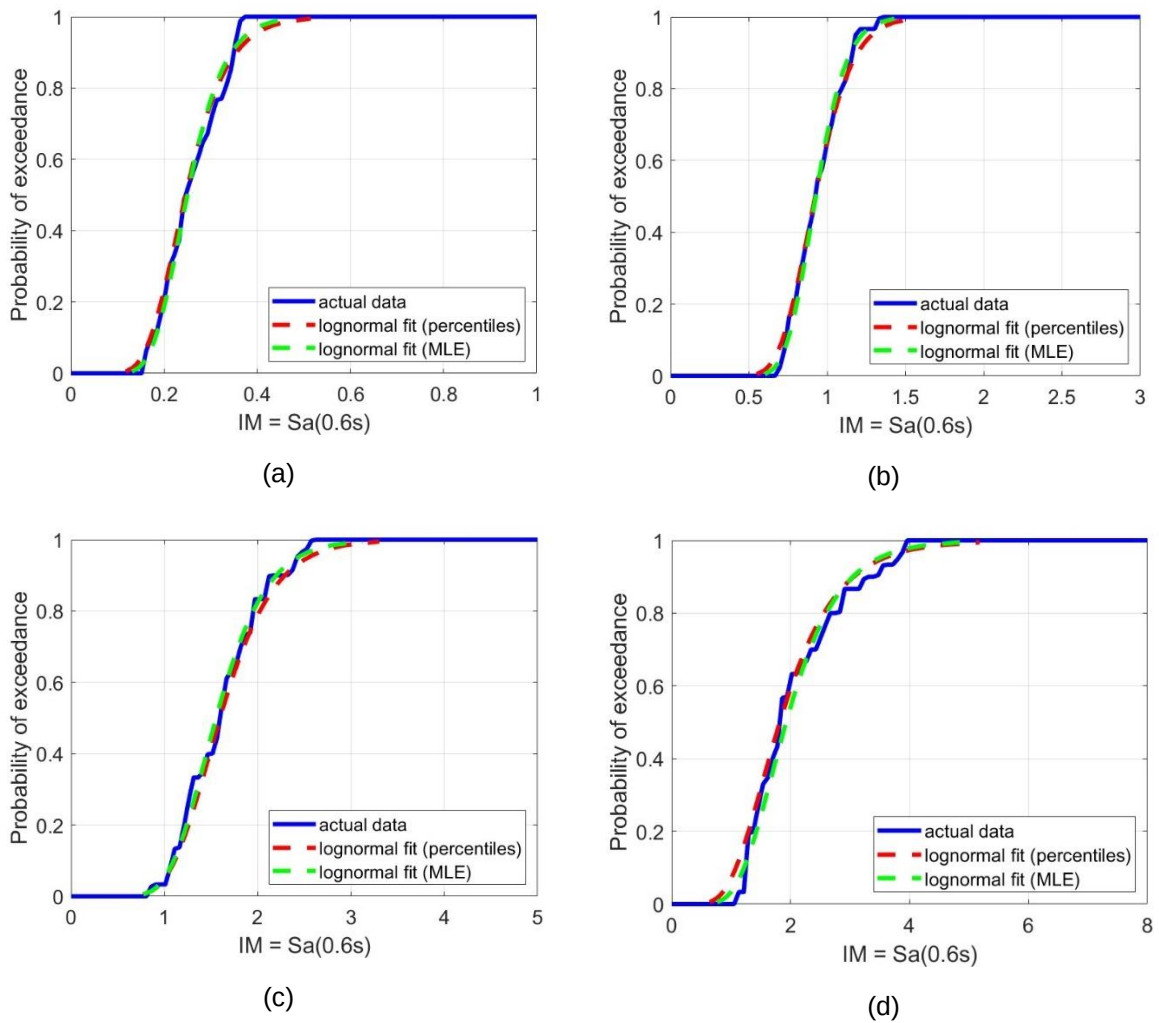


Figure C.3. Fragility Curves derived from IDA for an L plan-shape MDOF for Case 1 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

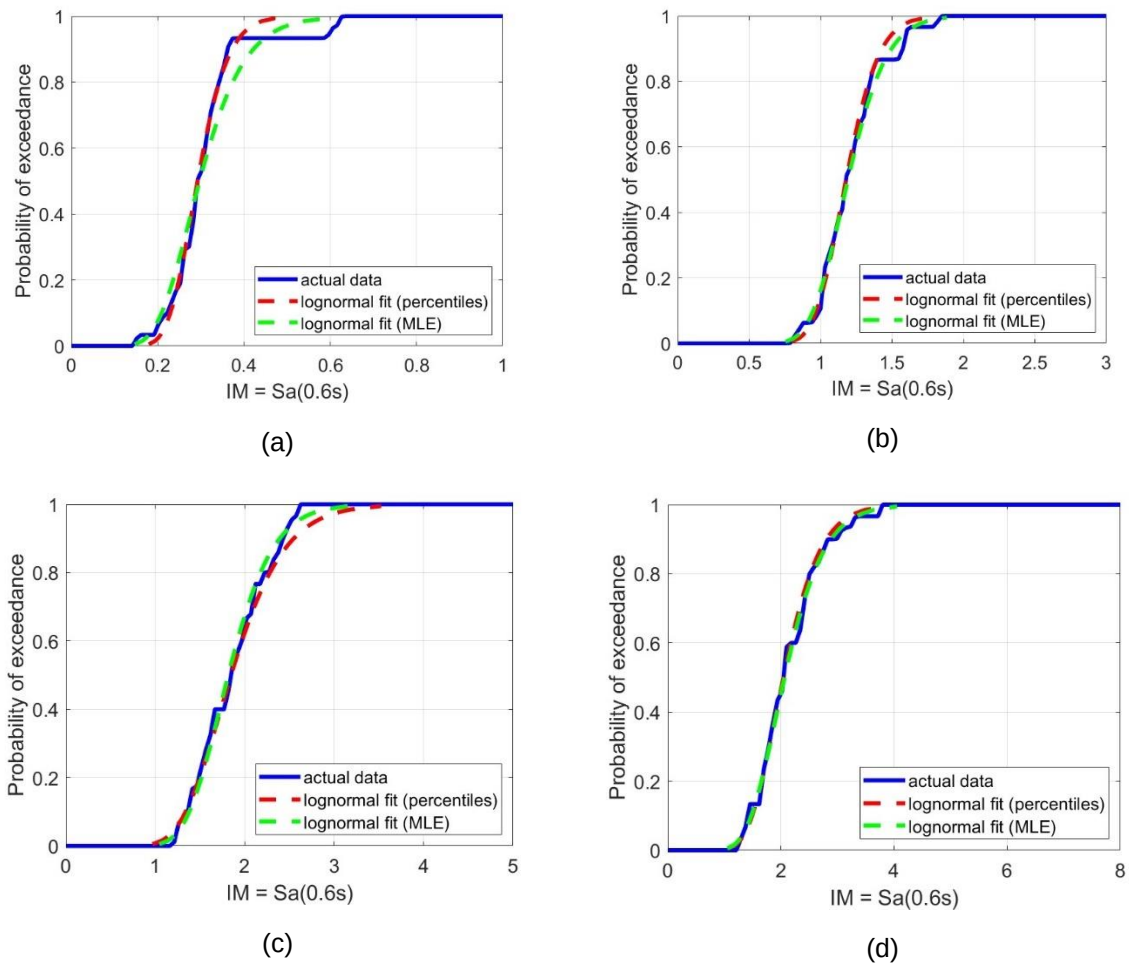
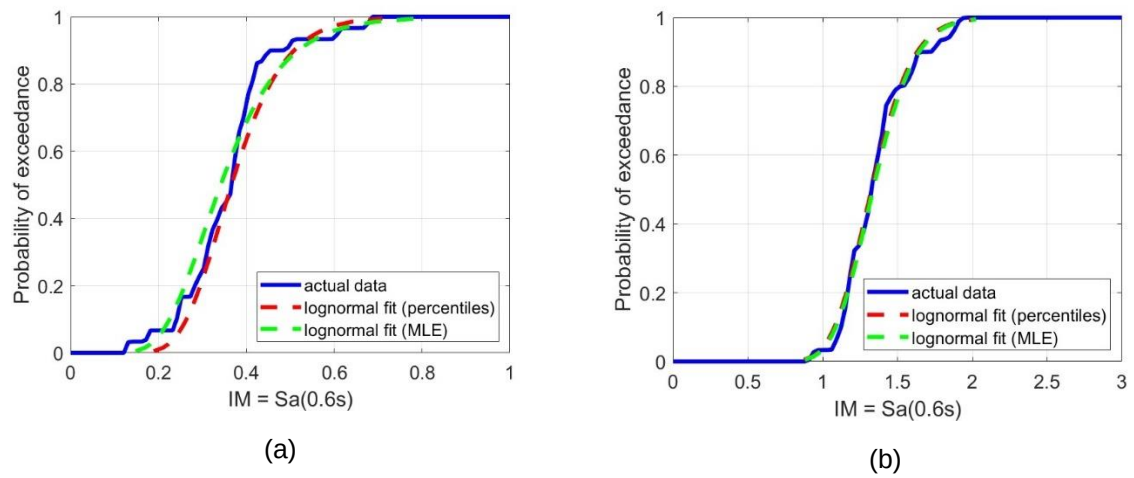


