

**DAMAGE FRAGILITY ASSESSMENT OF LOW-RISE
MASONRY AND REINFORCED CONCRETE BUILDINGS
UNDER FLOODS AND FLOW-TYPE LANDSLIDES**



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by
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To my family



DECLARATION OF ORIGINALITY

I hereby declare that I am the sole author of this thesis and that this is the true copy of my thesis, including the final revisions, approved by my thesis committee. All data and information have been obtained, produced, and presented in accordance with the rules of research ethics and principles of academic honesty. As required by these rules, to the best of my knowledge I have acknowledged ideas, thoughts, and any copyrighted material in accordance with the standard referencing rules. I certify that any part of this thesis has not been submitted for a degree or diploma in another educational institution.

Yasamin Moaiyedfar

ABSTRACT

This study investigates the damage potential of low-rise buildings exposed to floods and flow-type landslides, which are typically triggered by heavy rainfall and often involve high-density flows and impact loads from floating debris. Unreinforced masonry (URM) buildings and low-rise reinforced concrete (RC) frames, which are prevalent throughout European cities, are particularly vulnerable to these hazards. The study first examines the collapse potential of URM buildings, which tend to collapse completely due to the failure of load-bearing walls under extreme lateral flow loads. It then evaluates and compares the damage potential of two RC frame design categories, earthquake-resistant and gravity load-designed structures, using rotation- and drift-based damage state definitions. This assessment is conducted by developing damage fragility surface curves, generated through incremental flow analyses on validated structural models, followed by modeling these curves using logistic regressions. These curves incorporate key uncertainties in both hazard and structural modeling, thereby providing reliable predictions for diverse flow conditions. In addition to offering a robust framework for predicting structural responses and damage levels under extreme flow conditions, the proposed fragility models provide critical insights for various stakeholders aiming to optimize design and retrofitting strategies to reduce damage in low-rise buildings subjected to flood and flow-induced landslide hazards.

Keywords: Low-rise buildings, floods, flow-type landslides, damage, structural performance

ÖZET

Bu çalışma, genellikle şiddetli yağışların tetiklediği ve yüksek yoğunluklu akışlar ile yüzen enkazların neden olduğu darbe yüklerini içeren sel ve akış tipi heyelanlara maruz kalan az katlı binaların hasar potansiyelini araştırmaktadır. Avrupa şehirlerinde yaygın olarak bulunan donatısız yığma (URM) yapılar ile az katlı betonarme (RC) çerçeve yapılar bu tehlikelere karşı özellikle savunmasızdır. Çalışma öncelikle, taşıyıcı duvarların aşırı yatay akış yükleri altında çökmesi nedeniyle tamamen yıkılma eğiliminde olan yığma binaların çökme potansiyelini inceler. Ardından, iki farklı betonarme çerçeve tasarım kategorisi, deprem dayanımlı ve sadece yerçekimi yüklerine göre tasarlanmış yapılar, için döndürme ve deplasman temelli hasar durumu tanımları kullanılarak hasar potansiyeli değerlendirilir ve karşılaştırılır. Bu değerlendirme, doğrulanmış yapısal modeller üzerinde gerçekleştirilen artımsal akış analizleri ile hasar kırılma yüzey eğrilerinin oluşturulması ve ardından bu eğrilerin lojistik regresyonlarla modellenmesi yoluyla yürütülür. Söz konusu eğriler, tehlike ve yapısal modelleme kaynaklı temel belirsizlikleri barındırarak çeşitli akış koşulları için güvenilir tahminler sunar. Aşırı akış koşulları altında yapısal tepkileri ve hasar düzeylerini öngörmek için sağlam bir çerçeve sunmanın yanı sıra, önerilen kırılma modelleri sel ve akış kaynaklı heyelan tehlikelerine maruz kalan az katlı binalarda hasarı azaltmaya yönelik tasarım ve güçlendirme stratejilerini optimize etmeyi amaçlayan çeşitli paydaşlar için kritik içgörüler sağlamaktadır.

Anahtar Kelimeler: Alçak katlı binalar, seller, akış tipi heyelanlar, hasar, yapısal performans

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LIST OF ACRONYMS AND ABBREVIATIONS

3S3B	Three-Story Three-Bay
CDP	Concrete Damaged Plasticity
COV	Coefficients Of Variation
CP	Collapse Prevention
DL	Damage Limitation
DSs	Damage States
EQR	Earthquake-Resistant
FE	Finite Element
FN	False Negative
FP	False Positive
FPR	False Positive Rate
GLD	Gravity-Load-Designed
IFA	Incremental Flow Analysis
IMK	Ibarra–Medina–Krawinkler
IO	Immediate Occupancy
LL	Live Load
LS	Life Safety
NC	Near Collapse
RC	Reinforced Concrete
SD	Significant Damage

TN	True Negative
TP	True Positive
TPR	True Positive Rate
URM	Unreinforced Masonry
YLA	Yield Line Analysis



1. INTRODUCTION

As a result of global warming and climate change, natural hazards such as floods and flow-type landslides have been occurring with increased frequency and intensity, significantly increasing the risk of damage and even collapse of buildings and critical infrastructure due to high velocity, high flow density, and floating debris. The destructive potential of such events has been demonstrated by recent incidents in Şanlıurfa and Adıyaman, Türkiye (March 2023) [1], where numerous Unreinforced Masonry (URM) and Reinforced Concrete (RC) buildings experienced severe damage or collapse. These events have underscored the vulnerability of the built environment under flow-hazard scenarios and the need for systematic methods to evaluate structural damage fragility under such circumstances.

To better understand and mitigate flood and flow-type landslide risks, fragility curves have emerged as critical tools in probabilistic risk assessment frameworks. Fragility curves quantify the probability of a structure reaching or exceeding specific damage states as a function of hazard intensity. Unlike deterministic evaluations, fragility-based approaches explicitly incorporate uncertainties related to structural properties, hazard characteristics, and modeling assumptions. This is especially important for older and non-code-compliant structures, which often lack sufficient lateral strength and may exhibit semi-brittle or unstable collapse mechanisms under lateral loads induced by flow and debris impacts.

Chapter 2 of this thesis presents a collapse fragility assessment for low-rise URM buildings subjected to flood and flow-type landslide actions. As a widely used construction typology in Europe and other parts of the world, URM buildings are especially susceptible to failure when one or more load-bearing walls are exposed to strong lateral flow forces. This chapter adopts a probabilistic modeling framework that integrates validated

Finite Element (FE) simulations of URM load-bearing walls under hydrostatic, hydrodynamic, and debris impact loads. For a representative set of 14 one-story and two-story URM building configurations, computational models are developed based on real-world case studies in a finite element program, Abaqus [2], and incremental flow analyses are carried out on these models to simulate collapse mechanisms of the selected walls under increasing hazard intensities. The collapse mechanism is considered to form when the base shear capacity of the walls is exceeded by the applied flow load demands, which is a common assumption in the literature for non-ductile structures. Using logistic regression applied to collapse simulation data coming from parametric studies (totally, 60,760 data points), fragility curves are derived that reflect the combined influence of key variables such as flow depth, flow velocity, debris mass, and wall boundary conditions. The modeling approach used in fragility curves considers the key factors that govern the collapse performance of walls and their relevant significant uncertainties, therefore offering a practical yet reliable methodology for collapse prediction. This chapter lastly highlights the benefits of the new collapse fragility curves for URM buildings exposed to flow-induced hazards by comparing the results with overly simplified or deterministic methods found in the literature.

Chapter 3 presents flow damage fragility assessment models for 8 representative cases of low-rise reinforced concrete (RC) frame buildings, another prevalent typology in urban and rural areas. RC frames are generally assumed to be more resistant to lateral loading; however, significant vulnerability remains, particularly for gravity load-designed structures lacking proper seismic detailing. This chapter proposes fragility assessment models for RC buildings under the same types of multiple flow-induced hazard scenarios, including floods and flow-type landslides. The fragility analyses conducted within this chapter contrast two structural categories: earthquake-resistant frames designed with

high-ductility and gravity load-designed frames representing older or substandard construction practices. The damage criteria considered in the fragility analyses align with both FEMA 356 [3] and Eurocode 8-Part 3 [4] guidelines, enabling internationally relevant and code-consistent damage assessments. Damage simulation data are developed through nonlinear static (pushover) analyses on the computational models built on a finite element program, OpenSees [5], providing a more realistic evaluation of damage progression under extreme flow scenarios. Fragility surface models are then created using the logistic regression analysis tool on the damage simulation data coming from parametric studies (totally, 1.2×10^6 data points), in consistent with the methodology proposed for Chapter 2. Different damage thresholds from international codes are considered in developing the fragility models, providing a broader perspective on vulnerability. Lastly, several comparisons of the developed fragility models for earthquake-resistant and gravity-load-designed RC frames are performed to underscore the importance of proper lateral design for withstanding flow loads, as well as to highlight their advantages over existing models in the literature.

The fragility models presented in Chapters 2 and 3 address multiple research gaps identified in the literature. Most existing fragility studies for buildings either focus on individual hazards (e.g., only floods) or neglect the interaction between flow velocity and debris mass for flow hazard events. This study overcomes these limitations by considering different yet commonly encountered flow hazard load actions (i.e., hydrostatic and hydrodynamic pressures under different flow densities, and impact loads of floating debris) and generating fragility models presented both as surfaces as well as functional equations, thus offering clearer insights into structural performance under various flow scenarios.

The fragility models developed in this thesis offer substantial value for researchers, engineers, planners, and policymakers. By providing reliable, uncertainty-aware predictions of collapse and damage probabilities for both URM and RC building structures,

this research contributes toward national and international objectives for hazard mitigation, particularly in regions with aging infrastructure and increasing exposure to extreme weather events. Ultimately, integrating advanced numerical modeling and probabilistic analysis advances the state of knowledge on structural vulnerability to flow-induced hazards and promotes the development of safer, more resilient built environments.



2. COLLAPSE FRAGILITY MODELS FOR UNREINFORCED MASONRY BUILDINGS UNDER FLOOD AND FLOW-TYPE LANDSLIDE ACTIONS

¹ Recent lessons from heavy rainfall-induced flood flows and flow-type landslides have demonstrated that buildings can sustain significant damage or even collapse at higher flow intensities. Unreinforced masonry (URM) buildings are particularly vulnerable, as the failure of one load-bearing wall directly exposed to flow actions may lead to the complete collapse of the structure. This underscores the urgent need to better understand the flow vulnerability of these structures. This study thus conducts a probabilistic structural collapse assessment of vulnerable URM buildings under flood flow and flow-type landslide actions using a proposed incremental flow analysis approach. After selecting two case studies from the literature, finite element (FE) models were first built and subjected to incremental flow analyses to assess their collapse levels, which were then validated against the reported results. Next, these validated modeling and analysis approaches were used to create a set of FE case models representative of typical one- or two-story URM building stocks in Europe. Comprehensive parametric simulation studies were then conducted on these case models to investigate how variations in key hazard and structural parameters influence their collapse performance under different types of physical flow actions, including hydrostatic, hydrodynamic, and debris-impact loads. Using a logistic regression approach on the simulation data, collapse fragility models were subsequently developed. The proposed fragility models are based on validated FE modeling and analysis approaches, which consider different flood or flow-type landslide scenarios while systematically incorporating associated uncertainties; therefore, they can provide reliable

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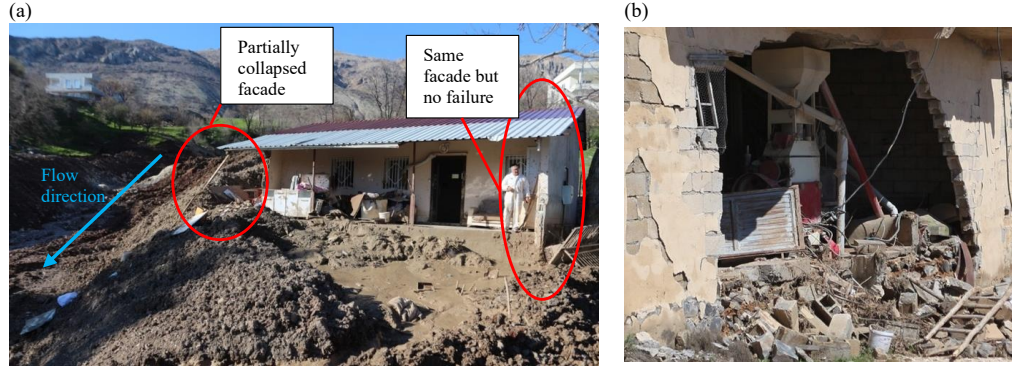
predictions of collapse performance for URM buildings under flow actions.

2.1 Introduction

Worldwide, heavy rainfalls have become more frequent under increasing climate change, often resulting in devastating flash floods or flow-type landslides. Compared to flood flows, flow-type landslides are typically described as the swift movement of a mixture of fine-grained or loose granular soil, rock, or organic matter with a significant amount in addition to water [6, 7]. Floods and flow-type landslides both have the potential to seriously damage or even bring down vulnerable structures [8, 9]. This potential is especially significant for unreinforced masonry (URM) buildings with insufficient or no lateral design, which still account for the majority of European housing stock [1, 10, 11, 12, 13]. Figure 2.1 shows examples of failures from recent catastrophic flow-type landslides and flooding incidents in the Turkish provinces of Sanliurfa and Adiyaman that occurred as a result of severe rains in March 2023. During the incidents, several URM buildings substantially collapsed on the side facing the flow direction (see Figure 2.1a), and the masonry facades of many framed buildings were severely damaged (see Figure 2.1b), highlighting how URM buildings and infill walls can be highly susceptible to flow actions. In most cases, the primary flow actions responsible for these failures have been identified as hydrostatic and hydrodynamic pressures, as well as impact loads from debris carried by the flow [14].

One of the key design goals of modern building code regulations is collapse prevention, ensuring an acceptable low probability of structural collapse under the most severe hazard loads evaluated throughout the building's design phase. To accurately and reliably assess the risk of structural collapse, a probabilistic evaluation is required by taking into account relevant significant uncertainties. The collapse fragility model is the key outcome of this evaluation that measures the collapse probability of a structure. By

Figure 2.1: Examples of the effects of a flow-type landslide event and a flooding event on masonry buildings, respectively: a) a URM building partially collapsed on one side [1] and b) partially collapsed infill walls due to debris impacts [1].



calculating the likelihood that the "hazard demand" imposed on the structure exceeds its "hazard capacity," collapse fragility curves show the conditional probability of collapse at a given hazard level. Several studies, particularly for URM structures, have been published in the literature to quantify the uncertainties in structural capacity and demand in order to calculate fragilities under specific hazard loads. However, the majority of these studies focus on seismic hazard events [15, 16]. In contrast, existing URM studies for flow actions [10, 11, 12, 17] primarily use deterministic methodologies and do not comprehensively evaluate the effects of significant uncertainties associated with flow hazards and building features on the structural collapse potential. To address this gap, it is essential to systematically quantify and incorporate these considerable uncertainties into fragility models for flow actions, using appropriate probabilistic approaches.

Assessing numerical building models and performance evaluations, along with identifying and incorporating significant uncertainties, is critical for establishing collapse fragility models. Due to the scarcity of field or experimental data, a variety of numerical models have been used in the literature to characterize the response of URM buildings to flow events, ranging from simple stick models to advanced finite element models. Most of these studies have focused on the out-of-plane failure of vulnerable URM

load-bearing walls, as the failure of one wall can result in the entire collapse of the structure. These studies usually employ simple analytical approaches, such as elastic beam analysis [10, 18, 19, 20] and yield line analysis [11, 17]. In particular, the yield line approach considers a linear kinematics analysis, modeling walls as rigid blocks with certain pre-assumed crack patterns that lead to failure through overturning or bending collapse mechanisms. This mechanism is evaluated using the virtual work principle, which compares the stabilizing moments provided by the wall's weight and any axial loads applied to the wall against the overturning moments produced by flow actions on the wall. Only a few researchers have used advanced finite element (FE) approaches [12, 21], but they generally assume elastic behavior for the masonry material.

Because of its low tensile strength, masonry material is often assumed to be a no-tension material in most studies [22, 23]. However, depending on the required level of details in the inputs, analyses, and outputs, some researchers consider the influence of tensile strength in masonry in their FE models [12, 24] or in simple elastic beam models [10, 18]. Also, while masonry materials are heterogeneous, the majority of these approaches assume continuous and homogeneous masonry material in wall models [24, 25, 26]. Additionally, masonry materials can be either interlocked or cohesive [27]. While masonry codes and studies in the literature mostly assume masonry to be an interlocked material with potential failure mechanisms arising from loss of equilibrium, there are several poorly-constructed masonry structural cases that may fail due to disaggregation (i.e., the masonry assembly breaks up into its components), which is often disregarded in the existing research [27].

Moreover, most of these existing performance models focus on flood load actions, generally evaluating the behavior of URM structures under hydrostatic flow actions caused by flow depth. They rarely consider velocity-driven actions, also known as hydrodynamic effects and impact loads caused by floating debris, such as tree logs [10, 18].

Also, similar to seismic or other hazard studies [15, 24], linear or nonlinear static analysis techniques are widely used in the performance evaluation of URM structures under these flow actions, as they are more efficient for a variety of computational tasks [12, 21, 28]. In these performance analyses, kinematic-based limits (i.e., overturning moments of flow actions exceeding the wall's stabilizing moments) [17, 29], as well as some displacement or force thresholds (e.g., base shear demands exceeding maximum base shear capacity) [12, 21] are frequently used to assess the safety of existing masonry buildings. All these findings indicate that numerical modeling and load analysis methodologies can even introduce uncertainty in predicting the response of masonry structures [15].

Fragility assessments require extensive parametric studies to carefully investigate and include relevant uncertainties in performance calculations, necessitating a delicate balance of computational efforts and modeling assumptions [16, 30, 31]. While advanced numerical approaches are detailed and require significant computational effort for parametric studies used in fragility evaluations, numerical procedures based on simpler assumptions can be practical. However, they may produce more uncertain or biased estimates of actual building capacities under flow actions. Given all these considerations, much work remains to accurately and reliably evaluate the collapse fragilities of vulnerable URM buildings under flood and flow-type landslide actions, while considering the appropriate numerical methods and significant uncertainties.

To address the aforementioned gaps in the literature, this study proposes reliable collapse fragility models for URM buildings subjected to flow actions of floods and flow-type landslides. The models were developed within a demand vs. capacity framework by systematically quantifying significant uncertainties and performing probabilistic analyses on sufficiently detailed yet practical numerical building models under varying flow actions (i.e., hydrostatic, hydrodynamic, and debris impact loads). These fragility models offer a more reliable and comprehensive way for assessing the collapse risk of URM

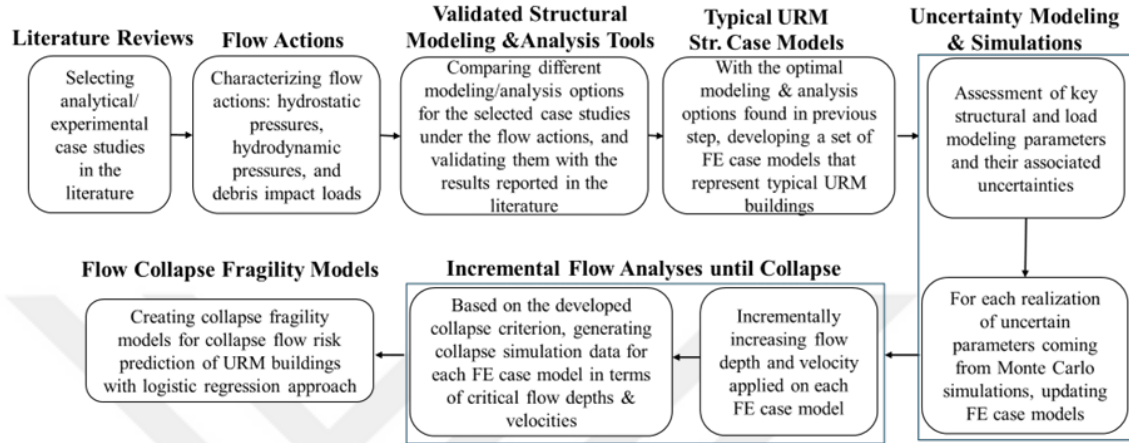
buildings as a function of key flow intensities (i.e., flow depth, velocity), considering the influences of debris mass, flow density, and varying wall boundary conditions. The proposed models thus can support risk-based decision-making by different stakeholders (e.g., engineers, policymakers, governmental agencies, insurance companies, and homeowners) to enhance societal hazard resilience, particularly in developing national construction regulations, regional emergency response plans, and risk management strategies.

2.2 Methodology

Following the probabilistic framework shown in Figure 2.2, this study proposes reliable collapse risk prediction models (i.e., collapse fragility models) for vulnerable URM buildings under flood flows and flow-type landslides. First, a comprehensive literature review was conducted to select a set of experimental and analytical case studies on URM buildings subjected to flow actions. These reviews also supported identifying the key flow actions (i.e., hydrostatic and hydrodynamic effects and debris impacts) essential to understanding how these structures respond under such extreme flow conditions. Detailed FE models were then built for the selected case studies using different modeling options in the FE software program Abaqus [2]. The models were subjected to a proposed incremental analysis approach to assess their collapse levels, which were then validated with the results reported in the literature. The validated modeling and analysis approaches were subsequently used to create a set of FE case models representative of typical one- or two-story URM buildings in Europe. Next, uncertainties associated with structural and hazard characteristics were systematically integrated into the FE case models using the Monte Carlo simulation approach. These FE case models were then subjected to incremental flow performance analyses under various flood and flow-type landslide actions to determine the critical flow intensity levels leading to structural collapse. Lastly, logistic regression analyses were performed on the collapse simulation data (i.e., the critical

flow intensity levels) to develop flow collapse fragility models. The following subsections provide more information on the methodology.

Figure 2.2: *The probabilistic framework considered for flow risk assessment of vulnerable masonry buildings.*



Notably, this study aims to create collapse fragility models (i.e., risk assessment models) for vulnerable URM structures. Developing these fragility models requires extensive parametric simulations. In order to reduce their computational demands, certain simplifications and assumptions were implemented in this study. This is a common approach in fragility studies to make several analyses more manageable. First, both flood flows and flow-type landslides are regarded as homogeneous, with a uniform flow density throughout the flow depth; however, flow-type landslides are assumed to have a higher flow density. Similarly, this study focuses solely on primary flow actions, such as hydrostatic and hydrodynamic pressures and debris impact forces, using simplified, statically applied loading formulations [10, 11, 17, 18, 32] to efficiently simulate numerous wall behaviors under varying flow conditions. Secondary actions such as uplift, wave pressures, and erosion (i.e., soil scour) were not considered within the scope of this study. Additionally, 2D shell models (i.e., homogeneous and interlocked masonry behavior) were utilized to practically represent the out-of-plane behavior of masonry walls under flow actions,

enabling efficient execution of numerous parametric simulations (in total, 60,760 simulations) in this study. The collapse state was also defined as the point where the structure reaches its maximum base shear capacity, a commonly used failure criterion in fragility studies [12, 33]. While these simplifications may introduce additional uncertainties in the development of fragility models, they are likely small or negligible compared to the uncertainties introduced by significant variability in key wall and flow parameters. Such simplified approaches are generally accepted in fragility literature [15, 16, 21, 30, 31], but users of the proposed fragility models should remain aware of these limitations. Further details are provided in the following sections.

2.2.1 Flow Actions

This study investigates the structural behavior of URM buildings under the main physical flow actions, including hydrostatic and hydrodynamic pressures, as well as impact loads from any floating debris in the flow (such as tree logs or similar items). In the literature [10, 11, 17, 18, 32], the triangular hydrostatic pressures, $p_{HS}(N/m^2)$, and the assumed-uniform hydrodynamic pressures, $p_{HD}(N/m^2)$, applied to an exposed building component are usually presented as follows:

$$p_{HS} = \rho_F g d \quad (2.1)$$

$$p_{HD} = \frac{1}{2} c_{drag} \rho_F v^2 \quad (2.2)$$

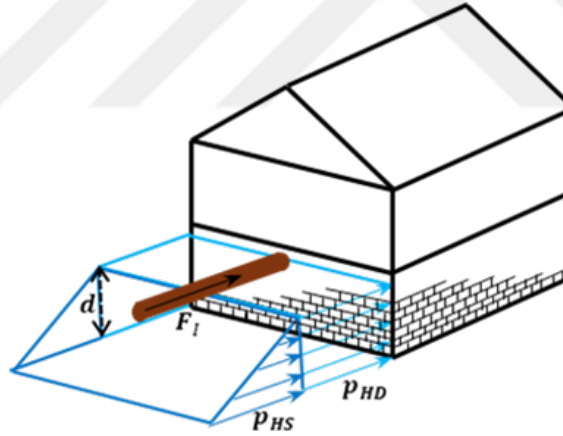
where, $\rho_F(kg/m^3)$ is the flow density assumed to remain constant throughout the flow depth; $d(m)$ is the difference in flow depths between the outside and inside of the building; $v(m/s)$ is the flow velocity generally taken to be uniform over the flow depth; g is the gravity acceleration ($9.81m/s^2$); and lastly, c_{drag} is the drag coefficient assumed to be constant and equal to 1.0 in this study following the literature [17, 18, 32]. Additionally, the debris impact force $F_I(N)$ is usually described by the formula in Equation (2.3) below [32, 34], where $w_D(N)$ is the debris weight and $\Delta t(s)$ is the time

of impact. Here, the velocity of debris is assumed to be the same as the flow velocity, and the factor $\pi/2$ is a consequence of the half-sine shape of the impulse load assumed in the formulation.

$$F_I = \frac{\pi w_D v}{2 g \Delta t} \quad (2.3)$$

Figure 2.3 provides a schematic view of the main flow actions, along with their application patterns and directions. One key difference between these flow actions in floods and flow-type landslides is the flow density (ρ_F). Flow-type landslides generally have higher density due to the entrainment of particulate matter, such as rocks, sediments, and other granular materials, significantly increasing the flow pressures exerted on the structure compared to floods.

Figure 2.3: Schematic representation of the main flow actions acting on a masonry building, including hydrostatic pressures (p_{HS}), hydrodynamic pressures (p_{HD}), and debris impact forces (F_I).



2.2.2 Structural Modeling and Analysis Approaches

Following the literature, this study focuses on the structural modeling of one load-bearing URM wall, assuming that the failure of one exposed wall under the flow actions typically causes the entire URM structure to collapse. In the literature, a variety of computational methods have been employed to analyze the out-of-plane failure mechanisms

of masonry walls under flow actions, including 3D continuum FE analysis [12] or 2D elastic FE analysis [21], yield line analysis (or linear kinematic analysis) [11, 17], and simple beam analysis [10, 18, 19, 20]. It is important to note that over-reliance on these computational models, as well as on standardized codes and formulations, may introduce inaccuracies into the structural analysis results—especially if these results are treated as certain [35]. This approach may also limit innovation and overlook the complexity of real-world challenges in structural engineering problems [35]. While software applications and regulations have their limitations, this study acknowledges these constraints and employs computational models that are carefully validated using case studies from the literature and evaluated for relevant uncertainties through the parametric studies, as detailed in the following sections.

This study first applied these most common modeling and assessment methodologies mentioned above to evaluate their impact on the outcomes. The trials revealed that critical loads in the kinematic analysis are determined using predefined wall failure mechanisms based on the most probable crack patterns. In contrast, the FE approach identifies crack patterns using strain results obtained from the analysis and provides additional responses and insights into wall behavior. Furthermore, various parameter values, such as boundary conditions, material properties, and load variables, can be easily entered into FE models to conduct parametric studies for the fragility development of walls under flow actions. Therefore, nonlinear FE analysis has been selected as a more appropriate and practical approach to assess the collapse fragilities of URM walls under hydrostatic and hydrodynamic pressures, as well as debris impact loads.

In Section 2.3, two typical URM wall case studies subjected to flow actions were chosen from the literature [12, 29] to validate the FE structural models and analysis approaches. The models built for these case studies were calibrated and validated against both numerical and experimental data reported in the literature, ensuring their accuracy

in replicating the complex collapse behavior of masonry wall structures under flow conditions. A range of nonlinear FE modeling techniques were explored, including nonlinear material properties, 2D or 3D modeling approaches, and analysis options. The most suitable 2D-shell modeling and nonlinear static analysis approaches were chosen based on their ability and efficiency in simulating numerous wall collapse scenarios under varying flow conditions. These approaches were found to be robust and well-suited for performing extensive parametric studies in developing collapse fragility models for URM structures. These validated tools were then employed in Section 2.4 to create a set of FE wall case models representative of typical European one- or two-story URM building stocks.

Lastly, it is worth noting that while various modeling and analysis approaches, including elastic beam analysis and kinematic-based analysis, were tested using data from the literature for the two case studies, only the details of the nonlinear FE modeling and analysis tools selected for this study are provided in Section 2.3.

2.2.3 Uncertainty Modeling and Monte Carlo Simulations

To thoroughly quantify flow collapse risk for URM wall structures, uncertainties in wall attributes (e.g., dimensions, strength) and flow hazards (e.g., flow density, debris mass) should be systematically incorporated into the fragility model development process. Therefore, using the approach of Monte Carlo simulations, this study incorporates a wide range of realizations of uncertain parameters—such as variations in structural wall properties and flow characteristics—into the FE wall case models. This approach ensures that the models capture the probabilistic nature of wall collapse under diverse flow conditions, reflecting the variability present in real-world scenarios.

Section 2.4.2 describes the distributions and statistical values (Means and Coefficients Of Variation (COV)) selected for these uncertain parameters in the Monte Carlo simulations. The distributions selected in the uncertainty modeling were assumed to be

uniform, normal, or lognormal and independent of one another. All statistical data were gathered from extensive literature reviews of URM structures and flow hazards.

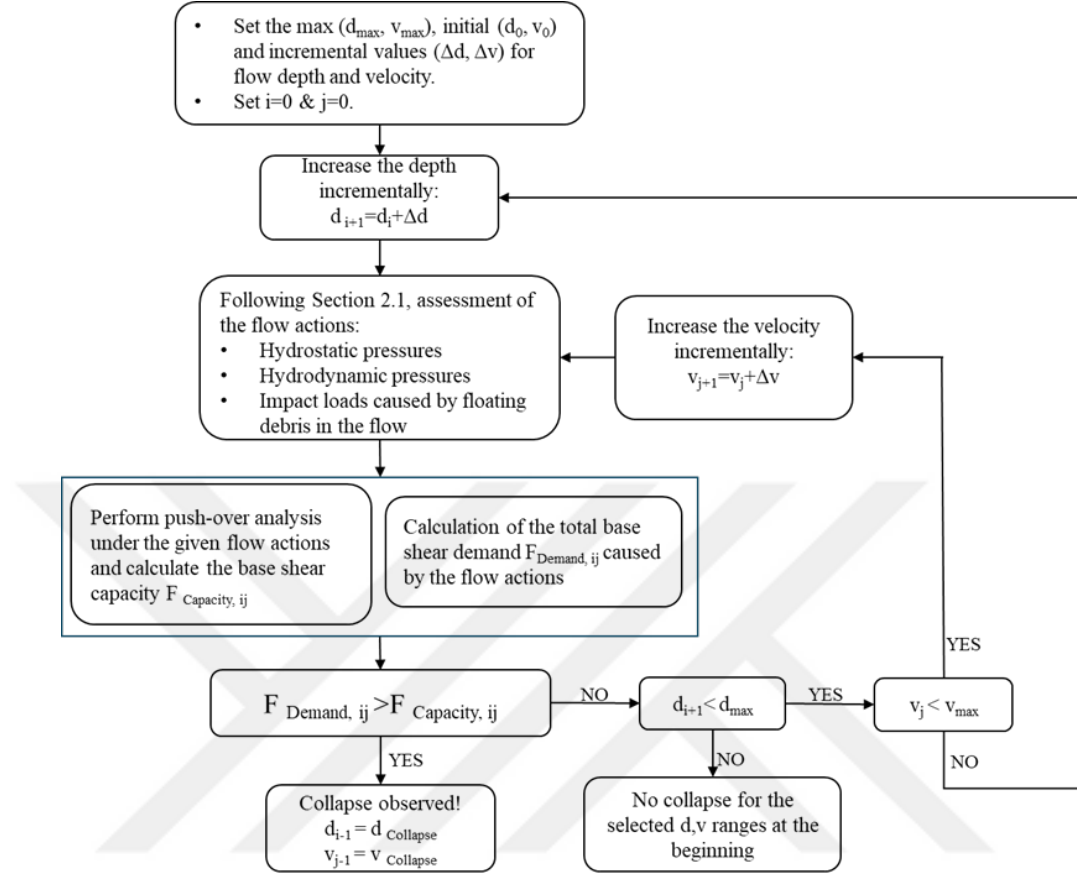
2.2.4 Collapse Assessment by Incremental Flow Analysis

This study proposes a new analysis method named "Incremental Flow Analysis (IFA)" to determine the structural collapse levels of masonry structures subjected to physical flow actions. While similar approaches exist in the literature [21, 36], the proposed IFA method can assess the collapse levels of masonry systems under various flow scenarios, such as flood flows or flow-type landslides.

