



PHYSICAL VULNERABILITY OF UNREINFORCED,
UNCONFINED, AND CONFINED MASONRY
BUILDINGS DUE TO MASS MOVEMENTS
PHENOMENA AND APPLICATION TO POPAYAN,
CAUCA, COLOMBIA

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Abstract

While climate change and urban development in mountainous terrain continue to take their toll on society through geo-mass transport phenomena, quantitative risk models must evolve and be transmitted as simply as possible to increase and improve the social appropriation of knowledge. Recent mass movement risk analysis developments equate external hazard intensity measures with inner building strength. However, procedures for obtaining building capacity and fragility curves are unclear, particularly for masonry buildings, perhaps the most common construction type worldwide (especially for less developed regions), and when laboratory or field data is scarce. Fragility curves relate the hazard intensity to the probability of reaching or exceeding a level of physical damage and have, therefore, become a fundamental part of vulnerability and risk studies due to natural phenomena, among other things. In this sense, a procedure to obtain structural damage thresholds through fragility curves for mass movements that include the following characteristics is presented: it is not based on seismic design parameters, considers the structural damage probability due to permanent deformations of the foundation ground, does not require an excessive computational effort, and is easy to implement. Geometrical and mechanical parameters are used with their uncertainty, trying to be representative worldwide; nevertheless, the methodology is easily replicable for specific cases. Damage state thresholds are given as a function of momentum flux and mass movement height for two typical thicknesses of unconfined and unreinforced clay brick masonry (URM), and confined clay brick masonry (CM).

Once the structural damage thresholds have been obtained, a specific application case for Popayán, Cauca, Colombia, is created to calculate the risk due to mass movements and generate social appropriation of knowledge.

Dedicatory

I dedicate this achievement to my family, my wife Lida Cristina, my daughters María Alejandra and Natalia, and my son Daniel, for inspiration and motivation. I dedicate this triumph to my parents, who guided me in knowledge and honesty; my mother, who bravely supported her family in difficult times; and my sister and her family, who have always been by my side. This achievement is also dedicated to my beloved teachers and professors for their patience, guidance and teaching, my students for allowing me to learn alongside them, and my friends for their friendship and loyalty.

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Nomenclature

$\overline{hv^2}_{DS}$	Median value of the impact index at which the building reaches a damage state threshold
A	Effective or net cross-sectional shear area of masonry
Au	The ultimate strength of the building in terms of seismic acceleration
Ay	The yield strength of the building in terms of seismic acceleration
b	Building plan dimension of the building. The direction of mass movements is parallel to this dimension.
B	Plan dimension of the building. The direction of mass movements is perpendicular to this dimension.
c	Cohesion
Cb	Building's replacement cost
cd	Drag coefficient
Cs	Dimensionless coefficient
db	Longitudinal reinforcement diameter
df	Foundation depth
E(Y)	Expected value of the function
Em	Masonry elasticity modulus
Es	Mass movements load
Fa	Hydrodynamic or hydrostatic acting force
fc	Specified compressive strength of the concrete
fc _b	Specified compressive strength of the bricks
f _m	Specified compressive strength of the masonry
Fr	The building-resistant force for different damage state thresholds
f _{tb}	Specified tensile strength of the bricks
Fu	The buildings' ultimate strength
f _y	Yield strength of the longitudinal reinforcement
F _y	The basal yield strength of the building
G _m	Modulus of stiffness in shear of the masonry
h	Height brick, mass movements' depth
h'	Vertical measurement of the landslide
hf	Foundation thickness
H	Wall effective height
hv ²	Momentum flux or impact index
I _c	Moment of inertia of the confining columns of the wall
I _m	Wall section inertia
I _s	Moment of inertia of the longitudinal steel of columns of the wall
K _a	Ground lateral pressure coefficient in an active fault state
K _d	Impact coefficient
K _e	Stiffness in the elastic range
K _p	Ground lateral pressure coefficient in a passive fault state
L	Wall length
l	Length brick

n	Number of longitudinal bars in the wall, number of variables to correlate in the Rosenbluth method, number of damage states examined
P_a	Active thrust
P_i	The probability that the building will reach damage state i
$P_{ij..n}$	Weighting factors
P_u	Passive thrust
q_{ult}	The ultimate strength of the out-of-plane walls
q	Gravitational load acting on the building
R	Risk loss
$S(Y)$	The standard deviation of the function
S_u	Ground undrained resistance
t	Wall effective thickness
V, v	Mass movements frontal velocity
V_1	Shear strength due to friction-shear failure mode. URM walls
V_2	Shear strength due to diagonal tension failure mode. URM walls
V_3	Shear strength due to compression failure mode. URM walls
V_{cr}	Cracking shear strength. CM walls
v_m	Diagonal compressive strength of masonry
V_{max}	Maximum shear strength. CM walls
V_{ult}	Ultimate shear strength. CM walls
W	Gravitational loads present in the structure
W_i	The repair percentage costs for damage state i
α	Modal parameter, angle
β	Factor that depends on the support conditions at the ends of the wall, Standard deviation of the natural logarithm of a variable. The terrain slope.
β_{DS}	Standard deviation of the natural logarithm of the impact index associated with a damage state
γ	Shape factor
γ_s	Soil unit weight
η	Efficiency factor
λ	Over-resistance factor, coefficients for fixed H/t ratios
μ	Friction coefficient
ρ	Mass movements density
ϕ	Friction angle
$\rho_{ij..n}$	Correlation coefficients
σ_v	Vertical compressive load on the wall
Φ	The standard normal cumulative distribution function

Abbreviations

CM	Confined clay brick masonry.
URM	Unconfined, unreinforced clay brick masonry.
sd	The standard deviation of a variable.

$P_{SDI}(DS \geq ds)$	Probability of reaching or exceeding a state of damage in the building, given that structural deformations occur in it due to the impact. (Rigid base assumption).
$P_{GF}(DS \geq ds)$	Probability of reaching or exceeding a state of damage in the building foundations, given that there are permanent ground deformations.
$P_{comb}(DS \geq ds)$	The combined probability of reaching or exceeding a state of damage, given that the structure shows deformations due to debris flow impact or loss of support due to permanent ground deformations.
$P_{comb}(DS = ds)$	The combined probability of being in a state of damage, given that the structure presents deformations due to debris flow impact or loss of support due to permanent ground deformations.
$P_{comb}(DS \geq C)$	The combined probability of reaching or exceeding complete damage state, given that the structure shows deformations due to debris flow impact or loss of support due to permanent ground deformations.

1. Introduction

Gravitational downslope movements of rock, debris or soil are called mass movements (D. M. Cruden, 1991). Debris flows, flash floods, and other related phenomena can be classified as mass transport rather than mass movements. However, from the perspective of the impacts on properties/buildings, the force intensities involved are related to fluid density and velocity, as is evident from the classical drag force equation. Thus, changing these parameters allows the analysis of different phenomena under the same framework. This consideration is implicit when physical vulnerability assessment methods for some natural events, e.g., Tsunamis, are used for other events, e.g., Debris Flows, (Prieto et al., 2018). Therefore, specific phenomena, such as landslides, debris flows, debris floods, etc., are not distinguished and are grouped as mass movements.

When mass movements events impact buildings, they cause incalculable social and economic losses (Aristizábal et al., 2015; Aristizábal & Sánchez, 2020; Gomez et al., 2023; Highland & Bobrowsky, 2008; Y. Huang et al., 2023; Santi et al., 2011). These phenomena accounted for 213 mass movements in 38 countries and almost 77,800 deaths between 1950 and 2011, making them the third most significant hazard after floods and earthquakes (Dowling & Santi, 2014). Mass movements in China cause economic losses of approximately USD 276 million and between 300 and 600 human losses per year (C. Peng et al., 2000). In South Korea, 1,728 people lost their lives, while economic losses ranged between USD 500 million and USD 1,000 million due to mass movement events that occurred between 1970 and 2017 (Lee & Winter, 2019). A mass movement occurred in Pakistan between June and July 2022, affected 33 million people and killed more than 1,500 (Khan et al., 2022). The United States Research Council reports that, on average, 25 to 50 people die due to mass movements each year in that country (USGS, 2023).

The spatial distribution of mass movements is heterogeneous, with high incidence in mountain areas of Central and South America, the Caribbean islands, East Africa, Asia, Turkey, Iran, and Europe (Froude & Petley, 2018). In Colombia, 30,730 events have been recorded between 1900 and 2018, resulting in 34,198 deaths and economic losses of around USD 654 million (Aristizábal & Sánchez, 2020). The latest tragic event in Colombia occurred on 12 January 2024 in the Carmen del Atrato municipality, Chocó Department, where a mass movement destroyed the road surface, vehicles, and single-story masonry dwelling, killing 38 people and leaving dozens injured (National et al. Unit, 2024). In recent decades, economic and human losses associated with mass movements have increased dramatically and have occurred more frequently (Aristizábal et al., 2020). The reasons that cause these catastrophic events are a combination of factors such as population growth, urban and rural development in mountainous areas prone to mass movements, informal construction, inadequate management of natural resources, and potent triggers such as unusually high rainfall led by climate change, tectonic plate movement, and volcanic activity.

Hazard is understood as the probability of occurrence of the magnitude of a phenomenon that may cause damage to people, their property, infrastructure, livelihoods, service delivery, and/or the ecosystem. Vulnerability is the physical, economic, social, environmental, or institutional susceptibility or fragility of a community to be affected by the magnitude of a given phenomenon. Risk can be defined as a measure of the likelihood and severity of an event to life, health, property, or the environment over a specified period. Therefore, it is determined by the interaction of hazard, exposure, and vulnerability (UNISDR, 2017).

In recent decades, studying hazards, vulnerability, and risks related to mass movements has been vital to mitigate this global scourge. However, research on physical vulnerability due to mass movements has not evolved at the same rate as in, e.g., earthquake engineering. This is due

to the phenomenon's complexity, which makes it challenging to have generalised criteria for interpreting and calculating the intensity of any event and its interaction with buildings. On vulnerability, Hu et al. (2012) affirm that most mass movement studies have focused on empirical relationships between the magnitude of the hazard and the degree of loss caused by a recent event. Similarly, other research stipulates that these empirical vulnerability functions do not consider the characteristics of the buildings at risk, such as their geometry, structural design, and geographic position (Mazzorana et al., 2014; Papathoma-Köhle et al., 2017). In addition, Luo, et al. (2019) comment that in most mass movements research, vulnerability has been determined by expressing it as a direct function of economic losses without including the structural behaviour. Therefore, considerations such as the structures' damage state, including their mechanisms, and failure processes due to this type of hazard are not yet well included. Damage states categorise the damage to the structure from a non-damage state to a complete damage state. Therefore, damage states can help understand the condition of the building at different hazard intensities (FEMA P-58, 2012). There is still a gap regarding the interaction of buildings and the natural process (Fuchs et al., 2019; Luo et al., 2023; Parisi & Sabella, 2017). Some researchers have tried to overcome this gap by trying to replace, where possible, empirical data with information from laboratory and numerical modelling.

Researchers such as Kean et al. (2019) and Prieto et al. (2018) have addressed the study of hazard, vulnerability, and risk of civil structures due to mass movements, analysing the factors involved, including uncertainty in the performance assessment of timber buildings and unconfined and unreinforced masonry buildings (URM), supported by the state-of-the-art and technical manuals such as (Hazus–MH 2.1: Technical Manual, 2012; Hazus Tsunami Model Technical Guidance, 2017). Nevertheless, in these researches, the structural median strength and their

uncertainty for each damage state were assumed considering parameters from earthquake analyses extracted from documents such as (Acevedo et al., 2017; Hazus–MH 2.1: Technical Manual, 2012), where, e.g. the lateral capacity parameters are in terms of spectral acceleration and modal mass coefficients. It is essential to illustrate that earthquake waves cause the earth's crust to oscillate in different directions over a specific time interval. The sudden movement of the ground is transmitted to the buildings through the foundations, giving rise to floor-to-floor inertial forces that oppose the movement. On the contrary, mass movements would directly hit the structures, generally on the lower floors, in the direction of the slope of the ground, without oscillating ground movements.

Numerical solutions have been implemented for frame-type structures (reinforced concrete buildings) where mass movements vulnerability is calculated using three-dimensional finite element models, including impact load interaction and probabilistic techniques such as Monte Carlo simulation to obtain structural damage thresholds or fragility curves, (H. Luo et al., 2020). However, although it is not a dynamic contact model where the impact force varies over time (Pu et al., 2023), a large amount of computer memory processing time, and costs prevent them from being easily replicable techniques on usual consultancy work (Cardoso et al., 2008; Pu et al., 2023).

Another aspect that has not been incorporated in the available models is the behaviour of the building foundations, i.e. to consider the possibility of permanent ground failures, for example, bearing capacity failure, sliding failure, loss of overall ground stability, and overturning failure of the structure. While damage due to the direct impact of a mass movement can, in some cases, be slight or moderate, failure of the foundation will cause complete damage. Current physical vulnerability models focus on the assumption that buildings rest on a rigid foundation. Therefore, expansion to flexible foundation models is essential.

There are no universally accepted models for quantifying the physical vulnerability of buildings to mass movements (H. Y. Luo et al., 2023). Many works have focused on qualitative methods, and more recently, some have used analytical and numerical methods, including uncertainty, to address the damage mechanisms of buildings interacting with mass movements. While these approaches have made strides in our understanding of structural performance, the absence of a general methodology for practising engineers remains a pressing issue that needs to be addressed. Even the methods for calculating vulnerability and risk due to mass movements in many countries (especially in underdeveloped countries) do not associate the level of damage of the structure with the intensity of the phenomenon, which is fundamental. Therefore, research into the physical vulnerability and risk of mass movements, considering buildings' structural behaviour, must continue to supply structural damage thresholds that update local regulations and provide practical and effective methodologies that are easy to use for engineering purposes.

Clay brick masonry is the most common construction material in many regions, including the Andean countries, representing more than half of the total buildings (Villar-Vega et al., 2017). In the Aburrá Valley, metropolitan Medellín and Bogotá, Colombia, with a population of approximately 3.93 and 7.18 million, respectively, more than 60% of residential buildings are unreinforced clay brick masonry (URM) constructions (Acevedo et al., 2017; Zeballos-Cabrera, 2007). Unfortunately, despite government efforts to approve regulations, some Andean countries still have high rates of self-construction that do not meet the minimum requirements (Yepes-Estrada et al., 2017). The situation is similar in other places, such as Iran, since more than 70% of residential buildings have been built using URM masonry with little shear strength (Khanmohammadi et al., 2014). In Indonesia, clay brick masonry remains a popular structural system for low-rise residential housing, particularly in developing regions, due to its low cost and

ease of construction (Habieb et al., 2023). In India, about 85% of existing buildings are made up of URM structures (Chourasia et al., 2019; Iyer et al., 2012). In South Africa, URM masonry is commonly used as a construction material for 2-5-story buildings (Haas & Kolf, 2014). In Malawi, most houses are URM constructions built by local craftsmen (Novelli et al., 2021). In Southern Europe, the structural performance of URM buildings was recognised as a significant problem, where urban and rural buildings were composed of old brick masonry buildings (Anthoine & Magonette, 1995). In the North Netherlands, most buildings are URM structures (Jafari et al., 2017). URM buildings are the most common constructions in the Mediterranean area (Celano et al., 2021). In Italy, these represent a large part of the existing buildings. The historic Centre of Ferrara comprises 92% URM buildings (Nale et al., 2021).

URM structures are typically the result of self-construction in urban and rural areas. These constructions are typically one - to four-story multi-family buildings in urban areas. At the same time, it is common in rural areas to find individual building units with one or two stories. Simple masonry brick walls in both directions characterise the lateral load-resisting system. The walls are frequently made of hollow or solid-fired clay bricks, non-structural, joined with lime or cement mortar. These buildings are under-designed structures, and their behaviour against lateral loads is considered poor. On the other hand, there are cases in which some elements of confinement in reinforced concrete (columns and beams) are evident. Nevertheless, most buildings do not meet the regulatory requirements to be considered confined masonry buildings (CM), as in the Aburrá Valley, Colombia (Fernández et al., 2019). CM buildings must meet requirements for wall confinement with reinforced concrete beams and columns and specific stiffness and strength parameters in plan and height (Ahmed et al., 2019; Alcocer-Martínez de Castro et al., 2013), which are usually omitted in self-construction. Over the last hundred years, CM technology has been

used to construct low to medium-rise buildings worldwide, particularly in regions characterised by high earthquake risk, including Latin America, Mediterranean Europe, the Middle East, Southeast Asia, and the Far East (Meli et al., 2011; Rankawat et al., 2021).

According to the World Bank Group, the information generated for disaster management has focused on the hazard, (Bank, 2007). Decision-makers lack sufficient information on vulnerability, exposure, and risk. Carrying out vulnerability studies, mainly on critical infrastructure as the basis of rehabilitation programs for risk reduction, is essential. Despite advances in the interaction between mass movement hazards and building fragility quantification, vulnerability assessment remains a weak component of quantitative risk analysis (X. Huang et al., 2023; H. Y. Luo et al., 2023). Clay brick masonry is the most used construction system in the world (URM and CM structures). Therefore, it is essential to recognise its high exposure to environmental hazards such as mass movements. This study aims to understand its behaviour, determine physical damage thresholds and uncertainties, and provide a practical, effective methodology for calculating physical vulnerability within reach of current practising consultants. The present methodology and fragility curves for URM and CM buildings are intended to contribute to the development by putting engineering, technology, and innovation at the service of the community.

1.1. Literature Review

Many studies regarding buildings' vulnerability against mass movements processes have been based on empirical data from areas impacted by these hazards, e.g. (Barbolini et al., 2004; Bell & Glade, 2004; Cardinali et al., 2002; Fuchs et al., 2007; Guzzetti et al., 1999; Kang & Kim, 2016; Liu et al., 2002; Liu & Lei, 2003; Michael-Leiba et al., 2012; Papathoma-Köhle et al., 2014, 2012; Sterlacchini et al., 2007) which, after a respective event, records the state of the infrastructure (buildings, bridges, and other services). This information allows them to conduct vulnerability and risk studies using qualitative or semi-quantitative methods. To improve the quantification of physical vulnerability and risk, some studies have involved parameters such as the resistance of structural elements, failure mechanisms, the geometry of the structure, interaction with the hazard, deterministic and probabilistic approaches of analysis, etc., taking advantage of relatively recent advances in experimentation and computational numerical analysis. Full development of fragility curves using numerical analysis would involve Monte Carlo simulations requiring expensive computational resources, e.g., several processors unavailable for usual consultants. In addition, little research has been done on the vulnerability of unreinforced clay-brick masonry, a widespread construction type worldwide. A summary of recent advances is presented in, Table 1.

Table 1*References on the Structural Vulnerability of Buildings to Mass Movement*

Reference	Input
(Walhorn et al., 2005)	Developed a finite element algorithm to determine the structural performance of individual elements when impacted by fluid flow, considering parameters related to geometry and flexural capacity only. This research does not include permanent ground deformation, i.e., flexible foundation. It is only valid for rigid foundations.
(Haugen & Kaynia, 2008)	Present a practical method to predict damage to structures impacted by debris flow. Using the concepts of dynamics and structural performance before seismic movements, they used the seismic fragility curves proposed in HAZUS-MH MR4 Technical Manual, 2003. The method was applied to a debris flow event in Sarno (Italy) in 1998. The proposed model evaluated the vulnerability of six impacted structures and compared it with actual damage states. The researchers did not find consistency in some results. They, therefore, suggested that the field information obtained, and the concepts applied should be reviewed and compared with phenomena that have occurred elsewhere. This research does not include permanent ground deformation, i.e., flexible foundation. It is only valid for rigid foundations.
(Mavrouli & Corominas, 2010)	Present an analytical methodology for evaluating vulnerability and calculating fragility curves for buildings in reinforced concrete frames subjected to rock blocks. This research does not include permanent ground deformation, i.e., flexible foundations. It is only valid for rigid foundations.
(Quan Luna et al., 2011)	FLO-2D was used to calculate the intensity of the debris flow event in Selveta (Italy). The intensity, expressed in height, velocity, impact pressure or flow viscosity, is used as an independent variable. The dependent variable is the economic vulnerability of the buildings, obtained from the relationship between the cost of repair and the value of the property. Finally, the vulnerability curves were determined through statistical analysis and compared with those proposed in previous research.
(Papathoma-Köhle et al., 2011)	Raise the limitation of empirical vulnerability functions by restricting their transferability to other building types or places. Furthermore, this technique does not address the damage mechanisms in

structures, restricting the application of the empirical approach in the planning of buildings subject to this type of risk.

(K. H. Hu et al., 2012)

Assert that most studies have focused on empirical relationships between the magnitude of hazard and the degree of loss caused by a recent event. These empirical vulnerability functions do not consider characteristics of the buildings at risk, such as their geometry, design, and geographical location. Using field surveys and remote sensing, the authors describe the characteristics and patterns of building damage in an event in Zhouqu, China. The researchers deduced that structural systems based on reinforced concrete frames suffered less damage than concrete masonry buildings. The authors worked under the configuration of five classes of damage: complete, extensive, moderate, light, and very light.

(Du et al., 2013)

Similar to Li et al., 2010 researchers determined the vulnerability involving parameters such as construction materials, susceptibility of the elements at risk, height and age of the buildings, foundation depth, slip depth and the intensity of the phenomenon to obtain the physical vulnerability of the buildings. However, these referents indicate that some parameters are still subjective and empirical. Therefore, they recommend that these quantifications be calibrated and documented gradually with the addition of objective data, experience, observations, and expert judgment. The authors include the study of the people's vulnerability in the structures.

(Mazzorana et al., 2014)

Propose a methodological and computational model (matrix and finite element methods) of scenario development techniques, considering the interaction of mudflow and structure. Initially, these authors quantify the time-varying impact spectrum based on a two-dimensional debris flow simulation (TRENT 2-D moving bed) and empirical formulations known to date. They then evaluate the structural performance of the building, including the damage mechanisms, using the SISMICAD 12.1 software (<https://www.concrete.it/prodotti/>). The authors consider two damage limit states specified by the Eurocode: the ultimate limit state in which, if exceeded, the building collapses and a serviceability limit state up to which the building can still provide its services. According to the researchers, a third state is the material intrusion limit state, which considers the possible ingress of mud and moisture through access points such as doors and windows. The damage susceptibility profile is the ideal starting point for the design process to provide optimum protection for structures. Does not include permanent ground deformation, i.e., flexible foundations. This research is only applicable to rigid foundations.

- (Mavrouli et al., 2014) Present an analytical quantification for the vulnerability assessment and calculating fragility curves of buildings in reinforced concrete frames exposed to landslides. Researchers base their analysis on the study of single-bay, one-story reinforced concrete frames, considering the variation of different physical and mechanical parameters, including the possibility of having weak, flexible foundations (e.g., isolated footings). However, this procedure does not apply to a structure where the foundation soil does not exhibit ground permanent deformations. This study shows the importance of including the possibility of foundation ground failure when a mass movement impacts a building. According to the authors, it is considered a first step towards expanding to more generic functions. This study highlights that vulnerability studies of structures on unstable or sliding slopes are scarce and that efforts should be made to fill this gap in the future.
- (Eidsvig et al., 2014) Researchers propose two methodologies to measure the buildings' vulnerability due to debris flows. S-shaped cumulative distribution functions are fitted concerning empirical data from the Martell Valley event in South Tyrol, Italy, in August 1987. The first model proposes vulnerability curves, and the second fragility curves take the intensity of the debris flow height into account. Researchers affirm that the methodology is replicable in other places. Nevertheless, it requires statistical analysis of data from properly recorded past events.
- (Kang & Kim, 2016) Analysed 11 mass movements in southern Korea in July and August 2011 due to high rainfall. They characterised the flow, obtaining height, velocity, and impact pressure parameters for 25 buildings. Each building was identified by its damage state (light, moderate, extensive, and complete). This research used field survey data, spatial data, and empirical formulae to analyse the relationship between the building damage state and the mass movement intensity. The authors obtained three different empirical vulnerability curves as a function of mass movement depth, flow velocity and impact pressure. They performed a non-linear regression analysis using an analytical expression to relate vulnerability to debris flow intensity. For the non-linear regression analysis, an S-shaped sigmoid function was used. This function is an asymptote from a small value close to zero to a particular finite value.
- (Papathoma-Köhle et al., 2017) Present, compare, and discuss the advantages and disadvantages of different approaches to assessing physical vulnerability to mass movements, considering matrices, vulnerability curves and so-called vulnerability indices. Present, compare, and discuss the advantages and disadvantages of different approaches to assessing physical vulnerability to mass movements,

	considering matrices, vulnerability curves and so-called vulnerability indices. They suggest that future studies combine the three approaches to have a more complete tool, including built environment resilience, conducting laboratory tests assessing the interaction between the flow and the buildings, and documenting damage to have a better database.
(Parisi & Sabella, 2017)	Carried out a fragility analysis on reinforced concrete buildings, assuming flow velocity as a measure of intensity. They proposed states of damage and different mechanical models for beams, columns, and infill walls by two-dimensional analysis. Affirm that the seismic design of the buildings plays a key role in mitigating the fragility due to mass movements. Two-story, double-bay, two-dimensional reinforced concrete frames are considered subsystems taken out from the overall three-dimensional framed structure of the building. These structures are assumed to be fixed at the foundation level to an immovable and rigid ground (i.e., no soil-foundation-structure interaction effects considered).
(Zhang et al., 2018)	Present a database of information from a catastrophic mass movement in Zhouqu County, Gansu Province, Northwest China, in August 2010 damaged more than 390 buildings. 116 reinforced concrete frames and 274 brick concrete buildings were damaged. Using software such as FLOW 2D, photo interpretation and field visits, they relate the phenomenon's intensity to the building damage. They propose empirical models of physical vulnerability curves to assess building damage. Unfortunately, the database does not include the structures' damage state.
(Sturm et al., 2018a, 2018b)	Developed an experimental physical study on a 1:30 scale simulating a flood event in Schnannerbach (Austria) 2005. They simulated the impact dynamics of the flow in three buildings. The measurement results validated a 3D numerical model (FLOW-3D) with real-time monitoring and multivariable regression. The authors obtained correlations between the intensity of the flow and the stresses generated in the elements.
(Cheung et al., 2018)	Through numerical analysis using the three-dimensional finite element software LS-DYNA (Livermore, 2011), the interaction of mass movement and flexible barriers was investigated using non-linear behaviour. The coupled analysis allowed a detailed study of the energy balance involved in the impact process.