Figure C.4. Fragility Curves derived from IDA for a Square plan-shape MDOF for Case 1 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



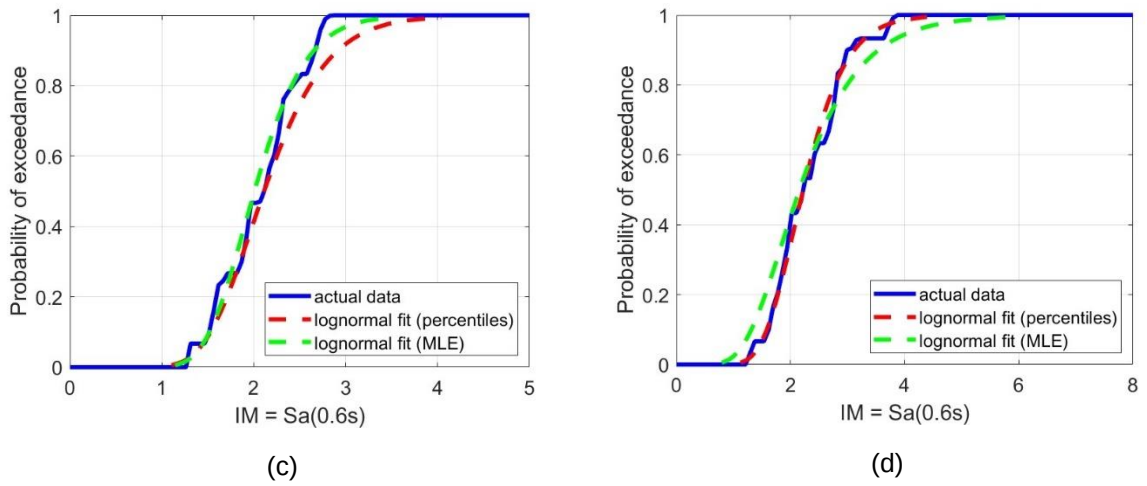


Figure C.5. Fragility Curves derived from IDA for a Rectangle plan-shape MDOF for Case 1 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

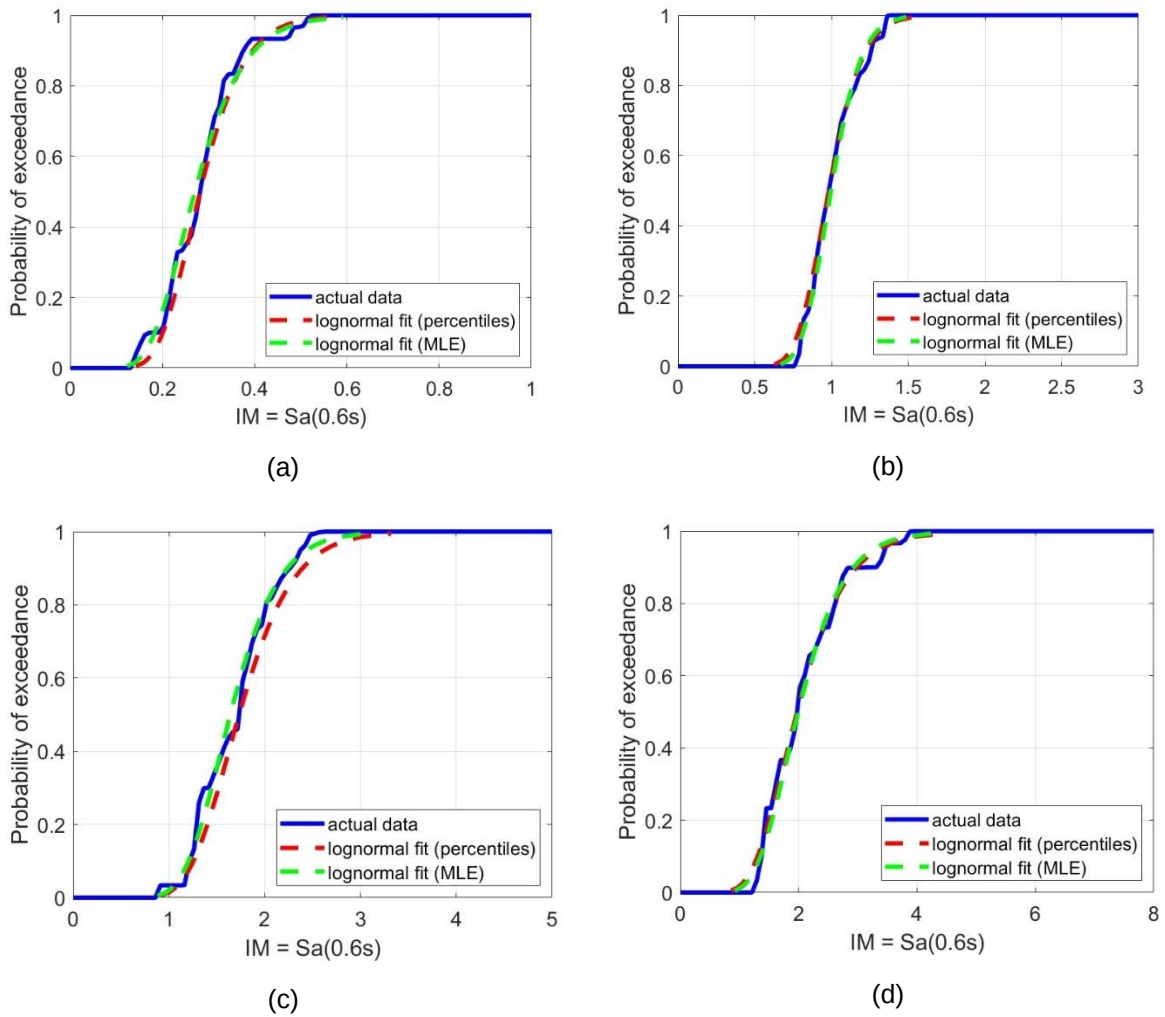


Figure C.6. Fragility Curves derived from IDA for an L plan-shape MDOF for Case 1 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