Figure 2.4 illustrates the steps followed in the IFA approach. In this procedure, flow depth and velocity are incrementally increased, and with each increment, the main flow actions (i.e., hydrostatic and hydrodynamic pressures, as well as impact loads; see Section 2.2.1) are calculated and applied to the structure. This process yields the out-of-plane capacity curve (i.e., the push-over curve of base shear versus maximum lateral displacement of the structure). The IFA continues through each increment until the collapse point is reached, where the base shear demands from flow actions exceed the structure's maximum base shear capacity (i.e., the peak on the capacity curve). This definition of the collapse point is commonly used in previous studies assessing the out-of-plane collapse capacity of brittle structures, such as URM buildings [12, 33], where the structure experiences an abrupt loss of strength at its ultimate base shear point without significant softening. Therefore, in this study, the collapse state is defined as the point where the structure reaches its maximum base shear capacity (the point where crack patterns are nearly fully formed), and no further load increments can be sustained, resulting in sudden failure.

In Section 2.3, the IFA procedure, along with the structural modeling approaches,

Figure 2.4: The flowchart considered for the proposed incremental flow analysis (IFA).



was tested and validated by applying them to two case studies selected from the literature [12, 29]. After testing and validation, a set of FE wall case models representative of typical European URM building stocks was developed. Numerous IFAs were then performed on these models by gradually increasing flow depth and velocity until structural failure occurred. This process facilitated the identification of critical flow thresholds (i.e., collapse capacity points), beyond which collapse became imminent. The IFA procedure was repeated for multiple random realizations of structural and load variables for the wall models in the case set, generating a comprehensive dataset of flow-induced collapse capacity points, which were then used to derive collapse fragilities in the following section.

2.2.5 Collapse Fragility Analyses and Models

This study conducts probabilistic collapse assessments to derive collapse fragilities for each building case model representative of European URM building stocks by considering uncertainties in wall and flow characteristics. A framework of demand vs. capacity was used to estimate collapse fragilities (i.e., conditional probability, P_{Cond}) by calculating the likelihood of a building's flow demand exceeding its associated flow capacity [37]. Based on the collapse criterion outlined in Section 2.2.4, the failure probability P_f for each building model is defined as in Equation (2.4), where the base shear demands (F_{Demand}) resulting from flow actions surpass the structure's maximum base shear capacity (F_{Capacity}) at specified hazard intensities characterized by flow depth d and velocity v .

$$P_{\text{Cond}} = P(\text{Collapse} \mid d, v) = P(F_{\text{Demand}} > F_{\text{Capacity}} \mid d, v) \quad (2.4)$$

In the literature, due to the scarcity of field or experimental data, collapse fragility curves are often evaluated by statistically examining the collapse and non-collapse simulation points resulting from numerical performance analyses. The likelihood of structural collapse for a particular hazard intensity level is often estimated as the ratio of the number of simulations (N) that led to collapse (i.e., demand exceeding capacity) to the total number of simulations performed at the same intensity level, as follows:

$$P_{\text{Cond}} = \frac{N_{\text{Collapse Points}}}{N_{\text{Total Simulation Points}}} \quad (2.5)$$

In the development of structural damage fragility models, several regression approaches have been introduced in the literature to estimate the failure probability in Equation (2.5), accounting for relevant uncertainties in capacity and demand based on simulation or field data. These approaches also help create smooth fragility models that identify

underlying main patterns while reducing outliers in the data [16, 30, 38, 39]. Logistic regression is one of the most widely used approaches, particularly when there are two alternative outcomes, such as collapse or no collapse, as shown in Equation (2.4) [38, 40, 41]. Following the literature [38, 40, 41], this study employs logistic regression analyses to develop collapse fragility models for each building case model representative of European URM building stocks. After performing numerous Monte Carlo simulations for each case (totally, 60,760 simulations across all cases), the IFA procedure was applied to each simulation to assess the hazard intensities (flow depth d and velocity v) corresponding to collapse and no-collapse points. Assuming the collapse state C is a binary output random variable (i.e., $C = 1$ for collapse and $C = 0$ for no-collapse), the relationship between the event $C = 1$ (collapse data points) and the independent flow intensity values d and v was quantified through logistic regression analysis for each case model. These relationships were then used to create a conditional probability function P_{Cond} , which estimates the collapse likelihood of each case at given values of d and v , as follows:

$$P_{\text{Cond}} = \frac{1}{1 + e^{-\phi(d,v)}} \quad (2.6)$$

where, the logit link function $\phi(d, v)$ converts probability values from 0 to 1 into real numbers ranging from negative to positive infinity. In this study, the logit link function was selected to be a quadratic function of the independent variables d and v (see Equation (2.7)), as it provides a more general form for both linear and nonlinear interactions between these variables.

$$\phi(d, v) = \ln \left(\frac{P_{\text{Cond}}}{1 + P_{\text{Cond}}} \right) = \alpha_0 + \alpha_1 d + \alpha_2 v + \alpha_3 dv + \alpha_4 d^2 + \alpha_5 v^2 \quad (2.7)$$

In regression analyses, the optimal estimate of the regression model parameters α in Equation (2.7) is often derived using maximum likelihood estimation [42]. This method quantifies how well the output estimates produced by the model parameters fit the observed data by calculating the probability of observing the data under various model parameter

values. Therefore, this study determined the optimal values of the regression coefficients $\alpha_0, \alpha_1, \dots, \alpha_5$ in Equation (2.7) by maximizing the log-likelihood function in Equation (2.8) using numerical optimization techniques. Here, the dependent collapse state c_i and the independent depth and velocity parameters d_i and v_i represent the data values at the i^{th} simulation (or observation), with i ranging from one to the total number of simulation data points, N .

$$LL(\alpha_0, \alpha_1, \dots, \alpha_5) = \sum_{i=1}^N [c_i \ln(P_{f,i}) + (1 - c_i) \ln(1 - P_{f,i})] \quad (2.8)$$

In logistic regression analyses, a confusion matrix is widely used to evaluate the predictive capabilities of the fitted models. More information regarding the confusion matrix and relevant fitness measures is given in Section 2.4.3.

2.3 Validated Structural Modeling and Analysis Tools

Following the literature [10, 17, 18, 19, 20, 21, 29], this study assumes that the collapse of a single exposed wall due to flow actions typically results in the collapse of the entire URM structure. This study, therefore, concentrates on the structural modeling of a single load-bearing URM wall. In order to verify the structural models, two typical URM wall case studies subjected to flow actions were selected from the literature [12, 29]. This section first demonstrates different structural FE models created in Abaqus [2] for each case study using the exact values of input wall and material properties and flow loading conditions reported in the literature. The built models were then compared and calibrated against the results reported in the literature for the two case studies. Particularly, the input values unclear or missing in the reports were tuned during these calibrations by assuming reasonable values from the literature, ensuring that the output results matched those reported in the previous studies [12, 29]. The IFA procedure for the collapse assessment was also tested and validated on the built models. Subsequently, the validated modeling and IFA methods were employed in Section 2.4 to develop a series of FE wall case

models that depict typical European one- or two-story URM building stocks. Additional information is provided in the subsections below.

2.3.1 The Selected Case Studies from the Literature

Two typical URM case studies were selected from the literature to validate the structural models: an advanced numerical case study by Milanesi et al. [12] and an experimental case study by Herbert [29]. Both case studies involved URM load-bearing walls subjected solely to hydrostatic water pressures until their collapse in the out-of-plane direction.

Milanesi et al. [12] developed an advanced 3D numerical model for a URM wall case of 3 m height by 6 m width and 0.3 m thickness to investigate the collapse mechanism caused by flood actions. It is worth noting that, despite using a detailed numerical FE model in this case study, it only accounts for the effects caused by hydrostatic water pressures, neglecting other major actions such as hydrodynamic actions and debris impact loads. The properties of the brick masonry wall used in this case study are shown in Table 2.1. This case study was conducted for a variety of boundary conditions, with P2, P3, and P4 representing two-edge, three-edge, and four-edge constrained walls, respectively. These three wall boundary configurations were selected to develop and validate the FE models in this study.

Herbert [29] experimentally studied the lateral load capacity of a 1/6 scale set of 40.86 cm by 79.25 cm masonry walls with a thickness of 1.7 cm under different boundary conditions. Two boundary cases from the experimental set, H9 and H7, were chosen for FE modeling validation. The former is a three-edge supported wall case. In contrast, the latter is supported on all four sides, including the upper floor effect via vertical axial loads applied at five points along the top edge.

Table 2.1: *Wall properties for the case studies selected from the literature.*

Reference for the Wall Case Study	Milanesi et al. [12]	Herbert [29]
Selected Wall Case No.	P2, P3, and P4	H7 and H9
Specimen scale	1	1/6
Number of edges supported	2 ^a , 3 ^b , and 4, respectively	3 ^b and 4, respectively
Height (m) × Length (m) × Thickness (m)	3 × 6 × 0.3	0.4086 × 0.7925 × 0.0171
Number of upper stories considered	0	0 and 1, respectively
Compressive strength of masonry, f_c (MPa)	3.00	6.43 ^c
Tensile strength, f_t (MPa)	0.03	0.05 ^d
Young modulus, E (MPa)	3000	1000 ^d
Poisson's ratio, ν	0.2	0.2
Fracture energy, G_f (kJ/m ²)	0.055	0.055 ^d
Density of masonry, ρ_M (kg/m ³)	1800	1828

^a Wall specimen with two supported edges at the top and bottom.

^b The only free edge of the wall specimen is at the top; all other edges are supported.

^c Calculated value from the formula by Ref. [28]: $f_c = 0.54 f_b^{0.7} f_m^{0.3}$, where $f_b = 21$ MPa and $f_m = 8.27$ MPa are brick's and mortar's compressive strengths, respectively [29].

^d In the case of missing data or the value is unclear, a reasonable assumption was made according to the literature and various trials.

2.3.2 Structural Modeling

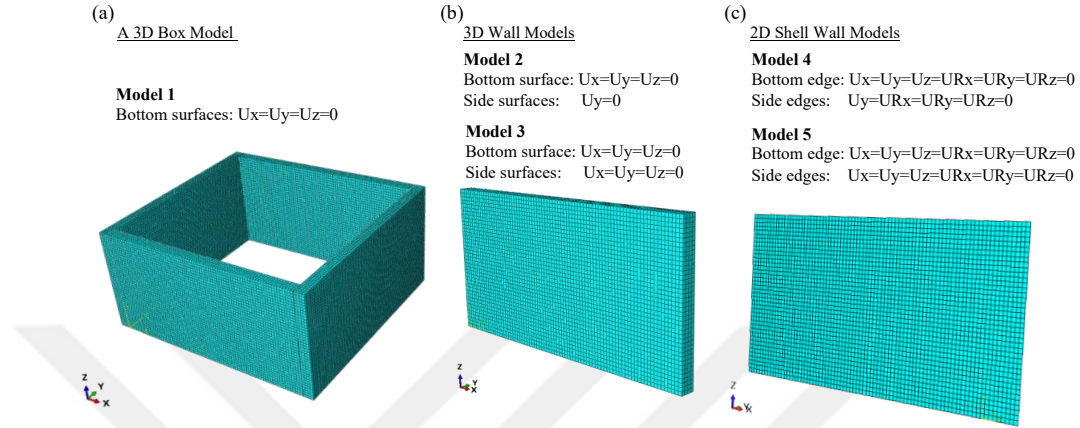
This section explores various structural modeling options using the Abaqus FE program [2] to find the optimal one for the case studies selected above. The walls were modeled taking advantage of the macro-modeling strategy, which assumes that the masonry material is homogeneous, isotropic, and interlocked. For each case study, the Concrete Damaged Plasticity (CDP) constitutive model [12, 43] was selected to model masonry material using the information in Table 2.1. To define the CDP model, while the compressive stress-strain behavior of masonry was taken into account using the constitutive model of Kaushik et al. [44], a linear tensile stress-strain behavior was assumed based on the fracture energy in Table 2.1. Following the literature [2], general values were assumed for the five parameters to define the plasticity flow capability of the CDP model for masonry, which include dilation angle ($\Psi = 20^\circ$) [12], eccentricity ($\epsilon = 0.1$), strength ratio ($f_{bo}/f_{co} = 1.16$; where f_{bo} and f_{co} are initial equiaxial and uniaxial compressive yield stresses), modification parameter ($K_c = 0.67$), and viscosity parameter ($\mu = 0$).

After material modeling, different FE modeling options were investigated and compared for the case studies selected above. Investigating different modeling options enables a precise evaluation of how boundary conditions and element types influence the

structural behavior of masonry walls under various support constraints, while also identifying the most practical and efficient approaches for this study. First, using the P3 case from Milanesi et al. [12] (see Table 2.1), a 3D solid box model for a complete structure of four walls (Model 1), as well as different 3D solid models (Models 2 and 3) and 2D shell models (Models 4 and 5) for one of the exposed walls were created in Abaqus [2] (see Figure 2.5). While the 3D models were built using *C3D8* solid linear hexahedral (eight nodes) elements, the 2D shell models were built with *S4R* linear quadrilateral (reduced integration shell with four nodes) elements. The *C3D8* elements have three translational degrees of freedom per node, while the *S4R* elements have six degrees of freedom per node (three translational and three rotational). The square element width was set to 0.075 m for all models, following the same mesh width in the case study by Milanesi et al. [12]. Regarding boundary conditions, the 2D shell and 3D models were fixed and pin-supported at the bottom, respectively, whereas all the models were set free at the top (P3 wall case). To properly include the sidewall effects on the wall models, different boundary conditions were tried, as shown in Figure 2.5. For example, in one of the trials for the 2D shell wall model (Model 4 in Figure 2.5), the sides were assumed to be translationally free in the in-plane directions (x and z axes) but translationally restricted in the other direction and rotationally restricted in all directions.

Likewise, several modeling options were built in Abaqus [2] to model the H7 (four-edge supported) and H9 (three-edge supported) wall cases in Herbert's experimental study [29], including 2D shell models with *S8R* eight-node reduced integration shell elements with a length of 0.037 m and a height of 0.013 m (equal to the brick size used in the experimental test). Following the experimental details, for the 2D shell models, the bottom edge (foundation level) was assigned as pin-supported, while the side and top edges were assumed to be translationally restrained in the out-of-plane direction but free in the other directions.

Figure 2.5: Structural FE modeling options considered for the three-edge supported P3-wall case study by Milanesi et al. [12]: a) a 3D box model of a single-story, single-bay masonry building composed of four load-bearing walls; and b) 3D wall models and c) 2D shell models for one of the exposed load-bearing walls (Note: U and UR represent the restrained translational and rotational support conditions, respectively).



2.3.3 Incremental Flow Analyses

Following the structural modeling, incremental flow analyses (IFA) were performed on the built models to assess the collapse intensity levels of the case studies and compare them to the results found in the literature. The IFA procedure enables tracking the progressive increase in base shear capacity as water depth rises, providing a deeper understanding of the structure's performance under flow-induced forces. Unlike traditional methods, which rely on a single push-over analysis, this approach conducts multiple push-over analyses to offer valuable insights into how walls behave under loads generated by gradually increasing flow depths and velocities (see Section 2.2.4). The IFAs in Abaqus were performed using Static-Riks analysis, a commonly used solution approach for non-linear problems [2]. Only hydrostatic flood pressures and flood depths were considered in this comparison below, in accordance with the loading protocols described in the two case studies, and the water depth applied to the wall models was increased incrementally by 0.1 m or smaller steps.

2.3.4 Comparisons of the Analysis Results

Following the same loading protocol in the case study by Milanesi et al. [12], IFA procedures were conducted, and maximum principal plastic strain contours were obtained for all models (Models 1-5; see Section 2.3.2) under increasing hydrostatic flood pressures. Figure 2.6 shows the model results for the P3 wall case (three edge-constrained but free at the top), illustrating only one push-over example of an increment for hydrostatic pressures corresponding to 1.3 m-flood water. In Figure 2.6a, compared to the push-over results for the 3D box (Model 1), the shell wall model (Model 4) generally provides comparable capacity values. In contrast, others either underestimate or overestimate the wall capacities. Furthermore, the maximum plastic strain contours of the 3D box model (Model 1) and shell model (Model 4) presented in Figure 2.6b and Figure 2.6c, respectively, which can be used to understand the development of crack patterns in the models [12, 28], are in good agreement. Additionally, the shell model results were similar to those reported in Milanesi et al. [12].

Similarly, modeling trials were conducted on Milanesi et al.'s P2 and P4 wall case studies [12] (see Table 2.1), taking into account all model options, including the shell models, with the only difference being the boundary conditions at the sides and top: while the P4 shell model was restricted at all edges, the P2 shell model was assumed to be completely free at the sides only. The findings from these cases were also consistent with the findings from the 3D box models and the values reported in the literature [12]. These comparisons suggest that a shell model with appropriate boundary conditions is a practical option for modeling masonry structures under flow actions.

The IFA curves for the P2, P3, and P4 wall case studies by Milanesi et al. [12] are given in Figure 2.7 (top-left), which show the relationships between total base shear vs. maximum out-of-plane displacements as water depth increases. The collapse intensity points (i.e., the last capacity points on the IFA curves, just before demand exceeding

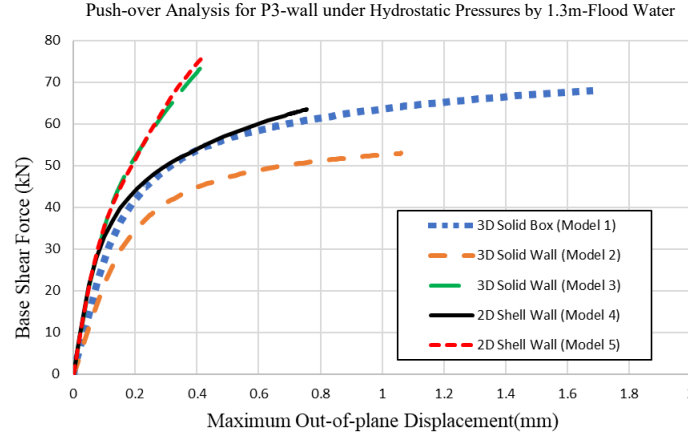
maximum base shear) were found to be 1.4 m, 1.3 m, and 1.57 m, respectively, based on the IFA procedure, which exactly match the results reported in the study by Milanese et al. [12]. Furthermore, as the depths increased along the IFA curves, the crack patterns—represented by maximum principal plastic strain contours—on the walls were obtained (such as those shown for the P3 wall case in Figure 2.7-bottom), giving patterns that were comparable to those seen in the literature (see Figure 2.7-top right side) [12, 29]. The P2 wall's cracks appear horizontally near the mid-height, indicating a flexural failure mechanism parallel to the bed face. In the P3 wall, diagonal cracks initiate from the bottom corners and extend toward the center. In the P4 wall, cracks originate from all four corners. Due to boundary effects, the flexural failure patterns in the P3 and P4 walls indicate a combination of bending mechanisms, both parallel and perpendicular to the bed face.

The same IFA analyses and crack patterns were also obtained for the case studies by Herbert [29] using different modeling approaches but the same loading protocol as in the experimental study. For example, Figure 2.8a and Figure 2.8b illustrate the results for the two shell models, corresponding to the H7 and H9 wall cases, respectively, compared with their experimental results. In Figure 2.8, the top-left images show schematic representations of the walls modeled in Abaqus, where the H7 wall is supported on all four sides, including the influence of the upper floor, and the H9 wall is supported on only three edges. The top-middle sections display the corresponding IFA curves, while the top-right images present the crack patterns observed in the experiments, as reported in Herbert's study. The bottom images depict the crack patterns simulated in Abaqus, corresponding to each increment on the IFA curves, and illustrate the damage progression as flow steps increase.

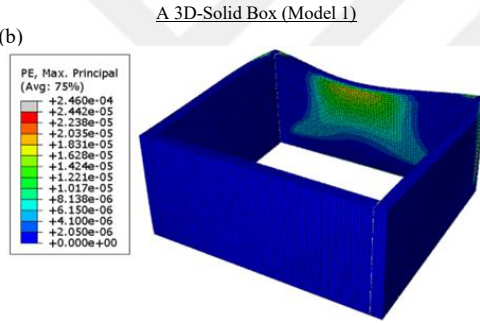
Compared to the results for other modeling options, the results for the two shell

Figure 2.6: For the three-edge supported P3 wall case study of Milanesi et al.[12]: a) push-over analysis results assessed under hydrostatic pressures for 1.3m-flood water; and maximum principal plastic strain contours at the level of 1.3m- flood water (corresponding to a base shear force of 49.7 kN) for b) the 3D box model (Model 1), and c) the shell wall model (Model 4).

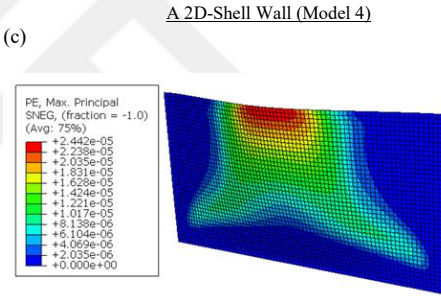
(a)



(b)



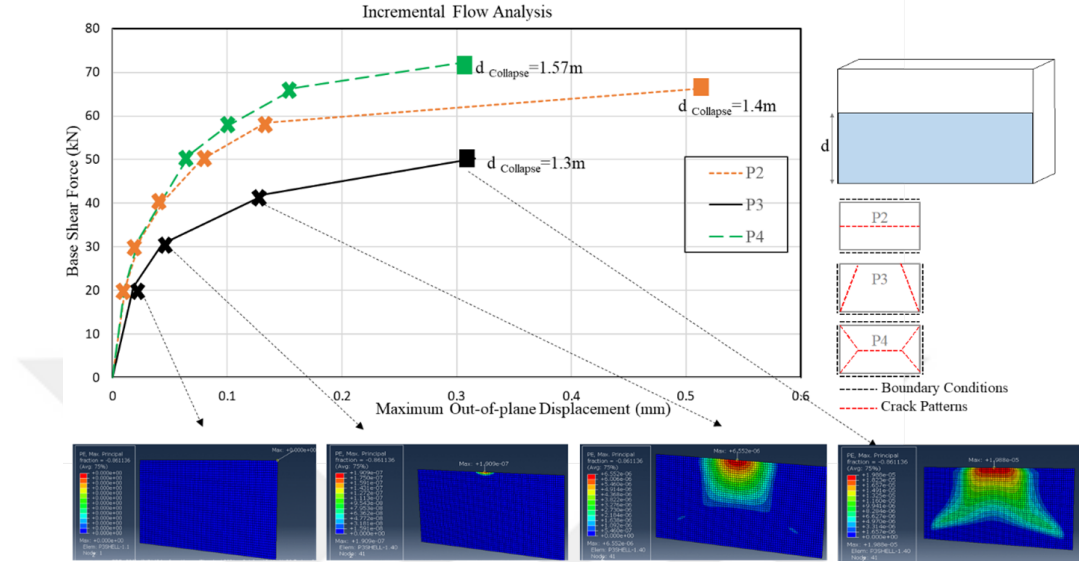
(c)



models presented in Figure 2.8 show a close agreement with the experimental data, confirming that shell models can be an appropriate option for masonry wall modeling once again. For example, the IFA curves and collapse intensity levels from the IFA analyses (the last point on the IFA curve just before the wall collapse, at 290 mm and 195 mm, respectively) were found to be comparable with the experimental results for the H7 and H9 masonry walls (247 mm and 209 mm, respectively). The slight differences between the model and experimental results in these case studies are likely due to the assumptions made for some material input values in Table 2.1.

It is important to note that the differences in maximum displacement and slopes

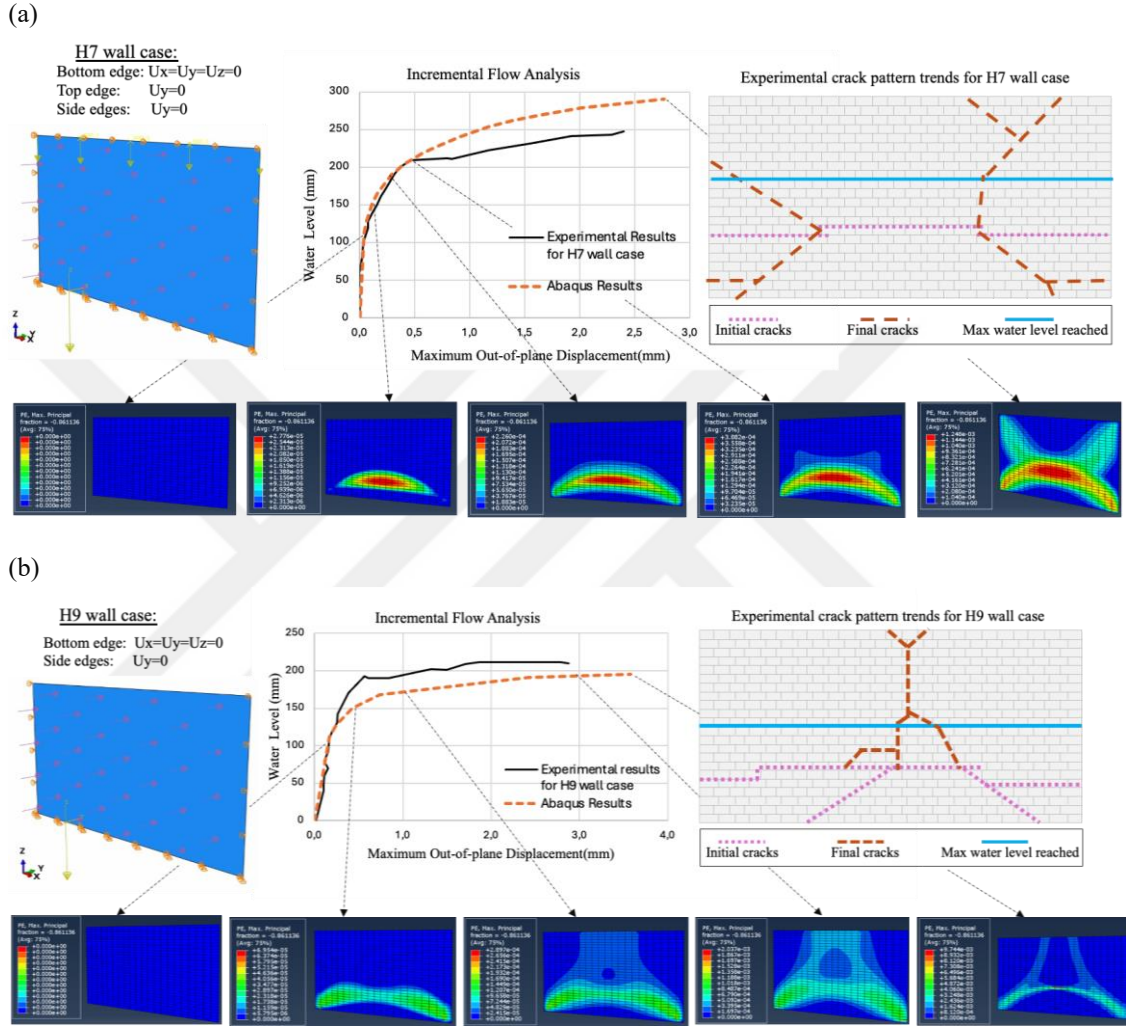
Figure 2.7: Abaqus results from incremental flood analyses (IFA) for the P2, P3, and P4 wall case studies by Milanese et al. [12] (top-left) and the maximum plastic strains contours for the P3 case study (bottom) in comparison with the crack patterns reported in the literature [12] (top-right).



between the IFA curves for the case studies by Milanese et al. [12] (Figure 2.7) and Herbert [29] (Figure 2.8) are mainly due to the different scales of wall samples used in each study (full-scale versus 1/6-scale) and the varying boundary conditions (see Figure 2.5 vs. Figure 2.8) [45, 46, 47]. Other factors, such as material properties and wall dimensions, also contribute to these differences.

As a result, in this study, shell modeling techniques were selected as the most effective way to simulate the behavior of unreinforced masonry (URM) structures under flow actions. The chosen structural modeling approach and the developed IFA methods were tested and validated using the two case studies discussed earlier. Building on these validations, the next section applies these methods to create a set of finite element (FE) models representing typical URM buildings across Europe. These models were then used to develop collapse fragility models.

Figure 2.8: Incremental flow analyses (IFA) results (top) and the corresponding maximum plastic strains contours (bottom) from the FE models built for the experimental wall case studies by Herbert [29], in comparison with experimental results, for: a) H7 wall case and b) H9 wall case (Note: U represents the restrained translational support conditions).



2.4 Collapse Fragility Models Proposed for Typical URM Buildings

This study aims to develop collapse fragility models for typical European masonry buildings subjected to floods and flow-type landslides. Since these structures are often composed of exterior load-bearing masonry walls, the failure of one wall could jeopardize

the overall structural stability. Therefore, this study focuses on the collapse performance of exterior walls exposed to flows to derive collapse fragility models.

2.4.1 Typical Wall Case Set

Below, Table 2.2 summarizes the fourteen wall cases created to evaluate the collapse performance of the walls under various scenarios. These cases provide valuable insights into how walls respond under different boundary conditions (i.e., three-edge supported and four-edge supported walls), changing flow actions (i.e., flood flow and landslide-flow types, and different debris masses), and different axial load conditions, contributing to better understanding of the walls' resistance to flow-induced collapse. In Table 2.2, it is noteworthy that three-edge supported walls (with a free top edge; W3) are typically found when the floor joists or roof rafters are parallel to the wall or when there are nearby stairs. Four-edge supported walls (W4), on the other hand, occur when there are no (or only marginal) openings adjacent to the edges or when the joists or rafters are not parallel to the wall [12].

For developing the collapse performance models for the fourteen wall cases in Table 2.2, a typical clay brick masonry wall with a height of 3 meters and a width of 6 meters was considered. As tested and validated in the previous section, a 2D FE model with four-node linear shell elements (*S4R*) and a square mesh size of 0.05 m were used to construct all masonry wall models in Abaqus [2]. The same material models as those in Section 3 were used. All wall models have fixed support at the bottom edge, proper restraint at the side edges (only translationally free in the in-plane directions), and the top edge either free (W3) or restrained like the side edges (W4).

Table 2.2: *Typical wall cases and flow cases considered in the collapse fragility assessment models.*

Case No	Total Story Num.	Case Acronym*	Boundary Conditions	Flow Type	Debris Mass	Presence of Axial Pressures from Upper Story		
1	One	W3-F-M0	Three-Edge Supported	Flood Flow	No	No		
2		W3-L-M0		Flow-type Landslide				
3		W3-F-ML		Flood Flow	Light			
4		W3-L-ML		Flow-type Landslide				
5		W3-F-MH		Flood Flow	Heavy			
6		W3-L-MH		Flow-type Landslide				
7	Two	W4-F-M0	Four-Edge Supported	Flood Flow	No			
8		W4-L-M0		Flow-type Landslide				
9		W4-F-MH		Flood Flow	Heavy			
10		W4-L-MH		Flow-type Landslide				
11		Two		W4-F-M0-UP	Four-Edge Supported	Flood Flow	No	Yes
12				W4-L-M0-UP		Flow-type Landslide		
13	W4-F-MH-UP		Flood Flow	Heavy				
14	W4-L-MH-UP		Flow-type Landslide					

* W3: Three-edge supported; W4: four-edge supported; F: flood flow; L: flow-type landslide; M0, ML, MH correspond to a debris mass value of zero light, and heavy, respectively; UP: Presence of axial pressures on the top wall edge coming from the upper story.

2.4.2 Modeling Parameters and Uncertainty Modeling

To improve the robustness and reliability of collapse risk assessment models for masonry walls, substantial associated uncertainties must be identified and incorporated. This study thus employs a series of Monte Carlo simulations that account for variabilities in several key modeling parameters for the wall cases and flow actions mentioned above. Table 2.3 and Table 2.4 show the statistical and distribution values for these key parameters, which were obtained through extensive literature reviews. The selected distributions were assumed to be independent of each other. The parameters with a higher coefficient of variation (COV) indicate significant variability, whereas those without COV are considered deterministic. For example, in Table 2.3, a high COV value of 30% to 35% is assumed for masonry material strength parameters, as these variations result from using various mortar compositions in masonry. However, Poisson's ratio and other material factors were found to have a minor impact on wall capacity predictions, so uncertainty was not taken into account for them. Similarly, wall width and height parameters were

assumed to be deterministic in this study, as changing them would require a very demanding effort and creating a new Abaqus model each time. While this issue limits the study, the chosen deterministic values are consistent with those commonly used in the literature for European masonry structures [10, 12, 15, 17, 21, 29].