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- (Prieto et al., 2018) Present a methodology that relates hazard intensity to the probability of structural damage, i.e., fragility functions that quantitatively estimate the probability of exceeding a damage state as a function of hazard intensity (debris flows) based on publications such as (Hazu-MH 2.1: Technical Manual, 2012; Hazus Tsunami Model Technical Guidance, 2017; FEMA P-58, 2012). The research presents a complete damage state threshold for URM structures in Medellín, Colombia. The fragility curve was performed in terms of momentum flux. These results were contrasted with field data of Buildings affected by debris flow in events that occurred in July and August 2011 in South Korea (Kang & Kim, 2016). This research does not include permanent ground deformation, i.e. flexible foundation. It is only valid for rigid foundations.
- (Papathoma-Köhle et al., 2019) Incorporate a method for calculating Physical Vulnerability Indices (PVI) for floods in the European Alps. Each characteristic of the physical environment is assessed as a weighted contribution according to experts and field observations. The sum of the indicators (vulnerability to mass movement intrusion, exposure, building characteristics, etc.) with the weighting of each of them results in an expression that calculates the physical vulnerability index. According to the authors, PVIs have the advantage of providing specific information on certain physical environment parameters, thus representing an alternative approach to better assess the physical vulnerability of buildings. Nevertheless, this method does not consider structural damage mechanisms due to interaction with mass movements.
- (H. Y. Luo, Shen, et al., 2019) (a) Reproduce the mass movement on 20 December 2015 in Shenzhen, China, through three-dimensional terrain analysis using an explicit integration software LSDYNA, considering three scenarios: green field, moving buildings (simulating ground sliding) and fixed buildings. The simulations show that the presence of densely located buildings significantly affects the flow pattern, displacement distance, deposition, and energy transfer mechanism. According to the research, the case of mobile buildings is closer to the flow reality as fixed buildings impose more constraints on the moving material, leading to higher deformation, higher internal energy, and lower kinetic energy of the soil mass. The most significant frictional energy dissipation occurs in moving buildings due to damaged buildings' considerable sliding travel distance. The internal energy of the slip mass dominates the energy transfer mechanism. The results presented here indicate the need to consider groups of buildings when performing hazard mapping and risk assessments for mass movements.

- (Feng et al., 2019) Present a numerical method to simulate the flow and its interactions with buildings in three-dimensional terrain considering the destruction process. In this proposed method, the flow is simulated using smoothed particle hydrodynamics, and the buildings are simulated using the finite element method.
- (Fuchs et al., 2019) Recent advances in vulnerability assessment methods for buildings subjected to torrential hazards demonstrate that more research is still needed. Researchers highlighted that despite the numerous studies on buildings' physical vulnerability, a gap exists regarding the interaction between buildings and the event.
- (Kean et al., 2019b) Record and map the 2018 Montecito debris flow event that affected California (USA) buildings. The hazard is quantified regarding height, velocity, and momentum flux between others. Fragility is calculated following the same methodology proposed in Prieto et al. (2018), obtaining light, moderate, extensive, and complete structural damage thresholds for wooden structures. The fragility curves were performed regarding momentum flux and debris flow height. This paper does not include permanent ground deformation, i.e., flexible foundations. It is only valid for rigid foundations.
- (Liang & Xiong, 2019) After considering the uncertainty of the occurrence of the phenomena due to mass flows on bridges in China, researchers determined a typical two-dimensional structural model (transverse frame of two columns in reinforced concrete) and the impact forces of mass movements (located in the plan of the frame). Then, they use the finite element software CSI-bridge (CSI, 2016) to calculate deformations using nonlinear pushover analysis. Based on the evaluation indices and damage exceedance probabilities, the authors quantitatively analysed the structure's vulnerability to debris flow by obtaining fragility curves, which were compared with those of other authors. This research does not include permanent ground deformation, i.e., flexible foundations. It is only valid for rigid foundations.
- (H. Y. Luo, Zhang, et al., 2019) (b)
(H. Y. Luo et al., 2020) Present, as a weak link in the mass movements risk analysis, the evaluation of the response and buildings' vulnerability. This research identifies the failure mechanisms and processes of reinforced concrete frame-type buildings due to flow impacts through an explicit analysis of time integration using the LS-DYNA software. They propose a five-class classification system based on field observations and numerical simulation results to assess building damage states.
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researchers comment that a single parameter, e.g., the impact pressure, is insufficient to indicate the damage status of the building. In this research, they visualise that if the structural and non-structural components survive the initial impact of the landslide, a safer condition would be achieved due to the rapid drop in impact pressure. The authors conclude that the structural capacity against landslides depends on the global lateral resistance of the building. However, they recognise the possibility of generating local failures in structural and non-structural elements, such as columns and walls perpendicular to the flow. It is worth noting that the impact cases in this paper are deterministic analyses. The fragility analysis, which involves the inherent uncertainties on the characteristics of both the mass movements and the building, has not been developed due to computational efficiency issues. Researchers determine that the applied method provides a powerful tool to investigate the key factors in the structural vulnerability analysis, which would be useful to develop fragility functions and vulnerability models based on reliability, considering variations in the properties of the materials and the impact load. The foundation of the building is not considered in the numerical model, and fixed boundary conditions are assigned for the bottom nodes of the columns and walls. This implies that only the impact of mass movement affects the superstructure.

(H. Luo et al., 2020)

It includes the interaction of intensity, structural damage mechanisms and uncertainty of the implicit variables using the OPENSEES software and the Monte Carlo simulation. Fragility curves for reinforced concrete frame buildings are presented. The researchers recognise that the intensity measurement must be chosen based on a specific failure mechanism, i.e., force-dominated or moment-dominated mechanism. Composite factors, e.g., impact force ($h v^2$) or overturning moment ($h^2 v^2$), are measurements used and visible in the presented fragility curves. This research does not include permanent ground deformation, i.e., flexible foundation. It is only valid for rigid foundations.

(Thouret et al., 2020)

Describe the current state of the art in mass movements risk management and affirm that damage state probabilities provide the necessary basis for estimating socioeconomic consequences and can be used with empirical loss data to derive quantitative risk assessments.

(Fang et al., 2021)

Through an explicit analysis of time integration using the LSDYNA software, study the failure mechanisms and processes of a typical reinforced masonry building due to flood impacts. This

	research does not include permanent ground deformation, i.e. flexible foundation. It is only valid for rigid foundations.
(M. Chen et al., 2021)	Used the FLO-2D program to reproduce the mass movements event in Cutou Creek, Wenchuan, Southwest China, on August 20, 2019. They found that the event's intensity in terms of momentum flow ($h v^2$) is a realistic indicator of the hydrodynamic forces that cause buildings to lateral shift and damage other assets. In parallel, researchers built a database of affected buildings in the area, determining the degree of damage to each building. Finally, the authors propose physical fragility curves based on (Eidsvig et al., 2014) for unreinforced masonry and wood buildings, providing a scientific basis for risk management and mitigation strategies.
(H. Y. Luo et al., 2023)	Present the current state of the art in studying vulnerability and risk due to mass movements, including recent advances in multi-hazard vulnerability assessment. Vulnerability assessment in a multi-hazard context remains an emerging research topic.
(X. Huang et al., 2023)	This paper presents a sensitivity analysis of the peak impact forces on a building considering the built environment parameters, such as the orientation, opening scale of the target building, and azimuthal angle and distance of surrounding buildings. The impact forces were obtained from the monophasic viscous debris flow with a synthetic and simplified hydrograph using the FLOW-3D model, a computational fluid dynamics approach, verified through the physical modelling results.
(Capparelli et al., 2023)	Researchers calibrate a model using numerical methods (using ABACUS) and semi-quantitative methods (giving weights and scores to the flooding, to the physical and mechanical properties of URM-type buildings, through probabilistic analyses using the Monte Carlo technique), to then obtain hazard and vulnerability index used to determine flood damage levels. The overall stability of URM buildings in the proposed model depends on the wall's out-of-plane structural behaviour. The structural capacity of the walls in its plane is not considered. They adopt the collapse of the building when the first panel or wall collapses. This research does not include permanent ground deformation.
(M. Chen et al., 2023)	Apply a model for different return periods to calculate the risk due to debris flows. The hazard is calculated using the FLO-2D program, and the vulnerability with adjusted empirical fragility curves using data from a specific event (intensity and damage to buildings), according to Eidsvig

et al., 2014. In calculating risk, they consider weighting factors related to structural damage and economic losses. The researchers analyse two types of buildings (brick concrete structures and masonry wood structures) but do not conduct a mechanical and physical analysis of them to calculate vulnerability. Instead, they involve damages observed after an actual event. The methodology is simple and helpful in calculating risk, but it is not easily replicable in other latitudes, precisely due to the lack of accurate information on past events.

(Pu et al., 2023)

Conclude that the most used methods for calculating vulnerability to mass movements, Monte Carlo simulation and Latin hypercube sampling, are not the most appropriate. Due to the high computational cost, both in terms of time and money, true dynamic contact models between the hazard and buildings have not been carried out. To overcome this difficulty, researchers propose to work with dynamic contact models based on the generalised F-discrepancy method (GF-discrepancy) to consider the interaction between the mass movements and the structure (J. Chen et al., 2016; J. Yang et al., 2020). In their research, they obtain damage mechanisms, vulnerability, and fragility curves for a specific two-story reinforced concrete frame-type structure. They assume that the foundation does not experience deformations, so full constraints are applied to it.

1.2. Scope and Limitations of the Research

A procedure is proposed for calculating the physical vulnerability and risk against mass movements, considering the possibility of direct impact on structures (rigid/fixed foundations), and possible damage due to permanent ground deformation. The present methodology and fragility curves for URM and CM buildings are intended to contribute to the development of the community by putting science, technology, and innovation at the service of the community.

This research implements numerical solutions for masonry structures (URM and CM buildings), initially calculating the structural performance due to mass movements using three-dimensional finite element models that include the interaction of the structure with the impact load and probabilistic simulation techniques such as Rosenblueth's method to obtain median building capacity curves. Still, it is not a dynamic contact model, where the impact force varies with time, as this would require large amounts of computer memory, processing time, and high cost, preventing it from being a reproducible technique in routine consultancy work. It is hoped that in the future, it will be possible to include computational tools that allow this interaction at a low cost and with easy access.

The research was validated with data from South Korea and Colombia's mass movements. Identifying intensity parameters (flow velocity and height), building structural typology, and complete damage thresholds were possible. However, it is recognised that more information on damage state thresholds from events worldwide would be ideal.

The methodology mainly considers the overall structural behaviour of the URM and CM buildings against mass movements. The walls parallel to the flow, with their high in-plane stiffness, provide lateral support to the buildings in case of impact. Due to their low contribution, the analysis has not considered the structural behaviour of walls subjected to out-of-plane loads.

It is recognised that, in principle, local failure damage may occur in façade walls perpendicular to the flow, especially on the first floors. Such damage can be included at the end of the risk calculation for further analysis, depending on the consultant's judgement of which walls would be affected.

The general procedure is helpful for any structure and mass movements. For a specific case, the uncertainty of each variable involved can be modified. However, regarding hazard, particularities, such as the individual rock block impact, may need to be analysed according to the event's kinetic energy. Concerning structural performance, the study focuses on low-rise URM and CM buildings, with damage concentrated on their shear response. Critical damage mechanisms should be analysed to determine damage thresholds and their uncertainty for taller masonry buildings or other structural systems.

The thresholds damage state given, with their uncertainty, are intended to represent informally constructed buildings (URM and CM) worldwide, specifically in underdeveloped countries, which unfortunately do not have good quality standards. Therefore, the mechanical variables of masonry, with their uncertainty, were analysed and collected from various sources, and the experience of suitable professional staff. For other contexts, the thresholds may not be representative. However, it should be noted that the methodology is easily applicable.

1.3. What is New in this Research

As shown above, many works have focused on qualitative methods, and more recently, some have used analytical, numerical methods, including uncertainty, to address the damage mechanisms of buildings interacting with mass movements. While these approaches have made strides in our understanding of structural performance, the absence of a general methodology for practising engineers remains a pressing issue that needs to be addressed. Even the methodologies for calculating vulnerability and risk due to mass movements in many countries (especially for less developed regions) do not associate the buildings' damage state with the phenomenon intensity, which is fundamental.

This research used three-dimensional finite element models, an equivalent non-linear model for masonry walls, and the non-linear progressive failure static analysis method to model URM and CM structures. The point estimations simulation method (Rosenblueth's method) has been incorporated to provide the structural performance of buildings with uncertainty.

The methodology proposed by Prieto et al. (2018) has been improved by not including seismic considerations to obtain the mass movement fragility curves, calculating through a three-dimensional model capacity threshold for different damage states regarding shears strength, including the uncertainty calculation.

Another essential aspect is the study of physical vulnerability due to mass movements, considering permanent ground failures. Most analyses to date have assumed rigid foundations in their models.

Physical damage thresholds and uncertainties for URM and CM buildings due to mass movement are provided for use by the engineering community, research, government agencies, insurance companies, local building regulations, and the overall community. It should be noted

that this type of information is scarce in the world. In Colombia, before the present research, only one data was extracted for complete damage state in URM buildings (Prieto et al., 2018).

This procedure benefits practising engineers and the wider community, as it provides a simple and efficient methodology for mass movement risk assessment, especially in areas where field data is unavailable, sophisticated equipment is not available, and time is pressing.

1.4. Report Outline

The research is configured as follows: 1. Literature review of physical vulnerability due to mass movements. 2. A brief introduction to hazards (mass movements). 3. A quantitative procedure (numerical and analytical) incorporating the variables' uncertainty is presented to assess the physical vulnerability of URM and CM buildings due to mass movements. The method provides fragility curves due to the direct impact of the flow against the structures and fragility curves due to the possibility of permanent ground deformation of the foundation. 4. Presentation of fragility curves for different damage states regarding wall thickness and number of storeys for URM and CM buildings. The phenomenon intensity is considered in terms of momentum flux and height. 5. Model validation with two events by mass movements in South Korea and Colombia. 6. Mass movements risk loss assessment. It includes an exercise applied to a Popayán, Cauca, Colombia neighbourhood. 7. Remarks and Conclusions.

2. Mass Movements Hazard Assessment

Mass movements include all downslope movements of a mass of rock, debris, or soil caused by gravity (D. M. Cruden, 1991). The mixture may be composed of large-diameter materials, such as rocks, gravel, and sand, with specific amounts of fine soils, such as silt and clay, as well as trapped water and/or air. They may also contain organic material, such as the remains of trees and other vegetation, and various materials dragged along in their path. This interpretation includes the fluvial processes that the phenomenon may experience associated with mass transport. Figure 1 shows a mass movement that occurred on November 3, 2019, affecting the population of Jericó, Antioquia, Colombia.

Figure 1

Jericó, Antioquia, Colombia Mass Movement.



Note. (a) Mass movement. 1st Avenue - 6th – 7th Streets. (www.eltiempo.com.co); (b) Mass movement 1st avenue – 6th Street. (www.eluniversal.com.co)

2.1. Mass Movement Classification

The velocity of the mass movement is an important parameter to characterise the phenomenon (D. Cruden & Varnes, 1996). Table 2 compiles the velocity thresholds and the damage caused to the population, infrastructure, and other properties.

Table 2*Mass Movements Velocity Scale, Cruden and Varnes (1996)*

Velocity	Description	Damage
> 5 m/s	Extremely rapid	Destruction of exposed buildings and death of the population due to impact or disintegration of the displaced material.
3m/min - 5 m/s	Very rapid	Loss of some lives due to the speed of movement being too fast to allow the entire population to escape, great destruction, irreparable damage, and evacuation is difficult.
1.8 m/hour - 3 m/min	Rapid	The population can escape and evacuate. The mass movements destroy structures, possessions, and equipment.
13 m/month - 1.8 m/hour	Moderate	Structures located in the displaced mass will be heavily damaged.
1.6 m/year - 13 m/month	Slow	Roads and some structures can be maintained through routine maintenance work. However, the damage is frequent, and repair is costly.
0.016 m/year - 1.6 m/year	Very slow	Some permanent structures are not damaged; if they have cracks or fissures, they can be repaired. Damage is rare and repairable.
< 0.016 m/year	Too slow	No damage to structures built to engineering standards.

The following is a classification of mass movements depending on the flow type. The magnitude of the velocity can be referenced in Table 2.

2.1.1. Falls

A fall begins with the detachment of soil, rock, or both from a steep slope along a surface where little or no shear displacement has occurred. The material subsequently descends mainly by falling, bouncing, or rolling (Highland & Bobrowsky, 2008).

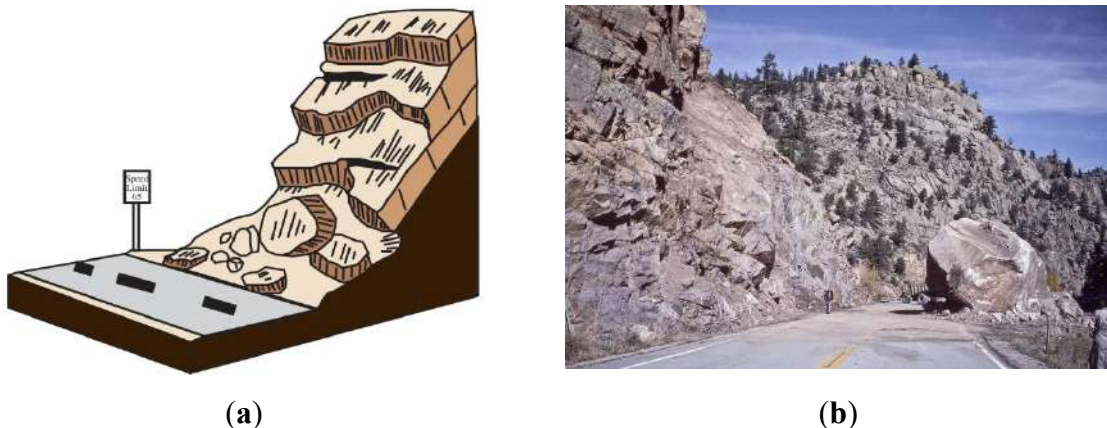
2.1.1.1. Rockfalls

Falls are abrupt, downward movements of rock, earth, or both that detach from steep slopes or cliffs. The falling material usually strikes the lower slope at angles less than the angle of fall,

causing bouncing. The falling mass may break on impact, begin rolling on steeper slopes, and continue until the terrain flattens, Figure 2. The rolling velocity is very rapid to extremely rapid and depends on slope steepness. (Highland & Bobrowsky, 2008).

Figure 2

(a) *Schematic of a Rock Fall*; (b) *Colorado Geological Survey. (2023, June 12). Rockfall - Colorado Geological Survey. <https://coloradogeologicalsurvey.org/hazards/rockfall/>*



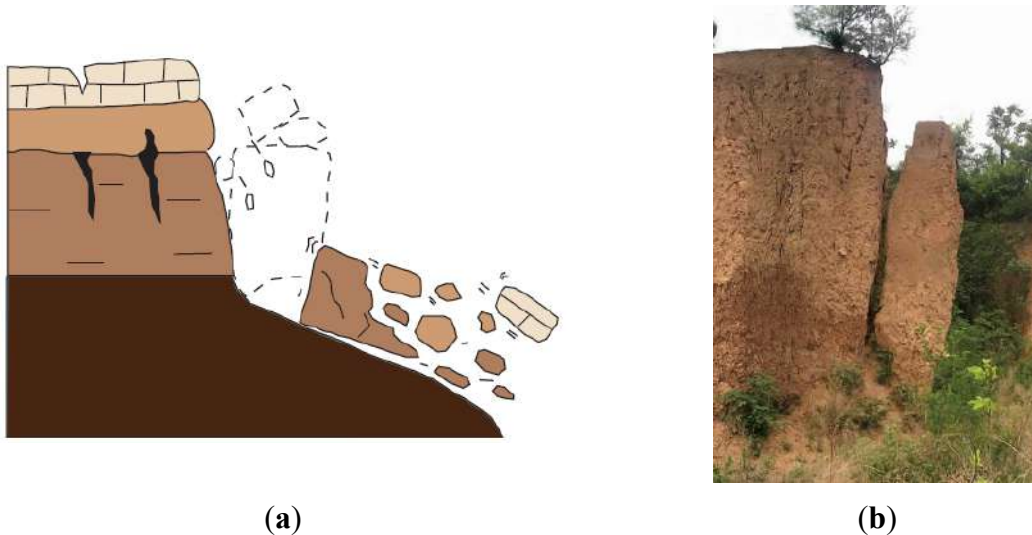
Note. (a) Adapted from the Landslide Handbook - A Guide to Understanding Landslides (p. 7), Highland, Lynn M., 2008, U.S. Geological Survey Circular. (b) Sourced from Colorado Geological Survey. (2023, June 12). Rockfall - Colorado Geological Survey. <https://coloradogeologicalsurvey.org/hazards/rockfall/>.

2.1.1.2. Topple

It is the forward rotation out of a slope of a mass of soil or rock around a point or axis below the centre of gravity of the displaced mass. Toppling is sometimes driven by gravity exerted by the weight of material upslope from the displaced mass. Sometimes, toppling is due to water or ice in cracks in the mass. Topples can consist of rock, debris (coarse material), or earth materials (fine-grained material), Figure 3. The travel velocity is extremely slow to extremely rapid, sometimes accelerating throughout the movement, depending on the travel distance.

Figure 3

(a) Schematic of a Topple; (b) Colorado Geological Survey. (2023, June 12). Rockfall - Colorado Geological Survey. <https://coloradogeologicalsurvey.org/hazards/rockfall/>



Note. (a) Adapted from. *The Landslide Handbook - A Guide to Understanding Landslides* (p. 9), Highland, Lynn M., 2008, U.S. Geological Survey Circular. (b) Sourced from. *A unified landslide classification system for loess slopes: A critical review* (Y. Li & Mo, 2019, p. 75).

2.1.2. Spreads

An extension of a cohesive soil or rock mass combined with the general subsidence of the fractured mass of cohesive material into softer underlying material. Spreads may result from the softer underlying material's liquefaction or flow (and extrusion). Types of spreads include block, liquefaction, and lateral spreads (Highland & Bobrowsky, 2008).

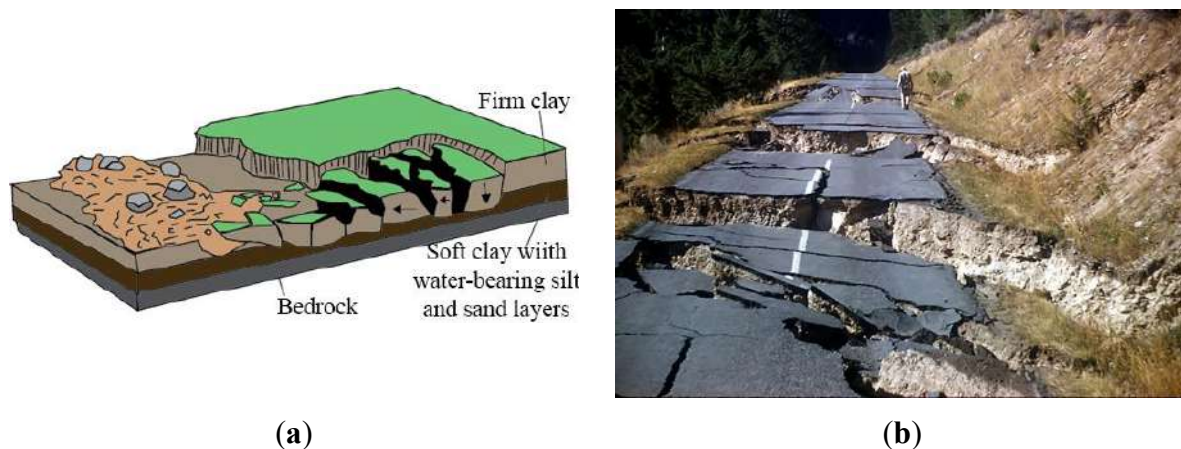
2.1.2.1. Lateral Spreading

Lateral spreads usually occur on very gentle slopes or essentially flat terrain, especially where a stronger upper layer of rock or soil undergoes extension and moves above an underlying softer, weaker layer. Such failures commonly accompany general subsidence into the weaker underlying unit, Figure 4. In rock spreads, solid ground extends and fractures, pulling away slowly from the stable ground and moving over the weaker layer without necessarily forming a

recognisable rupture surface. Under certain conditions, the softer, weaker unit may squeeze upward into fractures that divide the extending layer into blocks. In earth spreads, the upper stable layer extends along a weaker underlying unit that has flowed following liquefaction or plastic deformation. If the weaker unit is relatively thick, the overriding fractured blocks may subside into it, translate, rotate, disintegrate, liquefy, or even Flow. The travel velocity may be slow to moderate and sometimes rapid after specific triggering mechanisms, such as an earthquake. Ground may slowly spread from millimetres to tens of square meters daily (Highland & Bobrowsky, 2008).

Figure 4

(a) *Schematic of a Lateral Spreads; (b) Colorado Geological Survey. (2023, June 12). Rockfall - Colorado Geological Survey. <https://coloradogeologicalsurvey.org/hazards/rockfall/>*



Note. (a) Adapted from the Landslide Handbook - A Guide to Understanding Landslides (p. 15), Highland, Lynn M., 2008, U.S. Geological Survey Circular. (b) Extracted from. Evaluation of Current Empirical Methods for Predicting Lateral Spread-Induced Ground Deformations for Large Magnitude Earthquakes Using Maule Chile 2010 Case Histories (Tryon, 2014, p. 3).

2.1.3. Landslides

Landslides are downslope movements of a mass of soil, debris, or rock which occur over a recognisable rupture surface; in other words, gravitational and other types of shear stresses within a slope exceed the shear strength of the materials that form the slope.

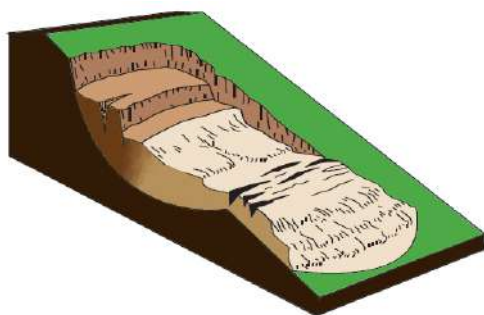
The two major types of landslides are rotational and translational landslides (Highland & Bobrowsky, 2008).

2.1.3.1. Rotational Landslides

This is a landslide in which the surface of rupture is curved concavely upward, and the slide movement is roughly rotational about an axis that is parallel to the ground surface and transverse across the slide, Figure 5 (a). Movement may initially be slow, but many are moderate in velocity to rapid. A landslide has occurred in the municipality of Rosas, Cauca, Colombia, which could cut off the south of Colombia and South America and cause local devastation, Figure 5 (b).

Figure 5

(a) Schematic of a Rotational Landslide; (b) Rotational Landslide Panamerican Highway (Rosas Cauca, Colombia).