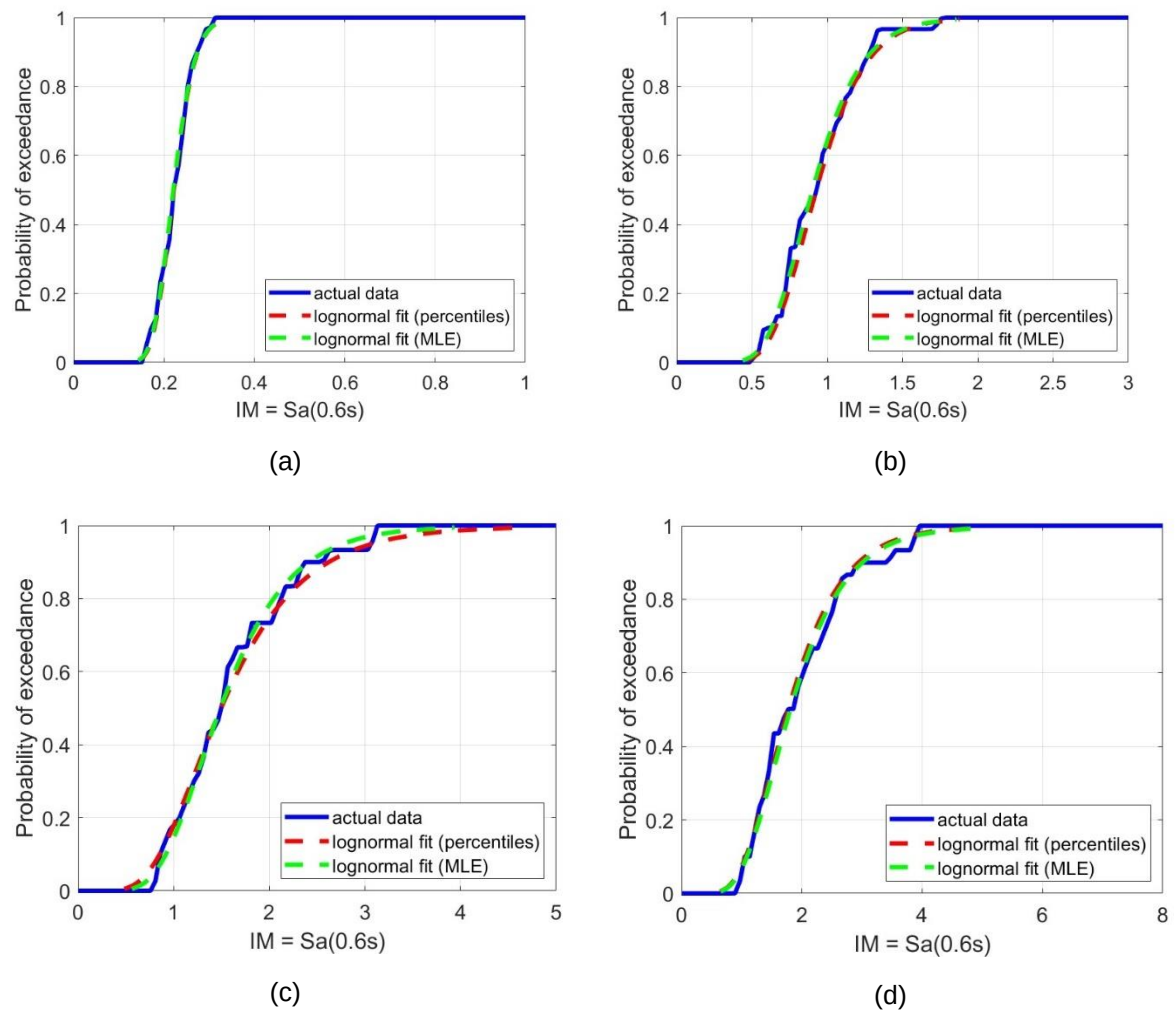
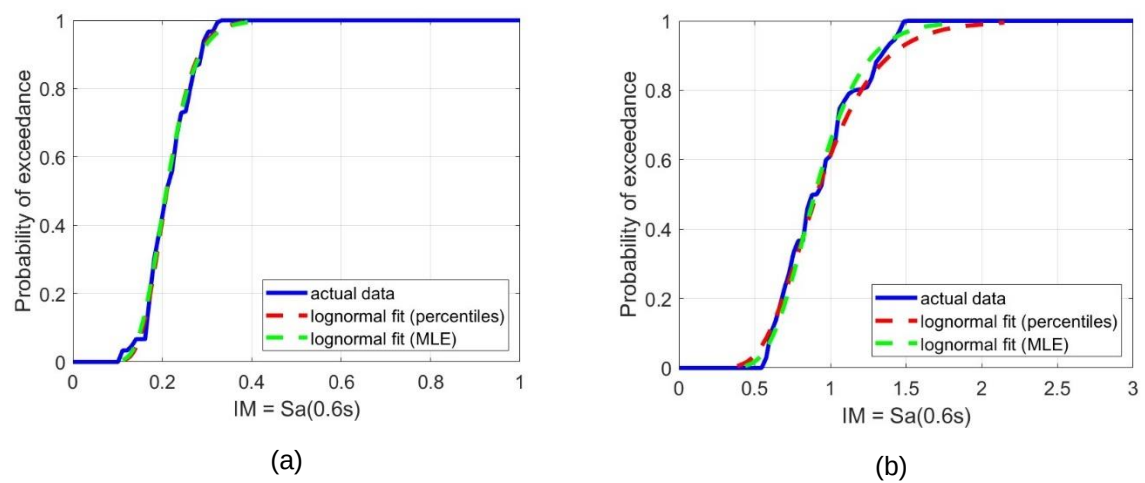


Figure C.7. Fragility Curves derived from IDA for a Square plan-shape MDOF for Case 2: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