As shown in Table 2.4, the flow was modeled as either flooding flow or flow-type landslide, potentially carrying no, light, or heavy debris. To characterize the flows, the flow density and debris mass were assumed to be uncertain parameters, while the impact time was set as a fixed variable. Furthermore, the floating debris load was assumed to strike the wall at the flow surface level, at the middle of the wall's bay length. Additionally, this study makes no assumptions about the hydrological relationship between flow depth and velocity, so they were considered independent.

Table 2.3: Typical statistical values and probability distributions assumed for wall modeling parameters.

Wall Modeling Parameters	Mean	COV	Distribution
Height of wall, H (m)	3 [10, 12, 18, 21, 48]	0	Deterministic
Width of wall, W (m)	6 [12, 17, 29]	0	Deterministic
Thickness of wall, t (m)	0.15 [12]	0.25 [11, 12, 18]	Uniform
Density of masonry, ρ_M (kg/m ³)	1800 [10, 11, 12]	0.15 [11, 21, 24, 29]	Lognormal
Compressive strength, f_c (MPa)	5.0 [49]	0.30 [6, 11, 29, 48, 49, 50]	Lognormal
Tensile strength, f_t (MPa)	0.2 [24, 49]	0.35 [10, 18, 49]	Lognormal
Coeff. for elastic modulus, a^a	775 [49]	0.30 [50, 51]	Lognormal
Fracture energy, G_f (N/mm)	0.02 [47, 49]	0.30 [50, 52]	Lognormal
Coeff. for axial pressures from upper story, c^b	0 and 7 [11]	0.20 [17]	Lognormal
Poisson's ratio, ν	0.2 [12]	0	Deterministic
Dilation angle, ψ^c	20° [12]	0	Deterministic

^a Elastic modulus E of masonry is often expressed as a function of its compressive strength f_c as follows: $E = a f_c$, with the coefficient “ a ” here assumed to be 775 on average, based on literature reviews [49, 50, 51].

^b In the cases of the presence of an upper story, the base walls are assumed to support the upper elements and bear their loads, including the upperstory's wall, floor, and roof. Based on the literature [11] (a floor with a dead load of 5.40 kN/m² and a live load of 1 kN/m²; and a roof with an inclination of 45° and a dead load of 0.75 kN/m²), the ratio “ c ” of axial load coming from the upper story to self-weight of the base wall is approximately 7. Here, c was then assumed to be either 0 (no axial pressures from the upper story) or 7 on average.

^c The most important parameter defining the plasticity flow behavior for the Concrete Damage Plasticity (CDP) model was only provided here. The other relevant parameters (ε , f_{bo}/f_{co} , K_c , μ) were assumed to have the same deterministic values as the default ones given in the literature [2] (see Section 2.3).

Table 2.4: *Typical statistical values and probability distributions assumed for flow intensity modeling parameters.*

Flow Intensity Modeling Parameters	Mean	COV	Distribution
Flow depth, d (m)	0.1m-increments btw. 0 and 3m	0	Deterministic
Flow velocity, v (m/s)	0.5m/s-increments btw. 0 and 3m/s	0	Deterministic
Flow density, ρ_F (kg/m ³)	Flood flow:1025 [17, 53]	0.05[10, 12, 18]	Uniform
	Flow-type landslide:1800 [6]	0.10[6, 12]	
Debris mass, m_D (kg)	No debris	0	Deterministic
	For light debris such as rubble: 50 [10, 54]	0.30 [54]	Uniform
	For heavy debris such as tree trunks:453.6 [32]		
Impact time duration for debris, Δt (s)	0.03 [32]	0	Deterministic

2.4.3 Logistic Regression Analyses on IFA Results

Following the uncertainty modeling described above, a suite of 20 Monte Carlo sample realizations was generated for each of the random variables listed in Table 2.3 and Table 2.4 at increments of 0.1 m for flow depth and 0.5 m/s for flow velocity, up to 3 m and 3 m/s, respectively. These realizations were obtained for each wall case in Table 2.2, resulting in a total of 60,760 simulations (14 wall cases x 31 depth values x 7 velocity values x 20 realizations per parameter). Incremental flow analyses (IFA) were then performed on each realization to examine collapse in each of the fourteen wall cases.

In the literature, logistic regression is frequently utilized to create structural fragility models, particularly when there are two alternative outcomes, such as collapse or no collapse [38, 40, 41]. Following the literature [38, 40, 41], this study utilizes logistic regression analyses to develop collapse fragility models for the fourteen masonry wall cases listed in Table 2.2. Once the IFA data were obtained for each wall case, numerous logistic regression analyses were conducted to determine the models that best fit the simulated IFA data (see Section 2.2.5 for details). The confusion matrix is widely used to evaluate the prediction capabilities of the fitted models. Assuming that collapse is a positive event, and no collapse is a negative event, the confusion matrix divides observed and predicted events into four categories (see Table 2.5): The first two are True Positive (TP) and True Negative (TN), which are predictions that hold true for positive and negative observations, respectively. The other two are False Negative (FN) and False Positive (FP), which denote

incorrect predictions for negative and positive observations, respectively.

Table 2.5: *The confusion matrix considered in logistic regression analyses*

Exact or observed from Abaqus simulations	Predicted (from the fitted logistic models)		
	Positive (Collapse) Negative (No-collapse)	Positive (Collapse)	Negative (No-collapse)
		True Positive (TP) False Positive (FP)	False Negative (FN) True Negative (TN)

For each wall case in Table 2.2, using the prediction results from the fitted logistic model and the observation dataset from Abaqus simulations, the confusion matrix was created to assess several commonly used fitting evaluation metrics described in Equations (2.9)-(2.13) [38, 39, 41, 55, 56, 57, 58], in the following order: the model's ability to distinguish between the positive and negative events (i.e., the area under the receiver operating curve, AUC_{ROC} , which tracks the relation between True Positive Rate (TPR) vs. False Positive Rate (FPR)); its ability to capture actual positive instances (i.e., recall or sensitivity); its ability to make positive predictions (i.e., precision); its overall correctness in classifying instances (i.e., accuracy), and lastly, its effectiveness as a harmonic mean of recall and precision (i.e., F1-score).

$$AUC_{ROC} = \text{Area under the curve of TPR} = \frac{TP}{TP + FN} \text{ versus FPR} = \frac{FP}{FP + TN} \quad (2.9)$$

$$\text{Recall (or Sensitivity)} = \frac{TP}{TP + FN} \quad (2.10)$$

$$\text{Precision} = \frac{TP}{TP + FP} \quad (2.11)$$

$$\text{Accuracy} = \frac{TP + TN}{TP + FN + FP + TN} \quad (2.12)$$

$$F_1 \text{ score} = \frac{2 \cdot \text{Precision} \cdot \text{Recall}}{\text{Precision} + \text{Recall}} \quad (2.13)$$

Table 2.6 shows the fitting evaluation results obtained for each wall case. The reported values, all greater than 0.9, indicate that the models can accurately predict and distinguish between positive and negative occurrences. Higher F1-score values demonstrate that the forecasts are exact and robust, emphasizing the models' overall effectiveness. In

summary, the results show that logistic regression models consistently outperform across multiple evaluation metrics, confirming the models' superior fit to the observation data.

Table 2.6: *Fitting evaluation results from the logistic regression models for the fourteen wall cases.*

Case No. (See Table 2.2)	Case Acronym	Accuracy	AUCROC	Precision	Recall or Sensitivity	F1 score
1	W3-F-M0	0.977	0.999	1.0	0.961	0.980
2	W3-L-M0	0.977	0.998	1.0	0.966	0.983
3	W3-F-ML	0.972	0.999	0.985	0.970	0.978
4	W3-L-ML	0.977	0.998	0.986	0.980	0.983
5	W3-F-MH	0.982	0.998	1.0	0.978	0.989
6	W3-L-MH	0.977	0.999	1.0	0.973	0.986
7	W4-F-M0	0.959	0.997	0.955	0.964	0.959
8	W4-L-M0	0.986	0.999	1.0	0.978	0.989
9	W4-F-MH	0.963	0.995	0.994	0.960	0.977
10	W4-L-MH	0.968	0.997	0.994	0.967	0.981
11	W4-F-M0-UP	0.968	0.997	0.958	0.945	0.952
12	W4-L-M0-UP	0.977	0.999	0.980	0.971	0.976
13	W4-F-MH-UP	0.926	0.981	0.952	0.939	0.946
14	W4-L-MH-UP	0.959	0.994	0.987	0.956	0.971

2.4.4 Proposed Collapse Fragility Models

Collapse fragility models that best fit the simulation data were proposed in the previous section, based on logistic regression analyses of the IFA simulation results. The key independent variables in these models are flow depth and flow velocity, while other factors, such as debris mass, flow density, and masonry material properties, were treated stochastically and incorporated using Monte Carlo simulations. The parameters and their respective statistical distributions are detailed in Table 2.3 and Table 2.4.

The collapse fragility models were categorized based on the fourteen wall cases defined in Table 2.2, considering two common wall boundary conditions (such as three-edge supported and four-edge supported walls), varying flow actions (including flood flow and landslide flow, with average flow densities of 1025 kg/m³ and 1800 kg/m³, respectively), different debris masses (ranging from 0 kg to 56 kg to 453.6 kg on average), and the presence of axial load conditions from the upper story (ranging from 0 to approximately 7 times the self-weight of the base wall). For each of these fourteen wall

cases, the corresponding collapse fragility contour models are presented in Figures 2.9-2.11. These contour lines, ranging from 0 to 1, represent the probability of wall collapse as a function of flow depth and velocity. The number (varying from 1 to 14) and acronym shown for each case in the figures indicate the corresponding wall case listed in Table 2.2. Specifically, Figure 2.9 illustrates the proposed models for one-story URM structures with three-edge supported walls (W3), while Figure 2.10 and Figure 2.11 show the outcome models for one- and two-story URM buildings with four-edge supported walls (W4), respectively. Only the models in Figure 2.11 account for the effect of axial stresses due to the presence of a second story on the walls' collapse potential.

The color mapping in the fragility contours further enhances visualization by illustrating the transition from safe conditions (dark blue, representing 0% collapse probability) to imminent collapse (yellow, representing 100% collapse probability). This color gradient allows for a quick and intuitive understanding of how the collapse fragilities of wall structures change under different flow scenarios.

The proposed models in Figures 2.9-2.11 can facilitate accurate estimation of the likelihood of structural collapse in URM buildings during flow events. This is particularly valuable for developing modern national building codes, regional emergency response plans, and risk management strategies, where the goal is to enhance occupant life safety by ensuring a sufficiently low probability of structural collapse during severe hazard events. These proposed models can be used in two ways: first, by referring to the fragility contours to estimate the likelihood of collapse for a specific wall among the fourteen cases based on selected flow depth and velocity values; or second, by identifying the critical flow depth and velocity values that correspond to an acceptable level of collapse risk for a particular wall. In summary, each contour line in Figures 2.9-2.11 represents a relationship between flow depth and velocity values that correspond to a specific risk of wall collapse, ranging from 0% to 100% collapse probability. In scenarios involving

additional variables, such as different boundary conditions, debris, flow density, or axial load, comparisons between wall cases with and without these variables (as shown in the plots from cases 1 to 14) offer valuable insights into how these factors influence collapse probability.

Overall, the results in Figures 2.9-2.11 demonstrate that collapse potential varies depending on each case's wall and flow properties. The fragility contours generally show an increased chance of collapse as flow depth and velocity rise, highlighting the sensitivity of collapse occurrence to both depth and velocity values. This sensitivity is particularly pronounced when debris is present in the flow, as it causes significantly more damage than hydrostatic and hydrodynamic flow actions. As the debris mass increases, the contours become more curved, shifting the collapse fragility contours to the left and leading to failures at lower depths and velocities.

Table 2.7 also provides the collapse depth values for each case at both the 50% and 90% collapse contours ($d_{50\%}$ and $d_{90\%}$, respectively), at a sample flow velocity of 1 m/s, to better emphasize the differences between the cases. For example, the results for W3 walls under flood flow conditions (with average flow densities of 1025 kg/m^3) show that in cases with no debris (Case 1), light debris (Case 3, with a mean debris mass of 50 kg), and heavy debris (Case 5, with a mean debris mass of 453.6 kg), the 50% collapse probability occurs at the depths ($d_{50\%}$) of 1.37 m, 1.23 m, and 0.57 m, respectively. It is important to note that the $d_{50\%}$ for Case 5 is roughly half of that for Case 1. This reduction becomes more pronounced at higher velocities. For example, the results in Figure 2.9 show that in the heavy debris case (Case 5), the $d_{50\%}$ value decreases from 0.57 m to 0.23 m (a 60% reduction) as the flow velocity increases from 1 m/s to 2 m/s. This demonstrates that walls exposed to significant amounts of debris are highly sensitive to flow velocity, as even a slight increase in velocity can substantially decrease the wall's collapse resistance.

Similar to the results for three-edge supported masonry walls (W3; Figure 2.9), four-edge supported masonry walls (W4; Figure 2.10) are also vulnerable to flows but exhibit greater resistance due to the additional constraints (i.e., their contours are generally shifted to the right), resulting in slightly higher stiffness against lateral flow loads. Moreover, comparing the W4 results in Figures 2.10-2.11 with and without axial loads shows that vertical loads from the upper story significantly strengthen the walls against lateral forces, further shifting the collapse fragility contours to the right. Table 2.7 further highlights these differences at a sample flow velocity of 1 m/s. In flood flow scenarios with heavy debris (average flow density of 1025 kg/m³ and debris mass of 453.6 kg), the $d_{50\%}$ values for Case 5 (W3 without axial load), Case 9 (W4 without axial load), and Case 13 (W4 with an axially distributed load of approximately 55 kN/m) increase from 0.57 m to 0.74 m (a 30% increase) and to 1.20 m (a 111% increase), respectively.

Furthermore, the collapse contours exhibit notable sensitivity for flow-type landslides, as they tend to have smaller contour interval widths and are usually shifted to the left. This results in quick collapse at lower flow depths and velocities (see Figures 2.9-2.11, right side). For example, at a flow velocity of 1 m/s, Table 2.7 shows that the collapse depth results for $d_{50\%}$ under flood flows range from 0.57 m (Case 5) to 2.08 m (Case 11). In comparison, the collapse depths are reduced for the same walls but under the flow-type landslides, ranging from 0.54 m (a 5% decrease for Case 6) to 1.69 m (a 22% decrease for Case 12), respectively. These differences become even more evident at higher probability levels. For instance, the $d_{90\%}$ values change from more than 2.90 m (Case 13) under flood flows to 1.86 m (Case 14) for flow-type landslides, reflecting a 36% decrease. However, when heavy debris is present in the flow, these changes are significantly smaller at higher velocities (as shown in Figures 2.9-2.11, bottom left vs. bottom right), with debris typically having a more significant impact than flow density.

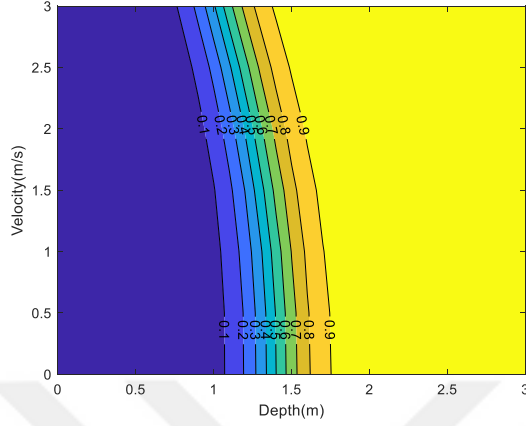
Lastly, there are considerable uncertainties in the collapse estimates. For example,

at a flow velocity of 1 m/s, Table 2.7 shows that the relative differences between the collapse depth results for $d_{50\%}$ and $d_{90\%}$, respectively, representing 50% and 90% collapse likelihood (i.e., $\frac{d_{90\%}-d_{50\%}}{d_{50\%}}$) range from 23.6% (Case 2) to 141.7% (Case 13). This indicates substantial uncertainty in key factors such as wall boundary type, flow type, debris mass, and axial load, as well as their influence on collapse estimates. Among these factors, variations in debris mass have the most significant impact on collapse predictions. These findings emphasize the importance of carefully considering these factors and their variability when assessing the collapse risk of masonry walls.

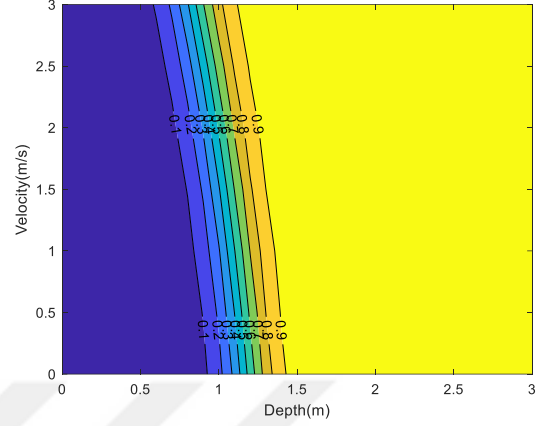


Figure 2.9: Collapse fragility contours for one-story URM buildings with three-edge supported walls (W3).

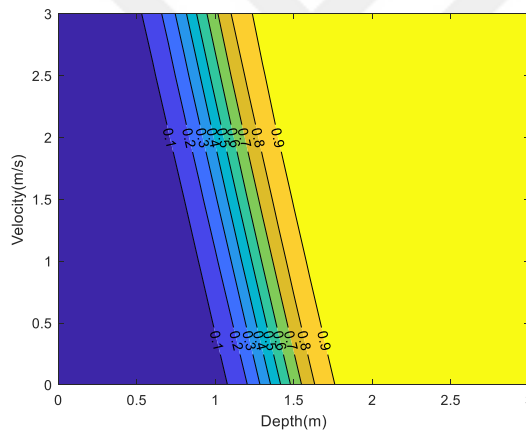
Case 1: W3-F-M0 Flood Flow, No Debris



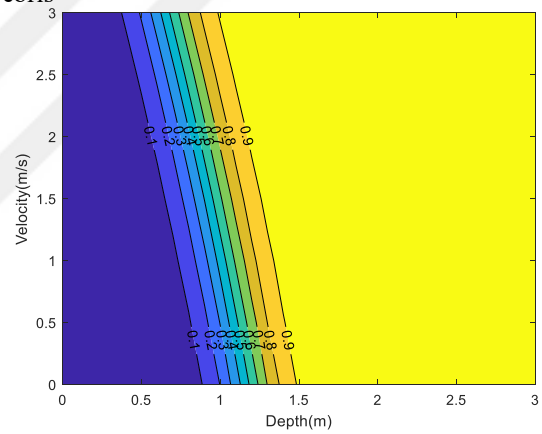
Case 2: W3-L-M0 Flow-type Landslide, No Debris



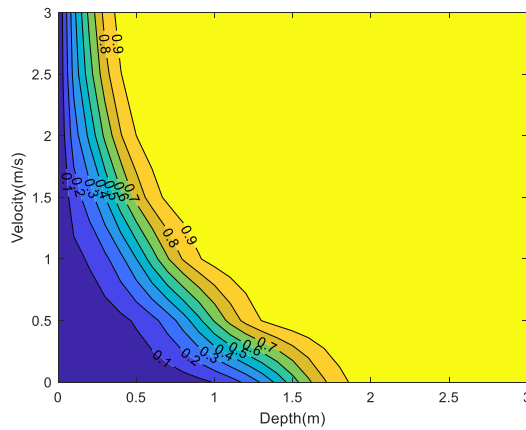
Case 3: W3-F-ML Flood Flow, 50kg-Debris



Case 4: W3-L-ML Flow-type Landslide, 50kg-Debris



Case 5: W3-F-MH Flood Flow, 453.6kg-Debris



Case 6: W3-L-MH Flow-type Landslide, 453.6kg-Debris

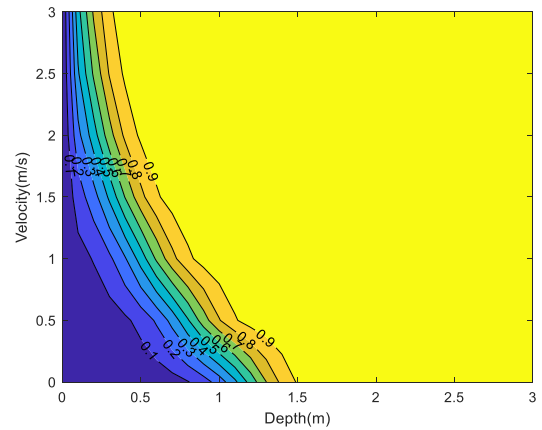
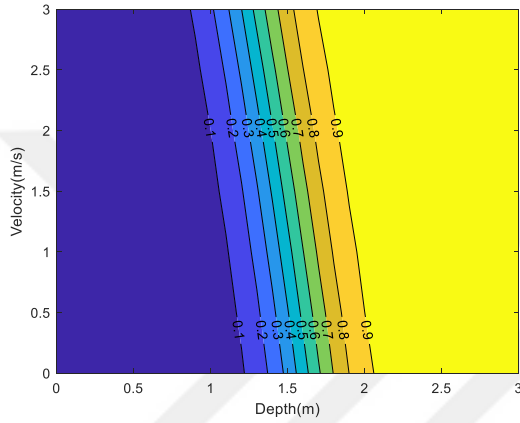
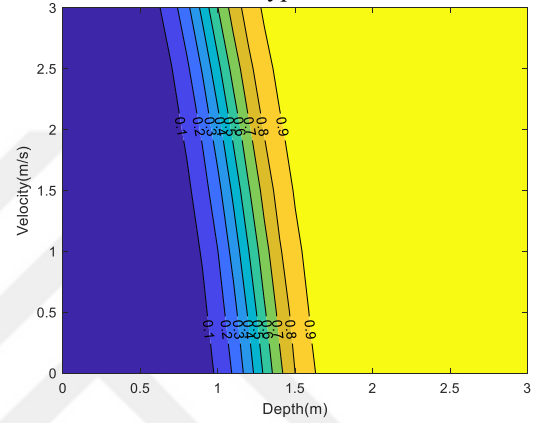


Figure 2.10: Collapse fragility contours for one-story URM buildings with four-edge supported walls (W4).

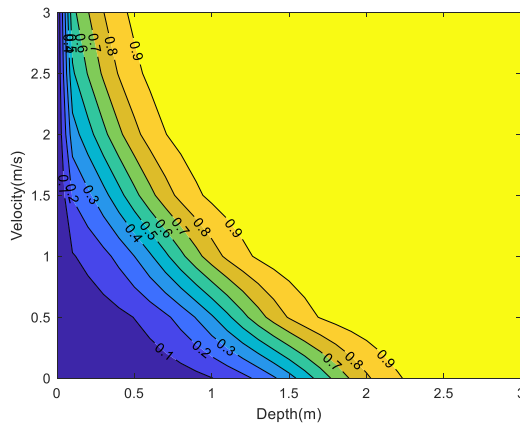
Case 7: W4-F-M0 Flood Flow, No Debris



Case 8: W4-L-M0 Flow-type Landslide, No Debris



Case 9: W4-F-MH Flood Flow, 453.6kg-Debris



Case 10: W4-L-MH Flow-type Landslide, 453.6kg-Debris

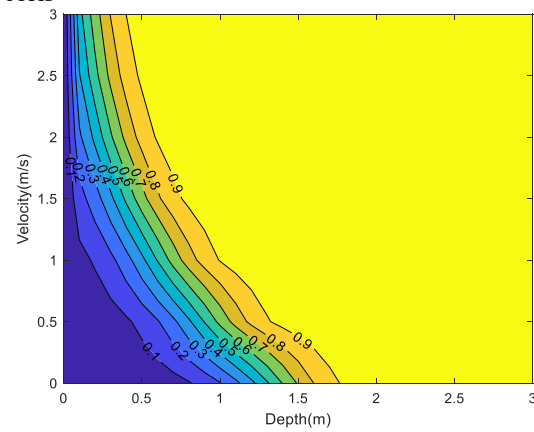
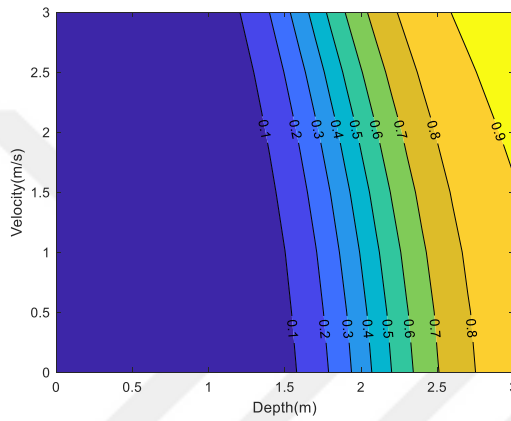
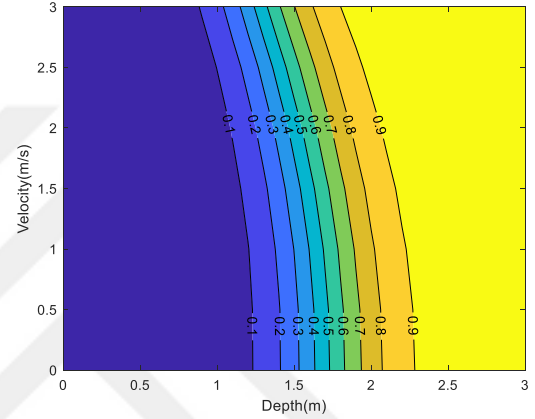


Figure 2.11: Collapse fragility contours for two-story URM buildings with four-edge supported walls (W4), considering the axial loads coming from the upper floors (UP)

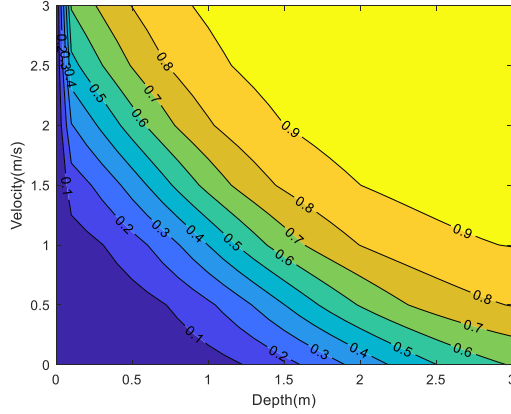
Case 11: W4-F-M0-UP Flood Flow, No Debris



Case 12: W4-L-M0-UP Flow-type Landslide, No Debris



Case 13: W4-F-MH-UP Flood Flow, 453.6kg-Debris



Case 14: W4-L-MH-UP Flow-type Landslide, 453.6kg-Debris

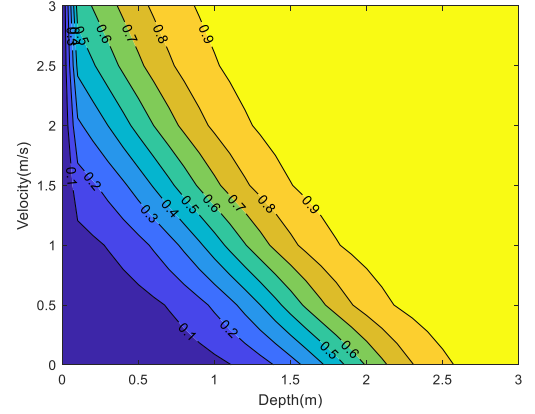


Table 2.7: The collapse depth values $d_{50\%}$ and $d_{90\%}$ for each of the fourteen cases that give 50% and 90% collapse likelihood, respectively, at a flow velocity of 1m/s.

Case No.*	Case Acronym**	$d_{50\%}$ (m)	$d_{90\%}$ (m)	$\frac{d_{90\%} - d_{50\%}}{d_{50\%}}$	Case No.*	Case Acronym**	$d_{50\%}$ (m)	$d_{90\%}$ (m)	$\frac{d_{90\%} - d_{50\%}}{d_{50\%}}$
1	W3-F-M0	1.37	1.71	24.8%	2	W3-L-M0	1.10	1.36	23.6%
3	W3-F-ML	1.23	1.58	28.4%	4	W3-L-ML	1.03	1.34	30.1%
5	W3-F-MH	0.57	0.95	66.7%	6	W3-L-MH	0.54	0.87	61.1%
7	W4-F-M0	1.53	1.95	27.4%	8	W4-L-M0	1.19	1.55	30.2%
9	W4-F-MH	0.74	1.28	72.9%	10	W4-L-MH	0.60	1.04	73.3%
11	W4-F-M0-UP	2.08	>3.00	44.2%	12	W4-L-M0-UP	1.69	2.22	31.3%
13	W4-F-MH-UP	1.20	2.90	141.7%	14	W4-L-MH-UP	1.10	1.86	69.1%

* see Table 2.2

** W3: Three-edge supported; W4: four-edge supported; F: flood flow; L: flow-type landslide; M0, ML, MH correspond to a debris mass value of zero light, and heavy, respectively;

UP: Presence of axial pressures on the top wall edge coming from the upper story.

2.4.5 Comparisons of the Proposed Collapse Fragility Models with Existing Ones

This section compares the proposed models for URM structures to existing collapse assessment models from the literature. Table 2.8 displays the selected existing models for comparison. It is important to note that most models in the literature primarily focus on flooding effects and use flow depth alone to account for these impacts, which were not included in this comparison. To ensure a fair comparison, the existing models in Table 2.8 were selected based on their inclusion of both flow depth and velocity, and their wall and flow properties being similar to those of the proposed models. The selected existing models employ a variety of approaches, including general collapse functions based on some collected field data [59, 60], simple elastic beam analysis [10, 18, 19, 20], and yield line analysis (or kinematic-based analysis) [11, 17]. However, none of these models consider debris impacts or landslide-related flow effects. Thus, Figure 2.12 compares the proposed collapse fragility models, specifically for flooding flow without debris effects (Case 7 and Case 11), to the existing models for one-story (left side of Figure 2.12) and two-story (right side of Figure 2.12) masonry structures, respectively.

When comparing the proposed models with existing models in Figure 2.12, a key difference is that the proposed models present a range of relationships for flood depth and velocity, each corresponding to different collapse likelihoods (with 10%, 50%, and 90%

collapse probabilities illustrated in Figure 2.12). In contrast, existing models provide only a single, fixed collapse prediction, disregarding variability in collapse estimates. By integrating significant uncertainties in structural and flow characteristics, the proposed models allow for a more comprehensive assessment, supporting a probabilistic and reliable evaluation of collapse likelihood across different flow depths and velocities. Conversely, existing models use deterministic approaches, yielding a single collapse prediction for each depth-velocity pair and potentially overlooking other possible outcomes under variations in wall and flow scenario conditions.

The existing models shown in Figure 2.12 each produce different results, likely due to varying simple assumptions about wall and flow characteristics, as well as differences in collapse assessment methods. Although these existing models are practical and rely on simple deterministic approaches, they may provide inaccurate or biased estimates of actual building collapse capacities under flow actions. For instance, the existing models that use simple analytical approaches, such as elastic beam analysis (orange lines) and yield line analysis (green lines), appear to underestimate the collapse levels for URM structures (i.e., staying toward the left in Figure 2.12) compared to the proposed models (black lines). Conversely, general functional models (red and purple lines) seem to overestimate these levels (i.e., staying toward the right top in Figure 2.12). On the other hand, the proposed models (black lines), which are based on nonlinear FE analysis approaches, systematically assess the effects of significant uncertainties associated with hazard and building features on building collapse evaluation. As a result, they provide more accurate and reliable collapse estimates along with their associated occurrence likelihoods (e.g., see 10%, 50%, and 90% collapse likelihood contours for the proposed models in Figure 2.12).

Lastly, while the existing approaches only consider flooding flow and ignore any debris effects, the proposed models in Figures 2.9-2.11 demonstrate their merits once

more by including various flow scenarios (flooding versus flow-type landslide with no, light, and heavy debris), different boundary conditions (W3 versus W4), and the influence of axial loads from the upper story on collapse estimates.

Figure 2.12: Comparison of the proposed models for one and two-story URM buildings under floods with existing ones in the literature, without considering any debris-related impact loads (note that the USACE model for the two-story building is out of the ranges, giving values more than 3m and 3m/s, so it was not shown on the right side).