(a)



(b)

Note. (a) Adapted from the Landslide Handbook - A Guide to Understanding Landslides (p. 11), Highland, Lynn M., 2008, U.S. Geological Survey Circular.

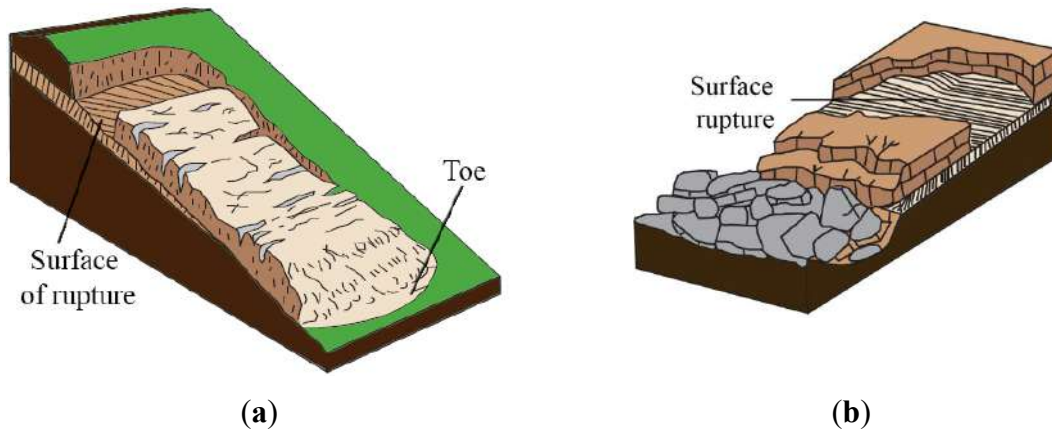
2.1.3.2. Translational Landslides

In this type of landslide, the landslide mass moves along a roughly planar surface with slight rotation or backwards tilting, Figure 6 (a). A block slide is a translational landslide in which the moving mass consists of a single unit or a few closely related units that move downslope as a relatively coherent mass, Figure 6 (b).

Movement may initially be slow, but many are moderate in velocity to extremely rapid. With increased velocity, the landslide mass of translational failures may disintegrate and develop into a debris flow.

Figure 6

(a) Schematic of a Translational Landslide; (b) Schematic of a Block Slide.



Note. Adapted from the Landslide Handbook - A Guide to Understanding Landslides (p. 13), Highland, Lynn M., 2008, U.S. Geological Survey Circular.

2.1.4. Flows

A flow is a spatially continuous movement in which the shear surfaces are short-lived, closely spaced, and usually not preserved. The component velocities in the displacing mass of a flow resemble those in a viscous liquid. Often, there is a gradation of change from landslides to flows, depending on the water content, mobility, and movement evolution (Highland & Bobrowsky, 2008).

2.1.4.1. Earthflow

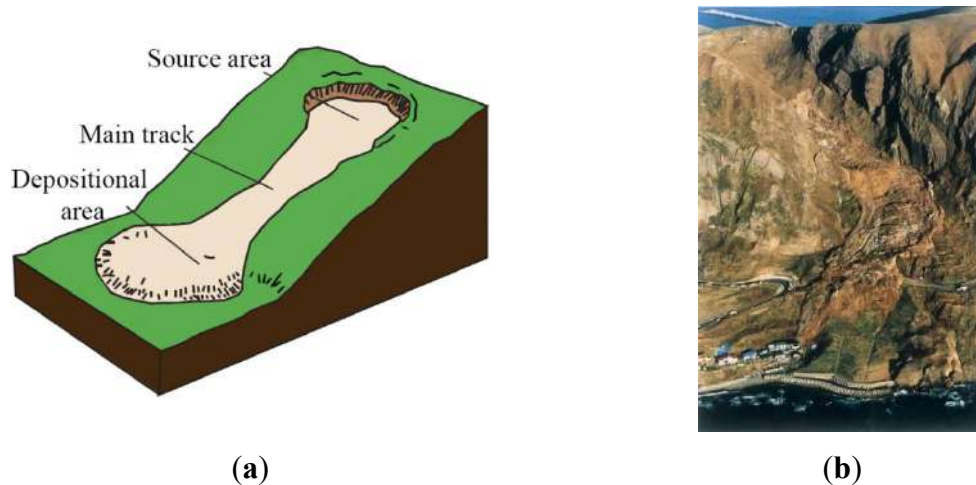
It is a movement that can be rapid or slow and intermittent in plastic clay or silt soils; the flow is rigid without shear deformation, and the water content is close to the plastic limit.

Landslides or lateral spreads may also evolve downslope into earth flows. (D. Cruden &

Varnes, 1996; F. Morassutti, 2016; Highland & Bobrowsky, 2008). (U.S. Geological Survey, 2004).

Figure 7

(a) Schematic of an Earthflow; (b) Motochi Earth Flow, October 11, 1994.



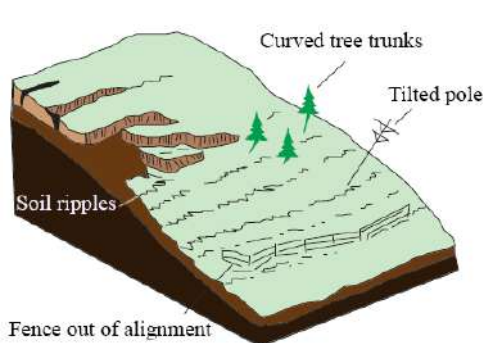
*Note. (a) Adapted from *Landslide Types and Processes* (p. 3), U.S. Geological Survey, 2004, USGS (<https://pubs.usgs.gov/fs/2004/3072/>). (b) Sourced from (Nakamura, 1996).*

2.1.4.2. Slow Earthflow (Creep)

Creep is the informal name for a slow earthflow and consists of the imperceptibly slow, steady downward movement of slope-forming soil or rock. Movement is caused by internal shear stress sufficient to cause deformation but insufficient to cause failure. Generally, the three types of creep are: (1) seasonal, where movement is within the depth of soil affected by seasonal changes in soil moisture and temperature; (2) continuous, where shear stress continuously exceeds the strength of the material, and (3) progressive, where slopes are reaching the point of failure for other types of mass movements. The velocity of travel is Very slow to extremely slow. Usually less than 1 meter per decade.

Figure 8

(a) Schematic of an Earthflow (Creep); (b) Slow Earthflow (Creep) and its Effect on Trees and Forests.

**(a)****(b)**

Note. (a) Adapted from *Landslide Types and Processes* (p. 3), U.S. Geological Survey, 2004, USGS (<https://pubs.usgs.gov/fs/2004/3072/>). (b) A Photo by NOAA/NGDC/Colorado Geological Survey, Public Domain <http://www.ngdc.noaa.gov/hazardimages/picture/show/1551>. (Sassa et al., 2017), p. 31.

2.1.4.3. Peat Flow

Peat Flow is the slow to very rapid movement of saturated peat under very high interstitial pressures (D. Cruden & Varnes, 1996; F. Morassutti, 2016).

Figure 9

Peat Flow Creag Mhor, UK, 24 March 2007.



Note. Sourced: Bill Copland / A peat-flow on the north side of Creag Mhor. / CC BY-SA 2.0. A peat flow on the north side of Creag Mhor, UK, after a sustained spell of heavy rain or snowmelt.

2.1.4.4. Mudflow

Mudflow is a very rapid to extremely fast flow of plastic-saturated debris within a natural channel where the moisture content in the parent material is significant (plasticity index greater than 5%) (D. Cruden & Varnes, 1996; F. Morassutti, 2016).

Figure 10

(a) Schematic of a Mudflow; (b) Mudflow Event in San Vicente, Paez, Cauca, Colombia, date 27 07 2022.



(a)



(b)

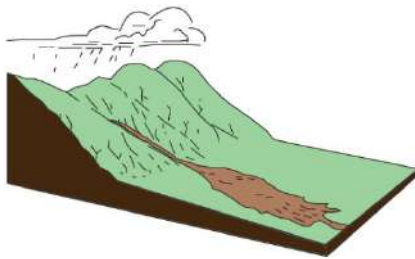
Note. (a) Adapted from *Landslide Types and Processes* (p. 3), U.S. Geological Survey, 2004, USGS (<https://pubs.usgs.gov/fs/2004/3072/>). (b) Sourced from Disaster Risk Management Advisory Office - OAGRD Cauca.

2.1.4.5. Debris Flow

Debris flow is a rapid to extremely rapid flow of saturated, non-plastic debris in a steeply sloping natural channel, Figure 11. The plasticity index is less than 5% in the sizes corresponding to the sands and finer fractions (D. Cruden & Varnes, 1996; F. Morassutti, 2016). Debris flowing downslope may comprise loose soil, rocks, and sometimes organic matter combined with water. Occasionally, as a rotational or translational landslide gains velocity and the internal mass loses cohesion or gains water, it may evolve into a debris flow. Dry flows can sometimes occur in cohesionless sand (sand flows). It can reach speeds such as 56 km/h depending on the consistency and angle of inclination (Highland & Bobrowsky, 2008).

Figure 11

(a) Debris Flows Schematic; (b) Debris Flow in Jerico, Antioquia (Colombia), 2019.

**(a)****(b)**

Note. (a) Adapted from the *Landslide Handbook - A Guide to Understanding Landslides* (p. 17), Highland, Lynn M., 2008, U.S. Geological Survey Circular, (b) Sourced www.eluniversal.com.co.

2.1.4.6. Lahars (Volcanic Debris Flows).

The word “lahar” is an Indonesian term. Lahars are also known as volcanic mudflows. These flows originate on the slopes of volcanoes and are a debris flow type. A lahar mobilises the loose accumulations of the airborne solids that erupted from the volcano and related debris. Lahars can be very rapid (more than 50 kilometres per hour), especially if they mix with a water source such as melting snowfields or glaciers. If they are viscous and thick with debris and less water, the movement will be slow to moderately slow (Highland & Bobrowsky, 2008).

Figure 12

(a) Lahars Schematic; (b) Armero City Destruction by the Eruption of the Nevado del Ruiz Volcano (Colombia), 1985.

**(a)****(b)**

Note. (a) Adapted from (Marshak, 2011), Chapter 16, (b) Sourced from armeroguayabal-tolima.gov.co, (Duque Escobar, 2015).

2.1.4.7. Debris Flood

A debris flood is a very rapid flow of water in waves in a natural sloping channel, with a large load of debris (D. Cruden & Varnes, 1996; F. Morassutti, 2016).

Figure 13

(a) Debris Flood, June 20, 2013, in Canmore, Alberta, Canada; (b) Houses Damaged Along the Edge of Cougar Creek are Shown June 20, 2013, in Canmore, Alberta, Canada.



(a)



(b)

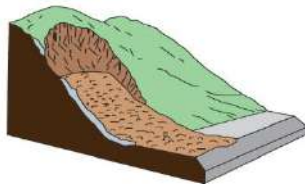
Note. (a) Adapted from (Church & Jakob, 2020), (b) Sourced from John Gibson/Getty Images. <https://globalnews.ca/news/1338351/15-compelling-images-of-canmore-in-the-2013-flood/>.

2.1.4.8. Debris Avalanche

A debris avalanche is a very rapid to extremely rapid surface flow of fully or partially saturated debris over an inclined slope without the confinement of an established natural channel. This flow can reach speeds up to 100 m/s (D. Cruden & Varnes, 1996; F. Morassutti, 2016; Highland & Bobrowsky, 2008).

Figure 14

(a) Debris Avalanche Schematic; (b) Debris Avalanche Chile South America in 2006.



(a)



(b)

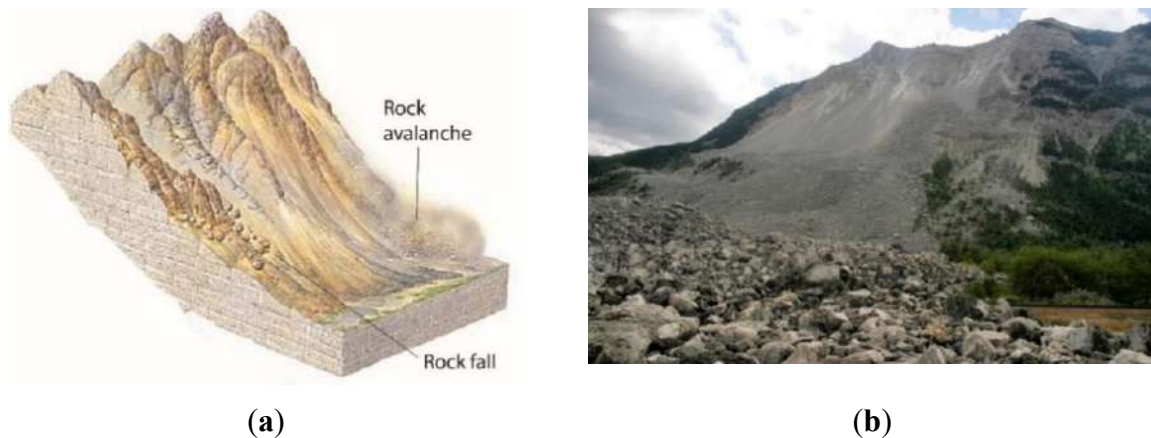
Note. (a) Adapted from the Landslide Handbook - A Guide to Understanding Landslides (p. 21), Highland, Lynn M., 2008, U.S. Geological Survey Circular, (b). Debris avalanche Las Amarillas is next to the Baños Morales in the San José de Maipo, Chile, Metropolitan Region (Photograph by Martin Mergili).

2.1.4.9. Rock Avalanche

A rock avalanche is an extremely rapid flow-like movement of a mass of fragmented rock from large landslides or rock falls. These avalanches can reach speeds of up to 25 m/s and spread over several kilometres in length (D. Cruden & Varnes, 1996; F. Morassutti, 2016; Highland & Bobrowsky, 2008).

Figure 15

(a) Rock Avalanche Schematic; (b) Rock Avalanche, Frank, Alberta, Canada in 2007.



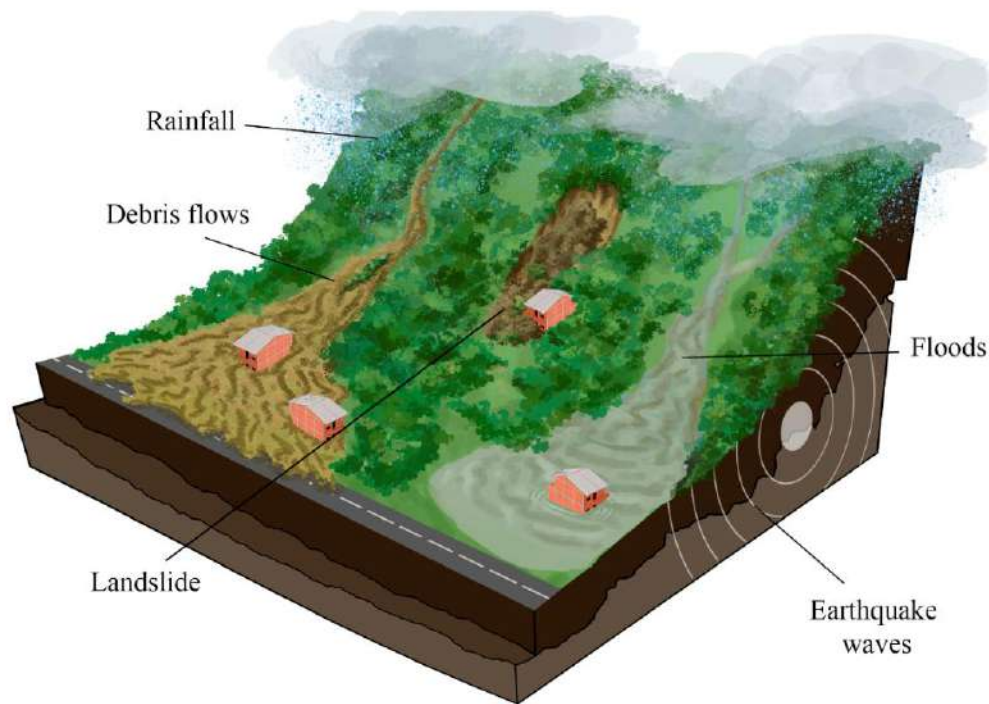
Note. (a) Adapted from (Marshak, 2011), Chapter 16, (b) Sourced from Ślusarczyk 2007.

2.2. Mass Movements Triggers

The three triggers of the most damaging mass movements around the world are as follows (U.S. Geological Survey, 2004):

Figure 16

Description of the Mass Movement Risk Including Triggers.



Note: Floods are not mass movements but can significantly increase the quantity of solids (sediments) along the path and become part of them.

2.2.1. Rainfall

Ground saturation by water is a primary cause of mass movements. This effect can occur in intense rainfall, snowmelt, groundwater levels, and water-level changes along coastlines, earth dams, and the banks of lakes, reservoirs, canals, and rivers.

2.2.2. Seismic Activity

Many mountainous areas vulnerable to mass movements have also experienced at least moderate rates of earthquake occurrence in recorded times. The occurrence of earthquakes in steep mass movement-prone areas dramatically increases the likelihood that mass movements will occur due to ground shaking alone or shaking-caused dilation of soil materials, allowing rapid water infiltration.

2.2.3. *Volcanic Activity*

Mass movements due to volcanic activity are some of the most devastating types. Volcanic lava may melt snow rapidly, causing a deluge of rock, soil, ash, and water that accelerates rapidly on the steep slopes of volcanoes, devastating anything in its path. These volcanic flows (also known as lahars) reach great distances once they leave the volcano's flanks and can damage structures in flat areas surrounding the volcanoes.

2.3. Fluid Models

2.3.1. *Newtonian Fluid (Manning Model)*

In Newtonian fluids, such as water and most gases, the unit strain is proportional to the shear stresses, showing a linear relationship where the slope of the straight line is the viscosity. The hydraulic representation of the water flow, according to the flow Newtonian characteristics, is done using the Manning model. This model is the closest to reality for mass movements with high water content and hyper-concentrated flows with viscous and turbulent Newtonian characteristics. A hyper-concentrated flow is considered when the volume concentration of solids in the flow is greater than 0.2 but does not exceed 0.6, a value that corresponds to a specific weight of approximately 2.0 ton/m^3 (Suarez, 2001). When inertial forces predominate, the flow is turbulent. As the velocity gradient increases, the friction between particles within the fluid increases, and the particles acquire appreciable rotational energy; the viscosity loses its effect, and due to the rotation, the particles change trajectory. As the particles move from one trajectory to another, they collide with each other and change course erratically (F. Morassutti, 2016).

2.3.2. *Non-Newtonian Fluid*

2.3.2.1. Bingham Plastic Fluid. An essential class of non-Newtonian fluids is the viscoplastic fluid. Shear stresses must exceed a critical value known as yield strength for the fluid to flow. For example, when we open a toothpaste, sufficient force must be applied to cause the toothpaste to flow. One of the simple viscoplastic models is the Bingham Plastic Model (F. Morassutti, 2016).

2.3.2.2. Pseudoplastic and Dilating Fluid. In the case of pseudoplastic fluids, there is less resistance to flow for higher stresses, i.e., the fluid drains more efficiently when the stress is higher (e.g., blood, mud, etc.). The dilatant fluids have a higher resistance to flow for higher stresses (F. Morassutti, 2016). For non-viscous debris flows where the collision effect between particles predominates, the Bingham fluid or Newtonian fluid conditions are not met, and the dilatant fluid model developed by Takahashi, (1981) is used.

However, it is worth noting the dynamism of these types of phenomena where, e.g., a debris flow can become a hyper-concentrated flow as the concentration of solid particles decreases, or a hyper-concentrated flow can become a debris flow as the concentration increases (Sanchez Morles, 2009).

2.4. Calculation of the Hazard Due to Mass Movements

2.4.1. *A Theoretical Framework for the Mass Movements Velocity Calculation*

For decades several professionals, due to the socio-environmental and economic problems caused by mass movements, have been concerned with understanding and mitigating the phenomenon, so researchers such as (K. Hu et al., 2013; Hungr et al., 1984; Prochaska et al., 2008; Rickenmann, 1999; Wang et al., 2018; H. Yang et al., 2014) have published several empirical formulations according to correlations with different mass movement phenomena that occurred in

specific places. In these investigations, the study of parameters such as the potential volume of debris, mean flow velocity, maximum discharges, and distance or path of the flow gained significant interest.

Mass movements can carry different materials; therefore, determining the appropriate roughness coefficients is difficult. It is essential to understand that the friction between the different materials that make up the mass movement and the supporting surface causes the energy to dissipate faster compared to the circulation of water flows (Suárez, 2001).

To calculate the stresses acting on any surface, it is important to deduce the mass movements velocity at the instant of collision. Taking into account the type of flow and its dynamic behaviour along the path, applicable experimental deductions have been made to date in analytical and numerical methodologies (Gong et al., 2020; Prochaska, Santi, Higgins, Prochaska, et al., 2008; Ramos Cañón et al., 2021; Suárez, 2001; H. Yang et al., 2014), Table 3.

Table 3*Equations for Estimating the Mass Movements Mean Velocity.*

Reference	Eq.	
Newtonian laminar flow	$\vec{V} = \frac{\rho g h^2 S}{k\mu}$	(1)
Dilatant grain shearing / Nonplastic grainfluid dispersions Hungr's Eq. (Hungr et al., 1984; Lo, 2000)	$\vec{V} = \frac{2}{3} \xi h^{\frac{3}{2}} S^{\frac{1}{2}} \quad \xi = 2.17 m^{-1/2} s^{-1}$	(2)
Newtonian turbulent flow: Manning – Strickler Eq.	$\vec{V} = \frac{1}{\eta} h^{2/3} S^{1/2}$	(3)
Newtonian turbulent flow: Yang's Eq. (Cui et al., 2013; H. Yang et al., 2014)	$\vec{V} = \frac{1}{\eta_1} h^{2/3} S^{1/2} \quad \eta_1 = 0.054 h^{0.42}$	(4)
Newtonian turbulent flow: Du's Eq. (Suárez, 2001)	$\vec{V} = \frac{1}{\eta_2} h^{2/3} S^{1/2} \quad \eta_2 = 0.035 h^{0.34}$	(5)
Newtonian turbulent flow: Chézy Eq. (Rickenmann, 1999)	$\vec{V} = \frac{h^{1/2} S^{1/2}}{C^{1/2}}$	(6)
Empirical Eq. (Rickenmann, 1999)	$\vec{V} = C_1 h^{0.3} S^{0.5}$	(7)
Lo'Eq. (Lo, 2000; J. Peng et al., 2015)	$\vec{V} = k_1 h^{2/3} S^{1/5}$	(8)
Prochaska's Eq. (Prochaska, Santi, Higgins, & Cannon, 2008)	$\vec{V} = 0.35 h^2 S + 5.36$	(9)
Hu's Eq. (K. Hu et al., 2013)	$\vec{V} = 8.94 h^{0.15} b^{0.35} S^{0.5}$	(10)
Yang's Eq. (H. Yang et al., 2014)	$\vec{V} = 6.54 p^{0.39} h^{\frac{1}{2}} S^{\frac{1}{3}}$ 2%<p<14%	(11)
Wang's Eq. (Wang et al., 2018)	$\vec{V} = 7.451 C_v^{0.391} h^{0.5} S^{0.011 \ln(C_v) + 0.054}$ 0.1<Cv<0.9	(12)
SGC 2016 (Ávila et al., 2016; Zhong & Yue, 2014)	$\vec{V} = \sqrt{2h'g \left[1 - \frac{\tan(\phi)}{S} \right]}$	(13)

Where:

k: coefficient that depends on the shape of the channel cross-section. For rectangular channels, it is equal to 3; for trapezoidal channels, 5; and semicircular channels, 8.

k₁: factor related to H, Table 4.

Table 4

Relationship between the Coefficient k_1 and the Mass Movements Depth.

h (m)	<2.5	2.75	3	3.5	4	4.5	5	>5.5
K_1	10	9.5	9	8	7	6	5	4

Note. Taken from Peng et al. (2015).

$\vec{V} \left(\frac{m}{s} \right)$: mean mass movements velocity.

$\rho \left(\frac{kg}{m} \right)$: mass movement density.

S : the bed slope where the stream flows.

$\xi \left(\frac{1}{s * m^{1/2}} \right)$: grouped coefficient dependent on grain size and concentration.

$h(m)$: mass movements depth

$h'(m)$: vertical measurement of the landslide.

η : mass movement's roughness coefficient

η_1 : mass movement's roughness coefficient. Yang's Equation.

η_2 : mass movement's roughness coefficient. Du's Equation.

$C \left(\frac{m^{1/2}}{s} \right)$: Chezy coefficient, Table 5.

$C_1 \left(\frac{m^{0.7}}{s} \right)$: empirical dimensional coefficient.

$p(\%)$: the clay content of sedimentary material.

C_v : sediment volumetric concentration.

Table 5

Chezy Coefficient, for Newtonian or near Newtonian Flows.

Type	C
Water, smooth channel	0.0025
Mudflow	0.002
Pinatubo - mudflow	0.0024
Sediment laden flow	0.005

Pinatubo - hyper-concentrated flow	0.009-0.014
Mangatoetoenui - lahar	0.02 - 0.1
Pinatubo - lahar	0.01 - 0.3
Lahars	0.01 - 1
Debris flow	0.06 - 0.577
Wet landslide	0.184

Note. Taken from Bulmer et al. (2002).

2.4.2. Theoretical Framework for the Calculation of Mass Movements Dynamic

Thrust Against Buildings

Hungr et al. (1984) presents perhaps the most famous hydrodynamic formulation to quantify mass movement hitting against a vertical barrier (Cui et al., 2015). The authors determine that the mass movements dynamic thrust can be calculated with Eq. (14) for barriers almost perpendicular to the flow direction. The hydrodynamic model's dimensionless impact coefficient (k) is 1.50. This value depends on the grain size distribution of the flow (Poudyal et al., 2020; Vagnon & Segalini, 2016).

$$F = k \rho A_c V^2 \sin \beta \quad (14)$$

β : the smallest angle between the barrier and the flow direction.

A_c : area of contact between the mass movement and the structure.

Below are some experimentally developed hydrostatic and hydrodynamic models for calculating the pressure from mass movements against vertical barriers.

Table 6

Hydrodynamic and Hydrostatic Hydraulic Models.

Reference	Equation
(Zhang, 1993)	$P = k \rho V^2 \cos^2(\alpha) \quad c_d = 3 \sim 5 \quad (15)$
(Armanini, 2008)	$P = c_s \rho g h \quad c_s = 9.0 \quad (16)$
(Scotton and Deganutti, 1997)	$P = c_s \rho g h \quad c_s = 2.50 \sim 7.50 \quad (17)$

Note. Mass movement impact (N/m^2). (Cui et al., 2015; K. Hu et al., 2011; Vagnon & Segalini, 2016).