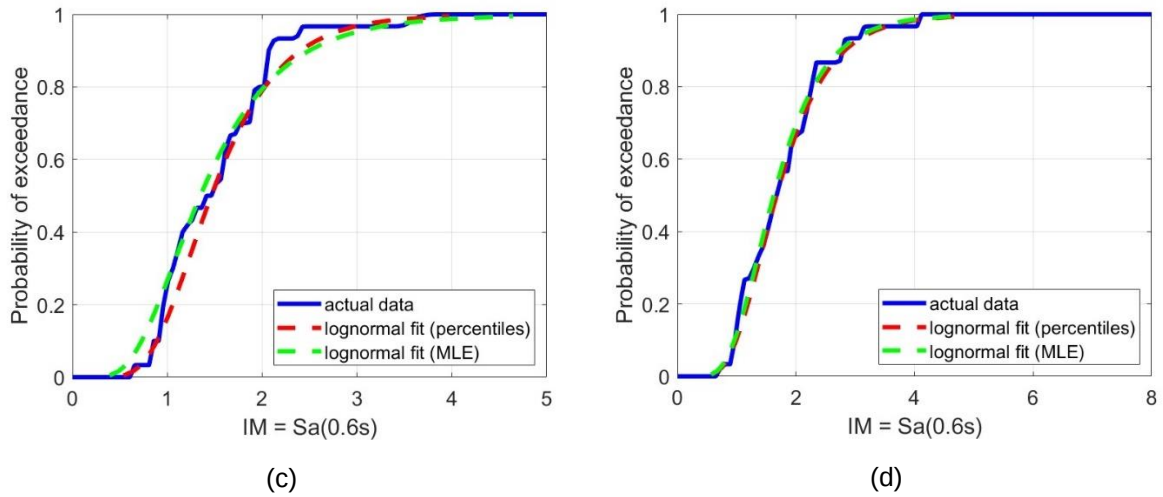


Figure C.8. Fragility Curves derived from IDA for a Rectangle plan-shape MDOF for Case 2: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

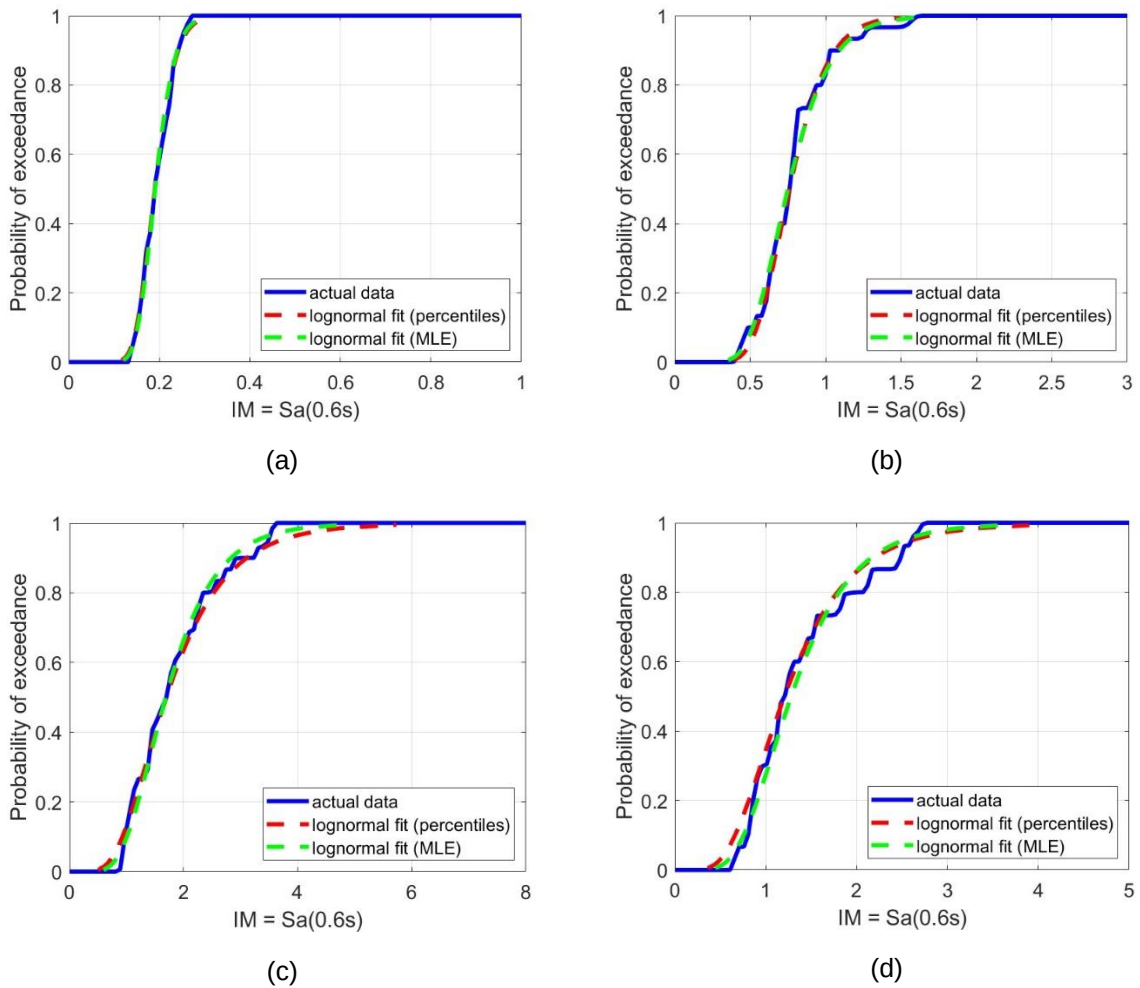


Figure C.9. Fragility Curves derived from IDA for a L plan-shape MDOF for Case 2: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