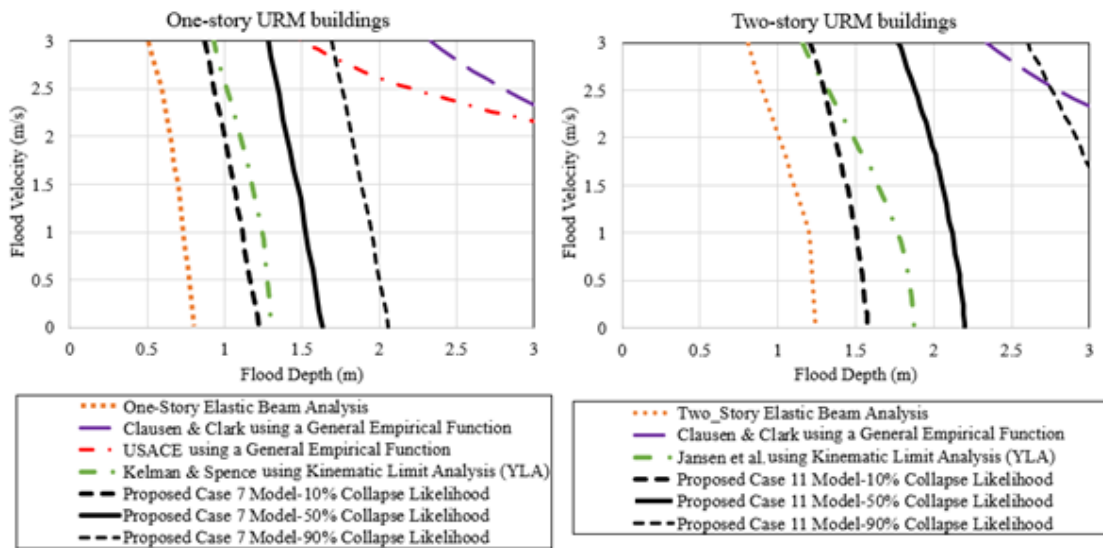


Table 2.8: Existing deterministic collapse models selected for comparison with the proposed models for URM structures.

Collapse Model Name	Number of Stories ^b	Wall Thickness (m), Height (m), Length (m)	Wall's Boundary Conditions ^c	Analysis Technique
Proposed Case 7-Model ^a	1	0.15 × 3.0 × 6.0	W4	Nonlinear FE
Proposed Case 11-Model ^a	2		W4	
Jansen et al. [11]	2	0.12 × 2.7 × 5.4	W4	Kinematic limit analysis (or Yield Line Analysis (YLA))
Kelman and Spence [17]	1	0.15 × 2.4 × 5.0	W4	Kinematic limit analysis (or YLA)
Elastic Beam Model for One Story [10, 18, 19, 20]	1	0.15 × 3.0 × 1.0	W4	Elastic analysis on one-span simply supported beam
Elastic Beam Model for Two Story [10, 18, 19, 20]	2	0.15 × 3.0 × 1.0	W4	Elastic analysis on one-span simply supported beam
USACE's criteria [59]	1, 2	A general collapse limit function was developed for masonry and concrete load-bearing wall buildings by the Portland District of the U.S. Army Corps of Engineers, using the results of the 1985 study on the flooding data for Willamette River System: $dv^{2.0406} > 14.169$ and $v > 1.92$ m/s in SI unit systems. No clear information was found on the structural properties used in their study.		
Clausen and Clarke's criteria [60]	n/a	Based on empirical data from mainly the 1864 Dale Dyke dam failure in Sheffield, a general collapse limit was defined for masonry and concrete residences in a functional form of a flow depth and velocity value: $dv > 7$ m ² /s and $v > 2$ m/s. No clear information was found on the structural properties used in their study.		

^a Proposed models for flood flows with no debris conditions

^b If the number of stories is 2, the models consider the axial loads coming from the upper floor, roof, and upper walls. Otherwise, no axial load is taken into consideration.

^c W4 stands for four-edge supported wall.

2.5 Conclusions

This study proposes reliable collapse risk assessment models (i.e., collapse fragility models) for vulnerable URM structures subjected to flood flows and flow-type landslides. The models depend on the collapse performance of a single load-bearing URM wall under flow actions, as failure of one wall typically results in the complete collapse of the structure. A nonlinear FE modeling approach, combined with an incremental flow analysis method, was first established to create structural collapse assessment tools for URM walls, which were subsequently validated using case studies selected from the literature. Using these validated tools, the proposed models were then developed for typical one- or two-story URM buildings in Europe, which are intended to be used as effective predictive models to assess collapse risk under flow actions.

Most building collapse assessment models for flow hazards in the literature use

simple deterministic analysis approaches, evaluating structural collapse under only hydrostatic pressures caused by flooding. They generally ignore any velocity-driven and debris impacts as well as landslide-related flow effects. Biased estimates and significant uncertainties in their collapse predictions highlight the importance and needs of using accurate assessment tools, incorporating the critical key factors, and accounting for existing uncertainties. The proposed models in this study are based on a more reliable nonlinear FE modeling and analysis approach, and they systematically assess the effects of significant uncertainties associated with hazard and building features on building collapse evaluation. Moreover, compared to the existing models, the proposed models demonstrate their merits by including various flow scenarios (flooding versus landslide-type flows with no, light and heavy debris impacts) and different structural characteristics (varying boundary conditions, with or without effects of axial loads from the upper story), therefore can provide more accurate and reliable collapse risk predictions for URM structures under flow actions.

The outcomes of this study show that both flood flows and flow-type landslides have the potential to seriously damage and even make vulnerable URM buildings collapse. Besides larger flow depth values, this potential is especially high for flow conditions with larger velocities and where heavy debris exists. Additionally, the study highlights the importance of considering proper support conditions in the collapse assessment of URM structures, as the four-edge supported walls were found to have larger resistance to collapse than three-edge supported walls. The study also examined the influence of axial loads transferred from the upper story to the walls, revealing it as a crucial factor that significantly augments the collapse resistance of walls.

Fragility assessments require thorough parametric studies to explore and account for relevant uncertainties in performance calculations, which involve balancing computational efforts and modeling assumptions. This study made several simplifications

and assumptions to efficiently conduct the computationally intensive parametric simulations. This approach is typical in fragility studies, provided the limitations are explicitly acknowledged. First, flood flows and flow-type landslides were assumed to be homogeneous throughout the flow depth. Second, the study focuses on primary flow actions—hydrostatic and hydrodynamic pressures, as well as debris impact forces—using simplified but widely used pressure or load formulations to model wall behavior under these varying flow conditions. Other effects, such as uplift and wave pressures, were not included in this study. Additionally, in order to efficiently facilitate parametric simulations, macro-models were used to represent the out-of-plane behavior of masonry walls under flow actions, omitting some potential collapse mechanisms, such as bricks sliding out of place due to weak joints or disaggregation of materials, particularly for poorly-constructed cohesive masonry structures. Moreover, the failure criterion was defined as the exceedance of base shear capacity by the demands induced by flow actions, facilitating the ease of identification of collapse in the parametric simulations. While these simplifications may introduce some additional modeling uncertainties in fragility development, they are likely minimal compared to the substantial uncertainties arising from the variabilities in key wall and flow parameters. Though such simplified approaches are widely accepted in fragility studies, users of the proposed models should be aware of these limitations. In addition to these limitations, the robustness of the models could be enhanced by expanding their application to include a wider range of wall dimensions, providing a more comprehensive risk assessment of URM walls. Furthermore, additional studies could explore advanced micro-modeling techniques, nonhomogeneous flow conditions, and their secondary effects on structures, such as uplift pressures and wave actions.

In conclusion, despite the aforementioned limitations, the proposed collapse fragility

models represent a comprehensive and significant advancement in understanding the collapse potential of URM buildings under flood and flow-type landslide actions. By systematically addressing significant uncertainties and various flow scenarios, these models provide reliable estimates of the likelihood of structural collapse during flow events. This makes them valuable tools for enhancing national building codes, regional emergency response plans, and risk management strategies, with a focus on improving occupant safety by minimizing the probability of building collapse during extreme hazard events. Additionally, these models support risk-based decision-making among different stakeholders (e.g., engineers, policymakers, governmental agencies, insurance companies, and homeowners) regarding retrofit and mitigation measures to improve the resistance of URM buildings to flow hazards. Such efforts are highly critical for safeguarding communities during extreme events and strengthening societal resilience to hazards.

3. COMPARATIVE DAMAGE FRAGILITY ASSESSMENTS OF EARTHQUAKE-RESISTANT AND GRAVITY-LOAD DESIGNED RC FRAMED BUILDINGS UNDER FLOOD AND FLOW-TYPE LANDSLIDE ACTIONS

In response to the growing threat of extreme flow events, this study evaluates the damage potential of reinforced concrete (RC) frames using numerical analysis. Focusing on low-rise RC structures, two design categories are considered: one featuring earthquake-resistant and the other employing a gravity-load design representative of structures typically encountered in suburban outskirts, rural areas, and mountainous valleys that are at high risk from hazardous flow conditions. Fragility surfaces were constructed using incremental flow analyses and fragility functions were developed through logistic regression to quantify damage probabilities under various flow conditions, with performance criteria assessed according to both FEMA356 and Eurocode8-part3 standards. This integrated approach systematically accounts for uncertainties in design criteria, material properties, geometry, flow intensity, and debris effects. These predictive models, besides providing a robust framework for forecasting the structural response and damage levels of RC frames under extreme flow scenarios, also offer critical insights for optimizing design and retrofitting strategies to mitigate damage in RC structures exposed to flood and flow-induced landslide hazards.

3.1 Introduction

Geohydrological hazards, such as floods and flow-type landslides, significantly impact regions and society globally. Compared to floods, flow-type landslides are characterized by high water content combined with solid materials. They usually exhibit higher flow density and exert substantial hydraulic forces on built environments. Both types of

Figure 3.1: *Damage to a reinforced concrete (RC) frame building due to flooding and flow-type landslide event in Adıyaman, Turkey (March 2023). The enlarged picture on the right highlights structural damage observed at the ground-floor columns[1].*



flow events can cause significant damage to buildings and infrastructure, especially when they involve debris impact. These hazards typically involve the rapid movement of flow water mixed with solid debris—such as floating tree trunks, aggregates, or similar materials—leading to extensive destruction, particularly in mountainous and urban areas. Figure 3.1 presents an illustrative example of a reinforced concrete (RC) frame building severely impacted, during the flooding and flow-type landslide events of March 2023 in Adıyaman, Türkiye [1]. This figure clearly demonstrates significant structural damage, including severe deformation and partial collapse of ground-floor columns, caused by the combined effects of flow pressures and floating debris collisions.

Since RC frame structures frequently suffer extensive structural damage under these extreme conditions, fragility models are essential tools within probabilistic risk assessment frameworks, quantifying the likelihood of exceeding predefined damage states under various hazard scenarios. Modern building design codes specify multiple damage states, and fragility assessments that explicitly incorporate these performance criteria provide engineers and policymakers with critical insights into structural vulnerabilities

across these states, enabling effective risk mitigation and targeted retrofitting strategies. These models systematically address uncertainties related to hazard characteristics and structural capacities, thus improving the reliability of structural performance predictions under hazard conditions.

In the literature, fragility studies for flow impacts on RC frame structures are classified into two distinct methodologies: numerical-based and data-driven approaches [61], as summarized in Table 3.1. Numerical-based approaches, such as those used by Parisi and Sabella [6], Zoppo et al. [62], Lue et al. [63], Prieto et al. [64], Roos [10] and Nadal et al. [53] apply simplified nonlinear analyses or computational modeling techniques to efficiently simulate structural responses under hazard conditions. Data-driven methods, as exemplified by the works of Kang and Kim [65], Zhang et al. [66], and Chen et al. [67], rely on empirical damage data collected from post-event surveys to establish direct relationships between observed damages and hazard characteristics.

Table 3.1 summarizes recent studies on fragility and vulnerability assessments of structures subjected to flow-induced hazards using both numerical-based and data-driven approaches. In these works, the primary flow hazards investigated include flooding or tsunamis [62], and flow-type landslides [6, 62, 63, 64, 65, 66, 67] often referred to as debris flows [63, 64, 65, 66, 67]. To quantify severity of these hazards, several intensity measures have been proposed, ranging from flow velocity (v) and depth (d) to momentum flux (dv^2) [62, 63, 64, 65, 66, 67], impact pressure [65], or overturning moment (dv) [66].

The buildings considered in these studies commonly employ RC frames in various configurations, including bare frames, masonry-infilled frames, or frames categorized by ductility level [62]. Some investigations additionally consider non-RC typologies, such as masonry or wood buildings. To assess the fragilities of these buildings, the studies based on numerical approaches typically employ computational methods, such as non-linear finite-element analysis [63] or lumped-plasticity models [6](often implemented

in platforms like OpenSees), or specialized frameworks like the SAFETY (VDPO-BI) method [62]. Uncertainty in material and structural properties is frequently addressed through Monte Carlo simulations (e.g., 10^4 – 10^5 samples) and Latin Hypercube sampling [62]. Meanwhile, data-driven studies derive empirical fragility or vulnerability curves from field-survey data and regression analyses, sometimes utilizing hundreds of damaged-building records to capture real-world variability [66, 67].

Damage States (DSs) in these studies generally align with established standards such as Eurocode (e.g., [6]), HAZUS Flood Manual [68] and HAZUS Tsunami Manual (e.g., [64, 65]). Commonly used states include slight, moderate, extensive (heavy), and complete damage. The specific corresponding quantitative threshold for each state varies: some studies employ moment-rotation capacities [6, 62] or displacement thresholds of 20%, 50%, and 80% of building or component capacity [63], whereas others use impact-pressure limits [65, 66], for instance, moderate damage at 35–100 kPa and complete damage above 100 kPa [65]. Such standardized criteria enable comparative assessments of vulnerability and resilience across different structural typologies, hazard types, and modeling approaches.

Despite significant research advances in the literature, notable gaps persist in understanding and quantifying damage fragilities for RC buildings under combined geohydrological hazards. Although previous studies have addressed vulnerability to flow scenarios (see Table 3.1), they generally focus on individual flow hazards only (e.g., only high-density flows), limiting their applicability to realistic multi-hazard scenarios frequently encountered in urban environments. Moreover, these existing studies do not explicitly account for the effects of floating debris such as tree logs impacting RC frames. They also usually focus on solely one standard or source for damage state thresholds, and do not consider diversity in damage descriptions given in different international codes.

This study addresses these gaps by introducing a novel analytical fragility assessment specifically developed for RC frame structures subjected to floods (flows with densities around 1025 kg/m^3) and flow-type landslides with high-density, offering a more comprehensive evaluation of RC frame vulnerability to flow events. Also, this study explicitly accounts for associated loads such as floating debris impacts in addition to flow pressures. Two distinct RC frame typologies are evaluated: earthquake-resistant frames designed for high ductility, and gravity load-designed frames lacking specific seismic detailing. Furthermore, this study develops and compares fragility models for RC frames based on different damage-state thresholds defined by two widely used international standards, US-based guidelines of Federal Emergency Management Agency [3] (FEMA 356) and European seismic design standards [4] (Eurocode8-Part 3). Employing damage thresholds in FEMA and Eurocode guidelines, this approach allows for a more comprehensive comparison and a broader perspective on structural vulnerability of RC frames, while also enhancing the practical utility and applicability of the proposed models in diverse international contexts.

Another primary innovation of this study involves systematically incorporating significant uncertainties of structural properties and multi-hazard loading conditions for RC frames into proposed fragility models. This is critical to obtain reliable predictions on their performance under different flow scenarios. Furthermore, proposed fragility models are presented through surface plots and fragility functions, described in terms of critical hazard characteristics, thus offering clearer and wider insight compared to traditional fragility curves. This advancement directly supports improved risk mitigation strategies and resilient engineering practices aligned with international disaster loss prevention objectives, such as those outlined in the European Union's disaster resilience goals [69].

Table 3.1: Summary of existing fragility and vulnerability studies for structures subjected to flow-induced hazards, including loading types, intensity measures, structural typologies, modeling methods, damage states, and analysis approaches.

Researchers reference	Load type	Structure type	Intensity measures	Model output, type of function	Damage states
Parisi & Sabella. [6]	Flow type Landslide	2-bay 2-story building: 1) Individual masonry infill wall 2) Bare RC frame 3) Masonry infilled frame	Flow velocity	Fragility curves derived via: - Monte Carlo simulations (10^4 samples) - Nonlinear static pushover analysis (OpenSees) for RC frames - Equivalent uniform pressure calculation (Mathematica) for infill walls	Structural (RC frames): In-plane damage: - DS1: Cracking of infill walls (slight damage) - DS2: Formation of plastic hinge in a column (moderate damage) - DS3: Rupture of plastic hinge (severe damage)
Zoppo et al. [62]	Tsunami Flow type landslide	RC frame buildings: 2 to 3 stories, 2 to 7 bays 1) Ductile 2) Brittle	Flow depth Flow velocity Momentum Flux (dx^2)	Fragility curves derived via SAFETI method (mechanics-based VDPO-BI analysis), validated against OpenSees FE models, Latin Hypercube algorithm with 105 Monte Carlo samples for portfolio analysis.	Structural: Ductile: - Slight: Column cracking (M_{cr}) - Moderate: Half-yielding (M_{hy}) - Extensive: Yielding (M_y) - Complete: Soft-story mechanism Brittle: - Complete: Shear failure of ground-story columns (EC8-3)
Lue et al. [63]	Debris Flow	2-bay 3-span 3-story bare RC frame	Flow depth Flow velocity Momentum Flux	Fragility curves derived via: - Monte Carlo simulations (1,000 samples) - Nonlinear finite-element pushover analysis (OpenSees)	Damage states (lateral displacement thresholds): - Slight: 20% of building capacity - Moderate: 50% of building capacity - Heavy: 80% of building capacity - Complete: Based on impact force
Prieto et al. [64]	Debris Flow	Six representative building types from the HAZUS taxonomy: - Wood buildings - Concrete shear wall buildings - Unreinforced masonry bearing walls buildings	Momentum Flux	Fragility curves derived via: - Monte Carlo simulations and SRSS uncertainty propagation.	Structural (Based on HAZUS Tsunami FEMA 2017): - Moderate: Yield capacity - Extensive: Average value of the yield and ultimate capacity - Complete: Ultimate yield force capacity
Roos [10]	Flood	1) Masonry (1B/1B2) buildings 2) Cast Concrete (CC) buildings 3) Prefabricated Concrete (PF) buildings	Flow depth Flow velocity	Fragility curves derived via: - GIS-integrated probabilistic thresholds	Structural: - Partial collapse: Failure of walls or foundation scour - Total collapse: 70% of partially collapsed buildings
Nadal et al. [53]	Flood	RC frame with infill concrete-block walls	Flow depth Flow velocity	3D vulnerability surface (EDP function) derived via: - Monte Carlo simulations (104 samples) - EFD expressed as a percentage of the building's total replacement value.	- Total loss: When EFD $\geq 60\%$ (repair cost replacement cost)
Kang & Kim. [65]	Debris Flow	Focus on Korean residential/commercial buildings: 1) Non-RC frame Masonry, Wood frame and Light steel frame 2) RC frame	Flow depth Flow velocity Impact pressure	Empirical vulnerability curves derived via: - field data (25 buildings, 11 debris flow events) - Sigmoid regression functions for RC/non-RC buildings.	RC (Aligned their damage classification with HAZUS Earthquake terminology) - Slight: Impact pressure < 35 kPa - Moderate: Impact pressure 35-100 kPa - Extensive: Impact pressure > 100 kPa
Zhang et al. [66]	Debris Flow	1) Brick-concrete buildings 2) RC frames	Flow depth Flow velocity Impact pressure Dynamic force Overtopping moment and relative intensity	Empirical vulnerability curves derived via: - logistic regression analysis of field data (390 damaged buildings) - and FLO-2D numerical simulations	Four damage states in Zone I (after Hu et al. [70]): - Slight: Impact pressure < 26.2 kPa - Moderate: Impact pressure 26.2-50 kPa - Heavy: Impact pressure 50-90 kPa - Complete: Impact pressure > 90.0 kPa
Chen et al. [67]	Debris Flow	1) Masonry-wood buildings 2) Brick-concrete buildings	Momentum Flux	Fragility curves Based on field survey data of 219 buildings	- Slight (D1- up to 25% insured loss) - Moderate (D2- 25%-75% insured loss) - Extensive (D3- >75% insured loss)

3.2 Methodology

Based on the probabilistic approach outlined in Figure 3.2, this study introduces fragility models (i.e., robust damage risk prediction models) for RC frame buildings subjected to flood and flow-type landslide hazards. Initially, a detailed literature review was performed, examining both numerical-based and data-driven approaches for fragility modeling of RC frame structures under flow-related hazards. This review identified critical hazard characteristics (e.g., hydraulic pressures and debris impacts) necessary to realistically evaluate the response of RC frames during severe hydrological events. Subsequently, a typical low-rise RC frame in Europe with high ductility was selected from existing literature [71], together with a corresponding gravity-based design frame. They were both modeled through the lumped plasticity approach using OpenSees [5] software, a widely used platform for nonlinear structural analysis. A pushover analysis was then conducted on this frame to validate the OpenSees modeling approach by comparing the results with the ones performed with a commercial finite element software, ETABS [72], as well as with ones coming from the referenced literature case study.

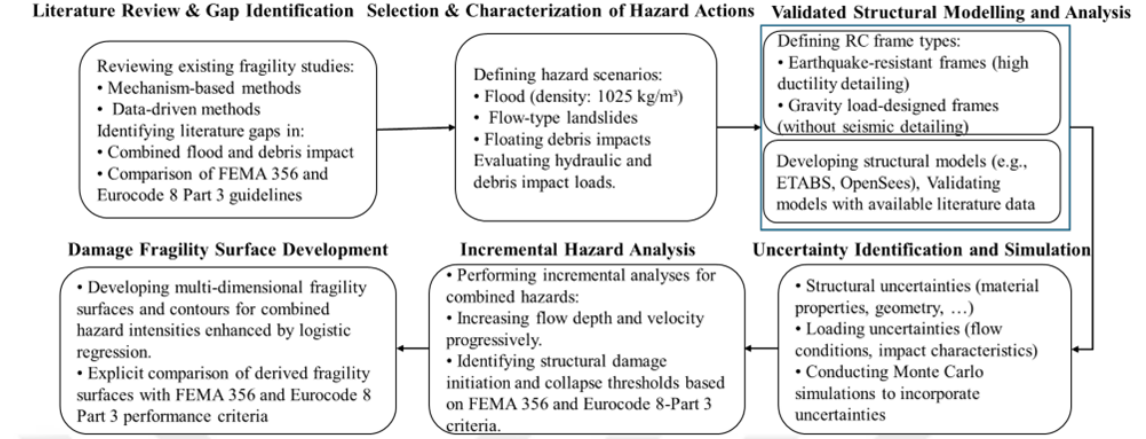
Following validation, this study comprehensively assessed key sources of uncertainty related to structural modeling, such as variability in material properties, component dimensions, and modeling assumptions. Hazard-related uncertainties, including variations in flow density and debris impact characteristics, were similarly identified. To quantify and incorporate these uncertainties effectively, Monte Carlo simulation techniques were utilized, allowing probabilistic representation of structural and load inputs. After integrating these uncertainties, incremental flow analyses (IFA) were carried out for various combinations of flood and flow-type landslide conditions by incrementally increasing their severity. Damage thresholds were carefully identified and classified according to criteria from FEMA 356 [3] and Eurocode8-Part3 [4] standards. Logistic regressions were then applied to simulation outputs to develop multi-dimensional fragility surfaces and

functions, effectively demonstrating the probability of reaching specific damage states under various hazard intensity scenarios. This research intends to produce accurate and practical fragility models for RC frame structures facing combined geohydrological hazards (i.e., floods and flow-type landslides). Considering the extensive numerical simulations required (totally 1.2×10^6 simulations), certain practical simplifications and assumptions were adopted, consistent with widely accepted practices in fragility studies (e.g., [6]).

This study adopts simplifications in hazard and structural modeling to balance computational efficiency and scope. Hazard actions (floods/flow-type landslides) are idealized as homogeneous flows with static primary loads, neglecting secondary effects like erosion, uplift, or wave dynamics. While lumped plasticity modeling efficiently captures flexural failure modes [73], it does not explicitly account for panel zone deformations at beam-column joints, potentially underrepresenting shear-dominated brittle failures. Consequently, the simulated failure modes assumed in this study (Earthquake-resistance & gravity-load-design) primarily originate from flexural mechanisms, which was often accepted and used in the literature [6].

Distributed plasticity models and dynamic analyses for any hydraulic or impact actions were omitted due to computational constraints, limiting the resolution of localized failure mechanisms. These assumptions, though consistent with common fragility modeling practices, warrant consideration when interpreting results for scenarios dominated by shear or dynamic loading. Detailed information on modeling assumptions, parameter selection, and analytical methods are discussed in the subsequent sections.

Figure 3.2: Probabilistic framework considered for the flow risk assessment of RC frames.

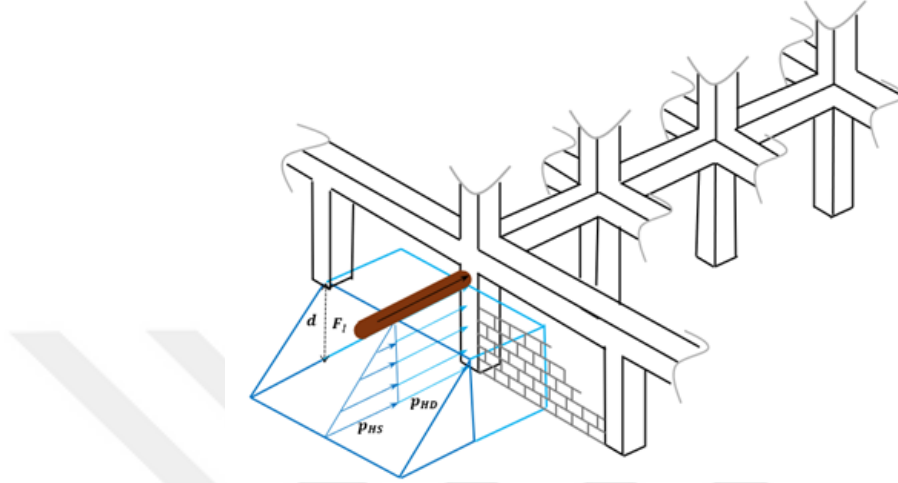


3.2.1 Flow Actions

In this research, RC frame structures subjected to flood and flow-type landslide actions are assessed by considering primary loading conditions, including hydrostatic pressures, hydrodynamic pressures, and debris impact forces. Figure 3.3 demonstrates schematically the distribution patterns and directions of these flow-induced loads. Landslide induced flows contain higher densities due to sediment transport, leading to greater forces exerted on structural components compared to conventional floods, highlighting the significant differences between typical flood events and flow-type landslides. The formulation and interpretation of these flow actions are based on the approach developed in Chapter 2 Section 2.2.1 (or see the paper by Moaiyedfar and Deniz [74]), where the structural response of unreinforced masonry (URM) buildings under the same loading mechanisms was explored. The same fundamental principles apply here; however, this study adapts the formulation to RC frame structures, which are characterized by distinct geometric and mechanical properties. In particular, for internal bare frames, the effective width of flow exposure on the RC frame (in the out of the plane direction) is estimated using the relation $b_{\text{effective}} = b + L_w$, where b is the column width and L_w is the clear length

of the wall or opening.

Figure 3.3: Schematic representation of flow-induced forces acting on RC frame elements, including triangularly distributed hydrostatic pressures (P_{HS}), uniformly distributed hydrodynamic (P_{HD}) pressures, and debris impact (F_I) force.



3.2.2 Analytical Modeling and Validation of RC Frame Structures

This study focuses on the structural modeling of RC frame structures using macro-modeling methods, specifically the lumped plasticity approach. Lumped plasticity modeling has been widely adopted in the literature due to its simplicity and computational efficiency in capturing the inelastic behavior of structural elements. For example, Harati & Van de Lindt [75] and Parisi & Sabella [6] employed lumped plasticity formulations to conduct parametric studies of RC frames under tsunami and flow-type landslides, respectively. In this work, two primary types of RC frames were considered: earthquake-resistant frames designed with high ductility detailing, and frames primarily designed for gravity loads without specific seismic detailing. For this purpose, a typical low-rise RC frame in Europe with high ductility was selected from existing literature [71], together with a corresponding gravity-based design frame. Both frames are three stories high, have three bays, and share the same member dimensions as well as gravity loading conditions. The main differences lie in the reinforcement detailing of the member sections.

In Section 3.3, OpenSees [5], was chosen to build the analytical models for both frames and exploration of their performance under various loading conditions. The earthquake resistant and gravity-load designed RC frames were modeled utilizing the lumped plasticity approach and subsequently validated. This validation was conducted by comparing the nonlinear static (pushover) analysis results obtained from the lumped plasticity models with those from a nonlinear modeling approach built using a commercial finite element program, ETABS [72]. The OpenSees results for the earthquake-resistant frame was also validated against the results reported ([76]) for the case study in the literature [71]. This comparative analysis ensured the accuracy and reliability of the modeling method applied in this study.

Following the successful validation, these verified analytical tools were employed in Section 3.4 to develop a representative case set of analytical models using lumped plasticity approach for three-story, three-bay RC frames, intended for use in damage fragility analyses. These RC frame models serve as representations of typical low-rise RC building stocks in Europe. The validated, practical yet sufficiently accurate lumped plasticity modeling approach thus enables conducting comprehensive parametric studies and robust fragility analyses under flood and flow-type landslide hazard conditions, as discussed in subsequent sections.

3.2.3 Probabilistic Assessment and Monte Carlo Simulations

Reliable assessment of the damage potential of RC frame structures under flood and flow-type landslide hazards require incorporating uncertainties in both structural properties and hazard conditions. Structural uncertainties come from differences in material properties, geometric configurations, and modeling assumptions, while hazard uncertainties result from variations in flow depth, velocity, density and debris impact characteristics. A probabilistic approach using Monte Carlo simulations was employed to integrate

these uncertainties, enabling a more comprehensive evaluation of RC frame behavior under different hazard scenarios.

The statistical distributions and parameters used to describe these uncertainties, including mean values and coefficients of variation (COV), are discussed in detail in Section 3.4.2. Depending on the nature of the variable, probability distributions such as normal, lognormal, or uniform were assigned. The statistical parameters were obtained from an extensive review of relevant literature and experimental studies focused on RC frame performance under flow actions.

3.2.4 Incremental Flow Analysis for Damage Assessment

Assessing how RC frames respond to increasing flood and flow-type landslide forces is crucial for understanding their structural vulnerability. This study applies an incremental flow analysis (IFA) approach to monitor how damage develops as flow intensity increases. Instead of focusing only on structural failure (collapse), this method captures different damage states, providing a more detailed evaluation of RC frame performance.

Figure 3.4 outlines the IFA process, where flow depth and velocity are gradually increased in small increments. At each step, flow-induced loads are applied to the RC frame model. The analysis is conducted using a pushover approach. The process continues until the structural responses to flow actions exceed predefined thresholds, classifying structural performance into different damage states according to drift-based damage states (FEMA 356 [3] and Ghobarah's study [77]) and Eurocode 8 Part 3 [4].

Table 3.2 provides an overview of the three structural damage states considered in this study. The DS1, which corresponds to Immediate Occupancy (IO) in FEMA and

Damage Limitation (DL) in Eurocode, is characterized by minor hairline cracks in structural elements, with non-structural components remaining functional. According to Ghobarah's study [77], non-ductile RC frames exhibit a drift limit of 0.2% at this stage, aligning with repairable light damage. DS2, which corresponds to Life Safety (LS) in FEMA and Significant Damage (SD) in Eurocode, includes moderate cracking and spalling in beams and columns, leading to partial strength degradation, but the structure remains repairable. Ghobarah's criteria specify a drift ratio of 0.5% for non-ductile RC frames at DS2, marking the damage beyond the yield point. DS3, which corresponds to Collapse Prevention (CP) in FEMA and Near Collapse (NC) in Eurocode, represents extensive plastic deformations and significant drifts imposed on the frame, and a severe loss of lateral strength, indicating a near-collapse condition. For non-ductile systems, Ghobarah identifies a critical drift limit of 0.8% at DS3.

The IFA methodology is applied to three-story, three-bay, two-dimensional RC frames, which are considered as subsystems taken out from the overall three-dimensional building. This approach is often employed to reduce computational efforts. Several incremental analyses are conducted on these frame models until the models reach the defined damage thresholds. This process is repeated for different variations in material properties and loading conditions to develop a comprehensive dataset for fragility assessment, as discussed in Section 3.4.

Table 3.2: Damage states considered in this study according to Ghobarah's study[77], FEMA 356 [3] and Eurocode 8 Part 3 [4] criteria.