Cs: hydrostatic model dimensionless coefficient.

α : angle between the direction of flow and the direction normal to the barrier or impact plane.

$g \left[\frac{m}{s^2} \right]$: gravitational acceleration.

$\rho \left[\frac{kg}{m^3} \right]$: maximum mass movement density.

A dynamic pressure coefficient (k) equal to 2.5 is adjusted to the mean impact pressure due to mass movement, considering boulders with diameters up to 0.5 meters (Koo, 2018; J. Peng et al., 2015). If larger diameters are present, the effect of the dynamic impact of mass movement and boulders should be calculated separately.

Table 7 shows experimentally developed mixed models for calculating mass movement pressure in vertical barriers. After identifying the strengths and weaknesses of hydrostatic and hydrodynamic models, researchers conclude that mixed models seem to be the best method to predict the pressure exerted by mass movements on a barrier (Gong et al., 2020). They establish that the impact pressure can be calculated using Eq. (21).

Table 7

Mixed Hydraulic Models.

Reference	Equation
(Arattano and Franzi, 2003)	$P = C_m(0.5\rho gh + \rho V^2)$ $C_m = 1.0$ (18)
(Hübl and Holzinger, 2003)	$P = C_m \rho (gh)^{0.6} V^{0.8}$ $C_m = 5.0$ (19)
(Cui et al., 2015)	$P = C_m F_r^{-1.5} \rho V^2$ $C_m = 5.3$ $F_r = \frac{V}{\sqrt{gh}}$ (20)
(Gong, 2020)	$P = 2.07 F_r^{1.64} \rho gh$ $F_r = \frac{\vec{V}}{\sqrt{gh}}$ (21)

Note. Mass movement impact pressure (N/m^2). Taken from Cui et al. (2015); Gong et al. (2020).

2.5. Remarks

This chapter compiles some basic concepts of hazard that are important for vulnerability and risk study due to mass movements.

The term mass movements includes all downslope movements of a mass of rock, debris, or soil because of gravity (D. M. Cruden, 1991). The mixture may be composed of large-diameter materials, such as rocks, gravel, and sand, with specific amounts of fine soils, such as silt and clay, as well as trapped water and/or air. They may also contain organic materials such as the remains of trees, other vegetation and various materials dragged along in their path. This interpretation includes the fluvial processes that the phenomenon may experience associated with mass transport.

A particular type of mass movement is torrential flooding. This phenomenon refers to a rapid flow through permanent or intermittent channels with high longitudinal gradients that can be generated by heavy rainfall. It involves transporting a mixture of water and a significant amount of sediment. The flow generates material deposition along its path when it reaches low-gradient areas. In the above definition, debris flows, mudflows, and hyper-concentrated flows (debris floods and mud floods) are considered to be the flows involved in the torrential flood concept, (Ramos-Cañón et al., 2021).

The velocity and height of the mass movements are fundamental variables in hazard quantification. We consider several experimental equations for velocity calculation. Their use depends on the nature of the phenomenon, e.g. the quantification of velocity for a debris flow is different from that for a landslide. Calculating the landslides' velocity can be done using Eq. (13). For other types of mass movements, it is possible to use the other equations given in Table 3, depending on the flow dynamics, sediment concentration and water quantity.

Hydrodynamic and hydrostatic models are considered for the forces' calculation acting on buildings due to mass movements. These are used in the following chapter. The hydrodynamic model is essential when the kinetic component is significant. The hydrostatic model may be relevant when the flow loses velocity and turbulence and the gravitational potential component increases.

Software is available to help define the hazard by simulating flows according to their characteristics, providing parameters such as height and velocity of mass movement along their path. These include FLOW 2D, FLOW 3D, HEC-RAS, etc.

Some studies focus on the susceptibility assessment and early warning of precipitation-induced mass movements: for example, using high-resolution digital elevation models and some soil parameters to predict ground behaviour in tropical and mountainous regions (SHALSTAB model), (Aristizábal et al., 2015) or more recently, satellite-based precipitation estimation and machine learning, especially in areas without ground-based rain gauge networks to correlate precipitation and mass movements, (Gómez et al., 2023).

Within the framework of the Disaster Risk Management Policy, the process of risk awareness is considered, which consists of the identification, analysis, monitoring and communication of risks to support the design and implementation of early warning systems, whose main objective is to preserve life in the event of a potentially catastrophic event. With the tragedy that occurred in Mocoa, Putumayo, Colombia, on 31 March 2017 due to mass movements, a disaster situation was declared. As a result, through the National Unit for Disaster Risk Management, the national government initiated a series of actions, including the knowledge base development to determine thresholds for early warning systems in micro-basins (Ramos-Cañón et al., 2023). Researchers emphasised the need for the early warning system to be redundant, i.e., to

work in terms of different physical variables, given the high level of uncertainty in the modelling processes (epistemic uncertainty) and the background information (random uncertainty) that feeds the models.

Building vulnerability to mass movements, expressed in quantitative terms and as a hazard intensity function, is an objective measure to be introduced in cost/benefit analyses to assess the efficiency of protection measures and risk reduction (Mavrouli, 2017).

3. Mass Movements Physical Vulnerability Assessment

Vulnerability can be defined as the degree of physical, social, or economic loss of an element or set of elements at risk due to a hazard for a given intensity (EERI Committee, 1986). Below are descriptions of social and physical vulnerability. In this case, economic vulnerability is understood as part of social vulnerability.

3.1. A Brief Introduction to Vulnerability

3.1.1. Social Vulnerability

The social vulnerability can be regarded as the partial outcome of social inequalities (Cutter et al., 2003). It measures the responsiveness of a social group to an environmental hazard, e.g., mass movement (Sung & Liaw, 2020). Social factors include gender, ethnicity, age, socio-economic status, disability, education, access to technology, government policies, etc. Usually, people and groups that are socioeconomically disadvantaged, such as the poor and immigrants, are often consigned to more vulnerable locations (Uitto, 1998).

This research does not include this topic. Nevertheless, it is crucial to recognise that social vulnerability studies are a prerequisite for assessing vulnerability to environmental hazards (Sung & Liaw, 2020). Social vulnerability to natural hazards increases in underdeveloped countries. Therefore, governmental, social, academic, and research efforts should be deepened to formulate risk reduction plans.

3.1.2. Physical Vulnerability

Physical vulnerability describes the ability of the infrastructure, e.g., (buildings, roads, bridges, etc.) to withstand impacts. This can be expressed as a function of the monetary value of physical assets in the hazard zone (Woodruff et al., 2017). The aim here is to contribute to

increasing physical vulnerability knowledge against mass movement. Two main functions have been used to describe this phenomenon: vulnerability curves and fragility curves.

3.1.2.1. Vulnerability Curves

These functions have been used in regions with data on an event's economic or social consequences. However, there are not enough events with reliable data. Vulnerability curves are challenging to apply retrospectively, even to the same site, as the essential characteristics of the system can vary considerably (Sanchez Silva, 2010). They have been used in the study of mass movement vulnerability, generally by contrasting the intensity of an event with the expected economic losses. These studies do not deal with the structural performance of buildings. Therefore, they do not predict the structural capacity and uncertainty and could not be used for integrated multi-hazard risk analysis (earthquake, tsunami, Natech, etc.). An important alternative to derive these loss functions is expert opinion. Qualified persons generally make a value judgment based on experience and information gathered. Nevertheless, it can be highly subjective.

3.1.2.2. Fragility Curves

Fragility curves provide the probability of reaching or exceeding a damage state given intensity level. Frequently, they are modelled as a cumulative lognormal distribution (Hazu-MH 2.1: Technical Manual, 2012). These functions, therefore, are a natural measure of a building's resistance to natural hazards. By their nature, they automatically provide the probability, i.e., the uncertainty of the damage. Fragility curves are ideal for multi-hazard risk analysis. These functions are crucial to assessing the physical risk from various phenomena, including mass movements.

Damage is the unwanted physical effect of an external event interacting with the infrastructure, and loss is the social or economic quantification of damage. Building damage generally occurs as

a continuum, increasing as the intensity of the hazard increases. However, for performance assessment, the damage is characterised as a series of discrete states called damage states (FEMA P-58, 2012). The damage state threshold is the median value of the building's structural condition for a damage state (Hazus–MH 2.1: Technical Manual, 2012).

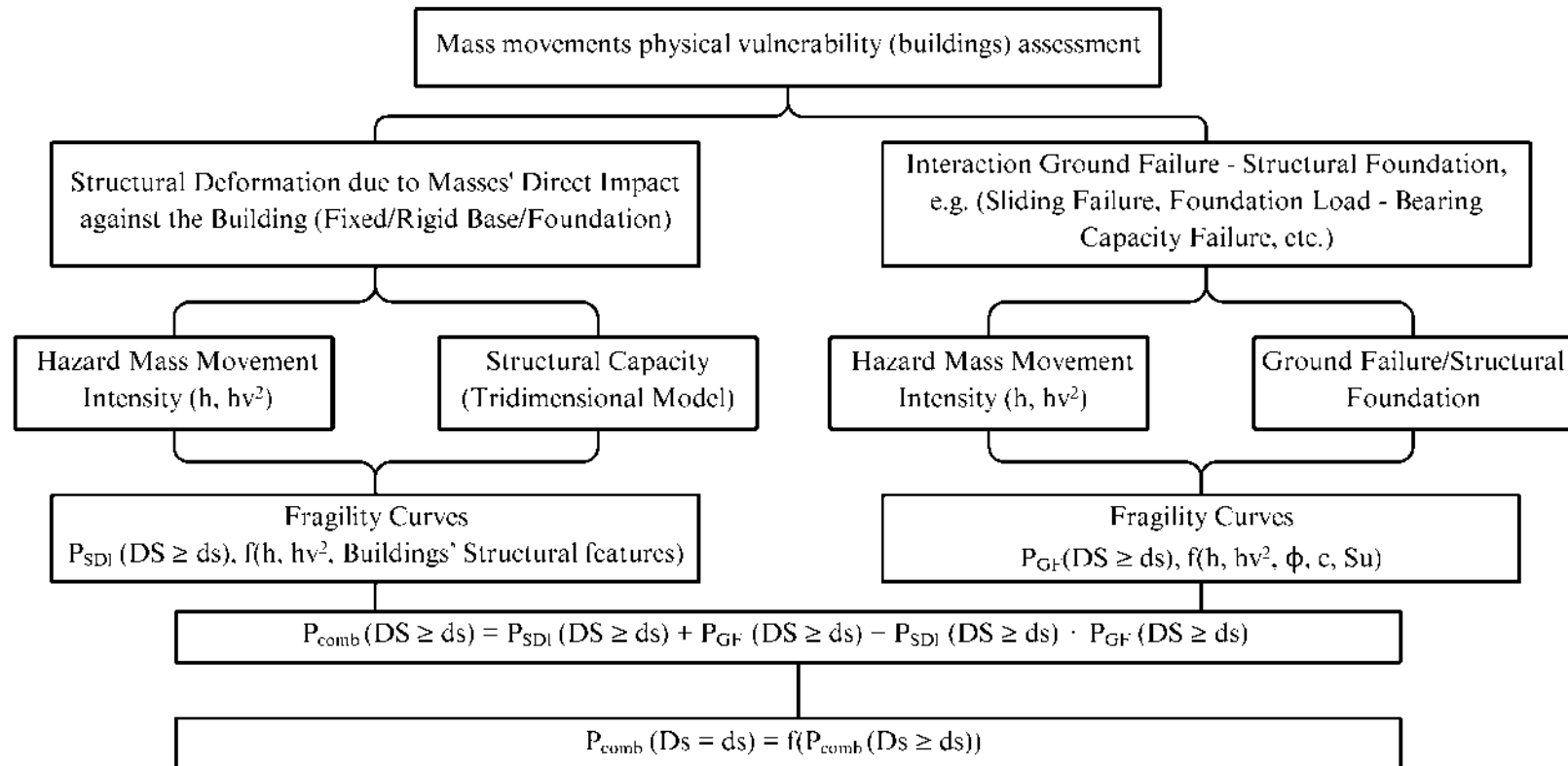
3.2. Buildings' Vulnerability

Buildings' vulnerability against mass movement events emerges as the direct or indirect result of two phenomena: the direct impact of masses against the building, which causes structural deformations, and permanent ground deformations/ground failure, which can affect the foundation. The first case can be treated as a fixed/rigid base/foundation structural problem, while the second implies different ground/foundation failure modes. Figure 17 describes a general procedure to evaluate buildings' vulnerability, including these two effects.

Section 3.3.1 considers the probability of damage due to the direct impact of masses against the building, and section 3.3.2 studies the probability of damage due to permanent ground deformation.

Figure 17

Building Vulnerability Assessment Methodology Due to Mass Movements.



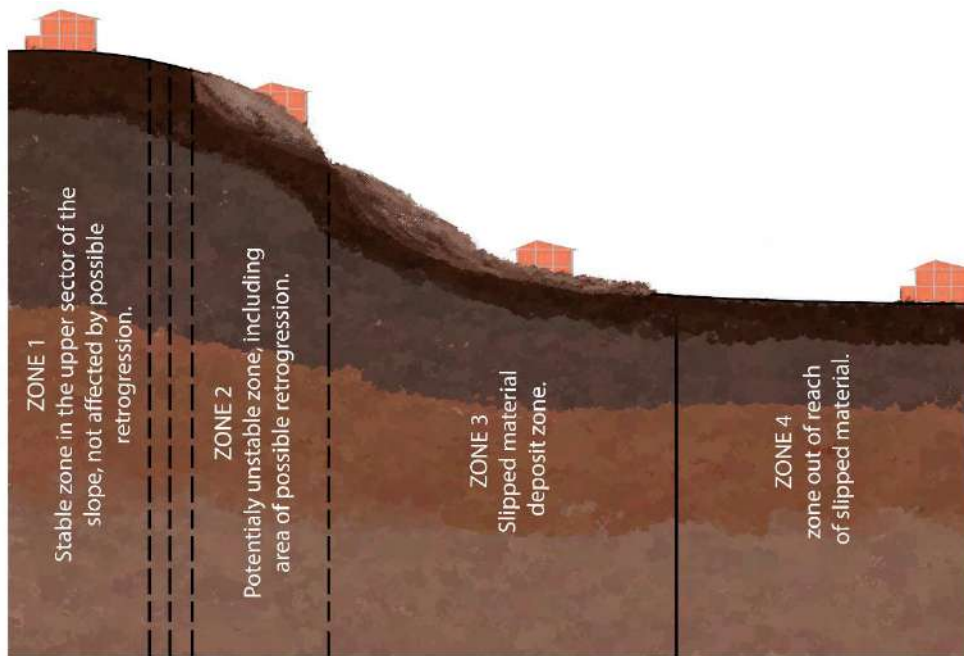
Note. Mass movements physical vulnerability (buildings) assessment methodology, adapted from Hazus (2012). $P_{\text{SDI}} (DS \geq ds)$: the probability of reaching or exceeding a given damage state due to the direct impact of the mass movement against the building, $P_{\text{GF}} (DS \geq ds)$: the probability of reaching or exceeding a damage state, given permanent ground deformation, $P_{\text{comb}} (DS \geq ds)$: the combined probability of reaching or exceeding a damage state, $P_{\text{comb}} (Ds = ds)$: the combined probability of being in a damaged state due to direct impact against the building or permanent ground deformation, S_u : soil undrained strength, c : effective cohesion, ϕ : effective friction angle, h : mass movement Height, $h v^2$: momentum flux.

3.2.1. Exposure Area and Types of Damage Due to Mass Movements

Four areas of building exposure to mass movement can be identified (Ávila et al., 2016),

Figure 18

Exposure Areas of the Elements in Case Mass Movements.



Note. Adapted from Ávila et al. (2016)

No damage is expected in zones 1 and 4. In zone 2, elements are located on a potentially unstable slope or potentially affected by retrograde effects. In this zone, collapse or immediate damage is expected due to loss of foundation support. Differential settlement, tilting and cracking associated with slow movement or collapse of the structures associated with rapid movement may be found. In Zone 3, buildings are in the path of mass movement or around the deposition of sliding material. Partial or complete damage can be expected due to impact, obstruction, foundation failure and burial. The research analyses and finds damage thresholds for masonry buildings according to these zones, where the probability of failure due to deformation of the

structure, assuming a fixed/rigid foundation and the failure probability due to foundation support loss are relevant.

3.2.2. Buildings' Vulnerability Due to the Direct Impact of Mass against the Structure (Fixed/Rigid Base/Foundation)

Following the methodology presented, a three-dimensional structural model is considered under the assumption of a rigid foundation, i.e., no deformations at the foundation. This makes it easier to model the system, concentrating on the parameters considered relevant to its functioning.

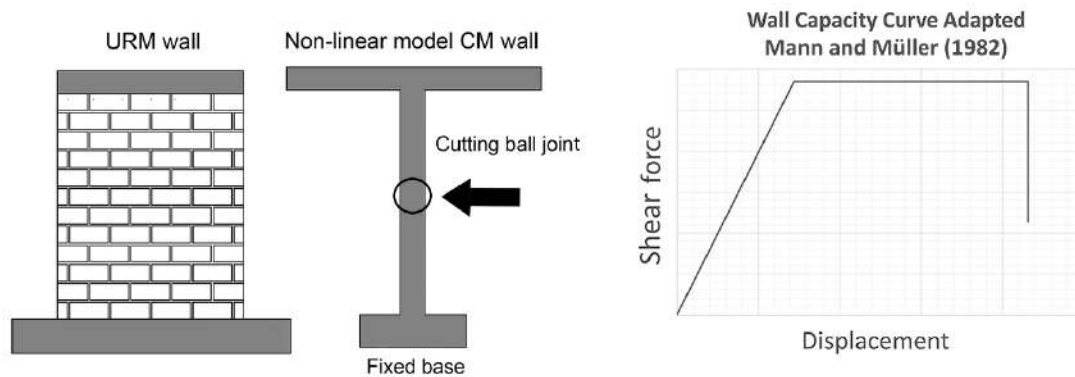
To understand the overall performance of masonry buildings due to mass movements, it is essential to recognise that mainly the walls parallel to the flow, based on their stiffness and strength, are responsible for resisting the impact. These elements' mechanical and probabilistic behaviour are studied in the following items. Additionally, the possibility of local damage to the walls perpendicular to the flow is considered in Section 3.2.2.8.

3.2.2.1. Mechanical Analysis of Masonry Due to Mass Movements.

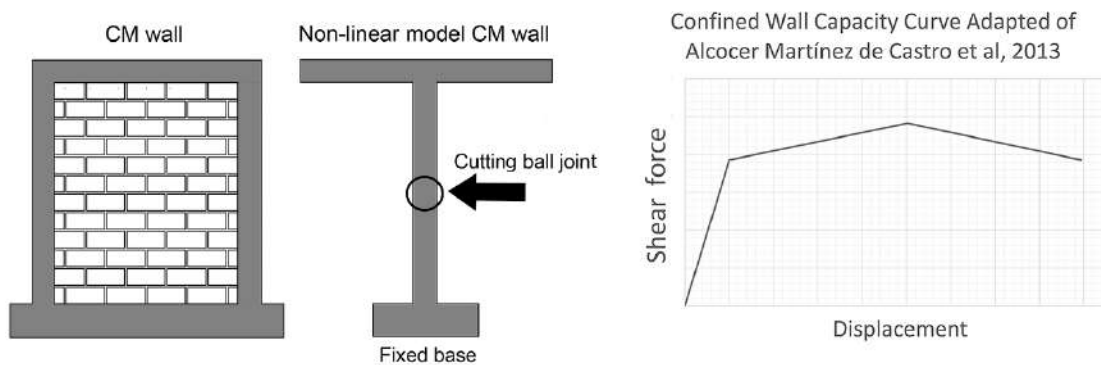
The structural behaviour of masonry buildings against external forces, e.g., mass movements, is controlled by the walls' shear strength (Kranzler et al., 2005), particularly in low-rise buildings. The shear strength of masonry buildings can be idealised using non-linear mathematical models for each building wall using equivalent structural frame elements (wide column method) (A-W et al., 2004; Borah et al., 2023; Gonzales et al., 2020; Pérez, 2014; Zuñiga, 2005). These equivalent frames have a half-height shear hinge that allows lateral deformations. In addition, a fixed base condition is assumed at the bottom of the frame, while a beam with infinite stiffness is supposed to exist at the wall top (Figures 19 and 20). The hinge (cutting ball joint) permits plotting a displacement versus shear force graph, i.e. the wall lateral capacity curve. This method ensures that the kinematic conditions are met (López-Ortiz, 2011).

Figure 19

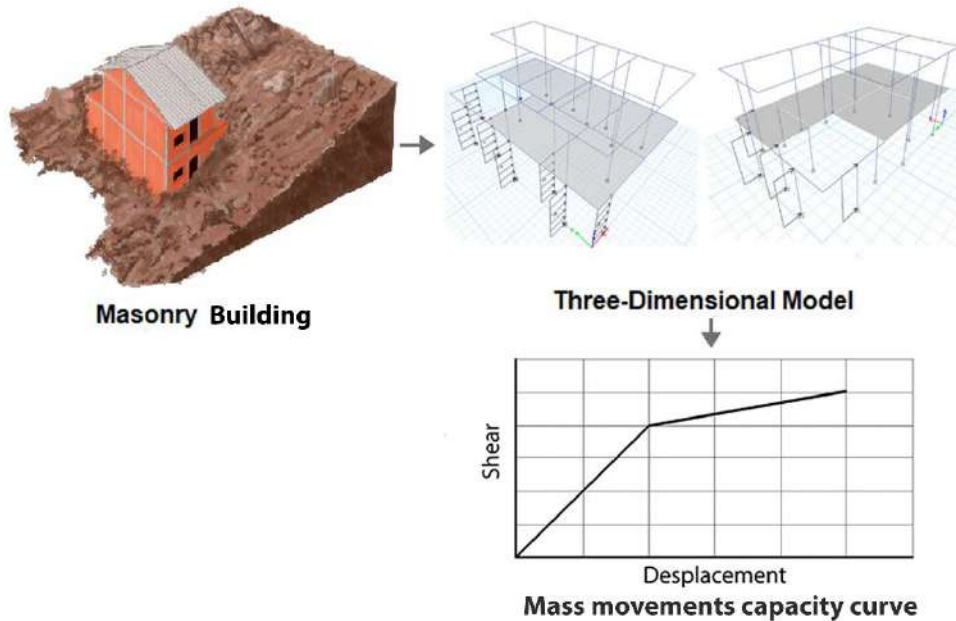
URM Wall and its Equivalent Nonlinear Model.

**Figure 20**

CM Wall and its Equivalent Nonlinear Model.



Each equivalent frame is then assembled to form a building three-dimensional model. The three-dimensional building performance model against mass movements is evaluated using a nonlinear static analysis procedure of progressive failure called the 'push-over' method, Figure 21. This technique is a powerful tool for evaluating the inelastic behaviour of structures under lateral loading (Khan, 2013). The conventional pushover method analyses how a structure deforms and collapses under a monotonic lateral load distribution (Panyakapo, 2014).

Figure 21*Conventional Push-over Analysis.*

3.2.2.1.1. Masonry Failure Modes.

Concerning the masonry failure modes under lateral loads such as mass movements, emphasis was placed on the shear strength behaviour of the walls with and without confinement. The shear strength of the masonry was investigated considering the effect of compressive stresses induced by gravitational loads in the direction normal to the horizontal joints.

Unconfined, Unreinforced Masonry (URM). This construction system usually consists of walls built with fired clay bricks joined with lime or cement mortar. URM building is a heterogeneous, nonelastic material. The bricks' properties vary from region to region depending on the nature of available soil and the technique adopted for moulding and burning (Joshi & Jain, 2013).

Failure Modes for URM Walls. For these walls, the most critical effect was taken into account according to the failure modes proposed by Mann and Müller, 1982 (Bakeer, 2012;

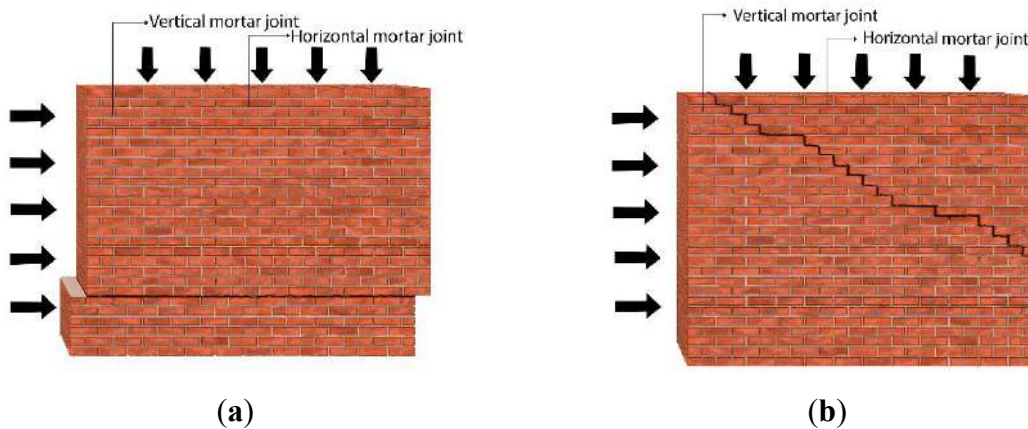
Bonett-Díaz, 1998; Kranzler et al., 2005), where the behaviour of the masonry is characterised by the brittle response of the bricks in traction and the weakness introduced by the mortar joints. Three failure modes are considered: friction-shear, diagonal tension, and compression (Figures 22 and 23). The friction-shear failure mode occurs on walls at low compressive stresses (σ_v). This failure type is produced by the weak union of the mortar-brick interfaces, giving rise to sliding shear stresses in the horizontal and vertical joints, Figure 22. The shear strength calculation is assessed following the Mohr-Coulomb failure criterium affected by an experimental coefficient.

$$V_1 = \left(\frac{c + \mu \sigma_v}{1 + \mu \frac{2h}{l}} \right) tL \quad (22)$$

where c is the cohesive strength of masonry bed joints, $\mu = \tan(\phi)$ is the bed-joints friction coefficient, ϕ is the bed-joints friction angle, σ_v is the gravitational load on the wall, t and L are the thickness and length of the wall, h , and l the height and length of the brick respectively.