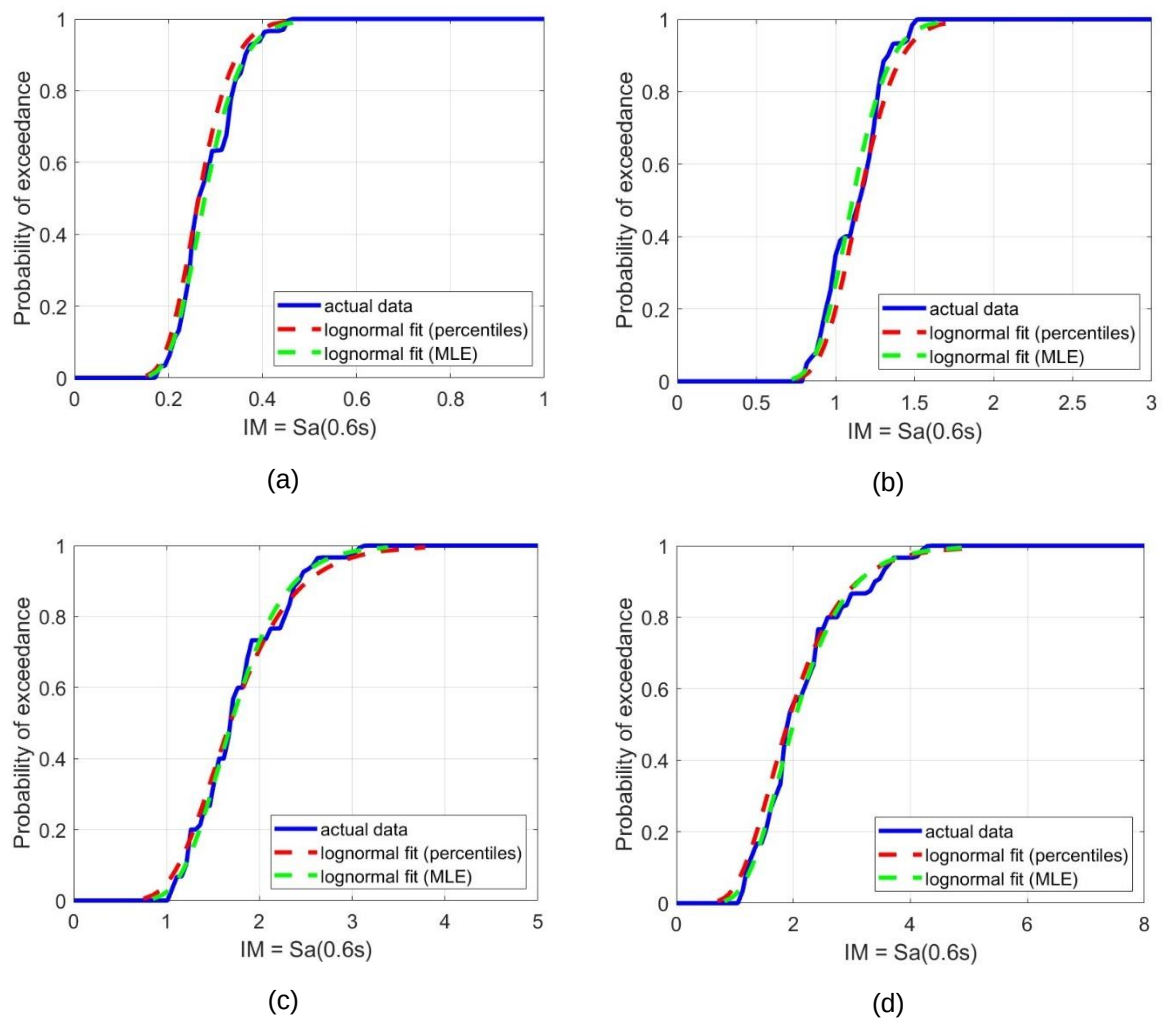
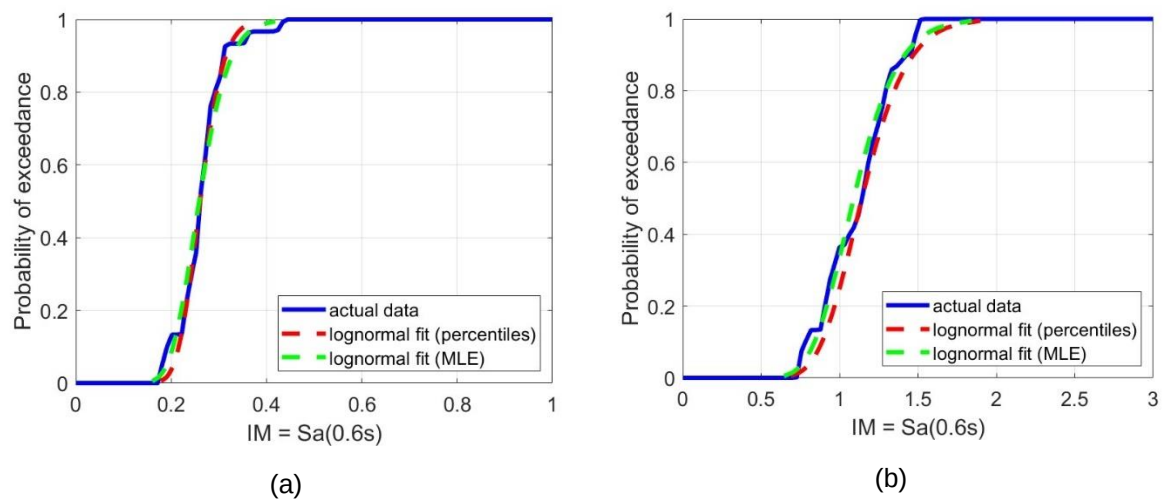


Figure C.10. Fragility Curves derived from IDA for a Square plan-shape MDOF for Case 3 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



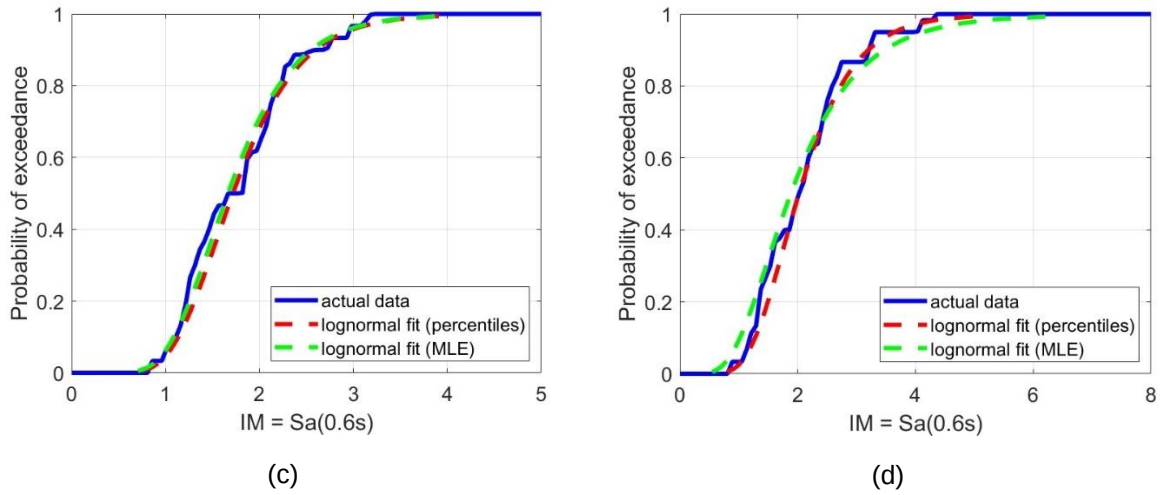


Figure C.11. Fragility Curves derived from IDA for a Rectangle plan-shape MDOF for Case 3 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