FEMA 356 Damage States	Drift Limits	Description	Damage States Based on Ghobarah's [77]	Drift Limits	Eurocode 8 Part 3 Damage State	Chord Rotation Limits	Description
[3]					[4]	(θ)	
DS1: Immediate Occupancy (IO)	1% transient drift, negligible permanent drift	Minor hairline cracks, ,no crushing, structure retains full strength and stiffness.	DS1: Repairable damage, light damage	0.2%	DS1: Damage Limitation (DL)	θ_y (Yield Rotation)	Light cracking in structural elements, non-structural components remain functional.
DS2: Life Safety (LS)	2% transient drift, 1% permanent drift	Significant damage to structural elements, moderate permanent drifts, structure may require repair but remain stable.	DS2: Beyond the yield point	0.5%	DS2: Significant Damage (SD)	$(3/4)\theta_u$	Moderate cracking, spalling in beams and columns, some loss of lateral strength, structure still repairable.
DS3: Collapse Prevention (CP)	4% transient or permanent drift	Extensive cracking, large deformations, significant loss of lateral strength, structure is near collapse.	DS3: Severe damage, Partial collapse	0.8%	DS3: Near Collapse (NC)	θ_u (Ultimate Chord Rotation)	Extensive plastic deformations, large permanent drifts, structure at high risk of collapse.

3.2.5 Fragility analyses and damage-state assessment models

This research utilizes a probabilistic approach to establish fragility curves for three specific damage states in RC frame structures, incorporating uncertainties related to structural properties and flow characteristics. The methodology closely follows the approach introduced in Chapter 2 (or the paper by Moaiyedfar and Deniz [74]), which focused on collapse fragility modeling of URM buildings under similar conditions. However, the current study extends that framework to account for multiple damage states and the distinct mechanical behavior of RC frames. The primary objective of fragility analysis is to determine the likelihood that structural responses under various flow depths (d) and velocities (v) will exceed pre-defined damage thresholds [37]. Based on the procedure described in Figure 3.4, the probability of surpassing a given damage state at specific hazard intensity levels (depth and velocity) can be defined by Equation (3.1). Since experimental data and field observations are limited, numerical simulations are typically employed to produce fragility curves. These simulations yield datasets that classify occurrences into binary outcomes: exceeding or not exceeding specific damage states. Thus, the exceedance probability for any damage state at a certain intensity is determined by the ratio between the number of cases indicating damage threshold exceedance (N_{DS_i}) and the total simulations (N_{total}) conducted at that intensity:

$$P_{\text{Cond}} = P(DS_i \mid d, v) = P(\text{Demand}_{d,v} \geq \text{Capacity}_{DS_i} \mid d, v) \quad (3.1)$$

$$P_{\text{Cond}} = \frac{N_{DS_i}}{N_{\text{Total}}} \quad (3.2)$$

Various regression techniques have been adopted in the literature to estimate probabilities of damage exceedance, accounting for uncertainties in structural demand and capacity derived from simulation outputs or empirical evidence [16, 30, 38, 39]. Among these, logistic regression is particularly suitable due to its effectiveness in modeling binary data such as exceedance specific damage state (*yes/no*) [38, 40, 41]. In this study,

logistic regression is employed to create probabilistic relationships linking flow depth (d) and velocity (v) to damage exceedance likelihood. A robust dataset consisting of totally 1.2×10^6 Monte Carlo simulation points was produced for comprehensive regression analyses.

The logistic regression model used to predict damage exceedance probabilities based on given intensity values (d, v) is expressed as:

$$P_{\text{Cond}} = \frac{1}{1 + e^{-\phi(d,v)}} \quad (3.3)$$

In Equation (3.3), the logit function $\phi(d, v)$ transforms the probability values from the $[0, 1]$ interval onto the real number axis, enabling regression modeling. For the current study, an interaction function between intensity measures depth and velocity was employed. Thus, the logit function is defined as:

$$\phi(d, v) = \ln \left(\frac{P_{\text{Cond}}}{1 + P_{\text{Cond}}} \right) = \alpha_0 + \alpha_1 d + \alpha_2 v + \alpha_3 dv \quad (3.4)$$

Detailed logit function formulations for each case and each damage state are comprehensively provided in Section 3.4.4 and Table 3.11.

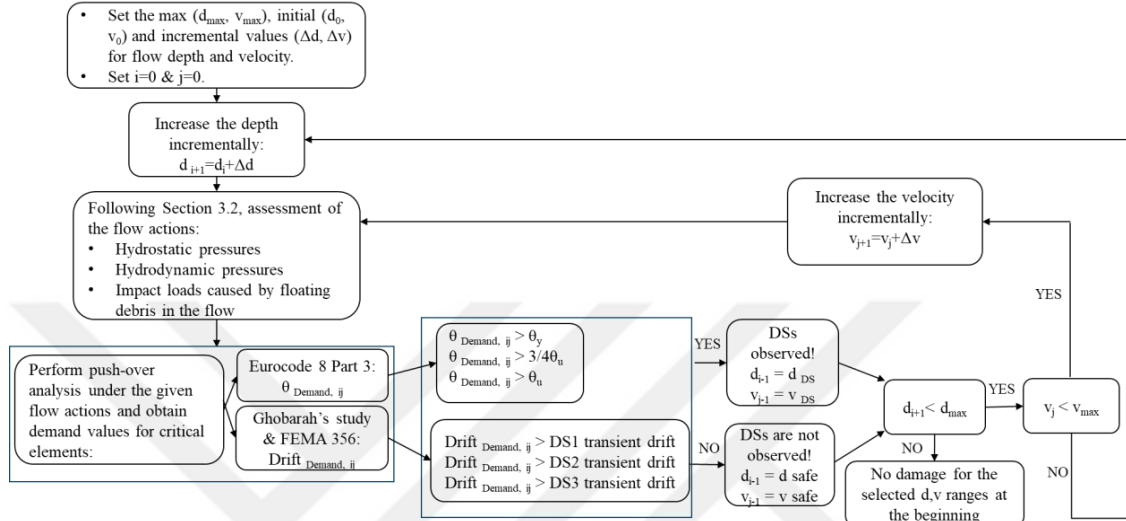
The logistic regression coefficients (α) are optimally estimated via maximum likelihood estimation [42], aiming to maximize the likelihood of obtaining the observed simulation data. Thus, the log-likelihood function used for estimating regression parameters $\alpha_0, \alpha_1, \dots, \alpha_3$, is formulated as follows:

$$LL(\alpha_0, \alpha_1, \dots, \alpha_3) = \sum_{i=1}^N [\text{obs}_i \ln(P_{\text{Cond},i}) + (1 - \text{obs}_i) \ln(1 - P_{\text{Cond},i})] \quad (3.5)$$

In Equation (3.5), obs_i denotes the observed binary outcome at the i^{th} simulation (1 for damage exceedance, 0 otherwise), and $P_{\text{Cond},i}$ is the modeled probability of exceedance based on intensity measures d_i and v_i . A confusion matrix and other relevant metrics

are subsequently employed to evaluate the regression models' predictive accuracy, as discussed in Section 3.4.3.

Figure 3.4: Flowchart illustrating the Incremental Flow Analysis (IFA) framework for RC frames.



3.3 Validated structural modeling and analysis tools

This section presents the structural modeling details of RC frames considered in the study. First, an earthquake resistant case study from Ferraioli [71] was selected and a corresponding gravity-based design case study (same building dimensions) was created to develop the structural models to be used later in deriving damage fragilities. To ensure the accuracy and reliability of the structural models, two different FE programs, both OpenSees [5] and ETABS [72], were utilized to build the models of the case study and to compare and validate their results under pushover analyses. The model results for earthquake resistant case study was also compared and validated against the ones reported in the paper by Ferraioli [71].

Following validation, the OpenSees modeling approach and IFA methodology were applied in Section 3.4 to develop more case models representing representative

Three-Story Three-Bay (3S3B) low-rise RC frames for further analysis. Additional details are provided in the following subsections.

3.3.1 The Selected Case Studies

A representative earthquake-resistant low-rise RC case study was selected from Ferraioli [71] to validate the structural models to be later used in the fragility analyses. This is one of the RC frames presented in Ferraioli [71], where its seismic performance was assessed through pushover analyses. The selected case study is a three-dimensional (3D) RC building (see Figure 3.5 a and b), consisting of three stories, with a first-floor height of 3.5 meters and second- and third-floor heights of 3 meters. It has three bays, each spanning 5 meters in both directions. Building upon this case study selection, a middle frame was chosen from the 3D RC building with two configurations: An earthquake-resistant frame as given in Ferraioli [71], designed with high-ductility detailing (see Table 3.3 and Figure 3.5c), and a corresponding gravity-load-designed frame, created of the similar geometry but without sufficient seismic detailing (see Figure 3.5d).

The earthquake resistant frame was constructed using *C25/30* concrete and *B450* steel reinforcement and was designed in strict compliance with the modern Italian Building Code (NTC-2018) [78] for seismic load calculations, structural analysis, and strength/drift verifications using strong-column weak-beam criteria. The behavior factor (q) and ductility requirements were derived from Eurocode 8 part1 [79], as explicitly adopted by NTC-2018 for high ductility class structures. The key seismic design parameters of the selected frame are summarized in Table 3.3 below.

Figure 3.5: (a) 3D view of the selected RC frame case study; (b) its floor plan [71]; and elevation views for (c) earthquake-resistant frame, and (d) gravity-load-designed frame [71].

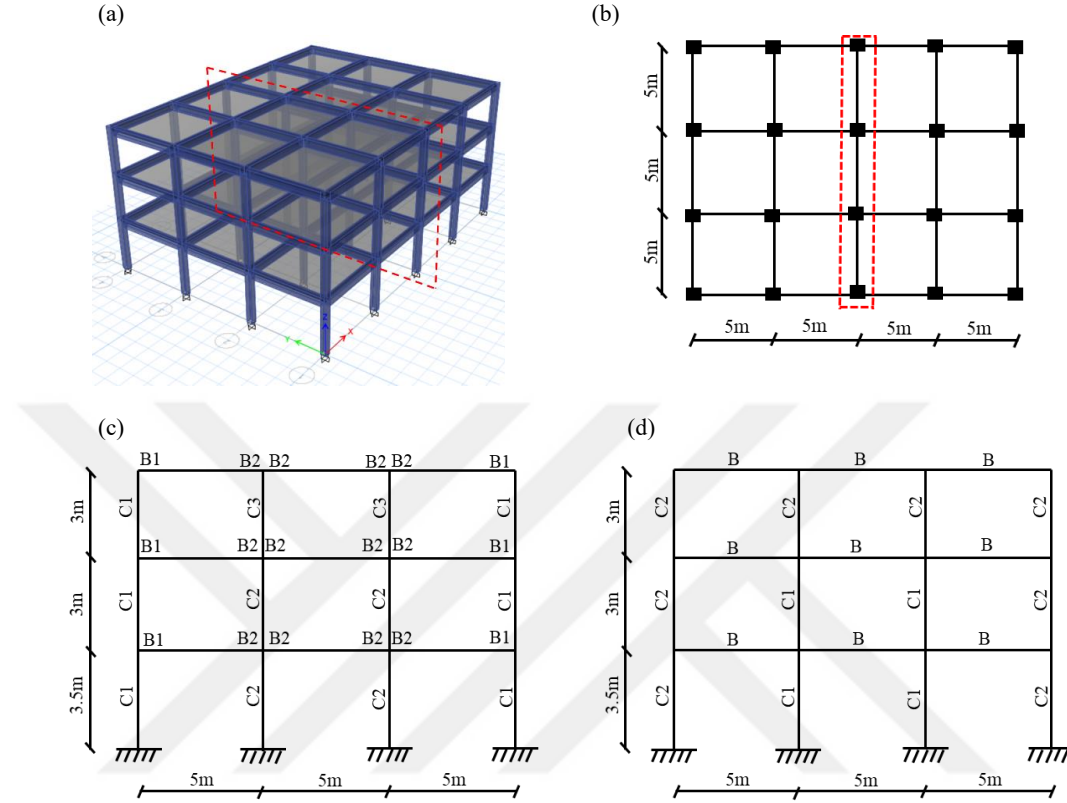


Table 3.3: Key seismic design parameters of the earthquake-resistant RC frame case study selected from Ferraioli[71].

Seismic Design Parameters	Value
Soil Class	A (rock or very stiff soil)
Peak Ground Acceleration (PGA)	0.25g
Damping Ratio	5% of critical damping
Behavior Factor (q)	5.85 high-ductility RC moment frame

The material properties and structural detailing of gravity-load designed frames in this study is classified as "precode" when compared to contemporary European codes, particularly Eurocode 8 (EN 1998), which mandates seismic design principles. The analyzed gravity-load designed frames aligned with the models (e.g., [6]) representative of Italian pre-1971 construction practices (Royal Decree 1939 [80]), lack seismic detailing and rely on older material specifications (e.g., *C10/12* concrete and *Aq42* steel.). These characteristics align with construction norms in many EU countries prior to the widespread adoption of modern seismic codes.

As illustrated in Figure 3.5, the supports of the both frames were defined as fully fixed and their beams (*B*, *B1*, etc.) and columns (*C1*, *C2*, etc.) were labelled according to their respective sections. Their detailed properties (e.g., section properties) are later comprehensively outlined in Section 3.4.2. Detailed structural design verification checks were performed for representative beams and columns from both earthquake-resistant and gravity-load-designed frames, including capacity/demand assessments using ETABS [72], are provided comprehensively in Appendix A.

3.3.2 Structural Modeling in OpenSees

As illustrated in Figure 3.6, structural models were developed in OpenSees [5] using *elasticBeamColumn* elements to represent the elastic behavior of columns and beams, while inelastic behavior was captured through a lumped plasticity approach. The lumped plastic hinge method offers a balance between computational efficiency and accuracy, making it an effective approach for capturing critical failure mechanisms, such as cracking, yielding, and degradation under extreme loading conditions [73] in large-scale fragility studies. This approach simplifies nonlinear behavior by concentrating damage at predefined plastic hinge locations along both ends of beams and columns. These hinges were

modeled using the *Bilin* uniaxial material [81], which provides a bilinear hysteretic moment–rotation (M – θ) response and accounts for different degradation types, as shown in Figure 3.7. Here, the inputs for the *Bilin* model parameters can be summarized as the yield moment (M_y), difference between yield rotation and rotation at maximum moment (pre-capping rotation; θ_p), the difference between rotation at maximum moment and rotation at complete loss of strength (post-capping rotation; θ_{pc}), ultimate rotation (θ_u), and residual moment (M_r).

Figure 3.6: Structural models of RC frames built in OpenSees incorporating lumped plasticity.

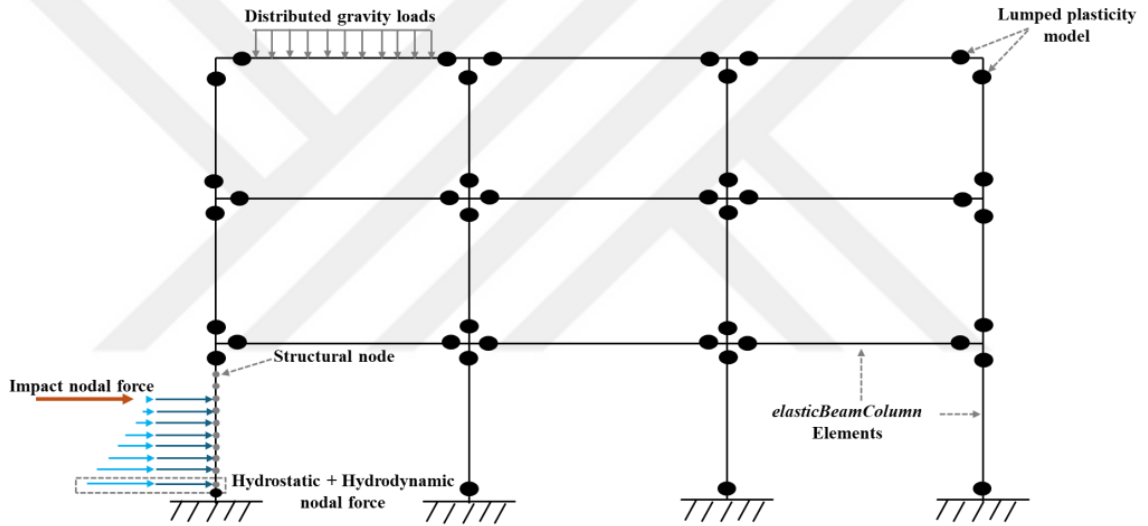
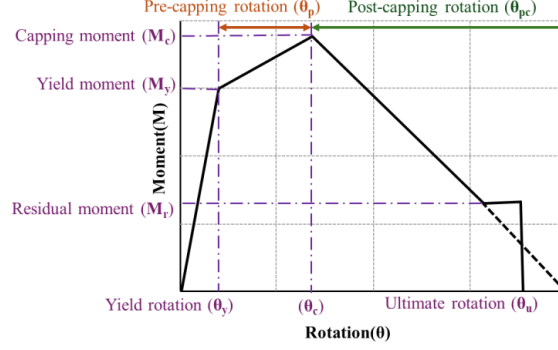


Figure 3.7: Idealized moment–rotation backbone curve used to model the nonlinear behavior of plastic hinges in beams and columns. (Note: the adopted values for the hinge model parameters are later summarized in Table 3.5 and Table 3.6).



3.3.2.1 Hinge Modeling

In order to evaluate the moment–rotation capacities required for nonlinear modeling in OpenSees, a detailed moment–curvature analysis was performed using ETABS [72] Section Designer. This analysis was based on accurate definitions of geometry, reinforcement, and material properties, and considered load combinations specified in NTC-2018. For each column and beam section, moment–curvature ($M-\phi$) curves were generated by applying an axial load and then exported. The plastic hinge length was calculated using the following empirical expression ([78, 79]):

$$L_P = 0.1L_V + 0.17h + 0.24 \frac{d_{bl}f_y}{\sqrt{f_c}} \quad (3.6)$$

where, L_V is the shear span, h is the section depth, d_{bl} is the longitudinal reinforcement bar diameter, f_y is the yield strength of the reinforcement, and f_c is the concrete compressive strength). $M-\phi$ curves were converted to $M-\theta$ using $\theta = \phi \times L_P$. The resulting yield, capping and ultimate rotations along with the corresponding moments were used to define hinge properties. Eurocode formulations were also used for validating these rotation capacities, which are comprehensively summarized in Appendix C. These formulations provided a basis for verifying the rotation values obtained through detailed moment–curvature analysis in ETABS, thus ensuring consistency and reliability of the

modeling approach adopted for the nonlinear structural analysis and subsequent pushover simulations.

Finally, these moment–rotation capacities derived from ETABS were incorporated as input parameters in OpenSees using the *bilin* (i.e., bilinear IMK) material model. The model parameters were defined in accordance with the backbone extracted from ETABS section analysis results. The elastic stiffness of the rotational springs K_s was defined as nK_{bc} , where $K_{bc} = 6EI/L$ is the stiffness of the elastic beam-column element under double curvature and n was set to 10. This approach follows the recommendations of Ibarra and Krawinkler [82] and is detailed in Appendix B.

The general configuration of the moment–rotation backbone curve used for all plastic hinges is shown in Figure 3.7. While Table B.1 in Appendix B provides a comprehensive summary of definitions for all input parameters used in the *Bilin* material command, their moment and rotation input values assigned for the gravity load designed and earthquake-resistant frames are summarized later in Section 3.4.2. The remaining input parameters required for the *Bilin* material model, including strength and stiffness degradation rates, strain hardening ratios, and residual strength ratios, are defined based on empirical relationships and modeling assumptions [82, 83] and are comprehensively summarized in Appendix B.

3.3.2.2 Load Modeling

To simulate gravity loading, a uniform distributed load was applied to the beam elements using the *eleLoad* command within a linear Plain load pattern. Following the study by Ferraioli [71], a dead load (DL) of 4.89 kN/m² and a Live Load (LL) of 2.0 kN/m² were considered and multiplied by the tributary slab width of 5 meters to determine the total uniform load applied to each beam. This load was assigned to nine beam elements within the frame, and a static analysis was carried out to evaluate the structural response

under gravity effects.

For the simulation of flow-induced loads, three load types were incorporated: hydrostatic, hydrodynamic, and debris impact. These were defined using additional *Plain* load patterns. As explained in Section 3.2.1, the magnitudes of these loads were computed based on flow conditions and applied directly to the built structural models. The loads targeted the exterior face of the RC frame, specifically the upstream side, as shown in Figure 3.3. The exposed column on this face (1st story's corner column), measuring 3.5 meters in height, was discretized into 70 elements, each with a length of 5 cm, to enable fine vertical resolution. The calculated flow forces were distributed across the nodes of the exposed column and applied as horizontal nodal loads using the load command. As the water depth increased, the corresponding nodal loads were updated accordingly to reflect the changing flow profile. In scenarios involving floating debris, an additional horizontal nodal load was applied at the node corresponding to the water surface elevation, representing the impact point load by debris carried with the flow. After all flow-induced loads were applied, a static analysis was performed to assess the structural response under each loading condition [15].

3.3.2.3 Geometric Nonlinearities

In structural analysis, geometric nonlinearities account for the effects of large displacements and rotations on the equilibrium and stiffness of a system. Three common approaches to model these effects are the linear transformation, PDelta transformation, and corotational transformation. The linear method assumes small displacements and rotations, neglecting geometric nonlinearities entirely. While computationally efficient, this approach fails to capture critical second-order effects, such as those induced by axial loads acting through lateral displacements (PDelta effects). The PDelta transformation partially addresses this by incorporating geometric stiffness terms to approximate second-order

behavior, making it suitable for moderate displacements. However, it remains limited in scenarios involving large rotations or severe nonlinearities.

The corotational transformation, adopted in this study, overcomes these limitations by dynamically updating the local coordinate system of elements to track large rotations and displacements [5]. Its robustness is particularly critical for push-over analyses, where structures experience progressive yielding and large deformations (see Figure 3.8).

3.3.3 Pushover Analyses and Validations

The developed OpenSees models were validated by comparing the nonlinear static (pushover) analysis results with those obtained from ETABS [72]. In ETABS, nonlinear hinges were assigned at both ends of the beams (*M3* hinges) and columns (*PM2M3* hinges), and the extracted moment–rotation parameters in Section 3.3.1 were input as backbone data. A pushover analysis was subsequently conducted in ETABS to assess the global capacity of the frame system, yielding results comparable to those obtained from the OpenSees models.

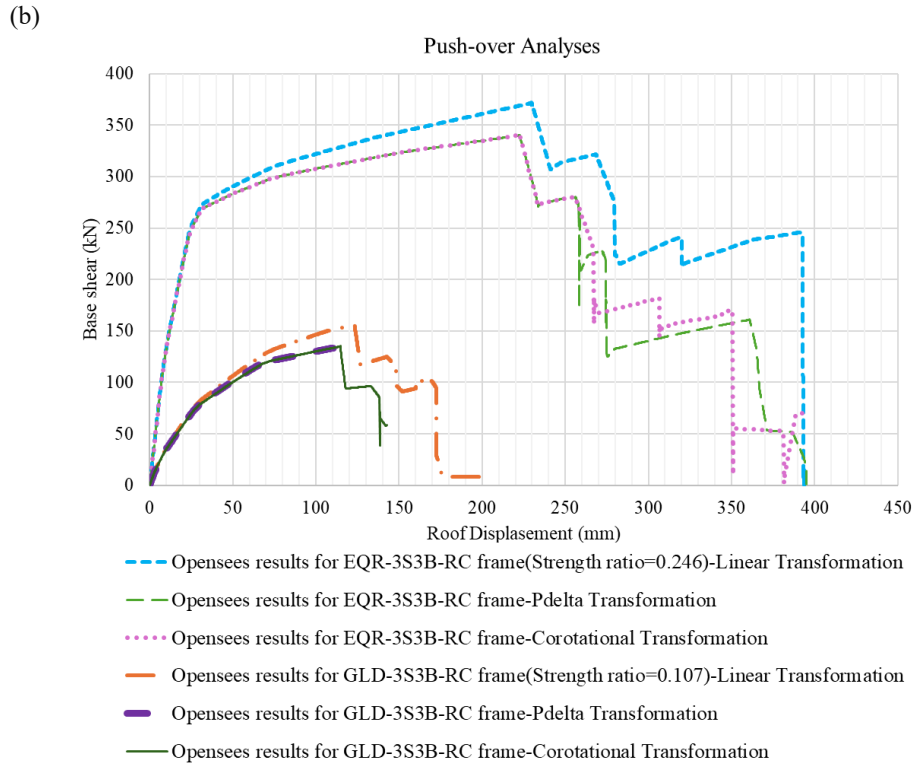
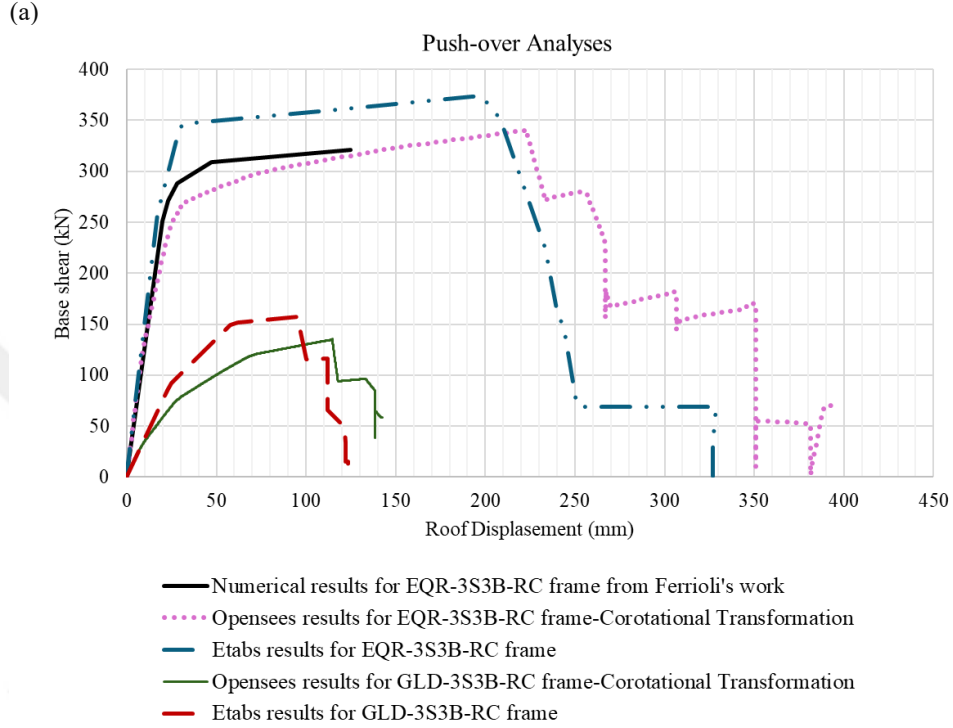
Additionally, the OpenSees results for the earthquake-resistant frame were cross-verified with the pushover findings for the same case study reported in the literature [71], which was performed based on the first-mode structural response through another FE program [76]. Figure 3.8a presents the results of the pushover analyses conducted in OpenSees for both the Earthquake-Resistant (EQR) and Gravity-Load-Designed (GLD) frames considered in this study. Overall, these comparative analyses confirmed the accuracy and reliability of the OpenSees modeling approach developed in this study.

Moreover, in Figure 3.8b, different geometric transformation options were considered for conducting the pushover analyses in OpenSees. Variations in the results highlight the significance of geometric nonlinearities on structural behavior, leading to a more pronounced degradation in failure response.

Additionally, in Figure 3.8, the OpenSees model results for GLD frame shows notably lower lateral strength and ductility compared to the ones for EQR frame, clearly highlighting the significant influence of seismic detailing on structural performance. The corresponding strength ratios for the EQR frame and GLD frame, which quantify the corresponding relative lateral capacity provided by seismic detailing, were also found as 0.246 and 0.107 respectively (as reported in the Figure 3.8 legend).

In summary, the lumped plasticity approach simplifies the nonlinear behavior of structural elements by concentrating deformations in predefined regions (plastic hinges), offering a computationally efficient alternative to distributed plasticity models [73]. Despite its simplifications, the results indicate that this method accurately captures key failure mechanisms, including yielding and cracking, without requiring the computational demands of more detailed models. This efficiency is particularly beneficial for parametric studies, where reducing computational time is critical for handling large datasets or multiple analysis cases. Thus, this validation demonstrates the effectiveness and computational efficiency of the lumped plasticity approach, especially suitable for probabilistic structural simulations.

Figure 3.8: Comparison of pushover analysis results, (a) Results from this study using the lumped plasticity model in OpenSees, ETABS, for earthquake resistance (EQR) and gravity-load-design (GLD) frames, and result from Ferrioli [71]; validating the structural modeling approach adopted in OpenSees. (b) Comparison of various geometric transformation used in OpenSees for performing pushover analysis.



3.4 Fragility models for RC frames

An outline of the eight cases developed to evaluate the damage performance of the typical low-rise RC frames under different scenarios is presented in Table 3.4. These cases offer valuable insights into the response of the frames across different design categories (i.e., earthquake-resistant versus gravity-load based), flow actions, and damage threshold criteria (i.e., FEMA [3] & Ghobarah's study [77] versus Eurocode [4]). Damage fragility models were obtained for each case. Details are given in the following sections.

3.4.1 *Low-rise RC Frame Case Set*

Table 3.4 presents the eight RC frame considered in this study to evaluate their structural response under flood and flow-type landslide conditions. The selected cases include both earthquake-resistant and gravity-load-designed frames subjected to different flow scenarios, incorporating variations in flow density and debris impact forces. These frame models provide critical insights into how structural design influences the vulnerability of RC buildings under extreme hydrostatic, hydrodynamic and impact loads. The earthquake-resistant frames are designed with high-ductility detailing, improving their capacity to withstand lateral forces, whereas the gravity-load-designed frames lack seismic reinforcements, making them more prone to damage under lateral loads. The inclusion of different flow densities (1025 kg/m^3 and 1800 kg/m^3) and debris impact conditions (0 kg and 453.6 kg) allows for a comprehensive assessment of hazard intensities on structural performance.

Table 3.4: *Frame cases and flow cases considered in the fragility assessment models.*

Case No.	Design Type	Case Acronym	Density (kg/m ³) (Mean Value)	Flow Type	Debris Impact (kg) (Mean Value)
1	Earthquake-resistant	EQR-L-M0	1800	Flow-type Landslide	0
2	Earthquake-resistant	EQR-F-M0	1025	Flood Flow	0
3	Earthquake-resistant	EQR-L-MH	1800	Flow-type Landslide	453.6
4	Earthquake-resistant	EQR-F-MH	1025	Flood Flow	453.6
5	Gravity-load designed	GLD-L-M0	1800	Flow-type Landslide	0
6	Gravity-load designed	GLD-F-M0	1025	Flood Flow	0
7	Gravity-load designed	GLD-L-MH	1800	Flow-type Landslide	453.6
8	Gravity-load designed	GLD-F-MH	1025	Flood Flow	453.6

F: flood flow; L: flow-type landslide; EQR: Earthquake-resistant; GLD: Gravity-load designed; M0, MH correspond to a debris mass value of zero, and heavy, respectively.

3.4.2 Modeling Parameters and Uncertainty Modeling

A reliable assessment of RC frame vulnerability under flood and flow-type landslide hazards requires the incorporation of uncertainties in both structural and loading parameters. To address this, a Monte Carlo simulation framework was used to evaluate the effects of variability in material properties, geometric dimensions, and flow characteristics on the structural response of RC frames.

The statistical input data, including the mean values, coefficients of variation (COV), and distribution types, were collected through an extensive review of relevant literature. Table 3.5 to Table 3.7 present the modeling parameters considered for the gravity-load-designed and earthquake-resistant frame types, as well as flow actions (flood and flow-type landslide, with and without debris impact), a total of 8 low-rise RC frame cases (see Table 3.4). The selected modeling parameters include properties of beam and column hinges, section geometries, material properties, as well as flow density and debris mass. These parameters were assumed to be mutually independent for simplicity in the simulations, and were treated as random variables with appropriate distributions (e.g., lognormal or uniform).

Parameters with considerable uncertainty were assumed to have high coefficient of variation (COV), while those considered less sensitive were modeled with minor COV or even they were treated deterministically. For example, rotations capacity of beam and

column hinges, which are critical for nonlinear response, were assigned to have high COVs due to their strong influence on fragility outcomes, as their variability stems from the influence of material properties such as concrete strength (f_c) and steel yield strength (f_y). Conversely, parameters like section dimensions (width and height), story height, and bay length were assigned to have a minor COV of 4%.