Figure 22

(a) *Friction-Shear Failure Mode Schema (Sliding in the Horizontal Joints)*; (b) *Friction-Shear Failure Mode Schema (Sliding in the Horizontal and Vertical Joints), URM Masonry.*



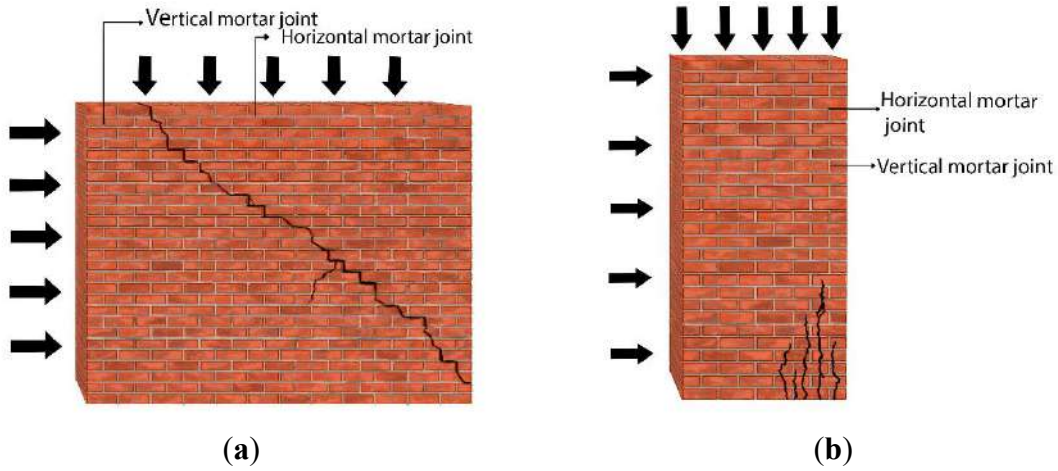
If the compressive stresses are increased moderately, the shear strength of the mortar joints increases. Therefore, it is probable that failure due to diagonal tension occurs on the brick units

of the URM walls, Figure 23 (a). The shear strength calculation considering this failure mode can be seen in Eq. (23). Where f_{tb} is the tensile strength of the brick, taken as 10% of its compressive strength, f_{cb} .

$$V_2 = \left[\frac{f_{tb}}{2.3} \sqrt{\left(1 + \frac{\sigma_v}{f_{tb}} \right)} \right] tL \quad (23)$$

Figure 23

(a) *Diagonal Tension Failure Mode Schema URM Masonry, (Including Bricks Cracking);* (b) *Compression Failure Mode Schema URM Masonry.*



For very high values of compressive stresses, the failure would be in compression and is similar to the one that occurs under direct compression, although the effect of the shear stress causes a reduction in the compressive strength of the masonry, Figure 23 (b). The shear strength calculation considering this failure mode can be seen in Eq. (24). Where f'_m is the compressive strength of the masonry.

$$V_3 = \left[(f'_m - \sigma_v) \frac{l}{2h} \right] tL \quad (24)$$

Regarding the calculation of deformations, the relationship between shear stresses and lateral stiffness of the wall allows for deducing the yield limit deformation. The stiffness in the elastic range is obtained according to Calderón (2005), Eq. (25).

$$K_e = \left(\frac{H^3}{\beta E_m I_m} + \frac{\gamma H}{A G_m} \right)^{-1} \quad (25)$$

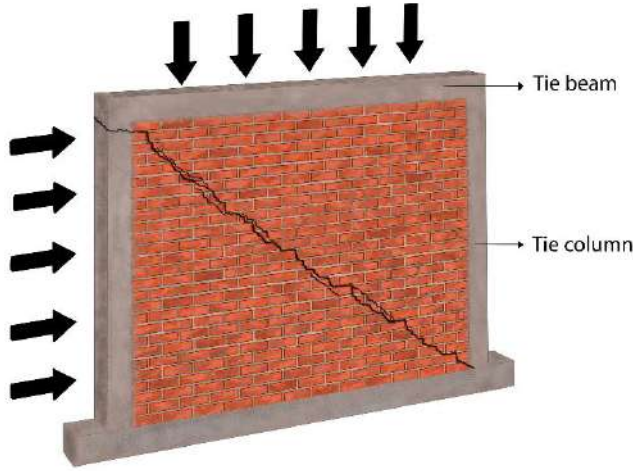
Where H is the effective height of the wall, E_m is the masonry elasticity modulus, G_m is the masonry shear stiffness modulus, A is the effective shear area, β is a factor that depends on the support conditions at the ends of the wall, considered equal to 3 for cantilever walls and 12 for built-in walls. A factor of 9 is used for the intermediate floors and 3 for the last floors since the roofs are light (Calderón-Ardila, 2005). I_m is the wall section inertia, γ is a shape factor that relates the average shear stress to the maximum stress for a given section. In the cases of rectangular sections, it is 1.2.

Recognising that URM walls subjected to lateral loads exhibit brittle structural performance, a capacity curve with perfect elastoplastic behaviour is assumed, where the ultimate strain is taken to be equal to 3 times the yield limit strain (FENTON, 1989).

Confined Masonry (CM). This is a construction system where the masonry structural walls are confined on its four sides by reinforced tie columns and tie beams (Alcocer-Martínez de Castro et al., 2013), Figure 24. The confining elements have the same thickness as the wall.

Figure 24

Diagonal Compression Failure Mode Schema CM Masonry and Disposition Reinforced Tie Columns and Tie Beams.



CM walls must be located correctly in the two main directions of the building, both in plan and height. This system configuration gives superior structural performance under lateral loads compared to URM buildings.

Statistically calibrated trilinear capacity curves for CM walls with or without transverse reinforcement, laterally loaded in a direction parallel to their plane, have been proposed (L.E. Flores & Alcocer, 1996). Based on the above research, the current state of the art, and experimentation, a trilinear curve has been fitted for the inelastic range (Alcocer-Martínez de Castro et al., 2013). They suggested expressions to determine the cracking shear strength expressed in Eq. (26), The maximum shear strength and the ultimate shear strength are shown in Eqs. (27) and (28) respectively.

$$V_{cr} = (0.5v_m + 0.3\sigma_v)tL \leq 1.5v_mtL \quad (26)$$

$$V_{max} = V_{cr} + 1.26\eta n d_b^2 \sqrt{f_c f_y} \leq 1.25V_{cr} \quad (27)$$

$$V_{ult}=0.8V_{max} \quad (28)$$

Where v_m is the masonry nominal shear stress, σ_v is the gravitational load on the wall, t and L the thickness and length of the wall, n expresses the number of longitudinal bars in the wall, η (efficiency factor) = 0.3, d_b is the diameter of the longitudinal reinforcement, f'_c is the concrete compressive strength, and f_y is the longitudinal reinforcement yield strength. Regarding the calculation of deformations, the relationship between the shear force and the wall lateral rigidity allows for the deduction of yield limit deformation. The stiffness in the elastic range is obtained according to Calderón-Ardila (2005), Eq. (29). I_c and I_s are the contributions of the columns and the longitudinal steel to the moment of inertia, respectively.

$$K_e = \left[\frac{H^3}{\beta E_m (I_m + I_c + I_s)} + \frac{\gamma H}{A G_m} \right]^{-1} \quad (29)$$

The displacement due to the maximum shear force, V_{max} , corresponds to the ratio between the maximum shear force and 25% of the wall lateral stiffness (K_e). The displacement due to the ultimate shear V_{ult} , is determined by the ratio between V_{ult} and 11.25% of the wall lateral stiffness (K_e) (Alcocer-Martínez de Castro et al., 2013).

3.2.2.2. Masonry Structural Performance Probabilistic Model

Masonry buildings performance analysis involves several variables, each with an uncertainty degree. This means that to account for a wide range of variability in these parameters in a three-dimensional model, it is necessary to use simulation methods such as the Monte Carlo technique. Nevertheless, this method requires a large number of simulations, i.e., in the order of hundreds, thousands and millions, to reach a reliable solution, making it highly computationally expensive and impractical for everyday use, particularly in the world's less developed regions

(Thinley & Hao, 2017). If there are n parameters, using Monte Carlo simulation and considering that each variable must generate at least 1000 values to define a probability density function (Heijungs, 2020), the 1000 values must be combined with each of the others; this would be 1000^n combinations. For example, 5 parameters would require 1000^5 runs of a three-dimensional structural model, whereas the procedure used here, i.e. the point estimation method, would require only 2^5 , i.e., 32 runs. Therefore, this research uses the methodology of point estimates for probabilistic moments originally proposed by Rosenblueth (1975) to include the effect of epistemic and aleatory uncertainty in the performance system of each structural model. This method simplifies estimating a function's mean and standard deviation and only slightly compromises accuracy when the variables' dispersions are very large (Hidalgo & Pacheco De Assis, 2011; Thinley & Hao, 2017). From the function $Y = f(x_1, x_2, \dots, x_n)$, where n represents the number of variables, it is necessary to have information on the mean value, μ_{xi} , and the standard deviation, σ_{xi} , of each of the variables x_i . According to the function, there are 2^n combinations $Y_i = f(\mu_{xi} \pm \sigma_{xi}, \mu_{xn} \pm \sigma_{xn})$. The expected value of the function and the standard deviation are calculated according to Eqs. (30) and (31) respectively.

$$E(Y) = \sum (P_{ij..n}) (Y_{ij..n}) \quad (30)$$

$$S(Y) = \sqrt{E(Y^2) - [E(Y)]^2} \quad (31)$$

The term $P_{ij..n}$ corresponds to the weighting factors of each estimate Eq. (32).

$$P_{ij..n} = \frac{1}{2^n} \left(1 \pm \sum \rho_{ij..n} \right) \quad (32)$$

The expression $\rho_{ij..n}$ alludes to the correlation coefficients between the variables of the function. The number of correlations between variables responds to the relationship, $\frac{n(n-1)}{2}$; with which 2^n weighting factors are calculated.

A general description of Rosenblueth's method application is given in Figure 25.

3.2.2.3. Uncertainty Parameters for the Study of Masonry

Mean mechanical properties values and corresponding standard deviations for URM and CM structures Tables 8 and 9, and the usual conditions of gravitational load, Table 10, were considered to carry out the probabilistic analysis from several references (Afanador-Garcia & Guerrero-Gómez, Gustavo Monroy-Sepúlveda, 2012; Ahmed et al., 2019; Flores et al., 2013; Joshi & Jain, 2013; Khanmohammadi et al., 2014; H. Luo et al., 2020; Magenes & Calvi, 1997; Meli & Reyes, 1971; Milkova et al., 2020; Muhit et al., 2022; Odacıoğlu & Doğan, 2019; Páez-M, 2006; Paulay & Priestley, 1992; Vamvatsikos & Pantazopoulou, 2010). Expert opinions were also accounted for. The parameters used are representative of existing masonry buildings. Generally, these types of buildings are the product of informal constructions with low-quality standards.

Table 8

Statistical Mechanical Parameters URM Structures.

Parameter	Mean value	sd
f_m (MPa)	3	0.9
c (MPa)	0.14	0.08
μ	0.6	0.3
f'_{cb} (MPa)	5	2

Table 9

Statistical Mechanical Parameters for the Study of CM Structures.

Parameter	Mean value (MPa)	sd
f_m	3	0.9
v_m	0.48	0.05
f_c	20	4
f_y	420	25

Table 10

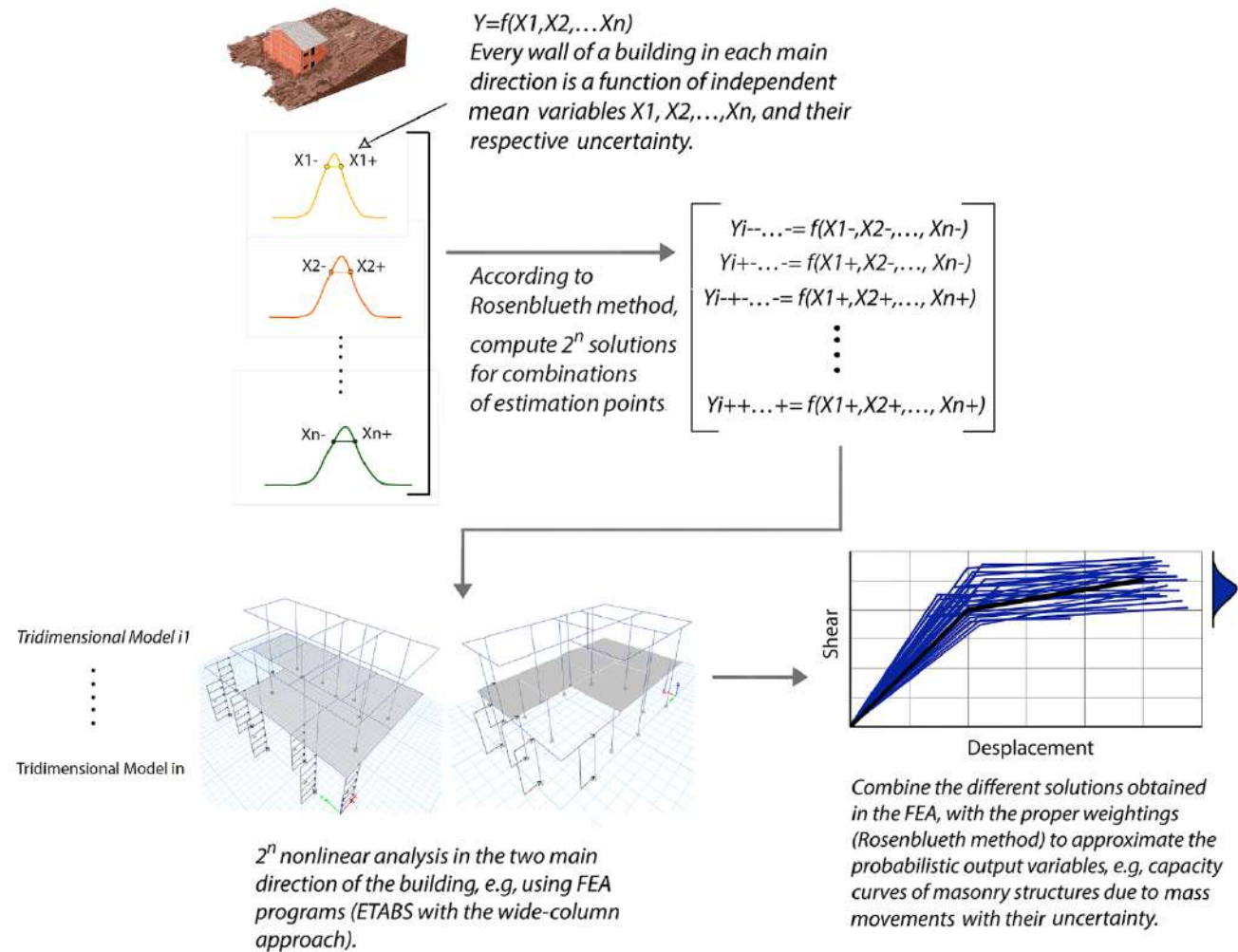
Statistical Mechanical Parameters for the Study of URM, and CM Structures.

Parameter	Mean value (MPa)	sd
σ_{v11}	0.065	0.015
σ_{v21}	0.20	0.050
σ_{v22}	0.065	0.015
σ_{v31}	0.38	0.09
σ_{v32}	0.20	0.050
σ_{v33}	0.065	0.015
σ_{v41}	0.49	0.110
σ_{v42}	0.38	0.09
σ_{v43}	0.20	0.05
σ_{v44}	0.065	0.015

Note. σ_{vij} : This refers to the mean afferent gravitational load acting on a wall of building i , located at level j , including the weight of walls, slabs, ceiling, roof, and 20% of live loads.

Figure 25

General Description for Obtaining Structural Capacity Curves Using the Rosenblueth Method Application.



3.2.2.4. Correlation Coefficients [ρ_{ij}] for Masonry

According to Rosenblueth's method, the probabilistic analysis must consider the correlations between the compromised variables. In general, the correlation between the physical and mechanical properties of the masonry varies depending on several factors, such as the type of masonry, the characteristics of the confinement (if any), the properties of the materials, the conditions of the contact surface, the roughness, curing conditions, the presence of lubricants or slip agents, construction quality, load conditions, among others. The different correlations used in the present research can be seen in Table 12. These coefficients are based on (Eurocode 6 - Design of Masonry Structures - Part 1-1: General Rules for Reinforced and Unreinforced Masonry Structures, 2011; 1994 Uniform Building Code, 1994; ACI et al., 2013; Elghazouli & Peppin, 2011; Gonen et al., 2022; Gumaste et al., 2007; Jafari et al., 2022; Odacıoğlu & Doğan, 2019; Pulatsu et al., 2020; Zimmermann & Strauss, 2011), and expert opinions. Table 11 allow the identification of each variable for its respective correlation.

Table 11

Identification of URM and CM Variables for the Calculation of Correlations.

Parameters (URM)	ID	Parameters (CM)	ID
f'_m	1	v_m	1
c	2	f'_c	2
μ	3	f_y	3
σ_v	4	σ_v	4
f'_{cb}	5	f'_m	5

Table 12*Correlation Coefficients for URM and CM Walls, Rosenblueth's Method*

Correlation (URM walls)	ID	Correlation (CM walls)	ID
1_2	0.3 – 0.5	1_2	0
1_3	0.1 – 0.2	1_3	0
1_4	0.3 – 0.5	1_4	0.3-0.5
1_5	0.3 - 0.5	1_5	0.3-0.5
2_3	0.0 – 0.2	2_3	0
2_4	0	2_4	0
2_5	0	2_5	0
3_4	0	3_4	0
3_5	0	3_5	0
4_5	0	4_5	0.3-0.5

3.2.2.5. Masonry Structures' Capacity Curves

Three-dimensional models of URM and CM buildings were analysed following the methodology of point estimates for probabilistic moments (Rosenblueth, 1975), Figure 25. The masonry's nonlinear behaviour was considered by idealising the equivalent frame (wide-column approach) (A-W et al., 2004; Gonzales et al., 2020; Zuñiga, 2005). In the ETABS software, 48 representative structures were modelled according to typology (URM and CM), architectural distribution (corner, party, and single buildings), number of storeys (1 to 4) and average wall thickness (12 and 20 cm), i.e., 24 URM buildings and 24 CM buildings. Considering the typical brick arrangements in wall construction, average wall thickness of 12 cm and 20 cm and an inter-story height of 2.4 m were utilised. In buildings with a wall thickness of 12 cm, an average wall density in the longitudinal direction per floor of $0.049 \text{ m}^2/\text{m}^2$ was used, and an average wall density in the orthogonal direction per floor of $0.029 \text{ m}^2/\text{m}^2$. For buildings with a wall thickness of 20 cm, an average wall density in the longitudinal direction per floor of $0.082 \text{ m}^2/\text{m}^2$ was used, and an average wall density in the orthogonal direction per floor of $0.048 \text{ m}^2/\text{m}^2$. The above geometric

arrangements are derived from documents such as (Acevedo et al., 2017; Brzev, 2007; Calderón-Ardila, 2005; Yekrangnia, 2023) and expert judgement. 2ⁿ models were studied following Rosenblueth's method, where n denotes the number of variables operated. In the case of the 24 URM structures, randomness was considered in five variables (c , f_m , μ , f_t , σ_v), for a total of 32 models per building type, i.e. 768 URM models were analysed. In the case of the 24 CM structures, randomness was also assumed in five variables (f_m , v_m , f_y , f'_c , σ_v), for a total of 32 models per building type, i.e. 768 CM models were analysed. In total, 1536 models were analysed using the progressive failure static nonlinear analysis method, "Push-over". The mass movement distribution pressure in the structures was considered uniform (H. Luo et al., 2020). The maximum contact height of the mass movements with the buildings was 4.8 m. Results of the lateral capacity curves parameters to mass movements of the masonry buildings according to rigid base assumption are presented in a tabulated format, Tables 13 and 14.

Table 13

Masonry Structures Capacity Curve Results. Wall Thickness: 12 cm.

Structure Group	Median value		β sdln
	F _y (kN)	F _u (kN)	
URM 1 Storey	102	114	0.41
URM 2 Storeys	189	221	0.36
URM 3 Storeys	302	333	0.31
URM 4 Storeys	352	386	0.31
CM 1 Storey	246	340	0.10
CM 2 Storeys	441	583	0.10
CM 3 Storeys	746	1004	0.10
CM 4 Storeys	868	1126	0.13

Table 14*Masonry Structures Capacity Curve Results. Wall Thickness: 20 cm.*

Structure Group	Median value		β sdln
	Fy (kN)	Fu (kN)	
URM 1 Storey	166	206	0.45
URM 2 Storeys	285	337	0.37
URM 3 Storeys	411	461	0.31
URM 4 Storeys	547	602	0.27
CM 1 Storey	444	681	0.10
CM 2 Storeys	741	1040	0.10
CM 3 Storeys	1323	1780	0.10
CM 4 Storeys	1571	2004	0.13

3.2.2.6. Clay Brick Masonry Buildings Fragility Curves (Fix/Rigid Foundations)

Fragility curves represent the probability of reaching or exceeding a specified limit damage state when a structure is subjected to a given hazard. Log-normal functions are proposed as mass movement fragility curves using the momentum flux, hv^2 (where h and v are the height and velocity of mass movement at a given site, respectively) as the hazard intensity parameter (Prieto et al., 2018). Each curve is characterised by a median value and its log-normal standard deviation (β). Therefore, the probability of reaching or exceeding a specific damage state, given a structural response due to a defined hazard (mass movements) can be calculated as follows:

$$P[DS/hv^2] = \Phi \left[\frac{1}{\beta_{DS}} \ln \left(\frac{hv^2}{\overline{hv^2}_{DS}} \right) \right] \quad (33)$$

where:

hv^2 : it is the mass movement impact index that represents the intensity of the hazard,

Figure 26.

$\overline{hv^2}_{DS}$: it is the median value of the impact index at which the building reaches the Damage State (DS).

β_{DS} : it is the standard deviation of the natural logarithm of the impact index associated with a Damage State (DS).

Φ : it is the standard normal cumulative distribution function.

The median value defines the point at which the probability of reaching or exceeding the damage state is 50%, i.e., the threshold value and the standard deviation give an idea of the dispersion.

3.2.2.6.1. Momentum Flux or Impact Index (hv^2)

The potential for tsunamis can be quantified by using the impact index (Hazard Tsunami Model Technical Guidance, 2017). This is defined mathematically as the product of the mass movement depth and the square of the maximum flow velocity at a given point, Figure 26. The momentum flux is a realistic indicator for hydrodynamic forces that result in the lateral displacement of buildings and other assets (Jakob et al., 2012). Physically, momentum flux represents the hydrodynamic drag force divided by the fluid density and building width.

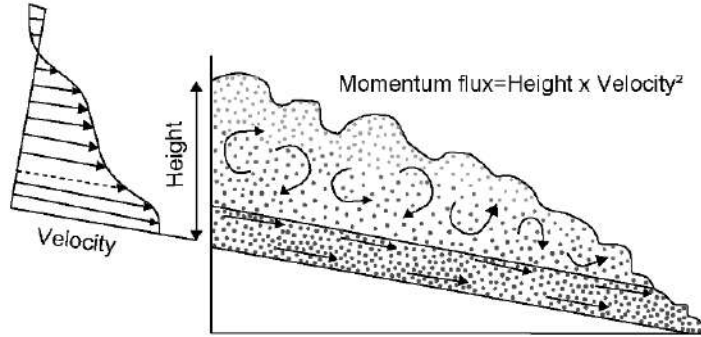
The hydrodynamic acting force is (Hazard Tsunami Model Technical Guidance, 2017):

$$F_a = \frac{K_d}{2} C_d \cdot \rho \cdot B (hv^2) \quad (34)$$

The impact coefficient (K_d) modifies the hydrodynamic force due to the size of the material dragged by the mass movements, which can cause damage to buildings. The drag coefficient (C_d) is used in the model to incorporate the drag effect between the mass movements and the structure. (ρ) represents the density of the material and varies with each type of mass movement. The building width (B) is measured perpendicular to the direction of mass movement. A lognormal distribution function represents each random variable.

Figure 26

Impact Index or Momentum Flux.



3.2.2.6.2. *Mass Movements Damage States*

The structural capacity due to mass movements, considering different damage thresholds, Figure 27, is referenced below:

Slight Damage State. This condition is found when the mass movements hazard is equal to the structural capacity corresponding to the average of the basal strength for the elastic behaviour of the building (Kean et al., 2019a).

$$F_{r \text{ elastic}} = \frac{F_y}{2} \quad (35)$$

Moderate Damage State. It is found when the mass movements hazard equals the building's basal yield strength.

$$F_{r \text{ yield}} = F_y \quad (36)$$

Extensive Damage State. This condition is achieved when the mass movements hazard is equal to the average of the yield strength and the ultimate strength.

$$F_{r \text{ extensive}} = \frac{F_y + F_u}{2} \quad (37)$$

Complete Damage State. It is deduced when the mass movements hazard exceeds the building's capacity.