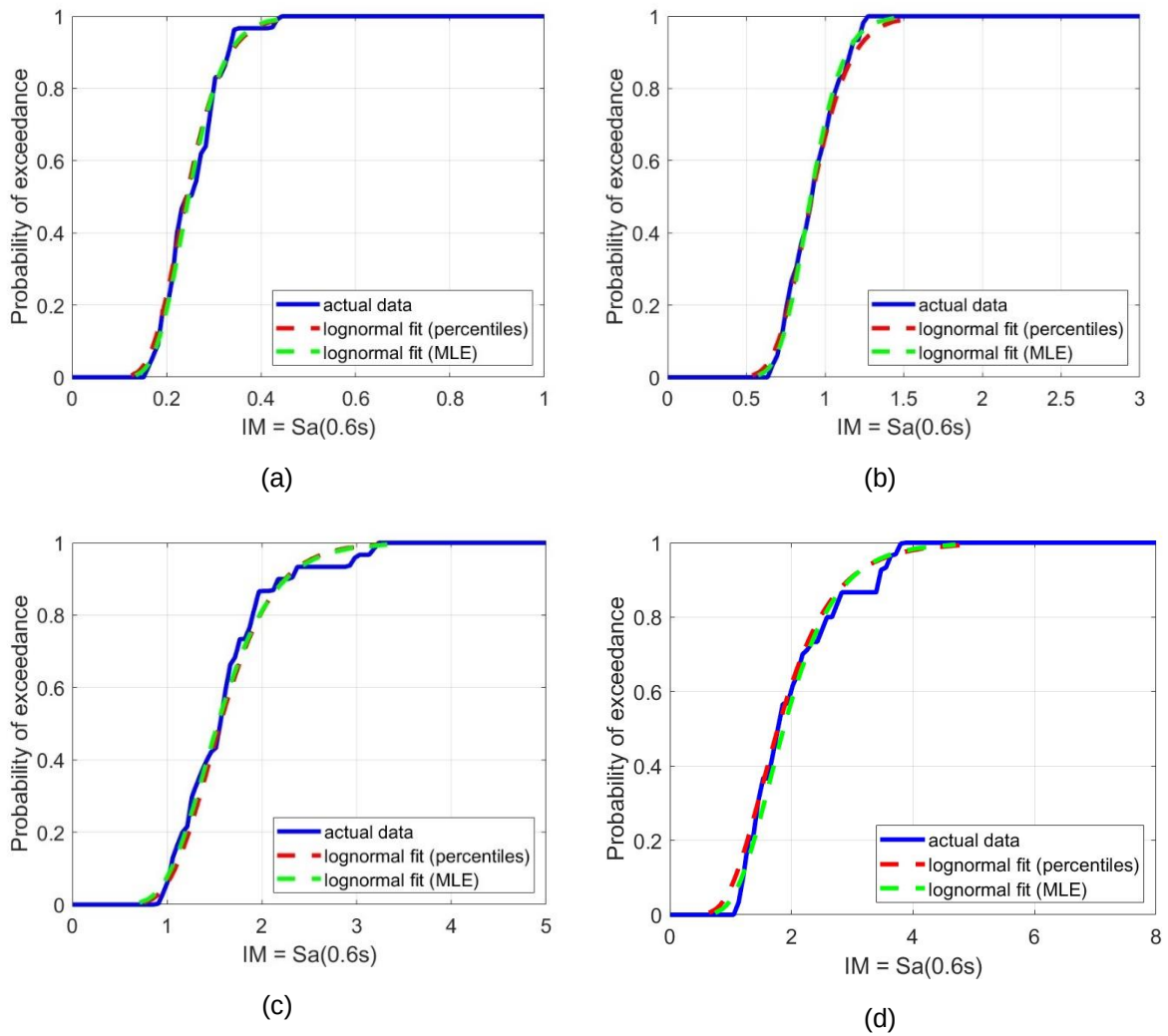


Figure C.12. Fragility Curves derived from IDA for an L plan-shape MDOF for Case 5 with period of 0.53s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

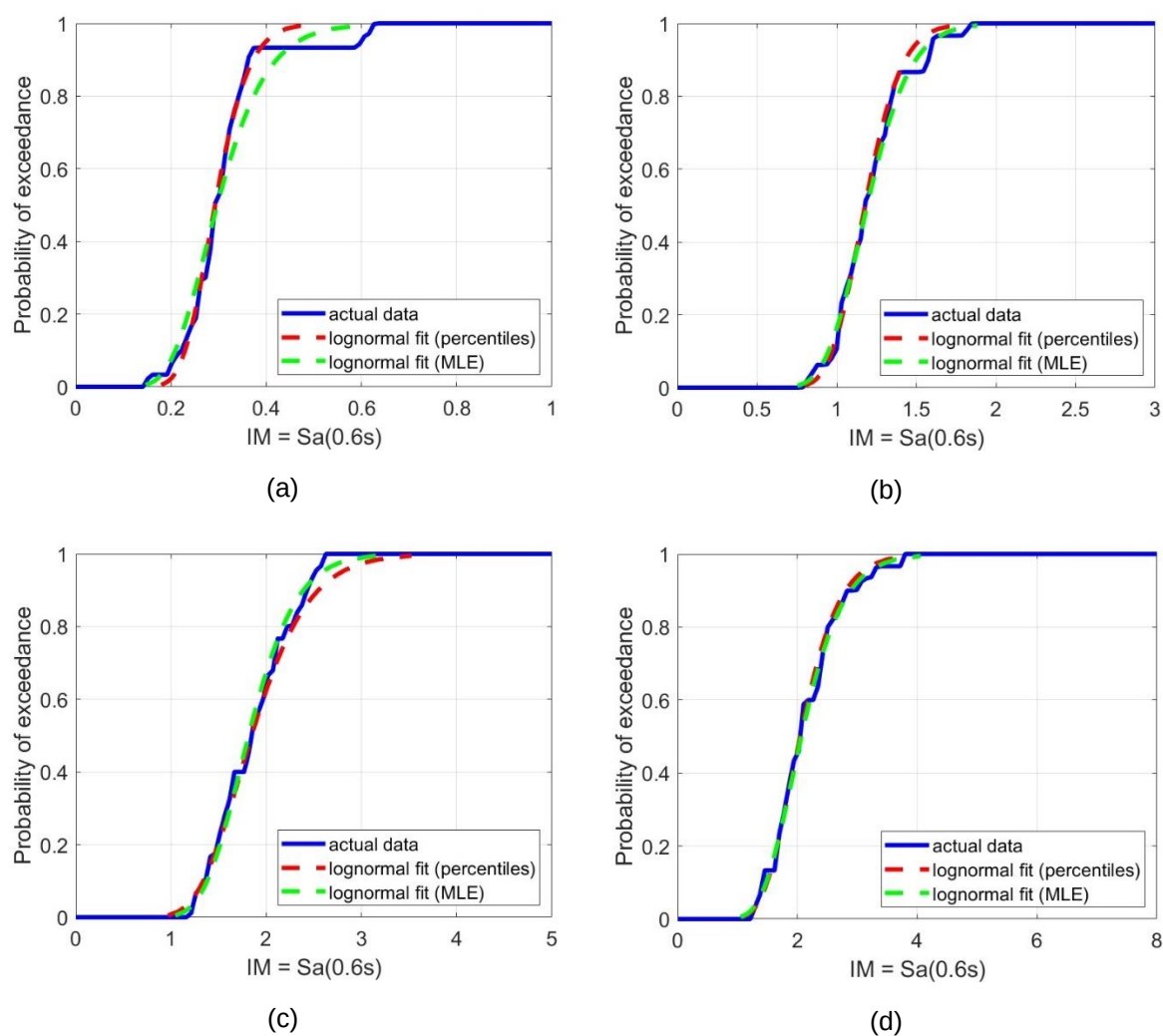
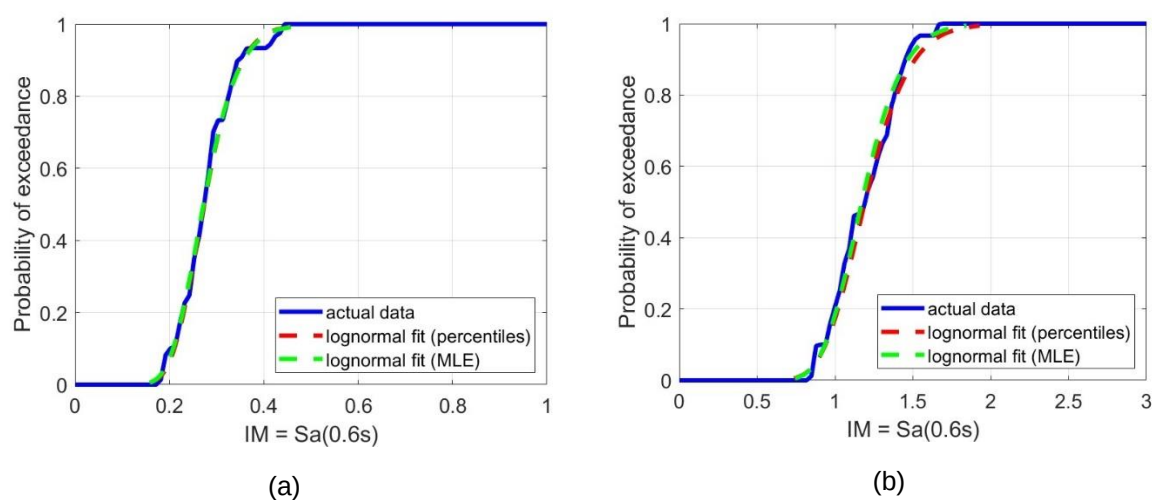