In modeling the external loads, flow depth and velocity were defined deterministically and varied incrementally over a specified range, while flow density and debris mass were treated as uncertain variables. Flow density was modeled with a uniform distribution, reflecting a range of values for flood and landslide scenarios. Debris impact mass of floating debris such as logs was considered to be a stochastic variable with a 30% value of COV and uniformly distributed values. Impact duration was treated as deterministic. These flow-related parameters were evaluated independently to isolate their individual effects on RC frame performance.

Table 3.5: Typical statistical values and probability distributions assumed for gravity-load designed building modeling parameters.

Category	Parameters	Mean	COV	Dist.
Material	Concrete young's modulus, E_c (GPa)	26.5	10%	LN
Beam Hinge Properties	Hinge Name	B1		
	Yield Moment, M_y (kN.m)	62	20%	LN
	Pre-capping Rotation, θ_p (rad)	0.0373	30%[6]	LN
	Ratio of Capping Moment to Yield Moment, M_c/M_y	1.45	-	Det
	Post-Capping Rotation, θ_{pc} (rad)	0.0142	40%	LN
	Ultimate Rotation, θ_u (rad)	0.0567	40%[6]	LN
	Residual Moment, M_r (kN.m)	24.8	20%	LN
Beam Section Properties	Section Height, b_{Beam} (mm)	400	4%[6, 84]	N
	Section Width, w_{Beam} (mm)	300	4%[6, 84]	N
Column Hinge Properties	Hinge Name	C1	C2	
	Yield Moments, M_y (kN.m)	67.6	37.5	20%
	Pre-capping Rotation, θ_p (rad)	0.0164	0.0228	30%[6]
	Ratios of Capping Moment to Yield Moment, M_c/M_y	1.45	1.45	-
	Post-Capping Rotation, θ_{pc} (rad)	0.0069	0.0092	40%
	Ultimate Rotation, θ_u (rad)	0.0279	0.0369	40%[6]
	Residual Moment, M_r (kN.m)	27.4	21.3	20%
Column Section Properties	Section Height, b_{Column} (mm)	400	350	4%[6, 84]
	Section Width, w_{Column} (mm)	400	350	4%[6, 84]
General Building Properties	Bay length, L (m)	5		4%[6]
	Story one height, h_1 (m)	3.5		4%[6]
	Story two and three height, h_2, h_3 (m)	3		4%[6]

Dist.: Distribution, N: Normal, LN, Lognormal, Det.: Deterministic

Table 3.6: Typical statistical values and probability distributions assumed for earthquake-resistant building modeling parameters.

Category	Parameters	Mean		COV	Dist.
Material	Concrete young's modulus, E_c (GPa)	33		10%	LN
Beam Hinge Properties	Hinge Name	B1	B2		
	Yield Moment, M_y (kN.m)	91.6	101	20%	LN
	Pre-capping Rotation, θ_p (rad)	0.0464	0.0461	30%[6]	LN
	Ratio of Capping Moment to Yield Moment, M_c/M_y	1.45	1.45	-	Det
	Post-Capping Rotation, θ_{pc} (rad)	0.0177	0.0177	40%	LN
	Ultimate Rotation, θ_u (rad)	0.0708	0.0708	40%[6]	LN
	Residual Moment, M_r (kN.m)	36.64	40.4	20%	LN
Beam Section Properties	Section Height, b_{Beam} (mm)	500	500	4%[6, 84]	N
	Section Width, w_{Beam} (mm)	300	300	4%[6, 84]	N
Column Hinge Properties	Hinge Name	C1	C2	C3	
	Yield Moments, M_y (kN.m)	125	177	143	20%
	Pre-capping Rotation, θ_p (rad)	0.0328	0.0245	0.0302	30%[6]
	Ratios of Capping Moment to Yield Moment, M_c/M_y	1.45	1.45	1.45	-
	Post-Capping Rotation, θ_{pc} (rad)	0.0137	0.0106	0.0123	40%
	Ultimate Rotation, θ_u (rad)	0.0545	0.0425	0.0492	40%[6]
	Residual Moment, M_r (kN.m)	50	70.8	57.2	20%
Column Section Properties	Section Height, b_{Column} (mm)	350	400	400	4%[6, 84]
	Section Width, w_{Column} (mm)	300	300	300	4%[6, 84]
General Building Properties	Bay length, L (m)	5			4%[6]
	Story one height, h_1 (m)	3.5			4%[6]
	Story two and three height, h_2, h_3 (m)	3			4%[6]

Dist.: Distribution, N: Normal, LN, Lognormal, Det.: Deterministic

Table 3.7: Typical statistical values and probability distributions assumed for flow intensity modeling parameters.

Flow Intensity Modeling Parameters	Mean	COV	Dist.
Flow Velocity, V (m/s)	0.25m/s-increments btw. 0 and 25m/s	-	Det.
Flow Depth, D (m)	0.1m-increments btw. 0 and 3m	-	Det.
Flow Density, ρ (kg/m ³)	Flow-type landslide:1800 [6, 62, 63] Flood flow: 1025	10%[64, 65, 66] 5%[62] 30% [54]	U U Det.
Impact Mass, M (kg)	0, 453.6 [32]		
Impact time duration for debris, Δt (s)	0.03 [32]	-	Det.

Dist.: Distribution, U: Uniform, Det.: Deterministic

3.4.2.1 Sensitivity analysis

Several trials on the key structural and flow parameters have been conducted to understand their significance on the critical flow capacities. This section presents an example of parametric sensitivity analysis to evaluate the influence of these key parameters on Near Collapse (NC) velocity predictions (DS3 in Eurocode 8 Part 3). The analysis focuses on Case 3 (EQR-L-MH), representing an earthquake-resistant RC frame subjected to a flow-type landslide with heavy debris impact (453.6 kg). The baseline velocity at the flow depth $D=1\text{m}$, computed using mean parameter values presented in Tables 3.5 and

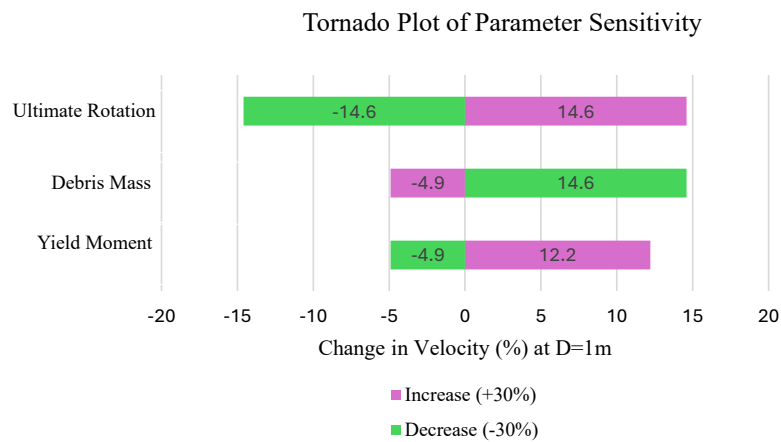
3.7, was 10.25m/s. A $\pm 30\%$ coefficient of variation (COV) was applied to top three key parameters—debris mass, yield moment, and ultimate rotation—while keeping other variables constant. Table 3.8 summarizes the results, presenting changes in velocity values at $D=1\text{m}$ under perturbed conditions.

Table 3.8: Sensitivity analysis for Case 3 (EQR-L-MH) at $D = 1\text{ m}$ and $COV = 30\%$.

Parameters	Change (%)	Velocity (m/s) at Near Collapse (NC)	$\Delta V/V$ (%)
Baseline		10.25	
Debris Mass	+30%	11.75	+14.6%
	-30%	9.75	-4.9%
Yield moment	+30%	11.5	+12.2%
	-30%	9.75	-4.9%
Ultimate rotation	+30%	11.75	+14.6%
	-30%	8.75	-14.6%

Among the parameters, ultimate rotation and debris mass exhibited the highest sensitivity, with velocity variations up to $\pm 14.6\%$ for a $\pm 30\%$ change in these parameters. The yield moment also demonstrated notable influences. This analysis is visualized using a tornado diagram (Figure 3.9), illustrating the relative influence of each parameter on the Near Collapse (NC) velocity. This supports robust probabilistic fragility modeling and uncertainty quantification, ensuring that key parameter uncertainties are systematically considered in the vulnerability assessment of RC frame structures.

Figure 3.9: Tornado plot showing the sensitivity of Near Collapse (NC) velocity $D=1\text{m}$ to $\pm 30\%$ variations in debris mass, yield moment, and ultimate rotation for Case 3 (EQR-L-MH).



3.4.3 *Logistic Regression Analyses on IFA Results*

To establish reliable fragility curves that accurately represent RC frame vulnerability under flood and flow-type landslide hazards, this study employs Monte Carlo simulations to systematically account for uncertainties in structural properties, geometry, and external loading conditions (see Table 3.5 to Table 3.7). Random variables, specifically flow depth (up to 3 meters) and flow velocity (up to 25 m/s [62]), were incrementally varied by 0.1 m and 0.25 m/s, respectively, with 50 samples generated for each increment. Consequently, the analysis produced an extensive dataset consisting of 1.2×10^6 simulations across 30 depth increments, 100 velocity increments, and 8 distinct structural RC frame cases (see Table 3.4). Subsequently, Incremental Flow Analysis (IFA) was conducted to evaluate structural responses at the three predefined damage states mentioned earlier in Section 3.2. These analyses thoroughly assessed for each RC frame case, capturing performance variations systematically across a broad range of hazard intensities.

Logistic regression analyses were then utilized on the simulation data to develop fragility models, a widely adopted approach in structural engineering for quantifying the probabilistic relationship between hazard intensities and the likelihood of damage occurrence [38, 40, 41]. For each of the three damage states and the eight RC frame cases, logistic regression was independently performed, utilizing the extensive simulation dataset. To validate and assess the predictive capabilities of these regression models, confusion matrices were constructed by comparing predicted damage states from logistic regression models against actual observed outcomes derived from numerical analyses. The confusion matrix, presented in Table 3.9, categorizes model predictions into four distinct classes: true positives (correct predictions of damage occurrence), true negatives (correct predictions of no damage), false positives (incorrect predictions indicating damage occurrence), and false negatives (missed predictions of actual damage occurrences).

The predictive accuracy and reliability of these logistic regression-based fragility

models were thoroughly evaluated using several statistical metrics commonly employed in classification problems [38, 39, 41, 55, 56, 57, 58]. Equations (2.9) to (2.13) which were discussed in Chapter 2 Section 2.4.3 define each metric explicitly, clarifying their role in evaluating model performance. These performance metrics were calculated comprehensively for all eight RC frame cases (see Table 3.4), across the three defined damage states (DS1, DS2, and DS3), considering the damage thresholds defined by FEMA 356, Ghobarah’s study [77] and Eurocode 8 Part 3 guidelines, as presented in Table 3.10.

Overall, the developed logistic regression models demonstrated strong performance, consistently achieving high scores (more than 0.9) across all statistical evaluation criteria (see Table 3.10). These results underscore the effectiveness and robustness of the fragility models in accurately predicting damage states for RC frames subjected to varying intensities of flood and flow-type landslide hazards.

Table 3.9: *Confusion matrix showing the categorization of predicted versus observed damage states for evaluating logistic regression model performance.*

		Predicted	
		Damage	No-damage
Observed	Damage	True Positive (TP)	False Negative (FN)
	No-damage	False Positive (FP)	True Negative (TN)

Table 3.10: *Fitting evaluation results from the logistic regression models for the 8 low-rise RC frame cases.*

Case No.	Case Acronym	DSs	Eurocode8Part3					FEMA 356, Ghobarah's study				
			Accuracy	AUC _{ROC}	Precision	Recall	F ₁ -score	Accuracy	AUC _{ROC}	Precision	Recall	F ₁ -score
1	EQR-L-M0	DS1	0.985	0.999	0.983	0.984	0.984	0.987	0.999	0.978	0.975	0.977
		DS2	0.986	0.999	0.973	0.971	0.972	0.990	0.999	0.968	0.966	0.967
		DS3	0.985	0.999	0.963	0.959	0.961	0.986	0.999	0.955	0.946	0.951
2	EQR-F-M0	DS1	0.985	0.999	0.979	0.978	0.978	0.990	0.999	0.971	0.971	0.971
		DS2	0.986	0.998	0.953	0.944	0.948	0.993	0.999	0.952	0.937	0.944
		DS3	0.988	0.998	0.925	0.917	0.921	0.991	0.999	0.923	0.911	0.917
3	EQR-L-MH	DS1	0.978	0.996	0.987	0.984	0.986	0.982	0.999	0.979	0.978	0.979
		DS2	0.985	0.999	0.990	0.989	0.989	0.982	0.999	0.978	0.975	0.976
		DS3	0.988	0.999	0.991	0.991	0.991	0.982	0.999	0.977	0.975	0.976
4	EQR-F-MH	DS1	0.98	0.997	0.988	0.986	0.987	0.979	0.999	0.973	0.971	0.972
		DS2	0.983	0.999	0.987	0.985	0.986	0.977	0.998	0.968	0.966	0.967
		DS3	0.987	0.999	0.989	0.988	0.988	0.980	0.998	0.971	0.967	0.969
5	GLD-L-M0	DS1	0.986	0.999	0.989	0.988	0.989	0.982	0.999	0.986	0.987	0.987
		DS2	0.977	0.999	0.977	0.976	0.977	0.984	0.999	0.986	0.986	0.986
		DS3	0.981	0.999	0.980	0.977	0.979	0.985	0.999	0.984	0.984	0.984
6	GLD-F-M0	DS1	0.982	0.999	0.983	0.983	0.983	0.984	0.999	0.988	0.986	0.987
		DS2	0.981	0.999	0.976	0.974	0.975	0.984	0.999	0.984	0.982	0.983
		DS3	0.979	0.998	0.972	0.970	0.971	0.985	0.999	0.982	0.980	0.981
7	GLD-L-MH	DS1	0.986	0.998	0.992	0.992	0.992	0.979	0.997	0.988	0.986	0.987
		DS2	0.988	0.999	0.993	0.993	0.993	0.975	0.998	0.979	0.976	0.977
		DS3	0.988	0.999	0.994	0.993	0.993	0.981	0.999	0.982	0.983	0.982
8	GLD-F-MH	DS1	0.986	0.999	0.992	0.992	0.992	0.979	0.998	0.988	0.987	0.987
		DS2	0.990	0.999	0.994	0.993	0.994	0.977	0.998	0.985	0.984	0.984
		DS3	0.986	0.999	0.993	0.991	0.992	0.976	0.997	0.983	0.981	0.982

The logit functions used in logistic regression models that define each damage state are provided Table 3.11, with coefficients obtained through probabilistic analyses. These functions are valuable tools for estimating damage probabilities under different hazard scenarios, supporting structural design, emergency response planning, and policy development.

Table 3.11: Logit functions from the logistic regression models for the eight RC frame cases.

Case No.	Case Acronym	DSs	logit function $\Phi(d, v)$ (Eurocode8Part3)	logit function $\Phi(d, v)$ (FEMA356, Ghobarah's study)
1	EQR-L-M0	DS1	-26.85 - 5.82 d - 0.09 v + 2.25 dv	-40.05 + 2.0 d - 0.58 v + 1.58 dv
		DS2	-8.17 - 15.09 d - 1.26 v + 1.97 dv	-35.9 - 3.15 d - 0.50 v + 1.37 dv
		DS3	-34.63 - 2.94 d - 0.46 v + 0.96 dv	-33.69 - 0.08 d - 0.17 v + 0.94 dv
2	EQR-F-M0	DS1	-29.33 - 5.25 d - 0.04v + 1.66 dv	-54.39 + 4.41 d - 0.29 v + 1.39 dv
		DS2	-5.72 - 14.78 d - 0.87 v + 1.36 dv	-39.47 - 2.74 d - 0.12 v + 0.96 dv
		DS3	-29.67 - 4.96 d + 0.21 v + 0.77 dv	-29.98 - 5.12 d - 1.01 v + 1.24 dv
3	EQR-L-MH	DS1	-2.65 - 7.84 d - 0.26 v + 2.77 dv	-45.15 + 3.78 d + 0.04 v + 1.59 dv
		DS2	-15.89 - 7.01 d - 0.11 v + 3.37 dv	-34.32 + 3.66 d - 0.30 v + 1.19 dv
		DS3	-21.63 - 8.41 d - 0.09 v + 3.80 dv	-33.13 - 3.02 d - 0.29 v + 1.13 dv
4	EQR-F-MH	DS1	-4.39 - 6.93 d - 0.17 v + 2.63 dv	-26.78 - 1.19 d - 0.78 v + 1.54 dv
		DS2	-17.23 - 7.79 d - 0.05 v + 2.81 dv	-23.32 + 1.34 d - 0.43 v + 1.18 dv
		DS3	-24.43 - 7.26 d + 0.13 v + 2.78 dv	-23.75 + 0.34 d - 0.34 v + 1.17 dv
5	GLD-L-M0	DS1	-30.94 + 2.11 d + 0.4 v + 2.68 dv	-28.80 + 10.06 d + 0.07 v + 1.97 dv
		DS2	-28.69 + 3.20 d + 0.42 v + 1.22 dv	-37.64 + 9.09 d - 0.02 v + 2.02 dv
		DS3	-35.83 + 6.16 d + 0.62 v + 1.10 dv	-35.33 + 6.34 d - 0.24 v + 1.89 dv
6	GLD-F-M0	DS1	-27.16 - 1.37 d + 0.16 v + 2.12 dv	-31.25 + 8.74 d - 0.03 v + 1.88 dv
		DS2	-42.29 + 7.45 d + 0.91 v + 0.82 dv	-36.38 + 6.10 d - 0.24 v + 1.88 dv
		DS3	-3.90 + 6.63 d + 0.59 v + 0.72 dv	-33.92 + 4.53 d - 0.35 v + 1.59 dv
7	GLD-L-MH	DS1	-6.20 - 7.19 d - 0.05 v + 5.37 dv	-13.26 + 4.24 d + 0.29 v + 2.25 dv
		DS2	-9.95 - 5.66 d - 0.01 v + 5.36 dv	-16.18 + 2.18 d + 0.20 v + 2.01 dv
		DS3	-11.08 - 5.31 d - 0.03 v + 5.42 dv	-16.81 + 2.67 d + 0.23 v + 1.45 dv
8	GLD-F-MH	DS1	-7.30 - 6.98 d - 0.07 v + 5.58 dv	-14.60 + 3.02 d + 0.288 v + 2.30 dv
		DS2	-12.13 - 4.94 d + -0.01 v + 5.28 dv	-13.90 + 2.46 d + 0.20 v + 1.15 dv
		DS3	-12.75 - 3.51 d + 0.08 v + 4.36 dv	-14.13 + 3.34 d + 0.14 v + 1.86 dv

3.4.4 Proposed Fragility Models

To assess the vulnerability of RC frames subjected to flood and flow-type landslide conditions, logit-based fragility functions were developed for eight different frame scenarios (see Table 3.11). These functions are based on logistic regression analyses that use results from IFA and Monte Carlo simulations. Flow depth and velocity were chosen as the main input variables in the fragility models, while the effects of debris mass, flow density, material properties, and geometrical parameters were handled using probabilistic methods, as detailed in Table 3.5 to Table 3.7.

The eight RC frame cases, summarized in Table 3.4, include combinations of two structural designs (earthquake-resistant and gravity-load designed), two flow types (flood and flow-type landslide), and two debris intensities (none and 453.6 kg). For each case, fragility contours were generated and are presented in Figure 3.10 and Figure 3.11. These contours represent damage states DS1, DS2, and DS3 reported in Table 3.2, corresponding

to Eurocode-based performance levels (see Table 3.2) “DL, SD, and NC (Figure 3.10)”, respectively; FEMA-based thresholds (see Table 3.2) “IO, LS, and CP”, respectively and Ghobarah’s study “0.2%, 0.5%, and 0.8%” [77], respectively (Figure 3.11).

Each subplot in Figure 3.10 and Figure 3.11 displays probability contours at 0.1, 0.5, and 0.9, showing the likelihood of reaching or exceeding a given damage state for a specific combination of flow depth and velocity. The x and y axes represent flow depth (m) and flow velocity (m/s), respectively, while the color gradients, from dark blue (low probability) to yellow (high probability of failure), offer a clear visual representation of damage potential. Labels on the curves reference the corresponding case numbers and acronyms listed in Table 3.4 and Table 3.2.

To systematically compare these results, the comparison first examines structural performance by holding flow depth constant and evaluating corresponding velocity thresholds, followed by the effects of hazard type and debris. Finally, the implications of different frame design types, flow situations, and performance standards (Eurocode vs. FEMA & Ghobarah’s study [77]) are discussed (see Figure 3.10, Figure 3.11 & Table 3.12).

In Figure 3.10, at a fixed flow depth of 1.5 m, earthquake-resistant frames (Cases 1–4) demonstrate markedly higher velocity tolerances compared to gravity-load-designed frames (Cases 5–8). For instance, under flow-type landslide conditions without debris (Case 1: EQR-L-M0, Figure 3.10), the frame requires a velocity of 21 m/s to reach a 0.5 probability of Near Collapse (NC), whereas its gravity-load counterpart (Case 5: GLD-L-M0, Figure 3.10) reached the same probability at just 12 m/s, a $\sim 43\%$ reduction in velocity resistance. This trend persisted across all damage states, with earthquake-resistant frames consistently requiring $\sim 40 - 60\%$ higher velocities to achieve equivalent failure probabilities at identical depths. The inclusion of debris in the flow further exacerbated these differences; for example, at 1.5 m depth, Case 3 with debris for earthquake-resistant

frames (EQR-L-MH, Figure 3.10) sustained velocities up to 7 m/s before reaching NC, while Case 7 with debris for gravity frames (GLD-L-MH, Figure 3.10) failed at 3 m/s (see Table 3.12).

When comparing hazard types while maintaining constant depth, flow-type landslides consistently produce higher failure probabilities than pure floods. As an example, in Figure 3.10, at 2.0 m depth, frames subjected to flow-type landslides (Cases 1, 3, 5, 7) reach the Significant Damage (SD) state at velocities $\sim 20\text{--}30\%$ lower than those in flood scenarios (Cases 2, 4, 6, 8). For earthquake-resistant (EQR) frames, the velocity threshold for SD under landslides (Case 1: 14 m/s) was 24.3% lower than the one in floods (Case 2: 18.5 m/s). Similarly, gravity-load-designed (GLD) frames exhibited a 27.3% reduction in velocity resistance during flow-type landslides (Case 5: 8 m/s) compared to one in floods (Case 6: 11 m/s) (see Table 3.12). These disparities underscore the heightened severity of flow-type landslides, which induce critical damage at lower velocities.

The debris effect is particularly pronounced in gravity-load frames; for example, at 2.0 m depth, the inclusion of 453.6 kg debris in Case 8 (GLD-F-MH) reduces the velocity threshold for DS3 under Ghobarah' standards to 4 m/s (Figure 3.11), a $\sim 60\%$ reduction compared to the debris-free Case 6 (GLD-F-M0), which requires 10 m/s to reach the same damage state (see Table 3.12).

Finally, the comparison of performance standards (Eurocode vs. Drift based performance standards) reveals notable differences in velocity thresholds for equivalent structural cases. For instance, under flow-type landslides at 2.0 m depth, the earthquake-resistant frame (Case 1: EQ-L-M0) requires 16.25 m/s (see Figure 3.10 & Table 3.12) to reach Near Collapse (NC) per Eurocode standards, but 20.5 m/s (see Figure 3.11 & Table 3.12) to achieve Collapse Prevention (CP) under FEMA criteria, a $\sim 21\%$ increase in required velocity. This discrepancy highlights the stricter definition of critical damage in Eurocode compared to FEMA for the same structural scenario, emphasizing the need

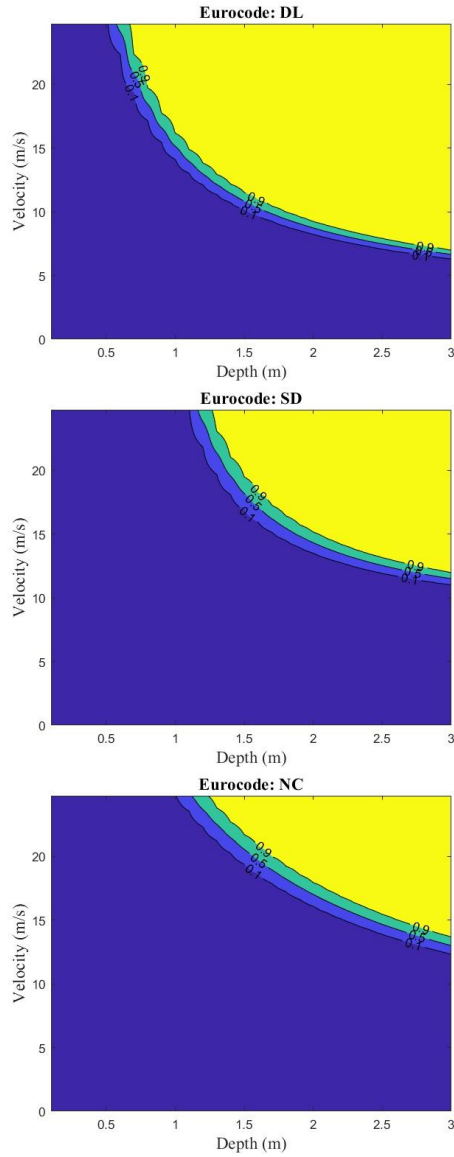
to align performance standards with regional hazard assessments and design philosophies when evaluating structural resilience.

Considering the logit functions in Table 3.11 and comparing Figure 3.11 with Figure 3.10 reveals that, while Drift-based fragility curves are aligned with different performance thresholds, they follow similar trends to the ones for Eurocode as described above. Regardless of the performance standard used, the fragility surfaces consistently show sensitivity to debris impact, flow density, and velocity, especially under severe flow conditions. These findings highlight again the importance of considering a range of hazard intensities in performance-based design and risk assessment strategies.

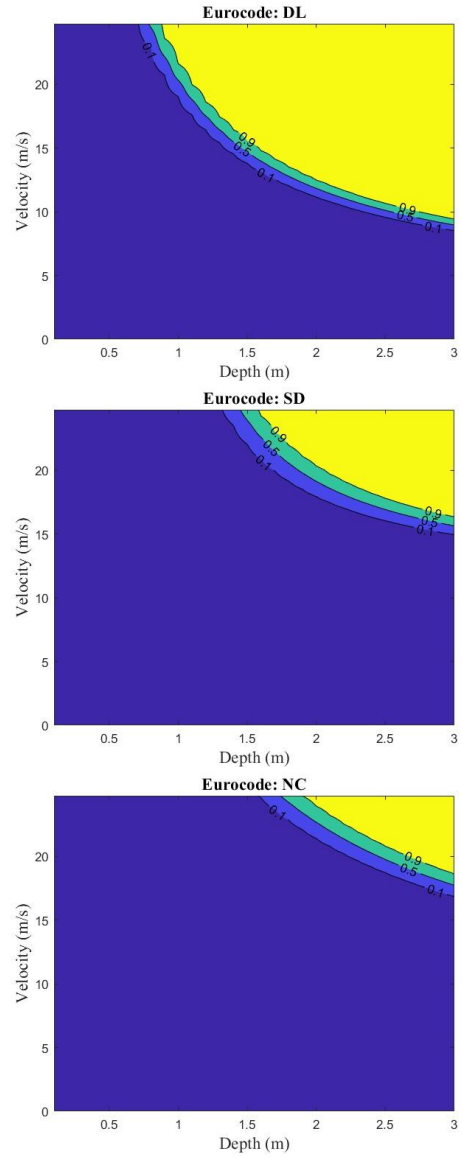
In summary, the fragility surfaces in Figure 3.10 and Figure 3.11 show significant variation depending on frame configuration and flow characteristics. Damage probabilities in all cases show sensitivity to both flow depths and velocities. Particularly, cases involving heavy debris (cases 3, 4, 7, and 8) show left-shifted contours, implying a higher likelihood of failure at lower flow depths and velocities. Also, earthquake-resistant frames (Cases 1–4) consistently demonstrate improved resilience compared to their gravity-load-designed counterparts (Cases 5–8), as evidenced by flatter and right-shifted probability contours. As a result, these proposed fragility models offer a deeper understanding of how RC frames respond to multiple interacting hazards. They support both forward prediction of a given damage state probabilities under known flow conditions and inverse estimation of critical hazard parameters for meeting specific performance goals.

Figure 3.10: Fragility contours for RC frames based on damage states by EuroCode8-Part3 [4].