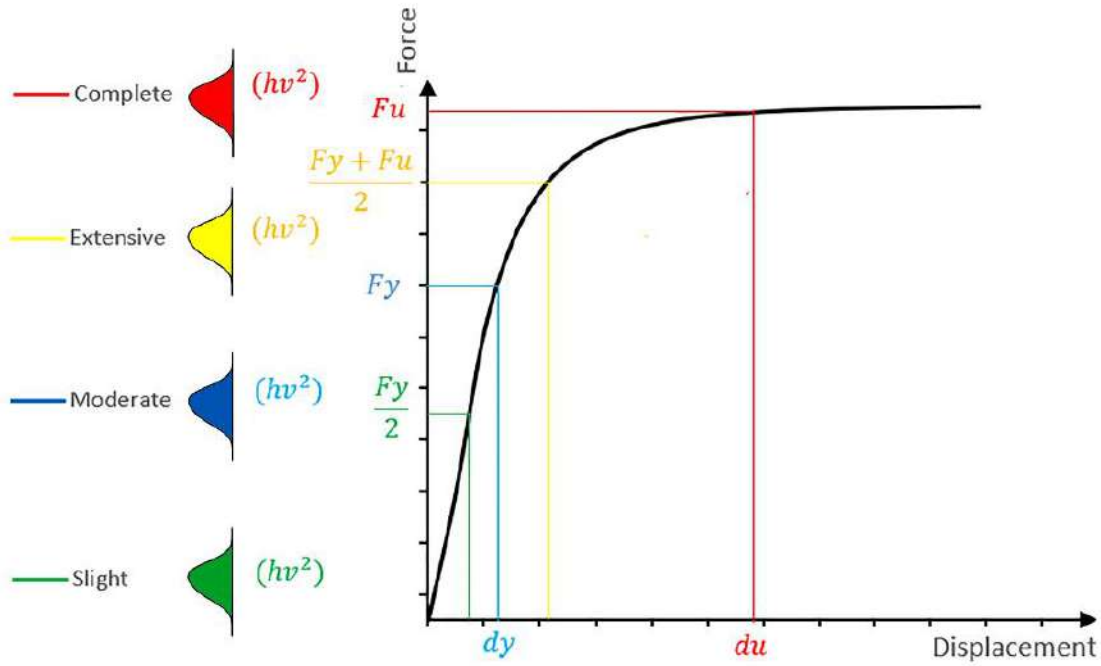
$$F_{r \text{ complete}} = F_u \quad (38)$$

The acting mass movement force Eq. (34) can be matched with the building lateral capacity expressions (Eqs (35), (36), (37) y (38)) to solve for the momentum flux threshold, hv^2 , for each damage state, i.e.,

$$hv^2 = \frac{2F_r}{K_d \cdot C_d \cdot \rho \cdot B} \quad (39)$$

Figure 27

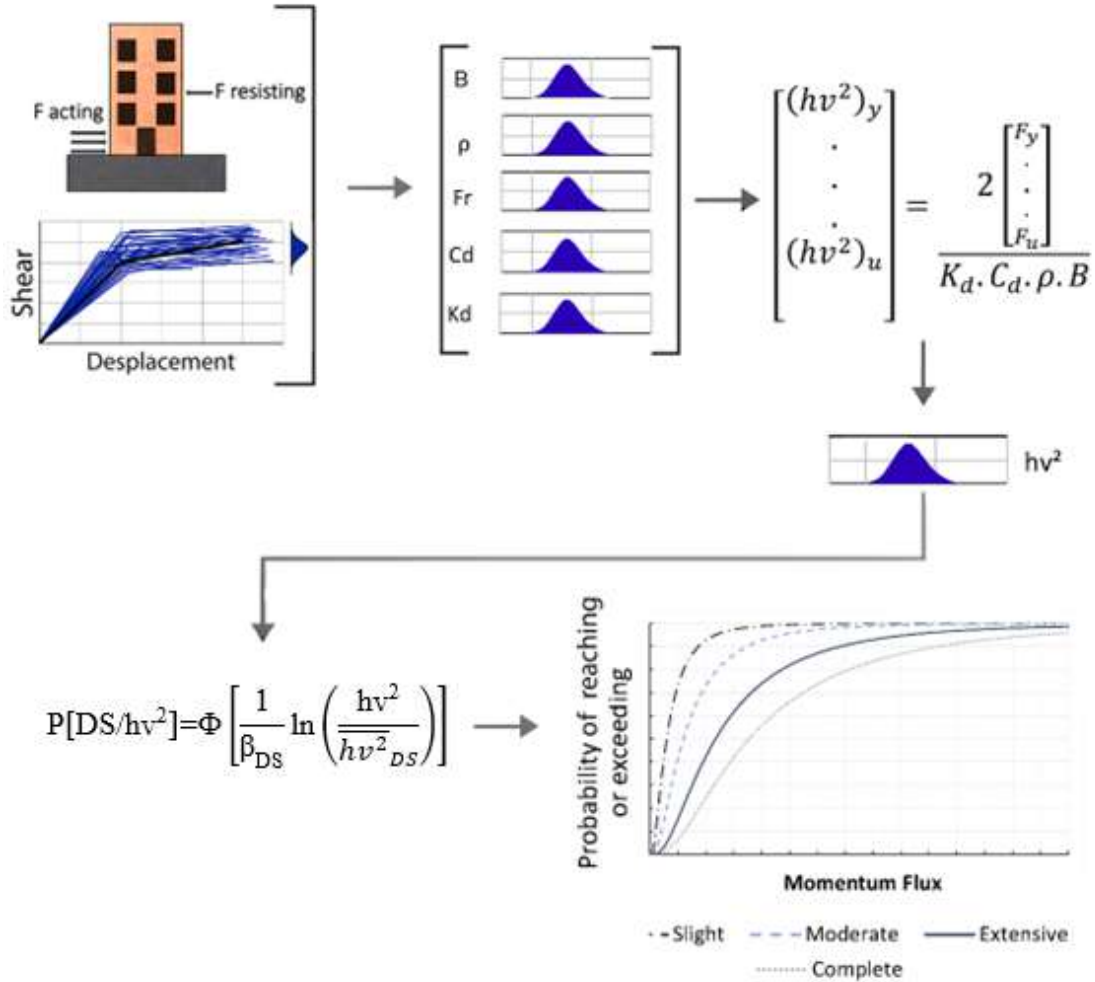
Median Structural Capacity Curve, Probabilistic Analysis.



The use of each variable uncertainty parameter allows for the estimation of the median momentum flux, $\overline{hv^2_{DS}}$, and its standard deviation, β_{DS} , Figure 28.

Figure 28

Methodology for Determining Buildings' Fragility Curves for Different Damage States.



3.2.2.7. Methodology's Application (Fix/Rigid Foundation)

Previous researchers estimated fragility curves for complete damage by mass movements in URM buildings (Prieto et al., 2018). A median, threshold value, moment flux of $23.32 \text{ m}^3/\text{s}^2$ and a lognormal standard deviation of 0.899 were obtained. The study was carried out in Medellín, Colombia, for 40 buildings from 1 to 4 stories with an average thickness of walls 20 cm. These researchers used typical earthquake analysis considerations regarding buildings' lateral capacity and uncertainty, taken from documents such as (Acevedo et al., 2017; Hazus–MH 2.1: Technical

Manual, 2012). The resistant force (F_r) for the complete damage state was calculated taking into account earthquake parameters (A_u , A_y), the gravitational loads present (W), the building width (B), the modal seismic parameters (α), the over-resistance factors (λ), according to (Hazu-MH 2.1: Technical Manual, 2012). Representative debris flow intensity parameters were also utilised in Table 15.

Table 15

Log-Normal Distribution Parameters Assigned to the Variables Used to Construct the Fragility Curve due to Momentum Flux in URM Buildings. Complete Damage State. Prieto et al., 2018.

Parameter	Median value	β (sdln)
K_d	1.2	0.52
ρ (kg/m ³)	2000	0.12
C_d	2	0.37
W (N)	812793	0.60
B (m)	7.1	0.24
W/B (N/m)	115349	0.53
$A_u = A_y$ (g)	0.53	0.33

The current methodology adjusts the former proposal, considering the structural behaviour of URM and CM buildings against lateral loads, as shown. Tables 16 and 17 show the parameters used for 12 and 20 cm wall thicknesses, respectively.

Table 16

Log-Normal Distribution Parameters Assigned to the Variables Used to Construct Mass

Movement Fragility Curves, Masonry Buildings (URM, CM), Walls Thickness 12 cm.

Parameter	Median value	β (sdln)
K_d	1.2*	0.52*
ρ (kg/m ³)	2000*	0.12*
C_d	2*	0.37*
B (m)	7.0	0.21
F_{yi} (kN)	Table 13	Table 13
F_{ui} (kN)	Table 13	Table 13

Note. * (Prieto et al., 2018).

Table 17

Log-Normal Distribution Parameters Assigned to the Variables Used to Construct Mass

Movement Fragility Curves, Masonry Buildings (URM, CM), Walls Thickness 20 cm.

Parameter	Median value	β (sdln)
K_d	1.2	0.52
ρ (kg/m ³)	2000	0.12
C_d	2	0.37
B (m)	7.0	0.21
F_{yi} (kN)	Table 14	Table 14
F_{ui} (kN)	Table 14	Table 14

Fragility curve parameters, i.e., median values and log standard deviations, obtained are shown in Tables 18 and 19. Fragility curves are also shown graphically in Appendix A.

Table 18

Fragility Curve Parameters, Masonry Buildings (URM, CM) from 1 to 4 Floors, Wall Thickness 12 cm. Momentum Flux Intensity.

Structure Type	Median $h v^2$ (m ³ /s ²)				β_{DS} sdln
	Slight	Moderate	Extensive	Complete	
URM 1 Storey	3.55	7.10	7.51	7.89	0.80
URM 2 Storeys	6.69	13.38	14.54	15.69	0.80
URM 3 Storeys	10.88	21.75	22.86	23.95	0.80
URM 4 Storeys	12.66	25.31	26.54	27.75	0.80
CM 1 Storey	9.21	18.42	22.09	25.52	0.70
CM 2 Storeys	16.52	33.05	38.56	43.68	0.70
CM 3 Storeys	27.98	55.95	65.96	75.26	0.70
CM 4 Storeys	32.50	65.00	75.17	84.16	0.70

Table 19

Fragility Curve Parameters, Masonry Buildings (URM, CM) from 1 to 4 Floors, Wall Thickness 20 cm. Momentum Flux Intensity.

Structure Type	Median $h v^2$ (m ³ /s ²)				β_{DS} sdln
	Slight	Moderate	Extensive	Complete	
URM 1 Storey	5.76	11.53	12.67	13.81	0.84
URM 2 Storeys	10.00	20.02	21.91	23.79	0.80
URM 3 Storeys	14.81	29.63	31.37	33.10	0.80
URM 4 Storeys	19.89	39.79	41.80	43.80	0.80
CM 1 Storey	16.65	33.30	42.40	51.06	0.70
CM 2 Storeys	27.80	55.60	67.15	77.99	0.70
CM 3 Storeys	49.61	99.23	116.94	133.43	0.70
CM 4 Storeys	58.81	117.62	134.76	149.78	0.70

Similarly, fragility curves using the mass movements' height upon impact were obtained as intensity parameters. The acting hydrostatic force used is (Armanini, 2008):

$$F_a = \frac{C_s}{2} \rho g h^2 B \quad (40)$$

Where gravity is equal to 9.81 m/s², C_s is a dimensionless coefficient that considers the effect of the overpressure due to the flow dynamic behaviour (K. Hu et al., 2011; HUEBL et al.,

2017; Vagnon & Segalini, 2016); ρ stands for density; h is mass movement height, and B is the building plan dimension of the building perpendicular to the flow direction. Because only extreme data are available, the coefficient C_s is represented by a continuous uniform probability distribution function. The mass movements density and the contact measure perpendicular to the flow direction were previously described with their setting functions. After matching the acting lateral expression Eq. (40) with those capacities for different damage states (Eqs (35), (36), (37) y (38)), height thresholds and their uncertainties were obtained, Tables 20 and 21. They are also shown graphically in Appendix B.

Table 20

Fragility Curve Parameters, Masonry Buildings (URM, CM) from 1 to 4 Floors, Wall Thickness 12 cm. Flow Depth Intensity.

Structure Group	Median h (m)				β_{DS} sdln
	Slight	Moderate	Extensive	Complete	
URM 1 Storey	0.40	0.56	0.58	0.59	0.67
URM 2 Storeys	0.54	0.77	0.80	0.83	0.67
URM 3 Storeys	0.69	0.98	1.00	1.03	0.67
URM 4 Storeys	0.75	1.06	1.08	1.11	0.67
CM 1 Storey	0.64	0.91	0.99	1.07	0.54
CM 2 Storeys	0.86	1.21	1.31	1.39	0.54
CM 3 Storeys	1.12	1.58	1.71	1.83	0.54
CM 4 Storeys	1.20	1.70	1.83	1.93	0.54

Table 21

Fragility Curve Parameters, Masonry Buildings (URM, CM) from 1 to 4 Floors, Wall Thickness 20 cm. Flow Depth Intensity.

Structure Group	Median h (m)				β_{DS} sdln
	Slight	Moderate	Extensive	Complete	
URM 1 Storey	0.51	0.72	0.75	0.78	0.70
URM 2 Storeys	0.66	0.94	0.99	1.03	0.67
URM 3 Storeys	0.81	1.15	1.18	1.21	0.67
URM 4 Storeys	0.94	1.33	1.36	1.39	0.67
CM 1 Storey	0.86	1.22	1.37	1.51	0.54
CM 2 Storeys	1.11	1.57	1.73	1.86	0.54
CM 3 Storeys	1.49	2.10	2.28	2.44	0.54
CM 4 Storeys	1.62	2.29	2.45	2.59	0.54

3.2.2.8. Local Effects on Walls Subjected to Out-of-Plane Loading

Due to their high stiffness and shear strength, masonry walls could efficiently resist in-plane loads. Therefore, the overall lateral load capacity of the URM and CM buildings in the present research has been addressed under this concept. Nevertheless, at least some walls building façades will initially be subjected to out-of-plane loading, leading to local failure modes. Due to their low out-of-plane flexural capacity, these walls can be affected. Since the 50s, different analytical, numerical, and experimental studies have been carried out regarding the out-of-plane direction strength of the URM walls (A-W et al., 2004; Abrams et al., 1996; FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Building, 2000; EN1996 Eurocode 6: Design of Masonry Structures, 2001; Angel et al., 1994; Dawe & Seah, 1989; Flanagan & Bennett, 1999; Jaramillo, 2002; Liberatore et al., 2020; Liberatore & AlShawa, 2021; Moghaddam & Goudarzi, 2010; Omote et al., 1977; Paulay & Priestley, 1992; Ricci et al., 2018; Shapiro et al., 1994; Simsir, 2004). The mass movement damage state thresholds in walls subjected to out-of-plane loads in the present research are calculated considering the research Parisi & Sabella. (2017).

The authors assumed that the wall resists a uniform lateral load through a one-way arch mechanism. Researchers determine the ultimate strength of out-of-plane walls, according to Eq. (41). λ is a best-fit coefficient (Asteris et al., 2017).

$$q_{ult} = \frac{0.7f'm\lambda}{\frac{H}{t}} (LH) \quad (41)$$

$$\lambda = 1.38 \left(\frac{H}{t} \right)^{-1.41} \quad (42)$$

Following the methodology presented in the research, the wall's ultimate capacity, Eq. (41), is equated with the mass movement demand, Eq. (34). A log-normal analytical function regarding momentum flux ($h\nu^2$) is obtained, which determines complete damage thresholds and uncertainty in walls subjected to out-of-plane loads. To consider the randomness of the wall's length (L) and height (H), a uniform function was used between 3 m to 4.5 m and 2.3 m to 2.6 m, respectively. The other variables have been defined previously in the research.

For an average thickness wall of 12 cm, a median ultimate capacity of 17.6 kN with a lognormal standard deviation of 0.32 and a median momentum flux ultimate intensity equal to $2.28 \text{ m}^3/\text{s}^2$ with a lognormal standard deviation of 0.94 was obtained. For an average thickness wall of 20 cm, a median ultimate capacity of 60.3 kN with a lognormal standard deviation of 0.34 and a median momentum flux ultimate intensity equal to $7.91 \text{ m}^3/\text{s}^2$ with a lognormal standard deviation of 0.93 was obtained.

In the same way, by equating the mass movement demand, Eq. (40), with the wall capacity to the out-of-plane loading, Eq. (41), it is possible to have a log-normal distribution function where the intensity is measurable in terms of mass movement height, obtaining complete damage thresholds and uncertainty for masonry walls.

For an average thickness wall of 12 cm, a median ultimate capacity of 17.6 kN with a lognormal standard deviation of 0.32 and a median flow depth ultimate intensity equal to 0.33 m with a lognormal standard deviation of 0.69 was obtained. For an average thickness wall of 20 cm, a median ultimate capacity of 60.3 kN with a lognormal standard deviation of 0.34 and a median flow depth ultimate intensity equal to 0.61 m with a lognormal standard deviation of 0.69 was obtained.

3.2.3. Buildings' Vulnerability Due to Permanent Ground Failure

The vulnerability study due to the buildings' foundation ground failure is essential for calculating the mass movement risk. A procedure is proposed to include this phenomenon by identifying the possible failure modes with their representative analytical functions, including the uncertainty of each variable involved. Fragility curves can then be determined using techniques such as Monte Carlo.

3.2.3.1. Typical Foundation Failure Modes

The mechanical behaviour of the main failure modes of building foundation ground is studied, with the hope of representing them through analytical functions in terms of intensity parameters such as momentum flux and mass movement height.

3.2.3.1.1. Loss of Overall Ground Stability

This refers to the loss of general stability of the ground on which the buildings are located, Figure 29 (a). It is a complex type of failure where the infrastructure is destined to collapse. Global instability conditions generally lead to the evacuation of these surfaces due to the high risk and exorbitant cost of stabilisation. However, in addition to the expertise of experts, finite element software is available that allows the study of this type of problem, reaching sufficient criteria to analyse and understand the reality of such a large-scale problem.

Figure 29

(a) Loss of Overall Ground Stability; (b) Loss of Overall Ground Stability, Rosas, Cauca, Colombia.

**(a)****(b)**

A particular case occurred around Rosas, Cauca, Colombia, where the ground lost stability on 10 June 2023, and more than 150 families were evacuated in time. The losses were total, Figure 29 (b).

3.2.3.1.2. *Sliding Failure*

This research study pure sliding in shallow foundations as most masonry houses are built on continuous spread footing grill type, Figure 30 (a).

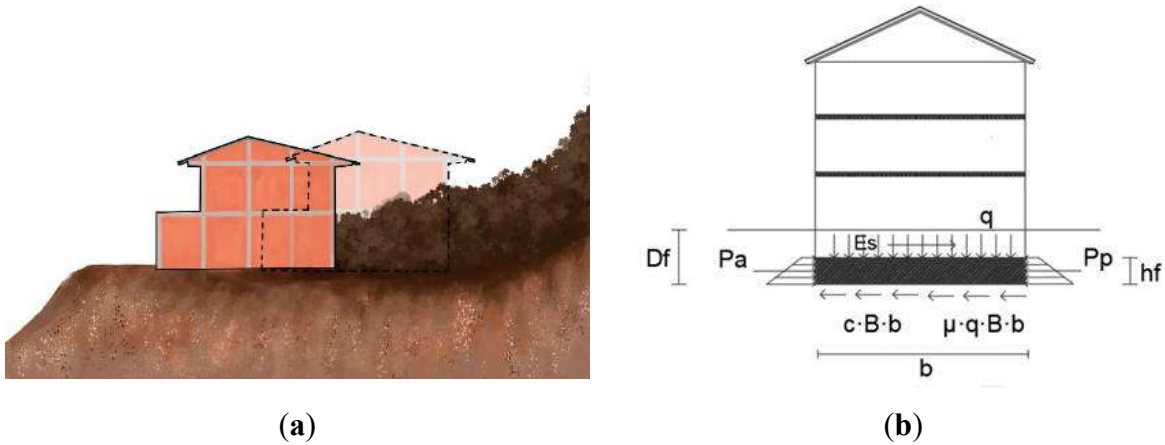
Figure 30 (b) shows the force balance where the mass movement thrust (E_s) is resisted by the adhesion and friction between the soil and the foundation (Mohr-Coulomb formulation), plus the difference between the passive and active soil thrust.

The shear strength of the soil depends on how the applied stresses are distributed among its components. The way the soil evacuates water is fundamental. The expected behaviour is drained in a high permeability soil, e.g. gravels and sands, whereas, in a low permeability soil, e.g. silts and clays, is undrained. These characteristics are considered in Eqs. (43) to (46). Particularly

for undrained behaviour, cohesion (c) is replaced by undrained resistance (S_u), and contact friction is zero.

Figure 30

(a) Building Sliding Failure; (b) Pure Slide Force Diagram.



Note. q : gravitational load acting on the building, E_s : mass movement load, P_a : active thrust, P_p : passive thrust, c : cohesion between soil and foundation, μ : $\tan(\phi)$ friction coefficient at the soil-foundation interface, ϕ : is the friction angle of the soil beneath the foundation, B : plan dimension perpendicular to flow direction, b : plan dimension parallel to flow direction, h_f : foundation thickness, D_f : depth of the foundation.

Mass movement impact against buildings is sudden and instantaneous, so all soil failure modes are expected to occur in undrained conditions. The present research is concerned with foundations supported on undrained soils. Nevertheless, for soils where high permeability is evident, a parameter equivalence in terms of S_u can be found, allowing the use of undrained formulations only (Bishop, 1954).

Equating the strength with the mass movements hazard (E_s) expressed in Eqs. (34) and (40), it is possible to have analytical functions in terms of momentum flux or mass movement height at the instant of impact.

- Slide strength in function momentum flux. Drained case

$$h_v^2 = \frac{2q \cdot b \cdot \mu + 2 \cdot c \cdot b + (2D_f - h_f) \cdot \gamma_s \cdot h_f \cdot (K_p - K_a)}{K_d \cdot C_d \cdot \rho} \quad (43)$$

- Slide strength in function momentum flux. Undrained case

$$h v^2 = \frac{2.Su.b + (2D_f - h_f). \gamma s. h_f. (Kp - Ka)}{K_d. C_d. \rho} \quad (44)$$

- Slide strength in function flow height. Drained case

$$h = \sqrt{\frac{2q.b.\mu + 2.c.b + (2D_f - h_f). \gamma s. h_f. (Kp - Ka)}{C_s. \rho. g}} \quad (45)$$

- Slide strength in function flow height. Undrained case

$$h = \sqrt{\frac{2.Su.b + (2D_f - h_f). \gamma s. h_f. (Kp - Ka)}{C_s. \rho. g}} \quad (46)$$

3.2.3.1.3. *Overturning Failure*

A possible overturning of the structure under the impact of the mass movement is analysed, Figure 32, two analytical expressions are deduced in terms of momentum flux and height of mass movements.

Figure 31

Building Overturning Failure.

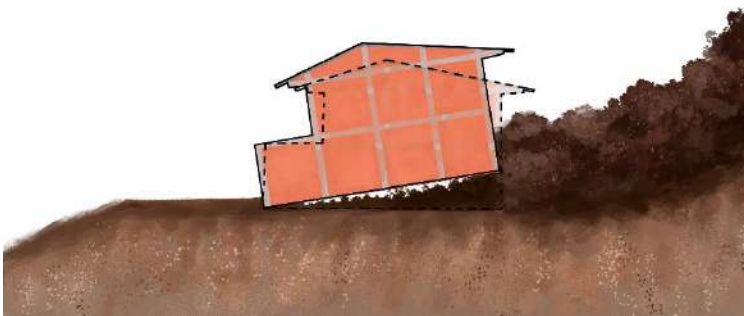
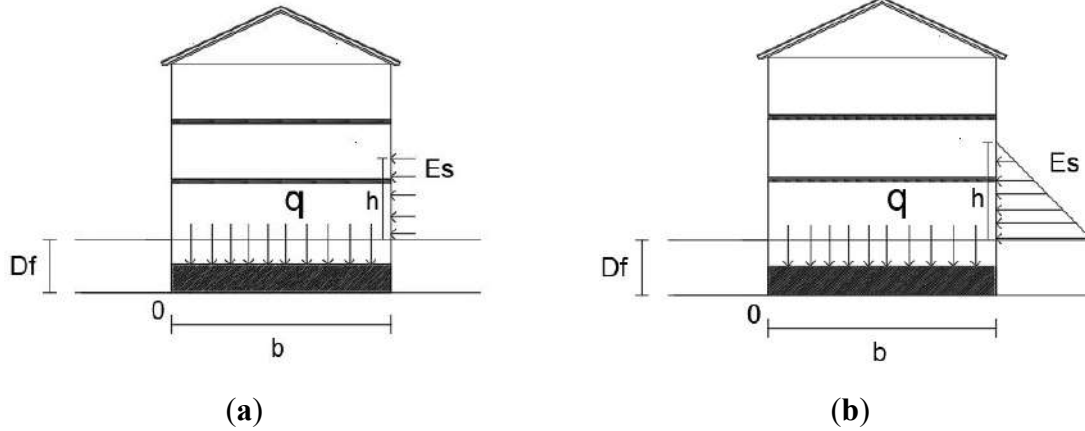


Figure 32

(a) *Overturning Diagram, Using Hydrodynamic Lateral Load (E_s); (b) Overturning Diagram, Using Hydrostatic Lateral Load.*



Note. The pivot point of the structure, "o".

$$h v^2 = \frac{q \cdot b^2}{K_d \cdot C_d \cdot \rho \left(\frac{h}{2} + D_f \right)} \quad (47)$$

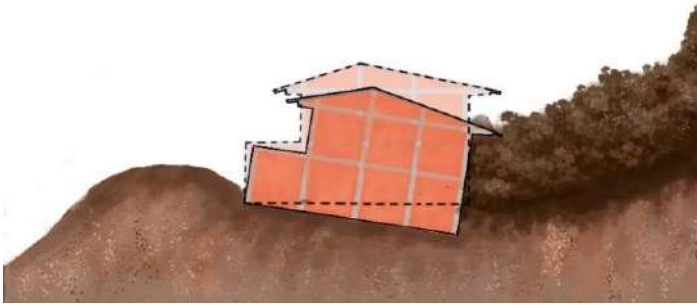
$$h = \sqrt{\frac{q \cdot b^2}{C_s \cdot \rho \cdot g \cdot \left(\frac{h}{3} + D_f \right)}} \quad (48)$$

3.2.3.1.4. **Ground Load-Bearing Capacity Failure**

Another possible failure mode is the loss of the soil's bearing capacity. The behaviour due to the impact of mass movement is sudden and instantaneous, so bearing capacity losses are calculated under undrained considerations.

Figure 33

Foundation Load-Bearing Capacity Failure.



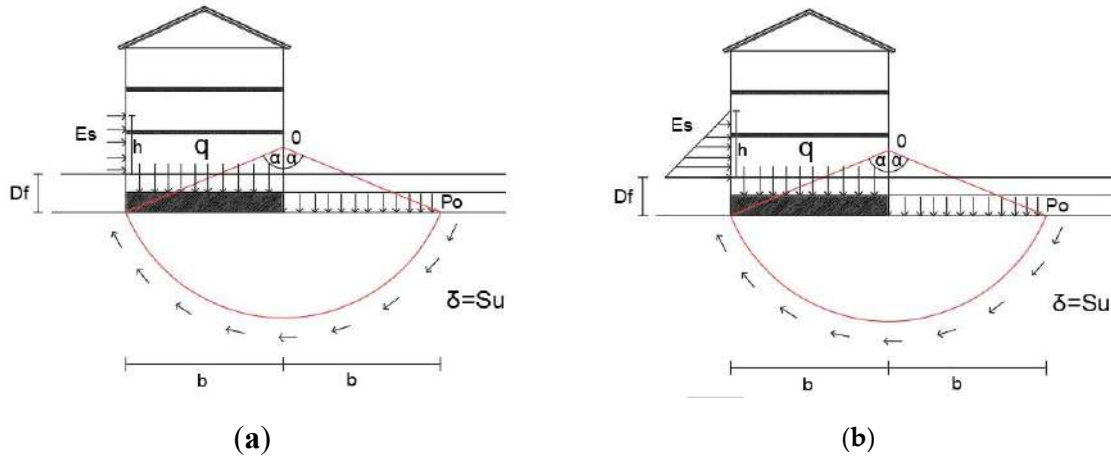
By adapting the derivation made by Prandtl and Tersaghi for undrained soils (fine soils of high and low compressibility), it is possible to equate ground capacity and demand concerning the phenomenon studied, Figure 34. The derivation of resistance in terms of momentum flux and mass movement height is visualised in Eqs. (49) and (50). In deducing these expressions, a maximum eccentricity of $b/6$ has been considered.