Figure C.13. Fragility Curves derived from IDA for a Square plan-shape MDOF for Case 3 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4



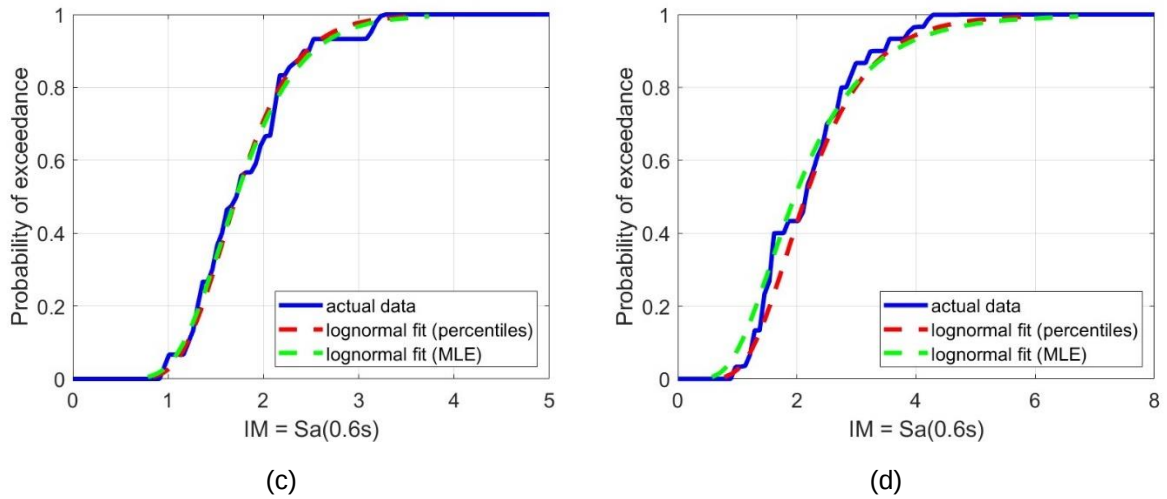


Figure C.14. Fragility Curves derived from IDA for a Rectangle plan-shape MDOF for Case 3 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

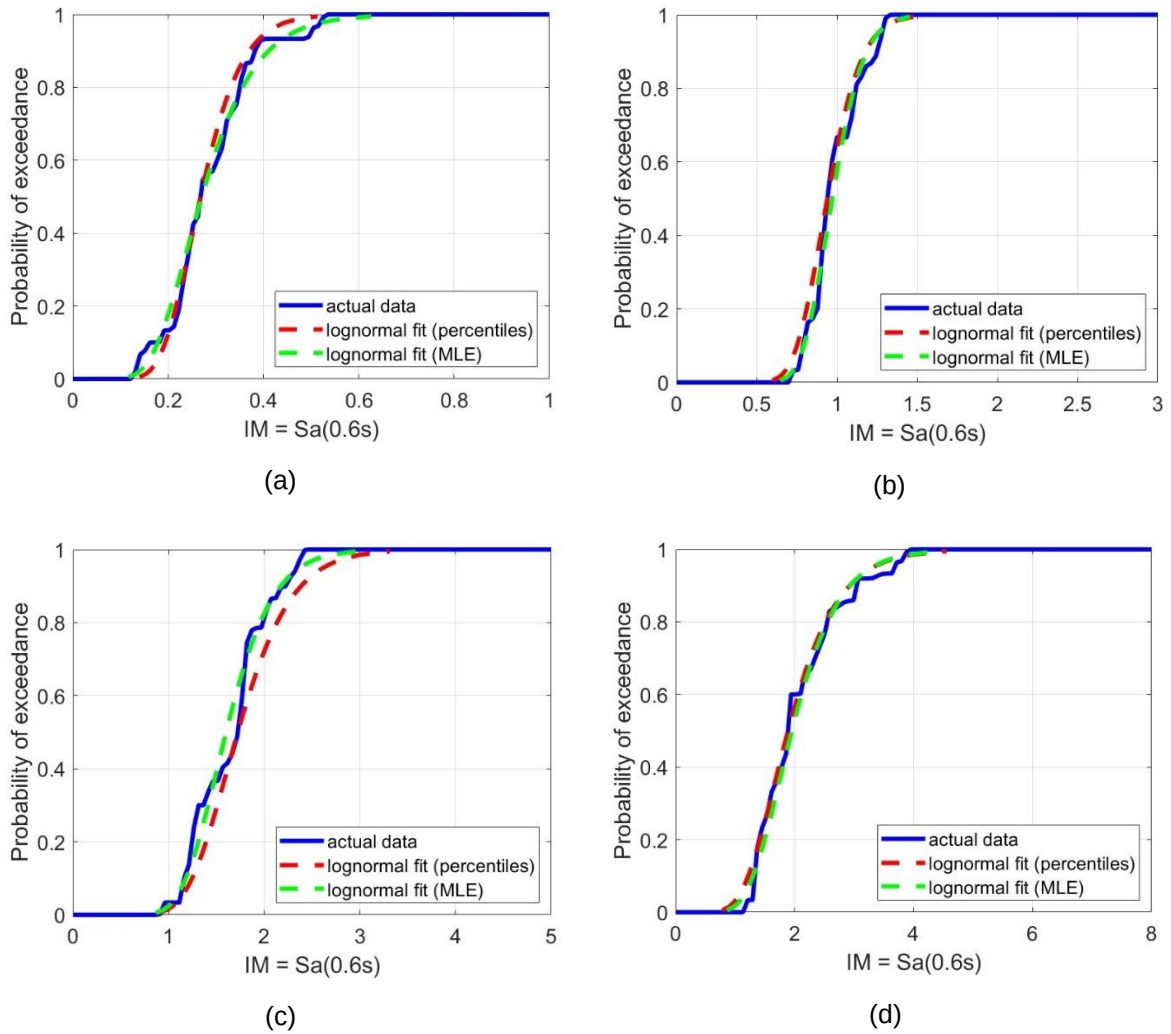


Figure C.15. Fragility Curves derived from IDA for an L plan-shape MDOF for Case 3 with period of 0.50s: (a) Damage state 1; (b) Damage state 2; (c) Damage state 3; (d) Damage state 4