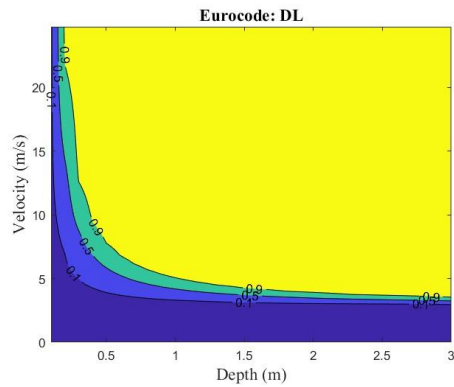
Case 1: EQR-L-M0, Flow-Type Landslide, No Debris



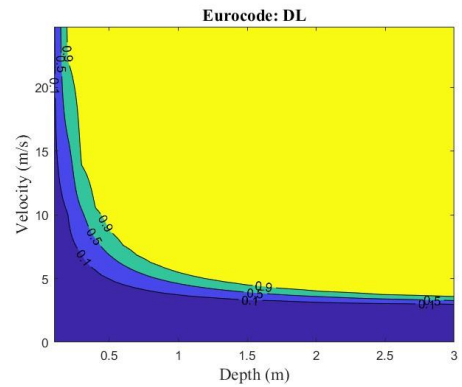
Case 2: EQR-F-M0, Flood, No Debris

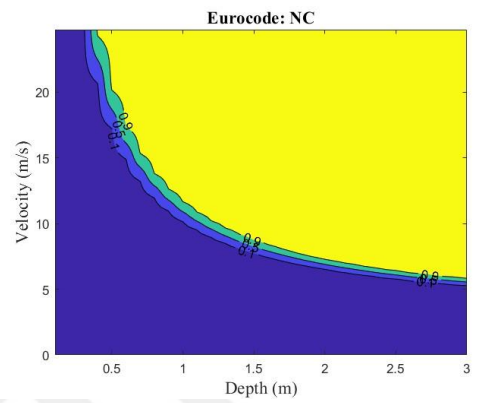
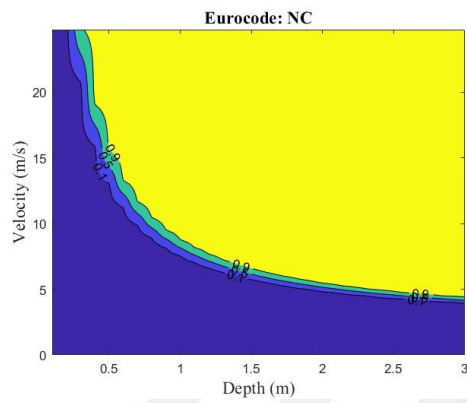
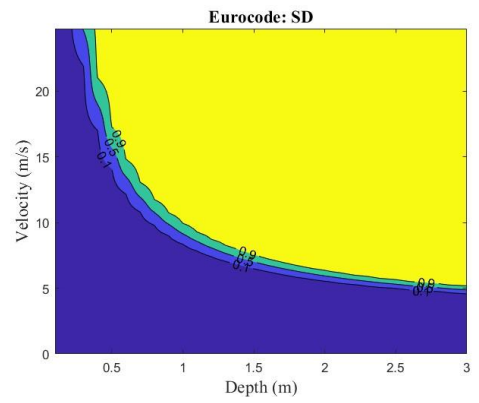
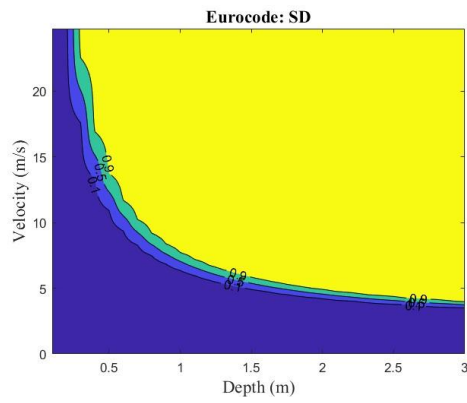


Case 3: EQR-L-MH, Flow-Type Landslide, 453.6kg-Debris



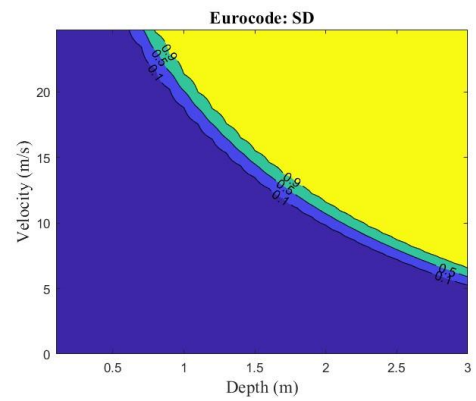
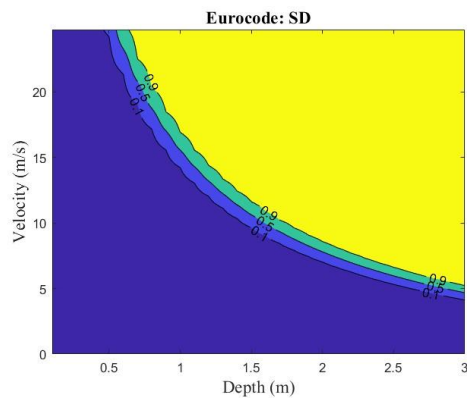
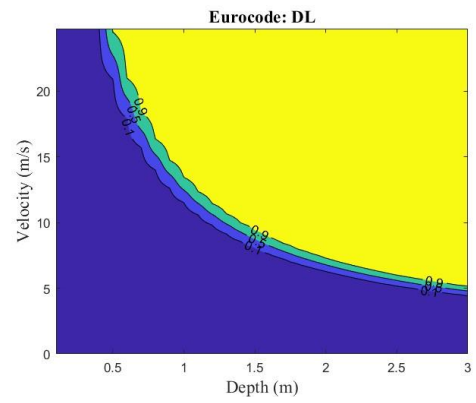
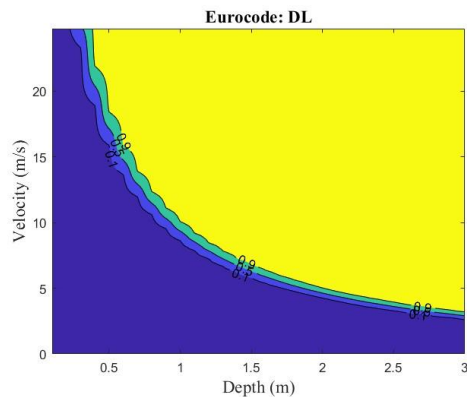
Case 4: EQR-F-MH, Flood, 453.6kg-Debris

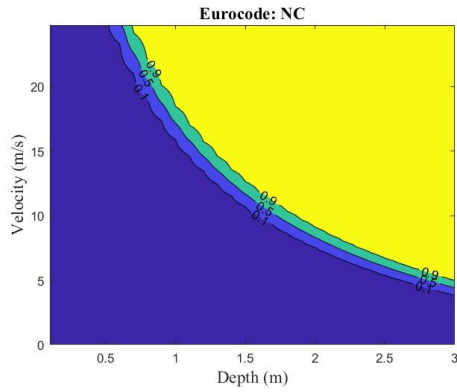




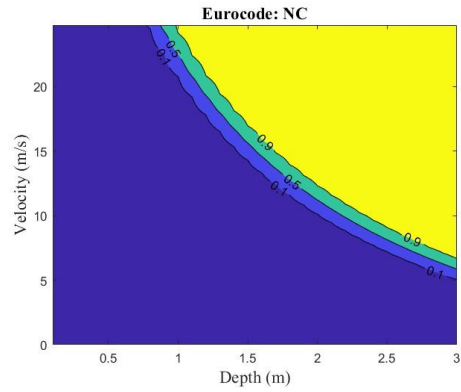
Case 5: GLD-L-M0, Flow-Type Landslide, No Debris

Case 6: GLD-F-M0, Flood, No Debris





Case 7: GLD-L-MH, Flow-Type Landslide, 453.6kg-Debris



Case 8: GLD-F-MH, Flood, 453.6kg-Debris

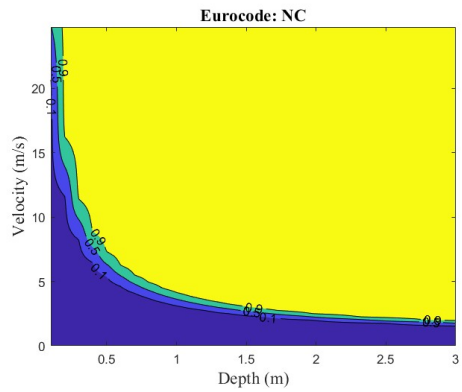
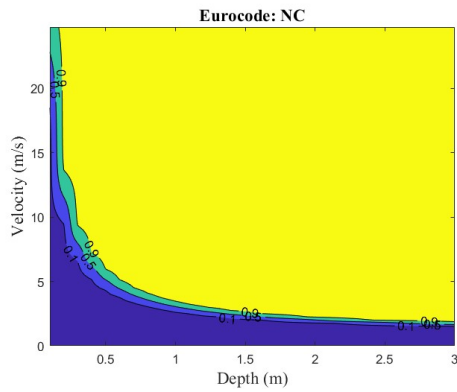
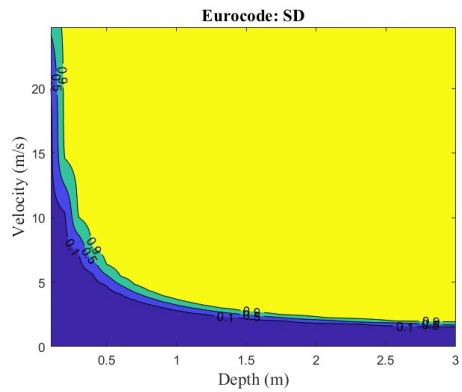
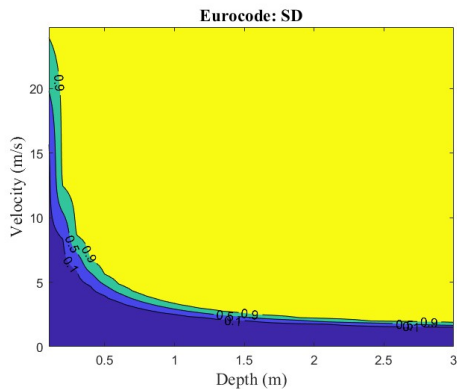
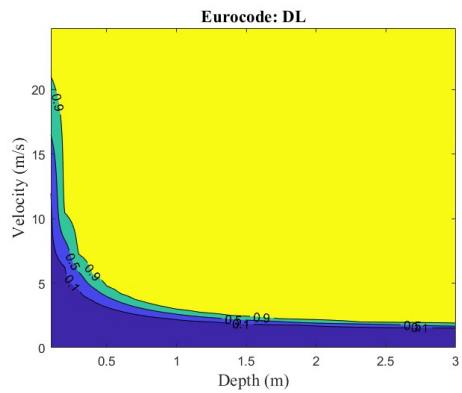
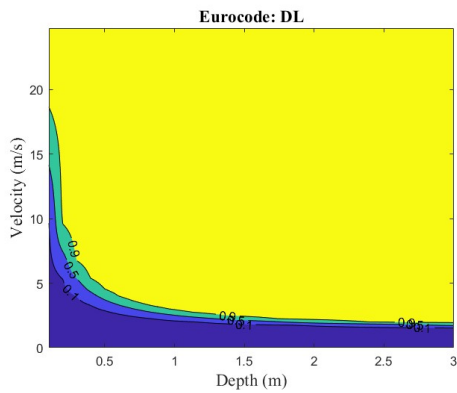
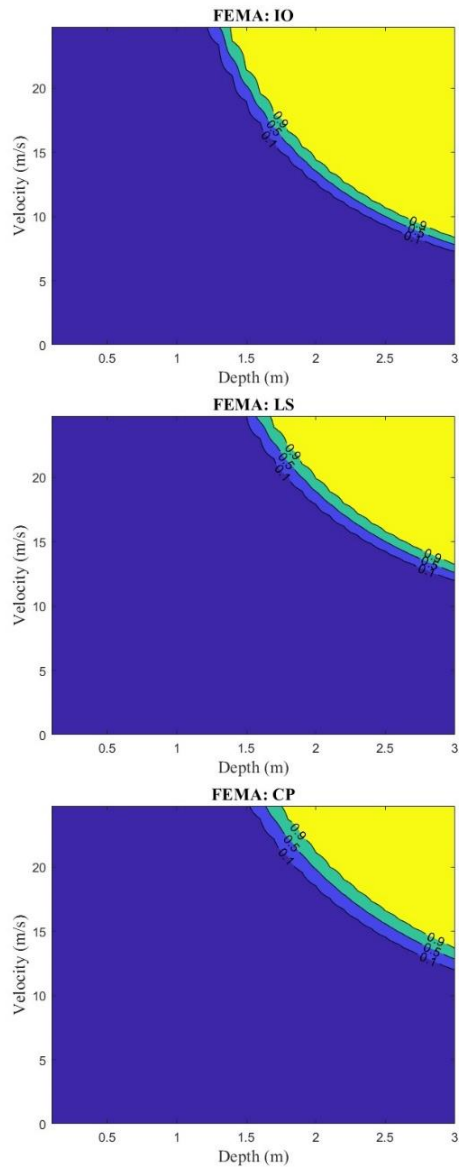
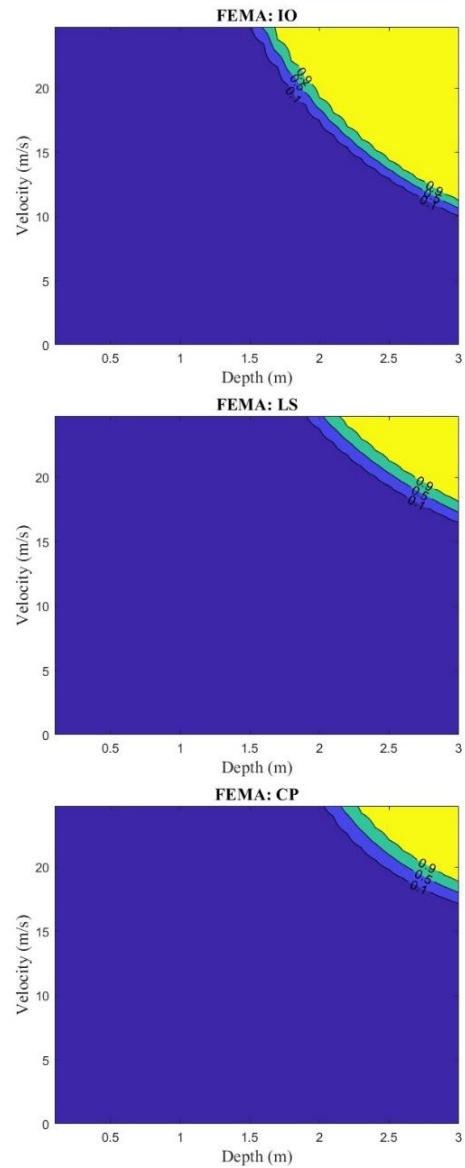


Figure 3.11: Fragility contours for RC frames based on damage states by FEMA 356 [3] and Ghobarah's study [77].

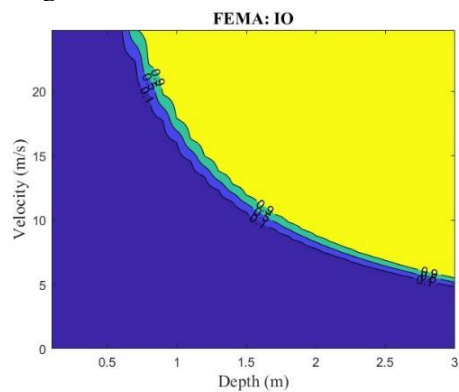
Case 1: EQR-L-M0, Flow-Type Landslide, No Debris



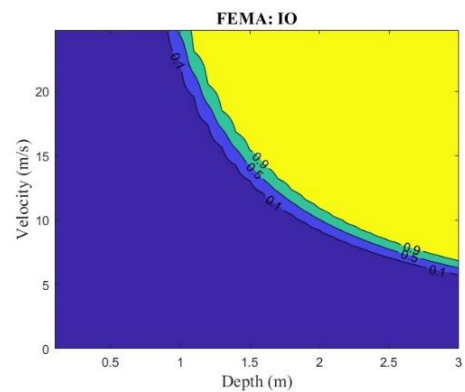
Case 2: EQR-F-M0, Flood, No Debris

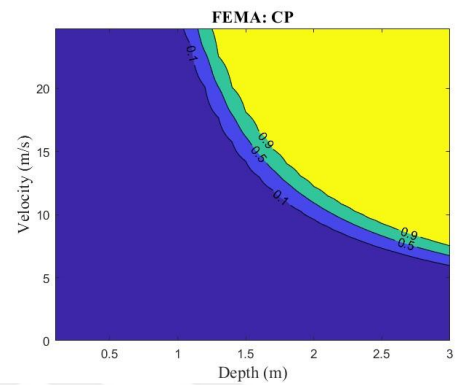
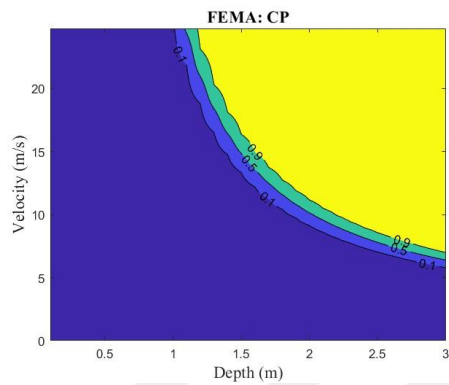
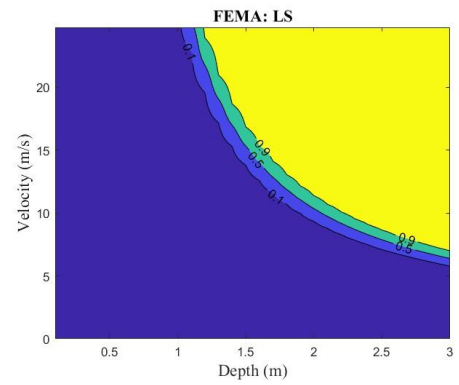
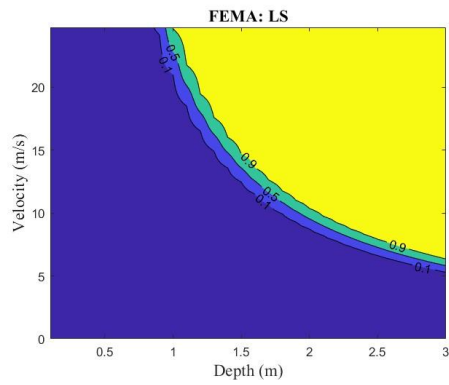


Case 3: EQR-L-MH, Flow-Type Landslide, 453.6kg Debris



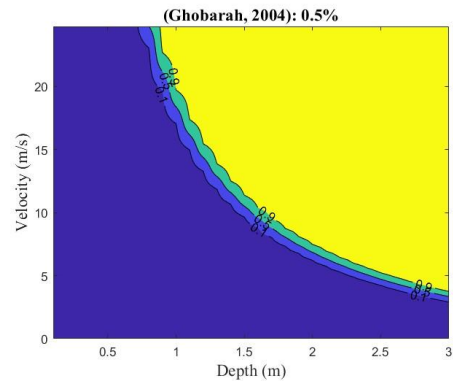
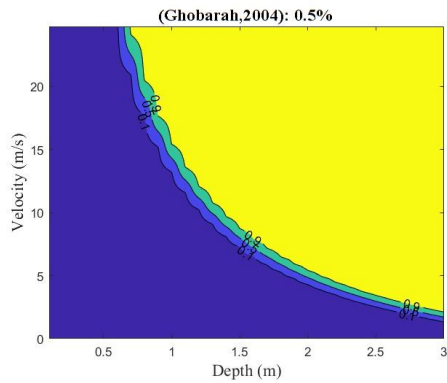
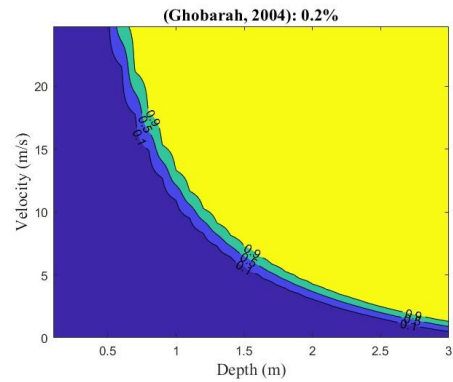
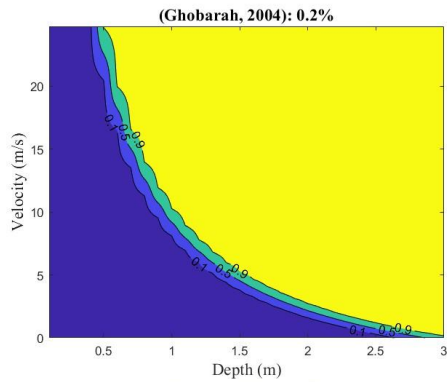
Case 4: EQR-F-MH, Flood, 453.6kg-Debris

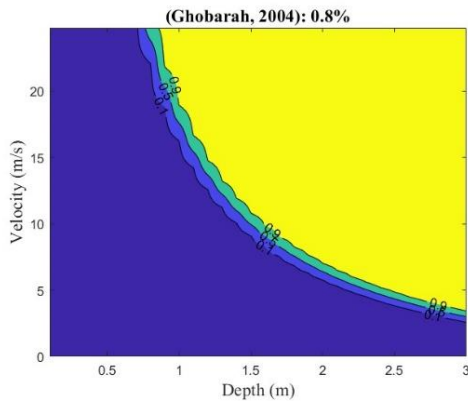




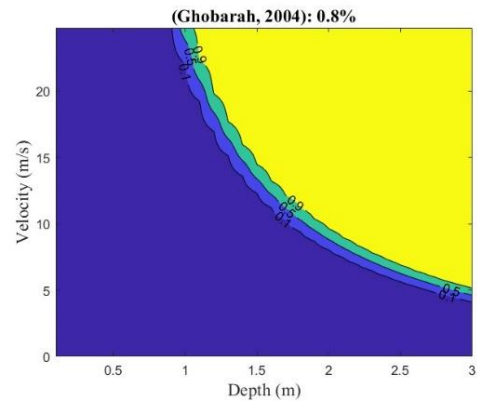
Case 5: GLD-L-M0, Flow-Type Landslide, No Debris

Case 6: GLD-F-M0, Flood, No Debris





Case 7: GLD-L-MH, Flow-Type Landslide, 453.6kg-Debris



Case 8: GLD-F-MH, Flood, 453.6kg-Debris

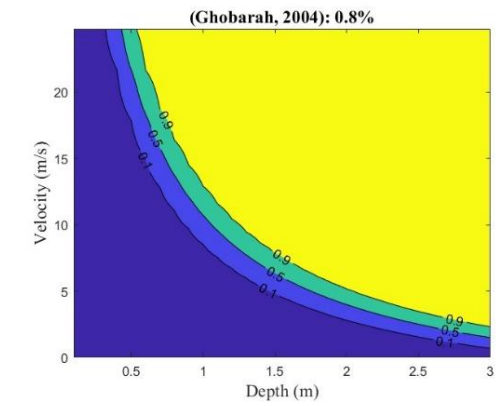
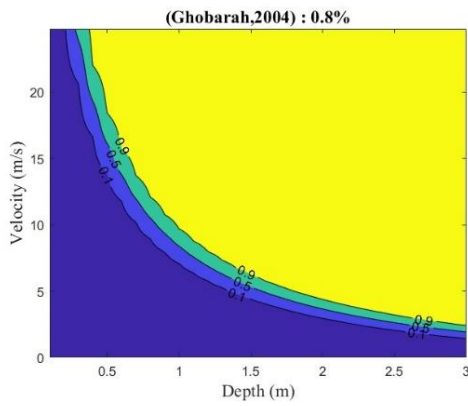
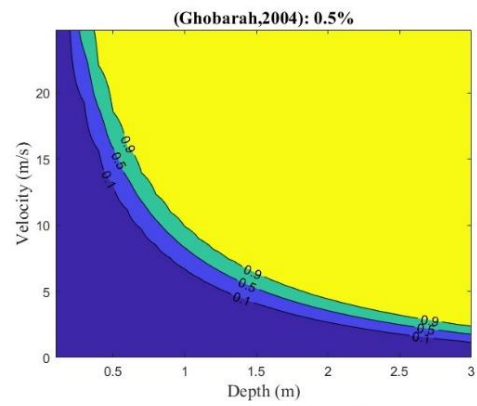
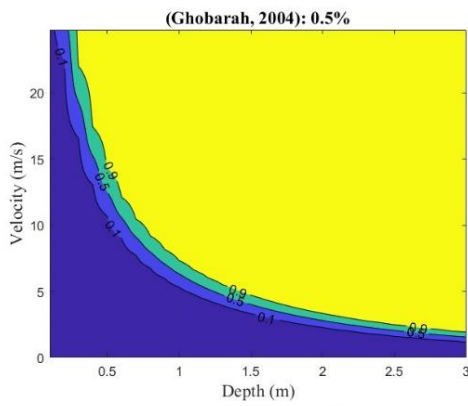
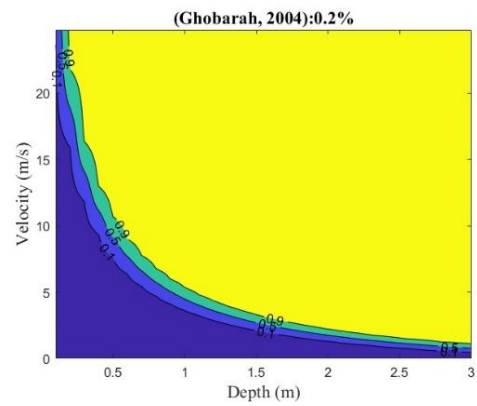
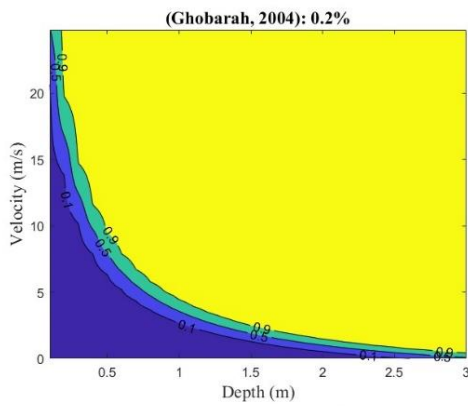


Table 3.12: Comparison of velocity thresholds and reductions for RC frames under flood/flow-type landslide hazards at fixed depths (1.5m and 2.0m), considering earthquake-resistant (EQR) and gravity-load-designed (GLD) frames, debris impact, and Eurocode/FEMA Ghobarah's study damage states[77].

Flow Depth (m)	Comparison Type	Case Pair	Case Acronyms	Design Type	Flow Type	Debris Mass (kg)	Damage State	Velocity Threshold (m/s)	Velocity Reduction
d=1.5	Structural Performance	Case1	EQ-L-M0	EQR	landslide	0	NC, Eurocode	21	~43%
		Case5	GL-L-M0	GLD				12	
d=1.5	Structural Performance	Case3	EQ-L-MH	EQR	landslide	453.6	NC, Eurocode	7	~57.1%
		Case7	GL-L-MH	GLD				3	
d=2	Hazard	Case1	EQ-L-M0	EQR	landslide	0	SD, Eurocode	14	~24.3%
		Case2	EQ-F-M0		Flood			18.5	
d=2	Hazard	Case5	GL-L-M0	GLD	landslide	0	SD, Eurocode	8	~27.3%
		Case6	GL-F-M0		Flood			11	
d=2	Debris	Case6	GL-F-M0	GLD	Flood	0	DS3(0.8%) Ghobarah	10	~60%
		Case8	GL-F-MH			453.6		4	
d=2	Performance Standard	Case1	EQ-L-M0	EQR	landslide	0	NC, Eurocode	16.25	~21%
		Case1	EQ-L-M0					20.5	
							CP, FEMA		

3.4.5 Comparisons

This section evaluates the proposed RC frame models by comparing them against some damage assessment approaches from the literature. Table 3.13 and Figure 3.12 provide an overview of the existing models used in the comparison. It is evident that many studies have predominantly focused on fragility curves based on a single intensity measure with limited consideration given to debris impacts or different flow hazard effects. Therefore, this makes the comparisons performed partial in Table 3.13 and Figure 3.12.

First, the proposed fragility models (Case 1 and Case 5 in Figure 3.10, based on Eurocode) were compared to two existing models from the literature for flow-type landslides without debris: two-bay, two-story bare RC frame studies by Parisi & Sabella [6], which are analytical studies for an earthquake-resistant and a gravity-load design RC frame under flow-type landslides but do not consider any debris impact loads; and a study by Kang & Kim [65] (see Table 3.1), which provides empirical vulnerability curves derived from field data through 25 buildings and 11 debris flow events.

In the proposed models, both depth and velocity serve as intensity measures,

whereas the study by Parisi & Sabella [6] use velocity alone with depth modeled as a random variable between 1.5 m and 4.5 m (a mean of 3 m). To facilitate direct comparison between the models, depth in the proposed models is fixed at 3 m, and the 50% exceedance velocities are extracted for two damage states: DL (moderate) and NC (extensive). When comparing the proposed models to existing models in Table 3.13 and Figure 3.12, comparable results were found. The results by Parisi & Sabella [6] show some variability, yet, the results are not too far from the predictions by the proposed models. This can be explained due to that fact that Parisi & Sabella [6] treat the depth as a random variable with a mean of 3 m in their study, while the proposed models take depth as a given deterministic value of 3 m. Also, while the study by Parisi & Sabella [6] uses a 3 m-story height, the proposed model uses a first story height of 3.5 m, which makes the first-story column more flexible and can explain why the proposed models show smaller levels of velocity for damage.

Again, the same outcomes of the proposed models but only for the gravity frame one Case5 (GLD-L-M0 in Figure 3.10, based on Eurocode) are compared with the results by Kang & Kim [65], which are in terms of impact-pressure ranges (without any debris). Since the buildings investigated by Kang & Kim [65] are located in South Korea, a region characterized by low seismicity, their empirical vulnerability curves are considered more representative of gravity-load designed structures. To compare the models, the velocity outcomes in this study are converted to impact pressures in Table 3.13. Again, comparable results were found. For example, the impact pressure value of 59.77 kPa found for DL in the proposed model Case 5 fall within their empirical range of 35–100 kPa.

It should be noted here that, in this section, the results for Case1 (EQR-L-M0, Eurocode) are compared solely with the ones by Parisi & Sabella [6], as their model represents one of the few options available for earthquake-resistant frames (see Figure 3.12a & c).

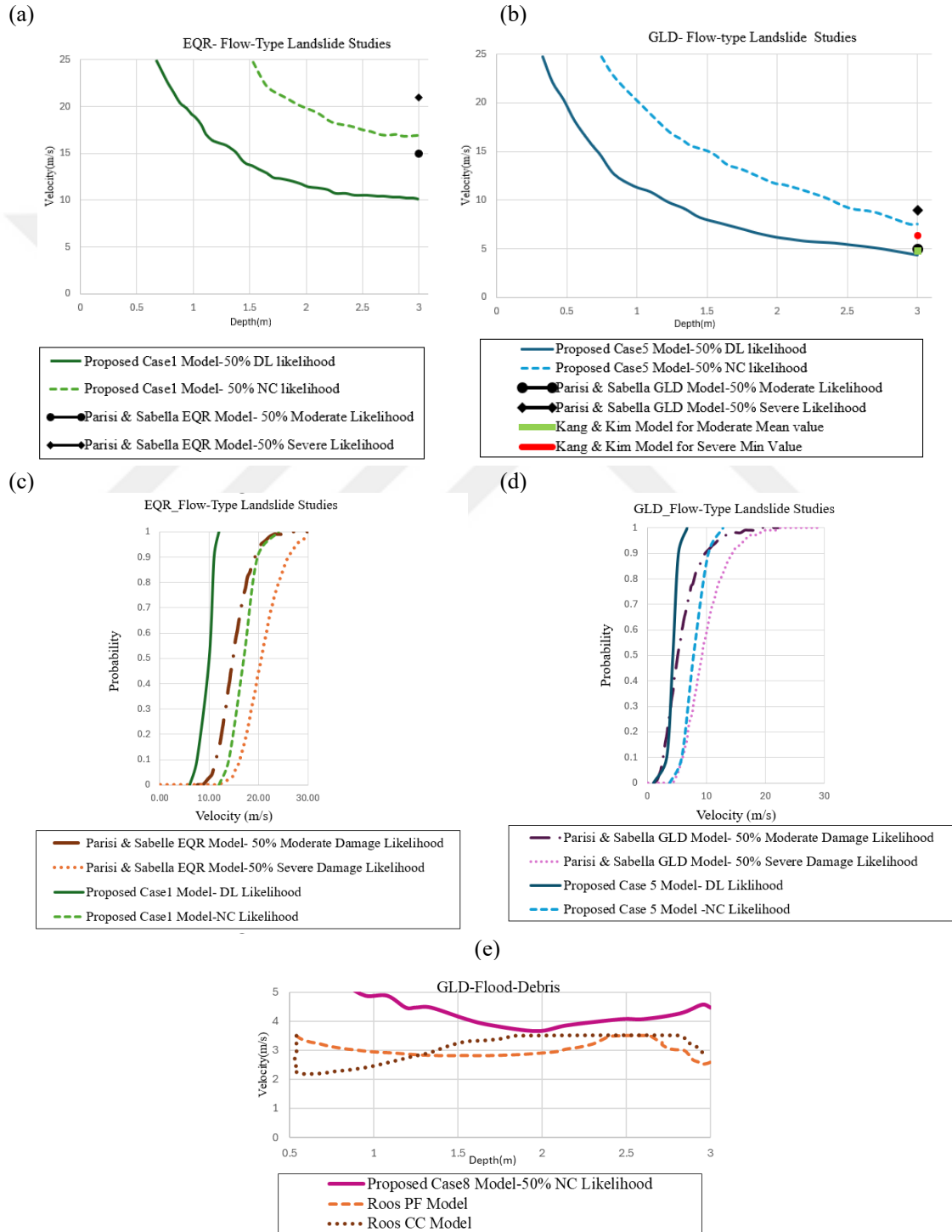
Also, in Figure 3.12e, the proposed fragility model for flood with debris load, Case 8 (GLD-F-MH, based on Eurocode), was compared to numerical fragility models developed by Roos [10] for prefabricated and cast concrete buildings subjected to combined flood and debris loading (50 kg). It should be noted that the existing models for Roos [10] are available for depth after 0.5 m and velocity until 3.5 m/s. Despite the larger debris mass considered in the proposed Case 8 model, its trends are observed to comparable results to the ones by Roos [10] for the specified ranges for this existing model, thus supporting the partial validity of the proposed models for debris scenarios.

In summary, despite differences in modeling assumptions and intensity measures, the comparisons show reasonable agreement between the proposed models and existing ones. The goal here is not to achieve exact numerical agreement but at least to partially validate the proposed methodology that it produces consistent trends with existing frameworks. Small discrepancies arise naturally from differing assumptions (e.g., fixed vs. random depth) and structural variations. Overall, all these comparisons underscore the proposed models' applicability for different flow scenarios for floods and flow-type landslides events, including debris impacts. Moreover, while existing models (e.g., Parisi & Sabella [6]) generally rely on one intensity measure-velocity, the proposed models in this study incorporate both depth and velocity as intensity measures, and also consider different flow densities. Lastly, the proposed models consider a systematic treatment of significant uncertainties, therefore, making them more robust and reliable.

Table 3.13: Comparison of proposed fragility models with existing models from the literature based on critical damage levels for velocity and impact pressure levels under flow-type landslides.

Proposed Fragilities (based on Eurocode)				The study by Parisi & Sabella.[6]		The study by Kang & Kim [65]	
	Damage State	Velocity at $P_{\text{Cond}} = 0.5$ (m/s)	Computed Impact Range(KPa)	Damage State	Velocity at $P_{\text{Cond}} = 0.5$ (m/s)	Damage State	Impact Pressure Range(KPa)
Case 1(EQR-L-M0)	DL	10	N/A	Moderate	15	N/A	N/A
	NC	17	N/A	Severe	21	N/A	N/A
Case 5 (GLD-L-M0)	DL	4.3	59.77	Moderate	5	Moderate	35–100
	NC	7.75	133.209	Severe	9	Severe	>100

Figure 3.12: Comparisons of the proposed models: (a) Earthquake resistant RC frames (EQR) exposed to flow-type landslide based on Eurocode (depth, velocity) , (b) Gravity-load design RC frame (GLD) exposed to flow-type landslide based on Eurocode (depth, velocity), (c) Earthquake resistant RC frames (EQR) exposed to flow-type landslide based on Eurocode (velocity, probability), (d) Gravity-load design RC frame (GLD) exposed to flow-type landslide based on Eurocode (velocity, probability). and (e) Gravity-load-design (GLD) RC frame exposed to flood and debris impact (depth, velocity).