Considering the flow behaviour at the moment of impact with a building, the hydrodynamic force is used to express the resistance of the foundation in terms of momentum flux ($h v^2$) or the hydrostatic force to represent the capacity in terms of flow height (h). The capacity of the foundation should also be expressed in terms of height, considering the possibility of a loss of flow velocity along the flow path, where the hydrostatic behaviour may be more representative. Nevertheless, the overpressure effect due to the dynamic effect is taken into account (Armanini, 2008).

Figure 34

(a) Foundation Load-Bearing Capacity Failure Diagram. Hydrodynamic Lateral Load (E_s).;

(b) Foundation Load-Bearing Capacity Failure Diagram. Hydrostatic Lateral Load (E_s).



$$hv^2 = \frac{(5.52Su + \gamma_s D_f - q)b^2}{3K_d \cdot C_d \cdot \rho \left(\frac{h}{2} + D_f\right)} \quad (49)$$

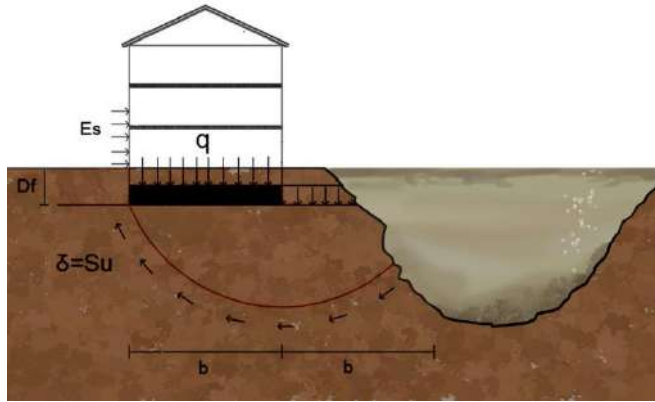
$$h = \sqrt{\frac{(5.52Su + \gamma_s D_f - q)b^2}{3C_s \cdot \rho \cdot g \left(\frac{h}{3} + D_f\right)}} \quad (50)$$

- Scour Failure Mode

Within the ground load-bearing capacity failure, situations such as the one shown in Figure 35 are expected at the edge of slopes and along riverbanks, where the foundation soil loses part of its bearing surface.

Figure 35

(a) Foundation Scoured Conditions. Hydrodynamic Lateral Load (E_s).; (b) Torrential Flood Salgar, Antioquia, Colombia, 2015 (DAPARD, 2015).

**(a)****(b)**

$$h v^2 = \frac{(2.76 S_u + \gamma_s D_f - q) b^2}{3 K_d \cdot C_d \cdot \rho \left(\frac{h}{2} + D_f \right)} \quad (51)$$

$$h = \sqrt{\frac{(2.76 S_u + \gamma_s D_f - q) b^2}{3 C_s \cdot \rho \cdot g \left(\frac{h}{3} + D_f \right)}} \quad (52)$$

3.2.3.2. Application Methodology (Ground Failure Foundation)

Once each phenomenon due to possible permanent ground deformations is studied, where is the building's foundation, obtaining analytical formulations strength in terms of moment flux and mass movements height. Next, the uncertainty of each variable involved is identified using information from studies such as the Hazus Tsunami Model Technical Guidance (2017), Jones et al. (2002), and expert opinion.

In this research, Table 22, an exercise is carried out to consider the possibility of ground failure foundation of the buildings, assuming an undrained condition during the mass movement process, as the duration of the flow and impact is short. It should also be noted that the gravitational load acting on the building (q) is for a single-story building.

Table 22

Statistical Parameters Assigned to the Variables Used to Construct the Fragility Curves Due to Permanent Ground Deformations.

Input parameter	Minimum	Probable	Maximum	Median	Standard deviation	Setting function
Df (m)	0.5		1			Uniform
hf (m)	0.2		0.5			Uniform
h (m)	0.3		4.8			Uniform
γ_s (kN/m ³)	16		18			Uniform
Cs	2.5		7.5			Uniform
b (m)				7	1.5	Log normal
ϕ (°)				28	6.92	Log normal
Kd				1.2	0.67	Log normal
Cd				2	0.77	Log normal
ρ (kg/m ³)				2000	250	Log normal
Su (kN/m ²)				10	2.9	Log normal
q (kN/m ²)				6.5	1	Log normal

Using the above analytical formulations, which express the behaviour of the building foundations due to mass movements, techniques such as Monte Carlo can be used to find cumulative distribution functions representing the probabilities of reaching or exceeding a given level of damage.

Table 23*Fragility Curve Parameters Due to Permanent Ground Deformations, Momentum Flux.*

Failure mode	Median $h v^2$ (m^3/s^2)	β_{DS}
	Extensive/Complete	sdln
Sliding	34.08	0.88
Overturning	41.16	1.13
Load-bearing capacity	123.08	1.21
Load-bearing capacity (scour)	68.69	1.19

Table 24*Fragility Curve Parameters Due to Permanent Ground Deformations, Mass Flow Height.*

Failure mode	Median h (m)	β_{DS}
	Extensive/Complete	sdln
Sliding	1.18	0.26
Overturning	1.44	0.34
Load-bearing capacity	2.53	0.36
Load-bearing capacity (scour)	1.89	0.34

3.3. Remarks

Physical vulnerability assessment due to mass movements is essential to reduce risk and make regions resilient (UNISDR, 2015). Despite advances in the interaction between mass movement hazards and building fragility quantification, vulnerability assessment remains a weak component of quantitative risk analysis (X. Huang et al., 2023; Luo et al., 2023). This gap is even more substantial for URM and CM buildings, significant components of dwellings and other constructions worldwide. Although, currently, there is no HAZUS module for mass movements, the methodology applied to tsunamis is a benchmark for calculating structural fragility due to these hazards, as shown by Kean et al. (2019a) and Prieto et al. (2018). Nevertheless, it is essential to clarify that these researchers used earthquake design considerations to get structural capacity (Luo

et al., 2023). This research presents a model for the structural behaviour of URM and CM buildings against lateral loads due to mass movements, without using seismic parameters. The procedures are intended to be especially useful in less developed regions where laboratory or field data are scarce, sophisticated, large capacity, equipment is unavailable, and time is pressing.

The three-dimensional mechanical behaviour of URM and CM buildings is studied, emphasising the responsibility of the global structural performance of the buildings in the shear resistance offered by the walls to in-plane lateral loads. The previous concept is based on the great rigidity and resistance these walls offer in the plane, providing lateral stability to the building against mass movements. Without knowing the flow's attack direction against buildings, three-dimensional models (assuming a fixed/rigid base) were analysed using the nonlinear pushover method. Each structure was loaded independently in its two main orthogonal directions, choosing the most critical response regarding shear capacity. To have an effective and simplified methodology for the study of vulnerability due to mass movements, macro models were used for the three-dimensional analysis of the structures, using frame-type elements with the plasticity concentrated in the centre of the vertical element (A-W et al., 2004; Borah et al., 2023; Gonzales et al., 2020; Pérez, 2014; Zuñiga, 2005). Macro-modelling in masonry focuses on representing the overall behaviour of masonry structures, considering the interactions between elements and the macroscopic response to applied loads. These models simplify the detailed characteristics of individual units and focus on providing results on a larger scale, efficiently reproducing the structural behaviour when compared to laboratory test results (Hidalgo-Leiva et al., 2018).

The Rosenblueth simulation method provides an effective and efficient probabilistic tool to determine the lateral structural capacity of three-dimensional models in terms of shear strength and uncertainty. Compared to the possibility of using micromodels or detailed models in finite

elements that include the dynamic interaction with the impact load and the Monte Carlo simulation, the difference in costs, resources, and computational time is enormous. It is still prohibited from being used to analyse complex structures (Hidalgo-Leiva et al., 2018). Finally, to obtain fragility curves or damage thresholds by equating the structural capacity with the hazard caused by mass movements, it is possible to have an analytical function regarding momentum flux, height, or another intensity measure of mass movements. At this point, by having an analytical function, using the Monte Carlo technique to provide a probabilistic response efficiently is possible. However, in this research, the cumulative distribution function used is the Log-normal, which is widely supported in engineering due to its excellent physical adjustment. This function reduces the analysis to a spreadsheet.

Confining the walls (CM structures) and/or increasing their thickness improves the structural performance of buildings, as confirmed by the damage state thresholds. For example, for one-story buildings, complete damage thresholds increase from $7.89 \text{ m}^3/\text{s}^2$ in URM constructions to $25.52 \text{ m}^3/\text{s}^2$ in CM buildings, Table 18. The complete damage thresholds for two-storey URM buildings with wall thicknesses of 12 cm and 20 cm are $15.69 \text{ m}^3/\text{s}^2$ and $23.79 \text{ m}^3/\text{s}^2$, respectively, Tables 18 and 19.

The existence or not of a rigid diaphragm (usually a reinforced concrete mezzanine slab) attached to the walls and the gravity loads present in the buildings substantially influence the behaviour of the structures against mass movements, e.g., in single-storey buildings with a light roof the momentum flux damage thresholds are about half the magnitude present for two-storey buildings, i.e., from $7.89 \text{ m}^3/\text{s}^2$ to $15.69 \text{ m}^3/\text{s}^2$ in URM buildings, Table 18.

Masonry structures, particularly URM buildings, respond to events like mass movements based on the stiffness and strength of their walls; the possibility of these buildings performing in

the inelastic range is limited, as evidenced by the minimal variation in moderate, extensive, and complete damage state thresholds for each building (Tables 18 to 21) and graphically illustrated in Appendices A and B.

It is worth noting that some facade walls might be subjected to out-of-plane loads during a mass movement. These walls will respond with low bending, leading to local failure modes. Due to the high fragility of the walls being subjected to this type of loading, only the possibility of local complete damage by out-of-plane loading was analysed. It is recognised that the possibility of the bidirectional arch mechanism as a structural response is not considered, and the beneficial contribution that the axial load can make to a wall is neglected. This type of damage due to local failure modes must be determined according to the risk analyst's expertise regarding the number of walls of the building that can be affected due to the mass movements, where the location of the structure and each element are fundamental. The methodology allows for its inclusion in the risk calculation for a specific analysis.

The structural damage probability due to permanent ground deformation is stated in the methodology and is calculated in this study. Most research and regulations only consider the probability of damage caused by direct impact against the structure, probably because, in many cases, the foundation soil is not damaged. It is likely that, for specific soil conditions, damage, in addition, is generated by the presence of foundation ground failure. With the derived analytical formulations representative of the ground failure modes and the uncertainty of each variable involved, cumulative distribution or fragility functions for extensive/complete damage states can be obtained using techniques such as Monte Carlo simulation. This methodology can be accompanied by relevant technical studies for each specific place for increased reliability.

Given the damage thresholds in this study for URM and CM buildings, it is advisable to review the damage thresholds used in the Methodological Guide for Torrential Flood Hazard Zoning (Ramos Cañón et al., 2021). This document uses qualitative information based on Jakob et al. (2012), to derive damage thresholds in terms of momentum flux. They determine a damage threshold of $1 \text{ m}^3/\text{s}^2$ for light damage, between $1 \text{ m}^3/\text{s}^2$ and $50 \text{ m}^3/\text{s}^2$ for moderate/severe damage and greater than or equal to $50 \text{ m}^3/\text{s}^2$ for complete damage. For example, the complete damage thresholds for one-storey and two-storey buildings of the URM type, Table 18, equal to $7.89 \text{ m}^3/\text{s}^2$ and $15.69 \text{ m}^3/\text{s}^2$ differ significantly from the thresholds found in the guide.

Comparing the fragility curves for direct impact on the structures and those obtained due to permanent ground deformations, it can be concluded that damage due to direct impact is dominant for fragile structures such as URM buildings. However, as the stiffness, strength and/or ductility of the structures increase, the thresholds given by permanent soil deformations may become more critical. For example, for 2-storey URM buildings, the complete damage threshold for direct impact is $15.69 \text{ m}^3/\text{s}^2$ (Table 18) and for permanent ground deformations, it is approximately $34.08 \text{ m}^3/\text{s}^2$ (Table 23). However, the complete damage threshold for direct impact for CM structures is $43.68 \text{ m}^3/\text{s}^2$, and for permanent ground deformations, it is approximately $34.08 \text{ m}^3/\text{s}^2$.

4. Validation

Model results are compared with field data obtained in South Korea, and Colombia. A significant shortcoming in testing mass movement fragility models is the scarcity of published data that provide both hazard intensities (depths and velocities) and the buildings' structural classification (including damage state). A database of 68 events worldwide was compiled by Jakob et al. (2012). The authors provided hazard parameters, such as depth, velocities, minimum, average, and maximum momentum flux. Unfortunately, they do not include the structural characteristics of the buildings that would allow them to be classified using, e.g., the HAZUS system. In southwest China, 61 buildings were damaged by the 20 August 2019 event in Cutou Gorge, Wenchuan, of which 19 constructions were URM buildings between one and two stories. Here, a mean threshold for a complete damage state of $26 \text{ m}^3/\text{s}^2$ could be verified (M. Chen et al., 2021). It is impossible to have detailed information for each building, but an average wall thickness of 20 cm can be observed in the images. We were able to compile valuable data from two sources: South Korea (Kang & Kim, 2016), and an event occurred in Colombia in 2015, as described below.

4.1. Data from South Korea

A dataset that includes hazard parameters and more specific structural data is provided by Kang & Kim. (2016). These authors used data collected and examined in detail from 25 buildings damaged in 11 events in South Korea. They provided the building sites' flow depths, velocities, and impact pressures. At the same time, they classified building types and building damage states according to the HAZUS system. We calculated momentum flux from the depth and velocity data they reported. Results for the URML buildings, the most abundant data (a group of 8 URM buildings between 1 and 2 stories, which reached complete damage), are shown in Table 25.

Table 25*Buildings Affected by Mass Movements in South Korea.*

Building N°	Flow depth (m)	Flow velocity (m/s)	Momentum flux (m³/s²)	Damage State
k-1	0.71	3.2	7.27	Complete
a-4	0.99	3.5	12.13	Complete
a-1	1.33	3.4	15.37	Complete
i-3	1.44	3.5	17.64	Complete
i-2	1.98	3.7	27.11	Complete
b-1	2.7	3.5	33.08	Complete
i-1	3.22	4.2	56.80	Complete
j-1	3.23	6.6	140.70	Complete

The approximate cumulative distribution function, using rank probability (taking the values of momentum flux, ordered from the lowest to the highest one, and dividing them by the number of values to obtain probabilities), is shown in Figure 36 (dots). A log-normal distribution, fragility curve, can be easily adjusted to the field data. The adjustment yields a median value of 24 m³/s² and a standard deviation equal to 1.0.

Figure 36 and Table 26 highlight a significant adjustment between fragility curves for complete damage in URML buildings obtained from field data in South Korea and for URM buildings in the current research. The median values of fragility curves are similar, ranging from 23 to 26 m³/s². These median values represent the thresholds at which a mass movement will start complete damage, i.e. the building resistance.

Although the S-type fragility curves proposed by Chen et al., (2021) do not match the fitted fragility curves with South Korean data and the functions proposed in this research, the complete damage state threshold is similar, Table 26.

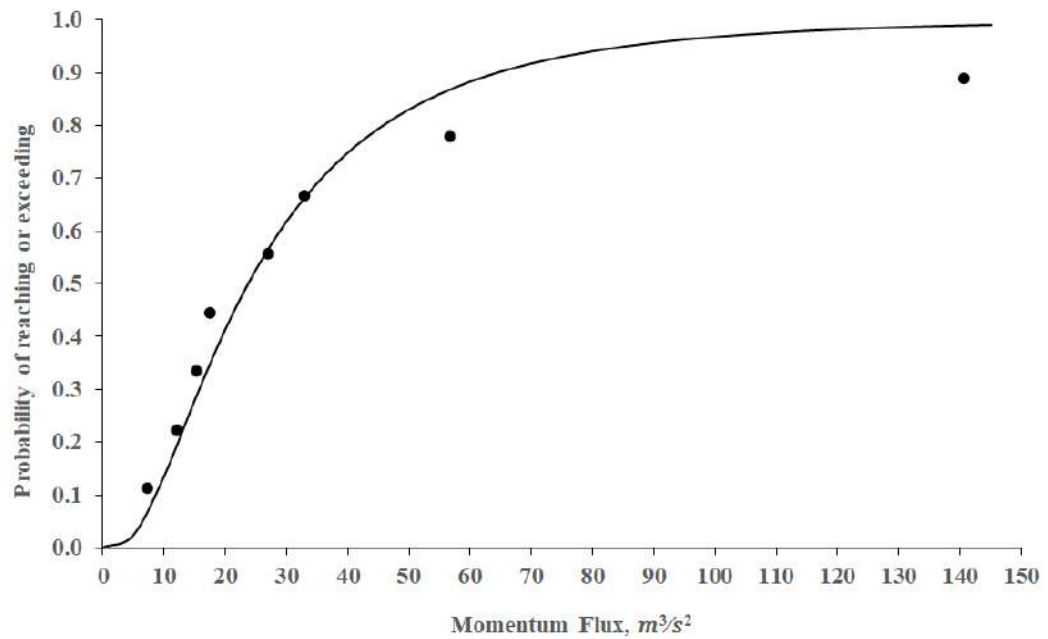
Table 26

Summary Parameters for Fragility Curves, Complete Damage, URM Buildings.

Type	Median $h v^2$ (m^3/s^2)	β_{DS}
	Complete damage	sdln
South Korea field data	24	1.0
URM fragility curve (current research), two stories, 20 cm wall thickness	23.79	0.80
China fragility curve	26	-

Figure 36

Fragility Curves for Complete Damage in URM Buildings for the Following 2 Cases: Field Data from South Korea (dots) and the Current Research Model (Solid Line), 2 Storeys, Average Thickness 20 cm.



4.2. Data from Salgar, Antioquia, Colombia.

On the early morning of May 18, 2015, heavy rains occurred in the upper basin of the Liboriana creek, Salgar municipality, Antioquia department, Colombia. These rains generated debris flows that were concentrated in the various channels and moved with great velocity to the lower part of the basin as a torrential flood. Along its course, the mass movements destroyed about 90 houses, killed 94 people, left 11 missing, and affected an estimated 1,397 people.

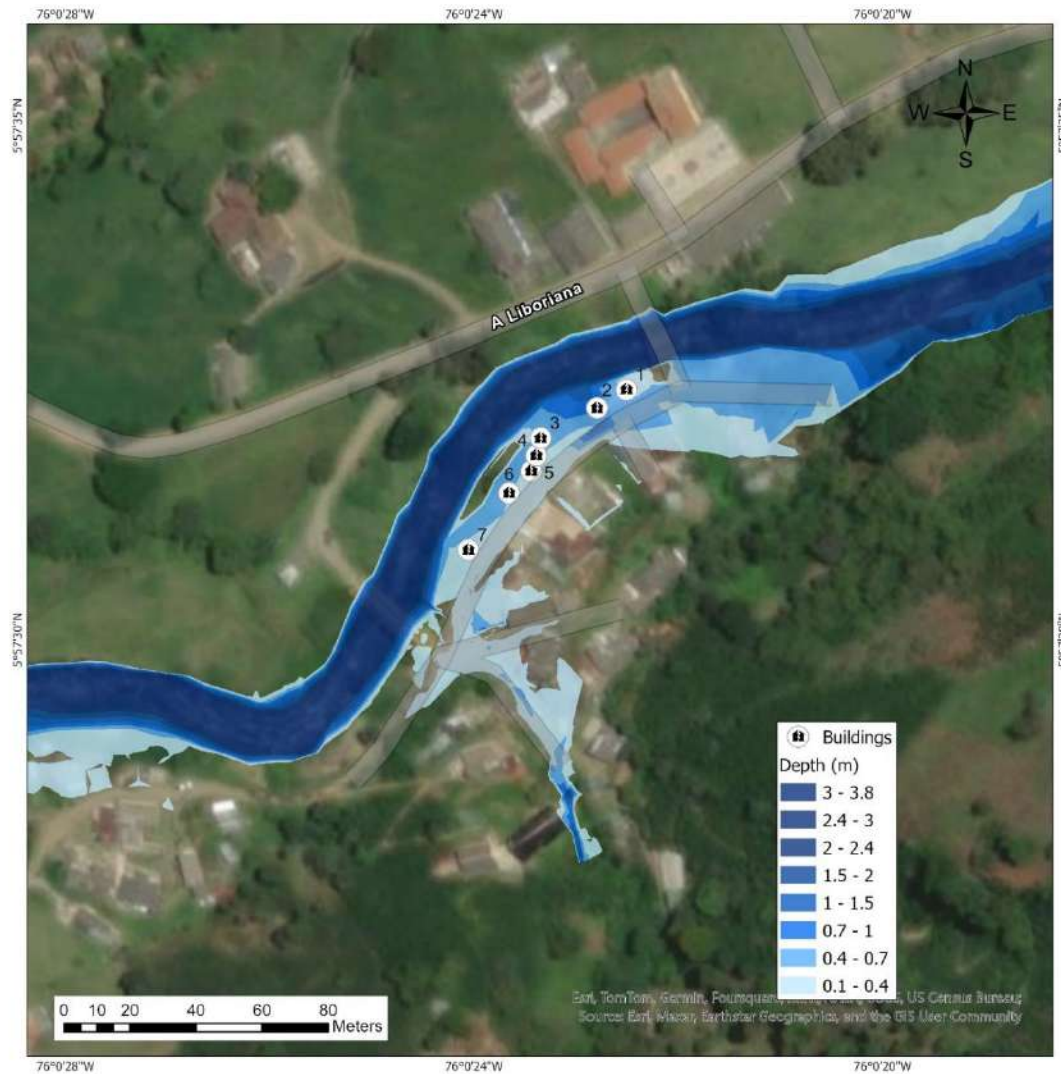
The potential impact of debris flow was modelled for a storm with a 300-year return period, increasing the flow by 40% to account for sediment input. Hydraulic transit modelling of peak flows in Liboriana Creek was carried out using the HEC-RAS version 6.5 program (Ackerman, 2002; Ford et al., 2002). The software requires input data on the boundary conditions, geometry or terrain, Manning's roughness value n , and design flow rates. It also solves the problem of water movement in two dimensions (2D) using finite elements. Flow depths are also calculated using the standard stepwise method for 1D modelling, and the model output is the free water surface associated with an imposed flow condition, which may vary along the channel.

The raster format model's height and flow velocity data (Figures 37 and 38) were used to calculate momentum flux values ($h v^2$).

It is important to note that the model was run on a recent topography (2023). According to photographic information before the 2015 event, the channel was not as deep at that time as it is today, which may reduce the modelled momentum flux values compared to the original event.

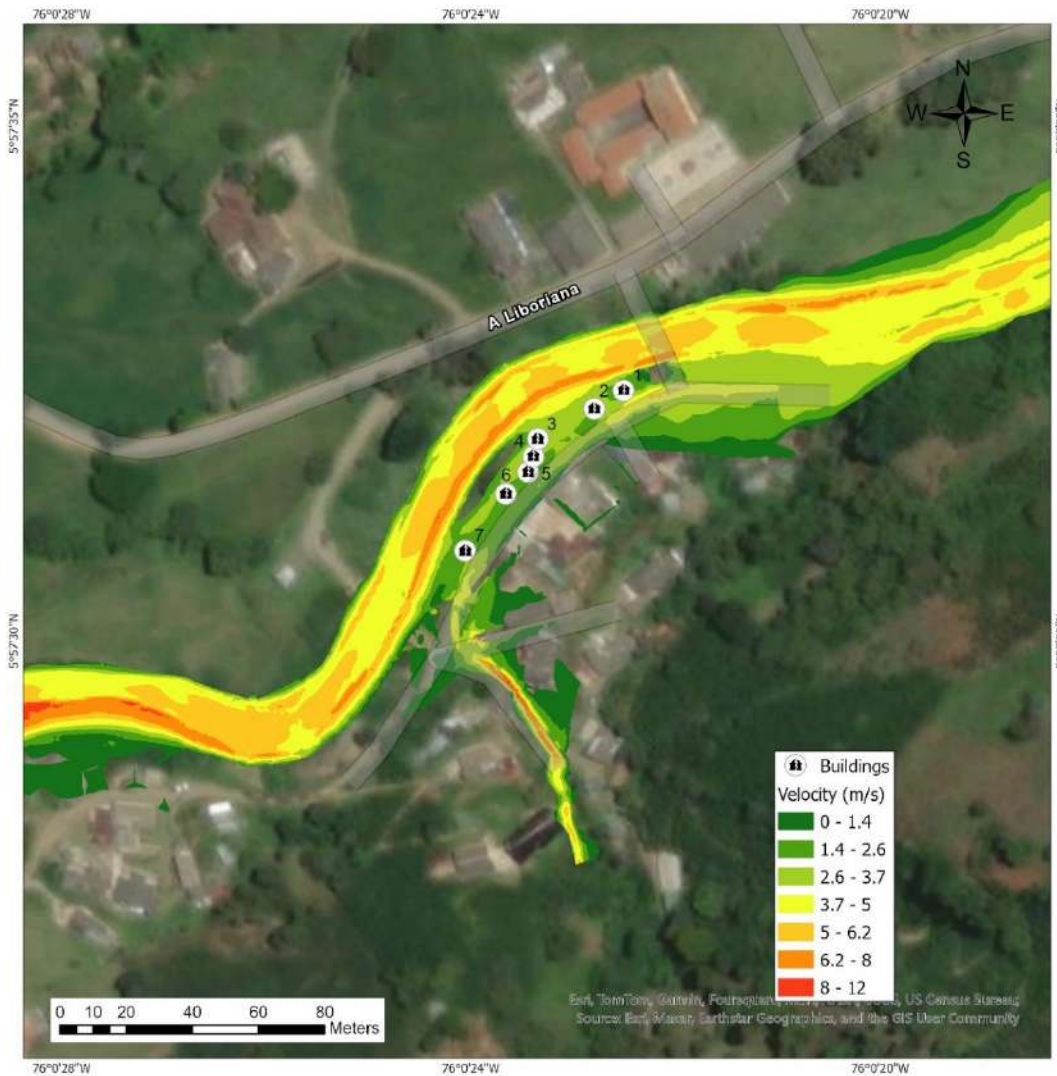
1 **Figure 37**

2 *Depth for the 300-Year Return Period Debris Flow Free Water Surface.*



4 **Figure 38**

5 *Velocity for the 300-Year Return Period Debris Flow Free Water Surface.*



The description of the buildings' damage was taken from the report carried out by the Administrative Department of the Disaster Prevention, Response and Recovery System of Antioquia, DAPARD (2015), during different visits between 18 and 22 March 2015. In the report, the destroyed or damaged houses in the different sectors of the municipality along the Liboriana Creek are located with coordinates. Report photographs also document the state of the buildings, Figures 39 to 41. Therefore, it was possible to locate the affected houses and assign them the maximum momentum flux values, Table 27.