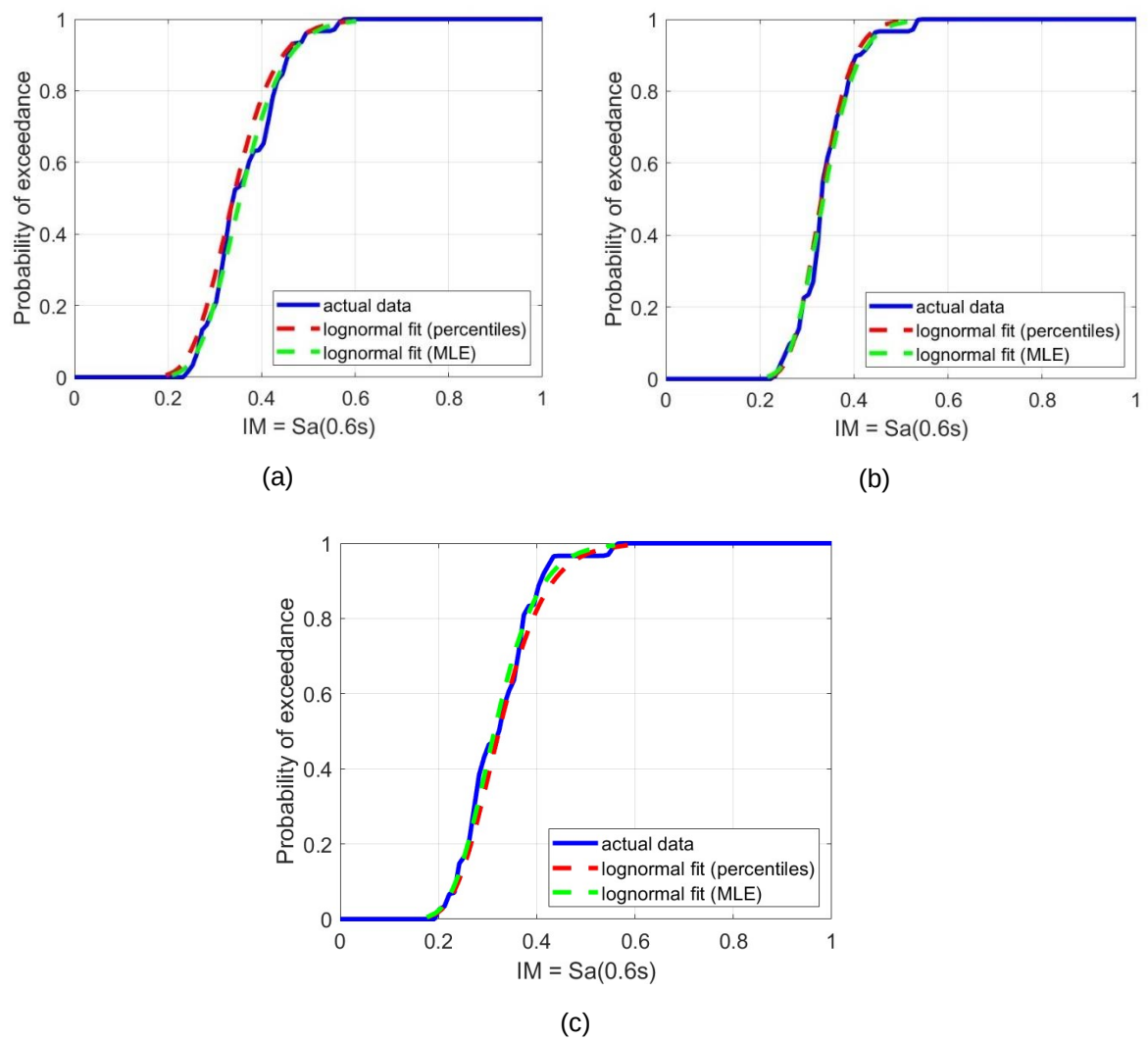
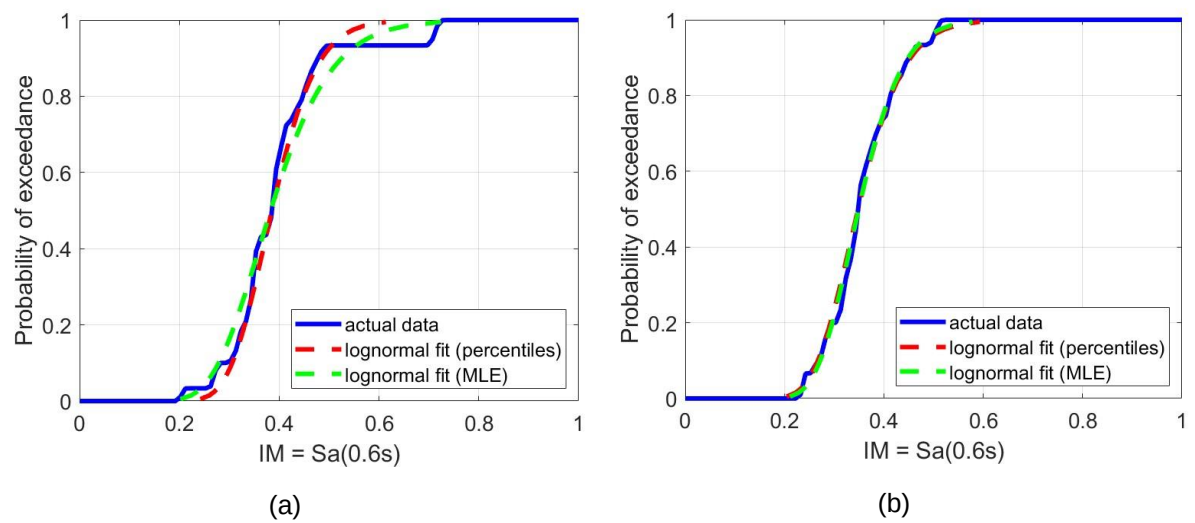


Figure C.16. Fragility Curves derived from IDA for MDOFs following case 3 for a period of 0.53s but for Damage State with threshold of 1% of IDR: (a) Square plan-shape; (b) Rectangle plan-shape; (c) L plan-shape



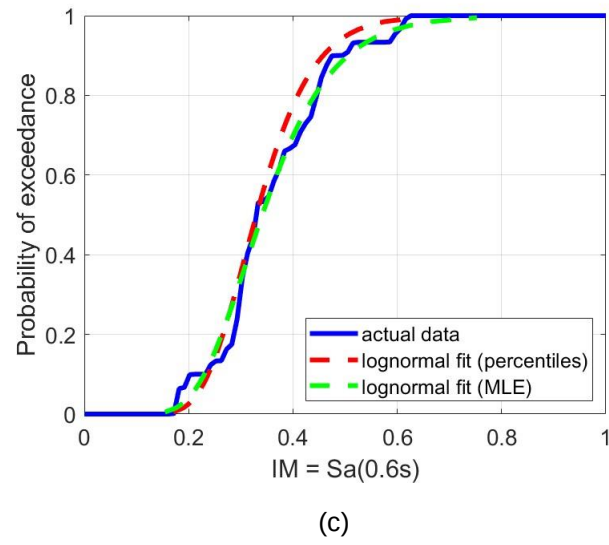


Figure C.17. Fragility Curves derived from IDA for MDOFs following case 3 for a period of 0.50s but for Damage State with threshold of 1% of IDR: (a) Square plan-shape; (b) Rectangle plan-shape; (c) L plan-shape