3.5 Conclusions

This study evaluates the damage potential of low-rise reinforced concrete (RC) frames under flood and flow-type landslide events, by comparing two design categories: one incorporating earthquake-resistant detailing and the other employing a gravity-load design. Performance thresholds were established according to drift-based studies (FEMA 356 and Ghobarah's study) and Eurocode8Part3 standards. Incremental flow analysis, enhanced through logistic regression, was used to derive fragility surfaces and functions that predict damage probabilities under varying flow conditions.

In this study, to manage computational demands while ensuring model accuracy, a lumped plasticity approach was adopted and validated against a distributed plasticity model. Although effects such as uplift wave pressure were not included, widely accepted formulations for flow actions were employed to maintain a robust and reliable analysis framework, balancing computational efficiency with predictive fidelity.

A notable contribution of this work is its novel approach that incorporates debris impact effects, diverse structural design cases, and detailed uncertainty modeling into the fragility assessment process. This research investigates the effects of floating debris (e.g., tree logs, vehicles) commonly observed during flow events. Moreover, the study explicitly considers different flow conditions with variable flow density. It also integrates a comprehensive assessment of uncertainties in material properties, geometry, and flow intensity effects into the fragility modeling process. Additionally, this work integrates two distinct performance codes, providing a broader perspective on vulnerability and an opportunity to make comparisons between different guidelines, in contrast with the predominant trend in the literature of employing a single code. By comparing outcomes from both drift-based standards (FEMA and Ghobarah's study) and Eurocode standards, the study reveals substantial differences in the predicted damage thresholds.

The outcomes of this investigation indicate that earthquake-resistant frames consistently exhibit superior performance compared to gravity-load-designed frames, as evidenced by fragility surfaces that require higher flow intensities to reach similar damage levels. Additionally, the study reveals that flow-type landslide conditions impose a significantly higher risk than standard flood scenarios, emphasizing the need for hazard-specific design strategies for these events.

Ultimately, the fragility surfaces and functions developed in this study provide critical tools for forecasting structural response under extreme flow conditions. Their detailed formulation, which accounts for extensive uncertainties, offers valuable insights for optimizing design and retrofitting strategies, thereby enhancing the resilience of low-rise RC structures against flood and flow-induced landslide hazards.

4. CONCLUSIONS

This research offers comprehensive fragility assessments for two prevalent building typologies exposed to flood and flow-type landslide hazards: one- and two-story unreinforced masonry (URM) buildings and low-rise reinforced concrete (RC) frames. By using incremental flow analyses enhanced with logistic regressions and Monte Carlo simulations, which cover uncertainties related to geometry, materials, and loading, a robust framework for flow risk assessment is established.

The developed collapse fragility models for URM structures specifically focus on single load-bearing walls, recognizing that the failure of these walls typically results in overall structural collapse. Detailed nonlinear finite element modeling and incremental flow analyses were performed in ABAQUS and validated using numerical and experimental studies from existing literature.

In evaluating RC frame buildings subjected to flow-induced hazards, this research compares earthquake-resistant frames with gravity-load-designed structures through nonlinear analyses performed using OpenSees and lumped plasticity modeling, further validated against reported results for a relevant case study from the literature as well as analytical analysis results obtained using a different FE program.

The proposed models reveal notable performance disparities for low-rise buildings under various flood and landslide scenarios. To accurately capture these disparities, different maximum flow velocities for URM (3 m/s) and RC frames (25 m/s) were intentionally adopted in this study, reflecting their fundamentally different structural resilience under flow hazards. Unlike conventional deterministic approaches, the models effectively incorporate diverse flow conditions—including flooding and landslide-induced flows with varying floating debris impacts—and multiple structural conditions and design criteria. An additional comparative layer in this research for RC frames, is considering and comparing FEMA, Ghobarah's study and Eurocode performance standards in developing the

fragility models, providing a broader perspective on vulnerability. This comparison addresses a gap in existing studies and reveals important differences in predicted damage levels and structural resilience between using rotation- and drift-based damage state definitions.

The results show that URM buildings are especially vulnerable, particularly when impacted by fast-moving flows carrying heavy debris. Also, the structural response of these buildings strongly depends on boundary conditions and vertical loads from upper floors, making these factors essential for accurately assessing collapse risks.

For RC frames, results demonstrate that earthquake-resistant frames can withstand significantly stronger flow intensities before reaching the same damage level in comparison with gravity load design frames, highlighting the importance of appropriate choice for lateral design methods for such vulnerable structures exposed to flow events. This difference was especially pronounced under flow-type landslide conditions and in the presence of debris, where gravity-designed frames exhibited significantly reduced capacity. Additionally, comparing the fragility results obtained from Eurocode and drift-based performance standards (FEMA and Ghobarah) revealed the critical influence of the predicted thresholds, with Eurocode generally indicating stricter criteria.

These observations emphasize the importance of selecting robust lateral design methods and context-appropriate performance standards when evaluating RC frame vulnerability in flood and landslide-prone areas.

The fragility models developed here significantly improve our understanding of structural vulnerability and offer actionable pathways to mitigate risks posed under flow-induced hazards. These models provide valuable tools for improving building codes, emergency response plans, and community safety, ultimately helping reduce risks associated with natural disasters. Engineers and urban planners can utilize these models to prioritize retrofitting of vulnerable URM buildings, such as reinforcing load-bearing

walls or improving boundary conditions, and to design new low-rise RC structures with flow-specific resilience criteria. Emergency management agencies can apply the models to generate risk maps that identify high-collapse-probability zones under varying flow intensities, enabling targeted evacuation protocols and resource allocation during disasters. Additionally, community safety programs can utilize these findings to educate stakeholders on the risks of non-engineered URM structures and gravity-load-designed RC frames, promoting safer construction practices in flood- and landslide-prone areas. By bridging the gap between academic research and practical implementation, this work provides multidisciplinary stakeholders with data-driven tools to enhance structural resilience, improve disaster planning, and ultimately save lives and livelihoods in the face of increasing climate-driven hazards.

While this study presents significant advancements, the proposed models include certain limitations. Flood and landslide flows were assumed to be homogeneous throughout the flow depth. Flow depth and velocity were treated as independent parameters in the fragility assessment framework; and secondary effects such as uplift pressures, wave actions, and foundation scour were not considered. In the URM building analysis, macro-model simplifications neglected localized failure mechanisms, such as brick dislodgment in weak-jointed masonry and clay brick masonry was considered as the representative URM typology. For RC frames, a lumped plasticity approach was used to capture structural response, which effectively models flexural failures but may underrepresent brittle or shear-dominated mechanisms at beam-column joints. While plastic hinge locations at member ends were validated for static flood/debris loading conditions, dynamic impact effects on moment distributions warrant further investigation in future studies. Although simplifications were adopted to ensure computational efficiency, these assumptions have been transparently acknowledged. They are consistent with methodological practices typical of fragility studies.

Future research should address these limitations by incorporating nonhomogeneous flow conditions, advanced micro-modeling techniques and diverse masonry types (e.g., stone or confined masonry), and secondary hydrodynamic effects—including scour—to enhance model robustness. Future work should also consider hydrological relationships between flow depth and velocity, treating them as dependent parameters to develop more realistic single-variable fragility models. Additionally, further studies should integrate dynamic loading effects and more detailed soil-structure interaction modeling to improve predictive accuracy and enable more comprehensive risk assessments.



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APPENDICES

APPENDIX A. STRUCTURAL DESIGN VERIFICATION

As part of the structural design verification process, all beams and columns within the earthquake-resistant and gravity-load-designed RC frames were evaluated in accordance with Eurocode 2 [85]. For conciseness, representative examples from each frame are presented below. These include one beam and one column from the earthquake-resistant frame and one beam and one column from the gravity-load-designed frame. The demand analyses for structural members were conducted under the governing load combinations specified by the Italian Building Code (NTC 2018) [78] for the earthquake-resistant frame (EQR), as this frame was validated against the work of Ferraioli [71], which originally employed NTC 2018. Given the alignment of NTC 2018 with Eurocode provisions for seismic design (e.g., load combination factors and partial safety factors), the adopted combinations ensure consistency with Eurocode-based verification methodologies. For the gravity-load-designed (GLD) frame, the gravity load combinations were retained from the EQR frame to maintain uniformity in comparative analysis, reflecting pre-code practices where seismic actions are excluded. The critical design actions (M_{Ed} , V_{Ed} , P_{Ed}) are identified as the maximum values arising from the most severe load cases. These demands are then compared with the corresponding member resistances (M_{Rd} , V_{Rd} , N_{Rd}) to ensure compliance with safety requirements. For columns, interaction diagrams are used to verify the combined effects of axial load and bending moment, ensuring that the design point lies within the acceptable domain for all load combinations.

A.1 Earthquake-Resistant Frame

A.1.1 Beam B1 (First Story)

Geometry and Material Properties:

- Span, $L = 5.0$ m
- Width, $b = 300$ mm
- Total depth, $h = 500$ mm
- Effective depth, $d = 445$ mm
- Concrete strength, $f_{ck} = 25$ MPa $\rightarrow f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{25}{1.5} = 16.67$ MPa
- Steel yield strength, $f_{yk} = 450$ MPa $\rightarrow f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{450}{1.15} = 391.3$ MPa
- Reinforcement area, $A_{s,top} = 615.75$ mm²

Flexural Capacity Check:

- $a = \frac{A_s f_{yd}}{0.85 f_{cd} b} = 56.68 \text{ mm}$
- Design moment resistance, $M_{Rd} = A_s f_{yd} \left(d - \frac{a}{2} \right) = 91.6 \text{ kN} \cdot \text{m}$
- $M_{Ed} = 40.37 \text{ kN} \cdot \text{m} \rightarrow M_{Rd} > M_{Ed} \quad \checkmark$

Shear Capacity Check:

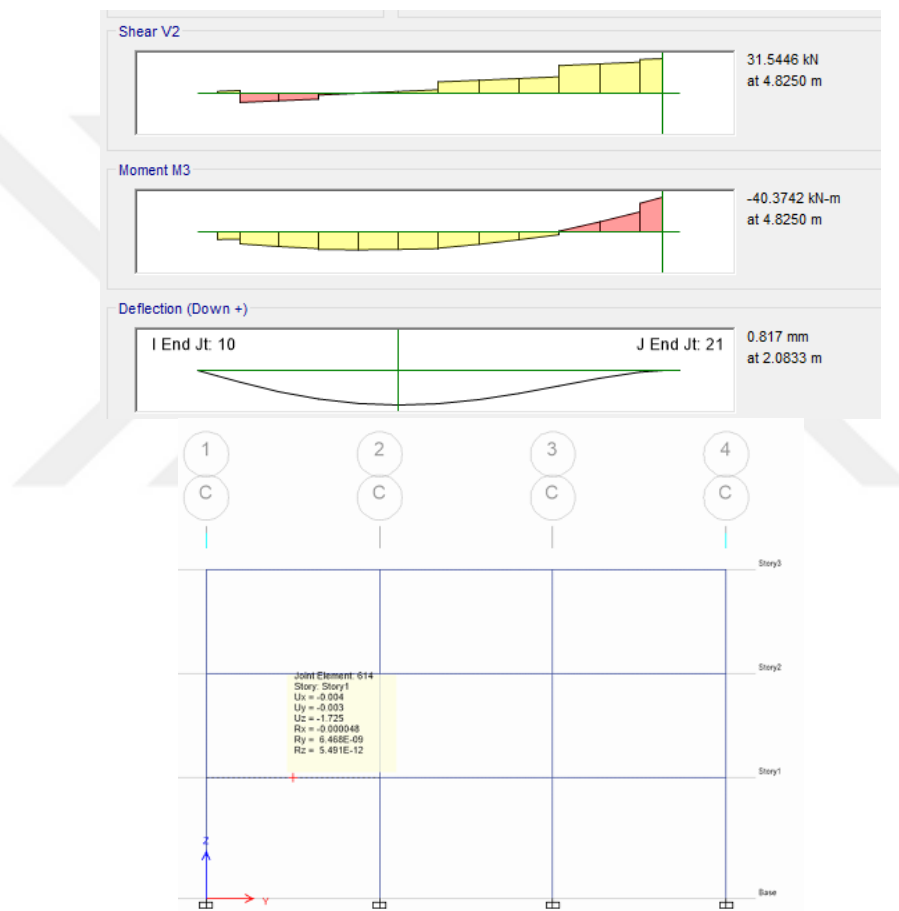
- $k = 1 + \sqrt{\frac{200}{d}} = 1.67 (\leq 2.0)$
- Design shear resistance, $V_{Rd,c} = \left[0.12 k (100 \rho_l f_{ck})^{1/3} \right] b d = 60.43 \text{ kN}$
- $V_{Ed} = 31.54 \text{ kN} \rightarrow V_{Ed} < V_{Rd,c} \quad \checkmark$

Deflection Check:

- Actual deflection = $1.725 \text{ mm} \times 5.85 = 10.1 \text{ mm}$
- Allowable deflection = $\frac{L}{250} = \frac{5000}{250} = 20 \text{ mm}$
- $\rightarrow 10.1 \text{ mm} < 20 \text{ mm} \quad \checkmark$

The figure below shows the shear force, bending moment, and deflection diagrams for Beam B1 extracted from ETABS under the governing load combination.

Figure A.1: Shear force, bending moment, and deflection diagrams for Beam B1 (Earthquake-Resistant Frame) under governing load combination.



A.1.2 Column C1 (First Story)

Geometry and Material Properties:

- Column height, $l = 3.5$ m
- Width, $b = 300$ mm
- Total depth, $h = 350$ mm
- Reinforcement area, $A_{s,\text{total}} = 1526.8 \text{ mm}^2$
- Net concrete area, $A_c = 103473.2 \text{ mm}^2$
- Stirrups, $\phi 8@150$ mm
- Effective cover, $a = 40 + 8 + 18/2 = 57$ mm

Slenderness Check:

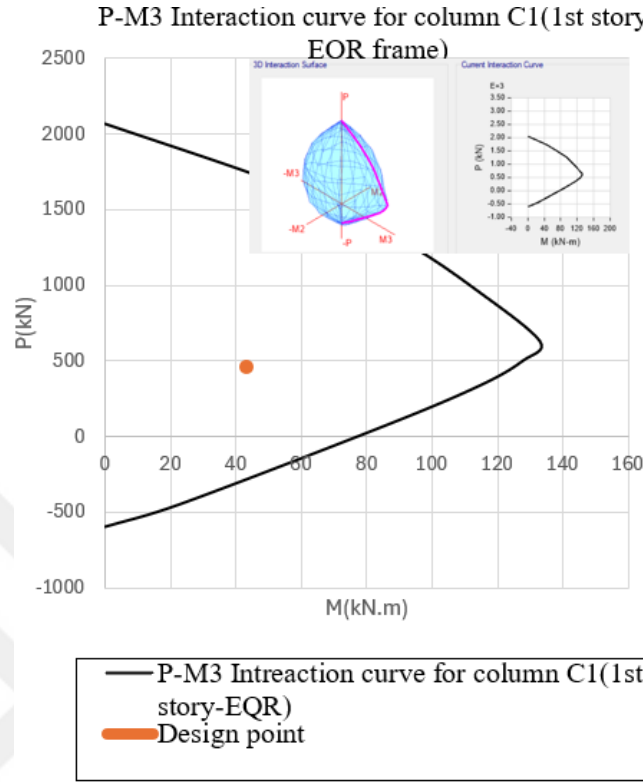
- Effective length, $l_0 = 0.7l = 2.45$ m
- Radius of gyration, $i = \frac{h}{\sqrt{12}} = 101.2$ mm
- Slenderness ratio, $\lambda = \frac{l_0}{i} = \frac{2450}{101.2} = 24.2 < 25$ ✓ (no second-order effects)

Axial and Bending Load Resistance:

- Concrete contribution, $0.85 f_{cd} A_c = 1466.16$ kN
- Steel contribution, $A_{s,\text{total}} f_{yd} = 597.44$ kN
- Total axial capacity, $N_{Rd} = 2063.6$ kN
- Design load, $P_{Ed} = 461.9$ kN $\rightarrow P_{Ed} < N_{Rd}$ ✓
- $a = \frac{A_s f_{yd}}{0.85 f_{cd} b} = 140.3$ mm
- Design moment resistance, $M_{Rd} = A_s f_{yd} \left(d - \frac{a}{2}\right) = 133.0$ kN · m
- $M_{Ed} = 43$ kN · m $\rightarrow M_{Rd} > M_{Ed}$ ✓

Interaction diagram for Column C1 under combined axial load (P) and bending moment (M_3). The design point lies safely within the interaction curve, confirming compliance with Eurocode 2.

Figure A.2: Interaction diagram for Column C1 (Earthquake-Resistant Frame).

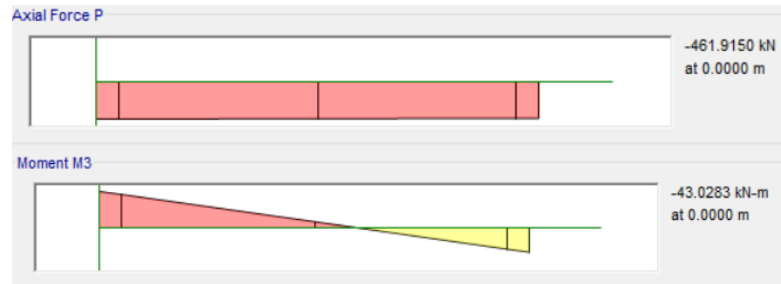


Reinforcement Limits:

- Minimum reinforcement, $A_{s,min} = \max\left(0.1 \frac{N_{Ed}}{f_{yd}}, 0.002 A_c\right) = \max(83.6 \text{ mm}^2, 206.95 \text{ mm}^2) < 1526.8 \text{ mm}^2 \quad \checkmark$
- Maximum reinforcement, $A_{s,max} = 0.08 A_c = 87786 \text{ mm}^2 > 1526.8 \text{ mm}^2 \quad \checkmark$

The following figures illustrate the moment and axial force distribution in Column C1 based on ETABS output under the governing load combination.

Figure A.3: Moment and axial force distribution in Column C1 (Earthquake-Resistant Frame) based on ETABS analysis.



A.2 Gravity-Load-Designed Frame

A.2.1 Beam B (First Story)

Geometry and Material Properties:

- Span, $L = 5.0$ m
- Width, $b = 300$ mm
- Total depth, $h = 400$ mm
- Effective depth, $d = 344$ mm
- Concrete strength, $f_{ck} = 16$ MPa $\rightarrow f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{16}{1.5} = 10.67$ MPa
- Steel yield strength, $f_{yk} = 250$ MPa $\rightarrow f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{250}{1.15} = 217.39$ MPa
- Reinforcement area, $A_{s,top} = 603.19$ mm²

Flexural Capacity Check:

- $a = \frac{A_s f_{yd}}{0.85 f_{cd} b} = 48.2$ mm
- Design moment resistance, $M_{Rd} = A_s f_{yd} \left(d - \frac{a}{2} \right) = 62$ kN · m
- $M_{Ed} = 23.62$ kN · m $\rightarrow M_{Rd} \geq M_{Ed}$ ✓

Shear Capacity Check:

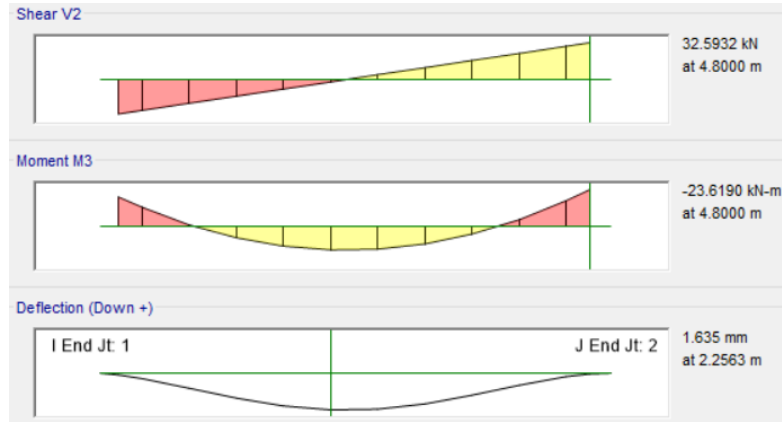
- $k = 1 + \sqrt{\frac{200}{d}} = 1.76$ (≤ 2.0)
- $V_{Rd,c} = \left[0.12 k (100 \rho_l f_{ck})^{1/3} \right] b d = 45.8$ kN
- $V_{Ed} = 32.59$ kN $\rightarrow V_{Ed} < V_{Rd,c}$ ✓

Deflection Check:

- Actual deflection = 1.635 mm $\times 5.85 = 9.56$ mm
- Allowable deflection = $\frac{L}{250} = \frac{5000}{250} = 20$ mm
- $\rightarrow 9.56$ mm < 20 mm ✓

The figure below shows the shear force, bending moment, and deflection diagrams for Beam B extracted from ETABS under the governing load combination.

Figure A.4: Shear force, bending moment, and deflection diagrams for Beam B (Gravity-Load Frame) under governing load combination.



A.2.2 Column C2 (First Story)

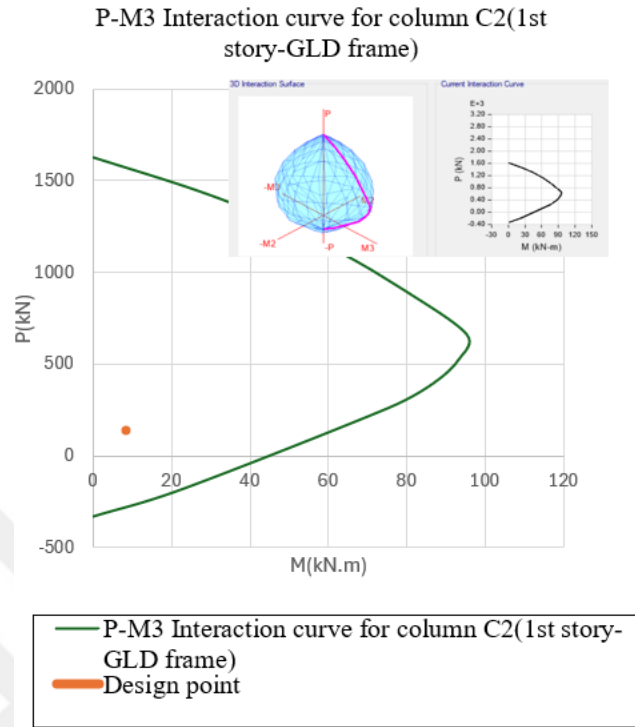
Geometry and Material Properties:

- Column height, $l = 3.5$ m
- Width, $b = 350$ mm
- Total depth, $h = 350$ mm
- Reinforcement area, $A_{s,\text{total}} = 1526.8 \text{ mm}^2$
- Net concrete area, $A_c = 120973.2 \text{ mm}^2$
- Stirrups, $\phi 8 @ 150$ mm
- Effective depth, $d = 293$ mm

Axial Load and Bending Resistance:

Interaction diagram for Column C2 under combined axial load (P) and bending moment (M_3). The design point lies safely within the interaction curve, confirming compliance with Eurocode 2.

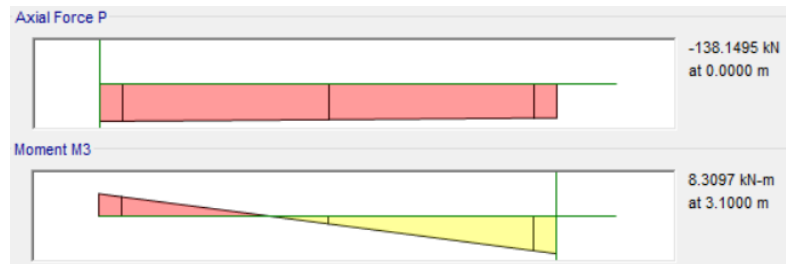
Figure A.5: Interaction diagram for Column C2 (Gravity-Load Frame).



Reinforcement Limits:

- Minimum reinforcement, $A_{s,min} = \max\left(0.1 \frac{N_{Ed}}{f_{yd}}, 0.002 A_c\right)$
 $= \max(61.7 \text{ mm}^2, 241.95 \text{ mm}^2) < 1526.8 \text{ mm}^2 \quad \checkmark$
- Maximum reinforcement, $A_{s,max} = 0.08 A_c = 9677.86 \text{ mm}^2 > 1526.8 \text{ mm}^2 \quad \checkmark$

Figure A.6: Moment and axial force distribution in Column C2 (Gravity-Load Frame) based on ETABS analysis.



APPENDIX B. PARAMETERS DEFINED FOR BILIN MATERIAL MODEL (MODIFIED IMK)

The following table summarizes the parameters used in the Bilin material model in OpenSees, based on the modified Ibarra–Medina–Krawinkler (IMK) deterioration model. These inputs were defined using values from moment–curvature analysis and empirical formulas. Rotational parameters and moments were extracted from Table 3.5 (gravity-load designed frame) and Table 3.6 (earthquake-resistant frame), and parameters were computed using the equations presented in Table B.1 below.

To accurately model the nonlinear behavior of structural members, rotational springs (representing plastic hinges) are connected in series with elastic beam-column elements. These springs capture inelastic deformations and energy dissipation. Following Ibarra and Krawinkler [82], the rotational spring stiffness K_s is set to nK_{bc} , where $K_{bc} = 6EI/L$ represents the nominal stiffness of the beam-column element under double curvature. This approach avoids numerical instability caused by infinite stiffness assumptions while ensuring realistic deformation sharing between sub-elements. A multiplier of $n = 10$ was recommended to preserve damping characteristics, accommodate moment gradient changes, and stabilize cyclic deterioration modeling. In this study, a similar formulation was adopted, with K_s scaled proportionally to K_{bc} , maintaining computational stability and physical accuracy in hinge behavior and member flexibility.

The strain hardening coefficient of the rotational spring, denoted as $\alpha_{s,s}$, was calibrated to reproduce the intended post-yield behavior of the full structural member. Since the member is modeled as a combination of an elastic beam-column element and a non-linear rotational spring, only part of the post-yield deformation occurs within the spring. To ensure that the desired global strain hardening ratio, $\alpha_{s,mem}$, is achieved, the spring's coefficient was adjusted using the following expression: $\frac{(n+1)\alpha_{s,mem}}{n+1 - n\alpha_{s,mem}}$. This formulation accounts for deformation sharing between the sub-elements and provides a consistent representation of the moment-rotation behavior [82].

Table B.1: Parameters used in the Bilin material model in OpenSees, based on the modified Ibarra–Medina–Krawinkler (IMK) deterioration model.

Parameter	Description	Value / Source
Ks ^a	Elastic stiffness	$\frac{n6E_cI}{L}$
as-Plus ^b	Strain hardening ratio for positive loading direction $\alpha_{s,s}$	$\frac{(n+1)\alpha_{s,mem}}{n+1-n\alpha_{s,mem}}$
as-Neg ^b	Strain hardening ratio for negative loading direction	$\frac{(n+1)\alpha_{s,mem}}{n+1-n\alpha_{s,mem}}$
My-Plus	Effective yield strength for positive loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
My-Neg	Effective yield strength for negative loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Lamda-S	Cyclic deterioration parameter for strength deterioration	Default
Lamda-C	Cyclic deterioration parameter for post-capping strength deterioration	Default
Lamda-A	Cyclic deterioration parameter for accelerated stiffness deterioration	Default
Lamda-K	Cyclic deterioration parameter for unloading stiffness deterioration	Default
c-S	Rate of strength deterioration	Default (1.0)
c-C	Rate of post-capping strength deterioration	Default (1.0)
c-A	Rate of accelerated reloading deterioration	Default (1.0)
c-K	Rate of unloading stiffness deterioration	Default (1.0)
Theta-p-Plus	Pre-capping rotation for positive loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Theta-p-Neg	Pre-capping rotation for negative loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Theta-pc-Plus	Post-capping rotation for positive loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Theta-pc-Neg	Post-capping rotation for negative loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Res-Pos	Residual strength ratio for positive loading direction	$\frac{M_{res}}{M_y}$
Res-Neg	Residual strength ratio for negative loading direction	$\frac{M_{res}}{M_y}$
Theta-u-Plus	Ultimate rotation capacity for positive loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
Theta-u-Neg	Ultimate rotation capacity for negative loading direction	See Table 3.5 (GLD) and Table 3.6 (EQR)
D-Plus	Rate of cyclic deterioration in the positive loading direction	Default (1.0)
D-Neg	Rate of cyclic deterioration in the negative loading direction	Default (1.0)
nFactor	Elastic stiffness amplification factor for concentrated hinges	Default (1.0)

^a where $n = 10$, E_c is concrete modulus of elasticity, I is moment of inertia of the member (e.g., beam, column) and L is member length (e.g., bay width or story height)[82, 83].

^b where $\alpha_{mem} = \frac{\Delta M_{in}}{k \Delta \theta}$ [82]

APPENDIX C. EUROCODE FORMULATION USED FOR VALIDATION OF ROTATION CAPACITIES

This appendix provides additional information on the equations coming from Eurocode [4], which were used for the validation of rotation capacities (yield and ultimate rotations) utilized in the structural modeling and analysis of reinforced concrete (RC) frames, as detailed in Section 3. The yield rotation (θ_y) was calculated using the following equation:

$$\theta_y = \frac{\phi_y L_v}{3} + 0.0013 \left(1 + 1.5 \frac{h}{L_v} \right) + 0.13 \phi_y \frac{d_{bl} f_y}{\sqrt{f_c}} \quad (\text{C.1})$$

where ϕ_y is the yield curvature ($\frac{\varepsilon_y}{d-a/2}$), L_v is the shear span, h is the section depth, d_{bl} is the diameter of the longitudinal reinforcement bar, f_y is the yield strength of reinforcement, f_c is the concrete compressive strength, d is the effective depth, and a is the depth of the equivalent rectangular stress block ($\frac{A_s f_y}{0.85 f_c b}$).

The ultimate rotation (θ_u) was determined using the following equation:

$$\theta_u = 0.016 (0.3)^\nu \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega)} f_c \right]^{0.225} \left(\frac{h}{L_v} \right)^{0.35} 25^{\alpha \rho_{sw} \frac{f_{yw}}{f_c}} 1.25^{100 \rho_d} \quad (\text{C.2})$$

where ν is the normalized axial load ($\frac{N}{f_c A_g}$), ω is the tension-reinforcement ratio ($\frac{A_s f_y}{A_g f_c}$), ω' is compression reinforcement ratio ($\frac{A'_s f_y}{A_g f_c}$), α is confinement effectiveness ($\alpha = \alpha_n \cdot \alpha_s$), ρ_{sw} is transverse reinforcement ratio, f_{yw} is stirrup yield strength; and ρ_d is steel ratio of diagonal reinforcement.

The rotation at maximum moment (θ_c) is defined as:

$$\theta_c = \frac{3}{4} \theta_u \quad (\text{C.3})$$

These formulations provided a basis for verifying the rotation values obtained through detailed moment–curvature analysis in ETABS, thus ensuring consistency and reliability of the modeling approach adopted for the nonlinear structural analysis and subsequent pushover simulations.

To validate the theoretical equations for yield rotation (θ_y) and ultimate rotation (θ_u), all structural elements (beams and columns) in the earthquake-resistant and gravity-load designed frames were analyzed. These tables present representative examples of one beam and one column from each frame.

Table C.1: Example of checks conducted for beam and column's rotation capacities used for the earthquake-resistant frame.

Element	Parameter	Equation value	Value from ETABS Section Analysis
Beam B1	Yield rotation (θ_y)*	0.00699	0.00673
	Ultimate rotation (θ_u)	0.0708	0.070796
Column C1	Yield rotation (θ_y)*	0.0084	0.008091
	Ultimate rotation (θ_u)	0.0487	0.054468

* Derived as $\theta_y = \theta_u - \theta_p - \theta_{pc}$

Table C.2: Example of checks conducted for beam and column's rotation capacities used for the gravity-load designed frame.

Element	Parameter	Equation value	Value from ETABS Section Analysis
Beam B	Yield rotation (θ_y)*	0.00531	0.00518
	Ultimate rotation (θ_u)	0.0567	0.056703
Column C2	Yield rotation (θ_y)*	0.00503	0.004879
	Ultimate rotation (θ_u)	0.0482	0.036906

* Derived as $\theta_y = \theta_u - \theta_p - \theta_{pc}$

VITA

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