Figure 39

The Margarita Village Centre Affected by the Torrential Floods.



Note. The figure shows one of the areas where the destroyed houses were located. (DAPARD, 2015)

Figure 40

Salgar, Antioquia, Colombia, Affected by the Torrential Floods.



Note. The figure shows the location of a URM single-storey dwelling with complete damage. Only the foundation remains.

Figure 41

(a) Salgar, Antioquia, Colombia, Avenue 24, Extensive Damage, URM Dwelling.; (b) Sector Gualanday, Salgar, Antioquia, Colombia, Extensive Damage, URM Dwelling (DAPARD, 2015).

**(a)****(b)**

Table 27

Buildings Affected by Mass Movements in Salgar, Antioquia, Colombia. Data from DAPARD.

Building N°	Latitude	Longitude	Flow depth (m)	Flow velocity (m/s)	Momentum flux (m^3/s^2)	Damage State
7	5.958577	-76.006682	0.45	2.52	2.84	Complete
5	5.95879	-76.006511	0.44	3.06	4.14	Complete
4	5.958832	-76.006497	0.57	2.95	4.98	Complete
6	5.958731	-76.006571	0.64	2.87	5.26	Complete
3	5.958879	-76.006485	0.7	3.44	8.25	Complete
2	5.95896	-76.006332	1.11	3.57	14.1	Complete
1	5.95901	-76.006252	1.19	3.76	16.88	Complete

Note. The URM buildings were single-story with an average wall thickness of 12 cm and a lightweight asbestos cement roof.

Figures 42 and 43 show the approximate cumulative distribution function using rank statistics. A good fit can be seen with the proposed fragility curves for 1-story URM buildings with an average wall thickness of 12 cm, Tables 28 and 29.

Figure 42

Fragility Curves for Complete Damage in URM Buildings ($h v^2$ Intensity), for 2 Cases: Field Data from Salgar (dots), and the Current Research Model (Solid Line).

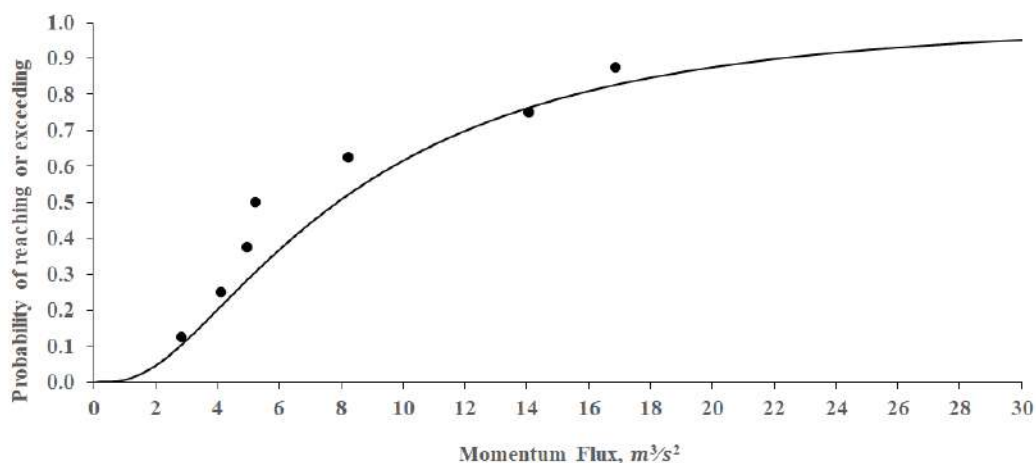
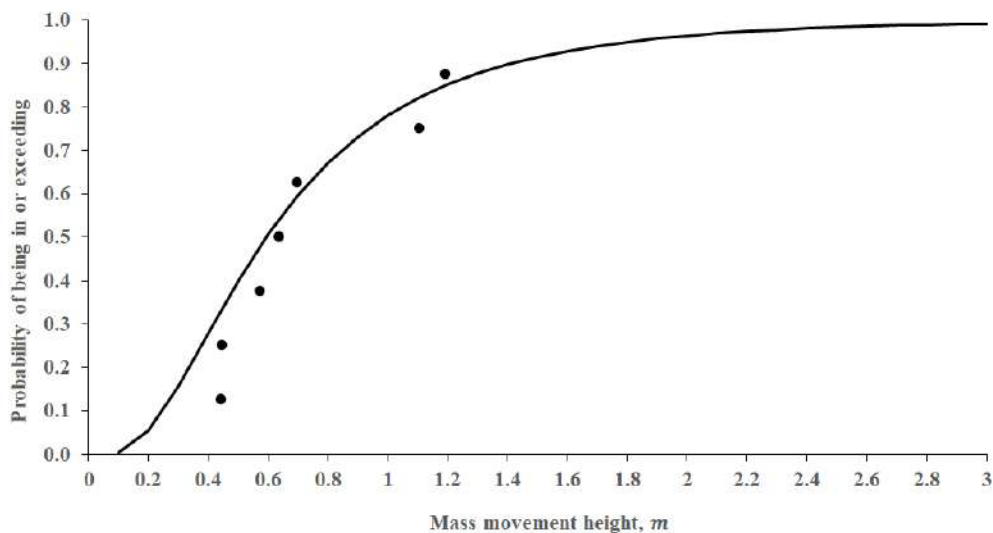


Figure 43

Fragility Curves for Complete Damage in URM Buildings (h Intensity), for 2 Cases: Field Data from Salgar (dots), and the Current Research Model (Solid Line).

**Table 28**

Summary Parameters for Fragility Curves, Complete Damage, URM Buildings ($h v^2$ Intensity).

Type	Median $h v^2$ (m^3/s^2)	β_{DS}
	Complete damage	sdln
Salgar, Antioquia, Colombia, data adjustment	7.0	0.86
URM fragility curve (current research), single story, 12 cm wall thickness	7.89	0.80

Table 29

Summary Parameters for Fragility Curves, Complete Damage, URM Buildings (h Intensity).

Type	Median h (m)	β_{DS}
	Complete damage	sdln
Salgar, Antioquia, Colombia data adjustment	0.70	0.55
URM fragility curve (current research), single story, 12 cm wall thickness	0.59	0.67

4.3. Remarks

Model results highlight a good agreement with damage field data. In the case of 2-storey thick walls (~ 20 cm) URM buildings, complete damage threshold values range between 23 and $26 \text{ m}^3/\text{s}^2$ for South Korea, Wenchuan in China, and this research, Table 26 and Figure 36. On the other side, data from Salgar, Antioquia, Colombia indicate, for 12 cm wall thickness URM buildings, a slightly low, but not that different, threshold momentum flux field value of $7 \text{ m}^3/\text{s}^2$ as compared to $7.89 \text{ m}^3/\text{s}^2$ for the model, Table 28 and Figure 42. Using the mass movement height, h , as an intensity parameter, no significant difference is noted between field data and model results, i.e., 0.70 m against 0.59 m respectively, Table 29 and Figure 43. The main differences between field data and model results can be attributed, e.g., to the fact that before the 2015 event, the Liboria Creek channel had different topographic sections than it has today, which may change the hydraulic modelled values compared to those of the original event. Another parameter that deserves some comment is the standard deviation. Previous research indicated log standard deviations in the order of 0.90 for momentum flux intensity thresholds in URM buildings (Prieto, et al., 2018), while lower values, e.g. 0.80, were obtained in this research, Tables 18 and 19. This decrease in epistemic uncertainty is believed to be due to the three-dimensional and structural model used in this research.

5. Mass Movements Risk Assessment

Risk is the expected degree of loss due to a potentially hazardous phenomenon (D. Cruden & Varnes, 1996). In other words, it can be defined as a measure of the likelihood and severity of an event to life, health, property, or the environment over a specified period. Therefore, it is determined by the interaction of hazard, vulnerability, and exposure (UNISDR, 2017).

5.1. Risk Loss Assessment Concepts

If the damage state thresholds and their uncertainties are available, estimating the structural repair costs is possible (Hazu–MH 2.1: Technical Manual, 2012). In the previous chapter, mass movement fragility curves were calculated for masonry buildings, considering the thickness of the walls and the number of storeys. Therefore, it is feasible to analyse the risk loss for this type of building.

The following equation calculates the risk loss (R) for each building.

$$R = \sum_{i=1}^n P_i \cdot L_i \cdot C_b \quad (53)$$

P_i is the probability that the building will reach damage state i , L_i is the loss per cent corresponding to the damage level i , C_b is the building's replacement cost, and n is the number of damage states examined.

The corresponding loss percentages are shown in Table 30 (M. Chen et al., 2021; Jakob et al., 2012). These same loss rates are reported in research such as M. Chen et al. (2024).

Table 30

Loss Percent Corresponding to Damage State i .

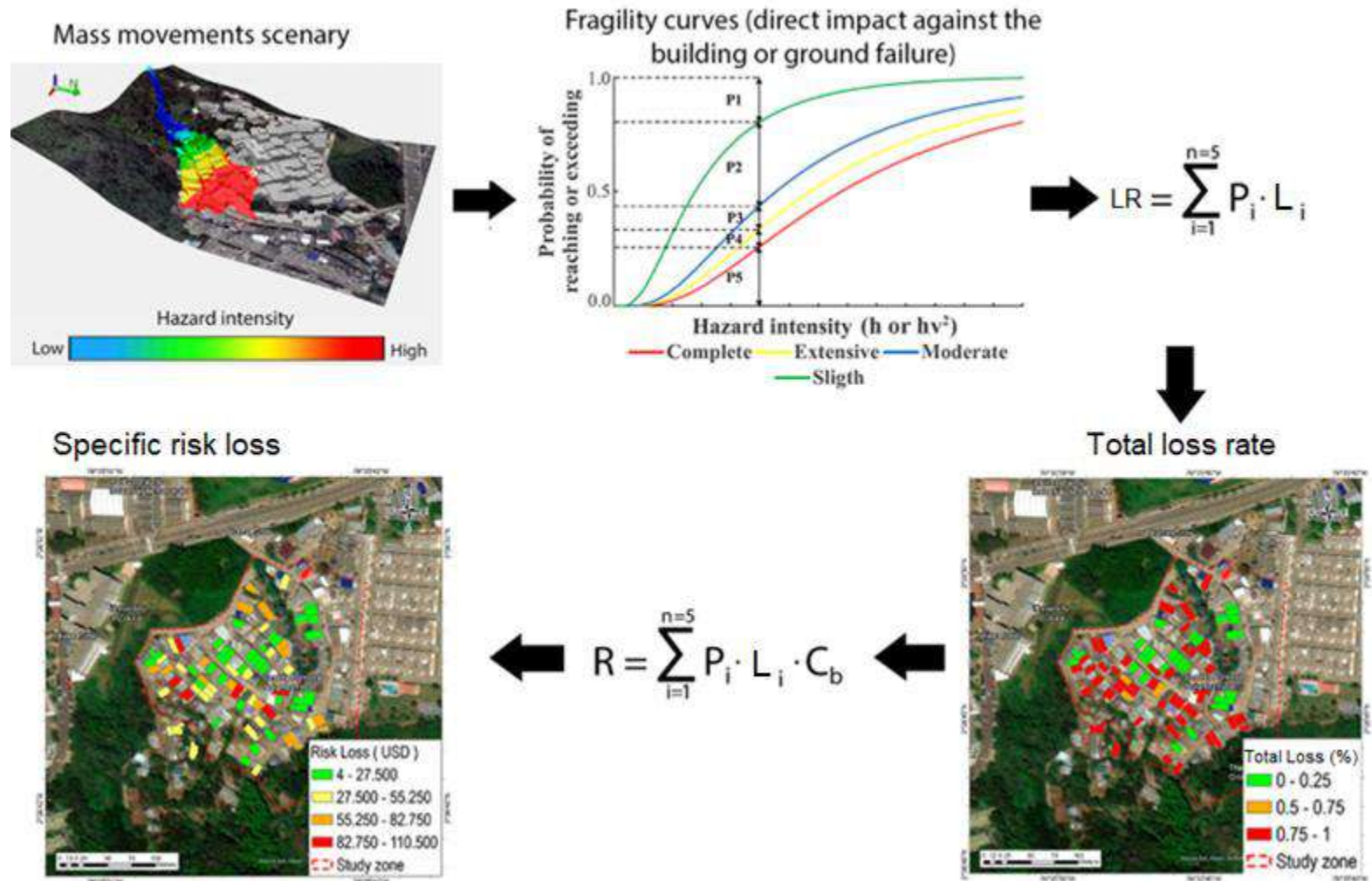
Damage State	L_i (%)
Slight	0 - 25
Moderate	25 - 75
Extensive	75 - 100
Complete	100

Table 6.1, which corresponds to the chapter on risks due to mass movements in the Colombian methodological guide (Ávila et al., 2016), deals with similar percentages of losses.

A description of the mass movement risk assessment procedure is given in Figure 44.

Figure 44

Flowchart of the Proposed Method for Carrying out the Risk Analysis of Buildings Due to Mass Movements.



5.2. Application Case: Risk Analysis Vereda Pomona Neighborhood, Popayan, Colombia

In Colombia and other underdeveloped countries, social, political, and economic difficulties mean that many human settlements are still vulnerable to events such as mass movements. Anthropogenic situations such as the deforestation of tutelary hills, informal construction on these sites, and construction on riverbanks, conditioned by the high rainfall and seismicity of our territories, exacerbate the community's risk of this type of event.

Popayán city is no exception. The mass movement vulnerability study carried out for the municipality by the Colombian Geological Service (Ruíz Peña et al., 2019), provides information on the hazard, which can be used to develop a practical exercise for calculating the risk using the methodology proposed in this research.

The Vereda Pomona neighbourhood, built informally near the Three Crosses Hill, was used as a pilot exercise. This place is northeast of the Popayán municipality, between the University of Cauca engineering campus and the Altos de Tulcán condominium on Avenue 2. It is not a legally constituted neighbourhood; the area of this zone is 3.55 ha, Figures 45 and 46. The population is around 900, and there are 178 buildings, most of which are clay-masonry (URM, CM). 1-storey (95) and 2-storey (57) buildings, up to 3-storey (21) or 4-storey (5). Lot dimensions of approximately 6 metres by 12 metres, generally built in their entirety and taking areas destined for pavements in most cases. The predominant type of use is residential use, with 95.31% of the buildings, followed by multipurpose use with 3.13%, which includes mixed residential and commercial use, and in lesser representation is commercial use, with 1.56%.

The areas of high hazard due to mass movements cover an area of 0.61 ha, corresponding to 17.2% of the total area of the study zone. The mass movements hazard zones cover an area of

0.76 ha, corresponding to 21.4% of the total area of the study zone. The low mass movements hazard zones cover an area of 2.18 ha, corresponding to 61.4% of the total area of the study zone, Figure 47. Low-hazard areas close to high- and medium-hazard areas may be affected depending on the travel distances of possible movements.

Figure 45

Vereda Pomona Neighbourhood, Popayan, Cauca, Colombia

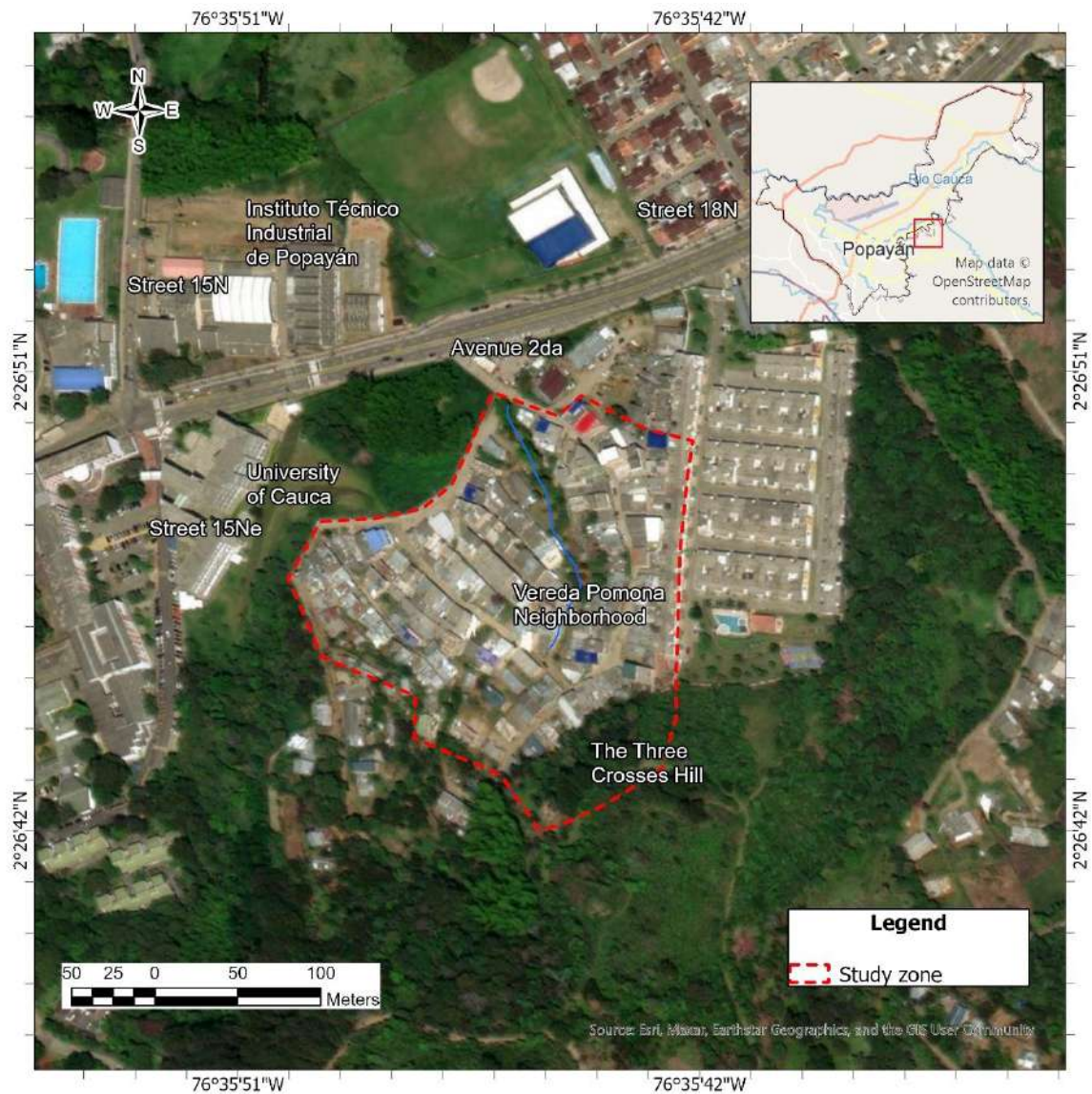


Figure 46

Three-Dimensional Image of the Vereda Pomona Neighbourhood, Popayan, Cauca, Colombia.

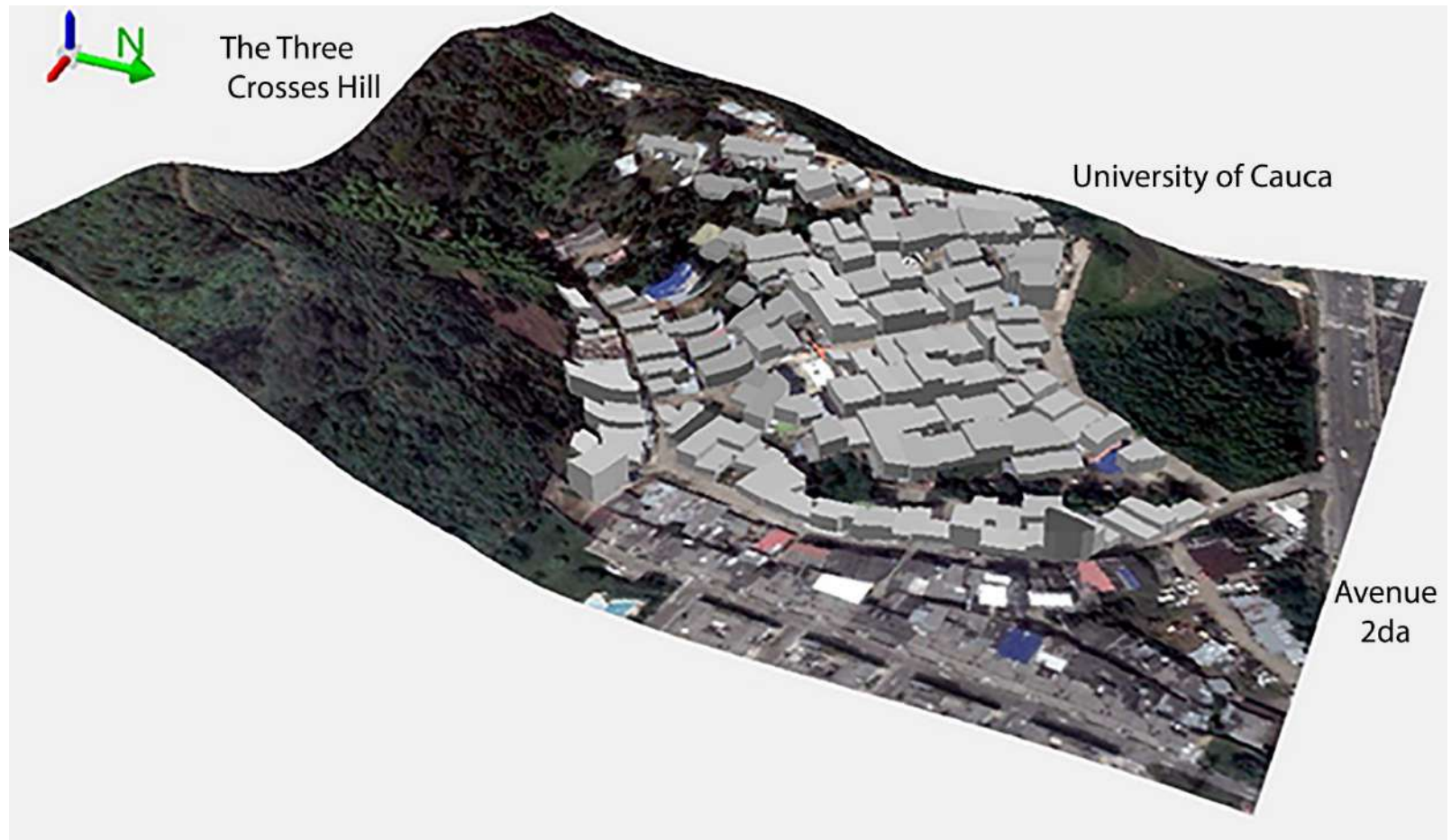
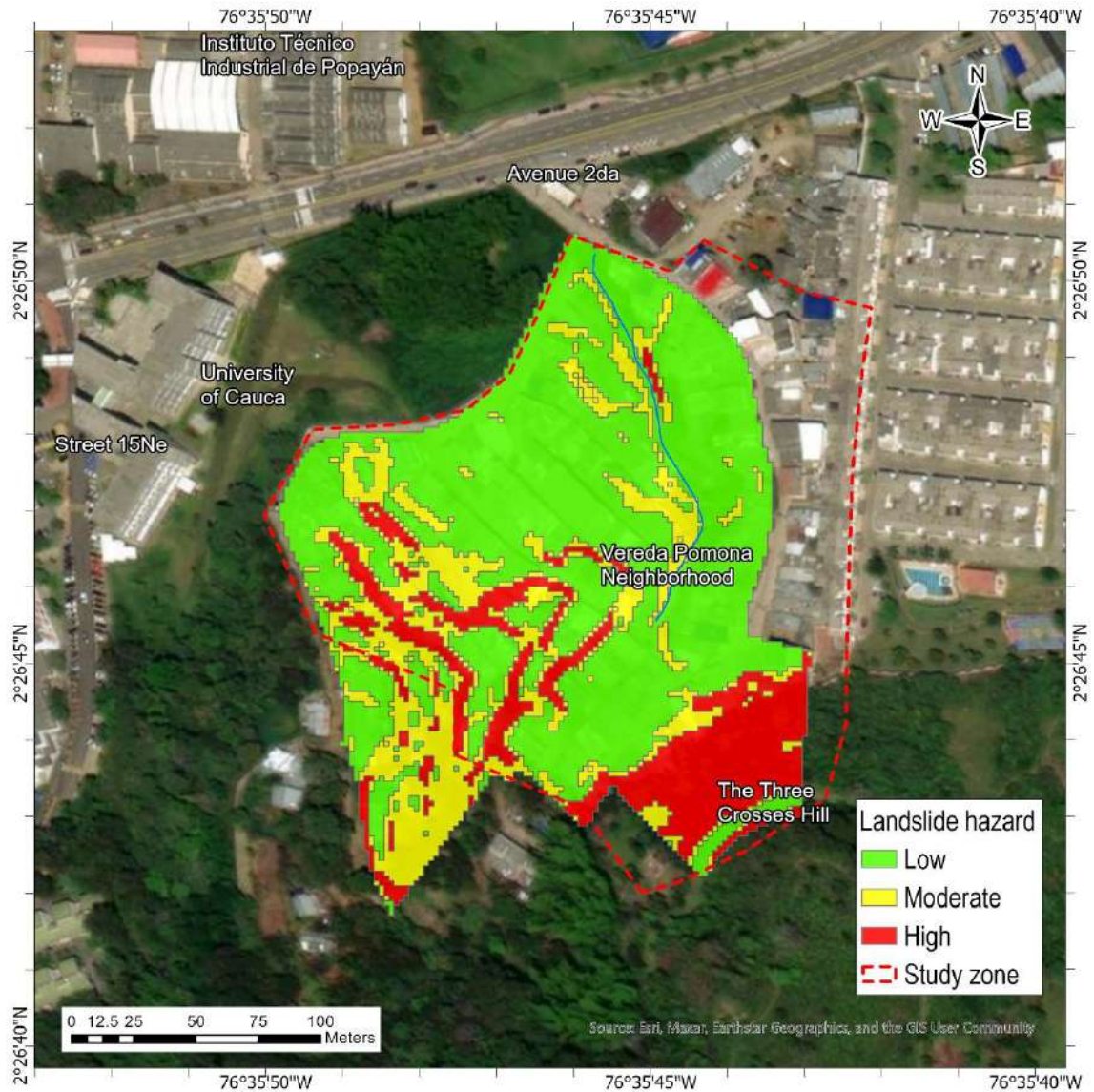


Figure 47

Ground Deformation model, (High Slope Areas), Vereda Pomona Neighbourhood, Popayán, Cauca, Colombia.



Note. Information obtained from the study of vulnerability and risk due to mass movements carried out by the Colombian Geological Service.

The sliding block method calculates the landslide intensity, Eqs. (13) y (55), Figure 48.

The first equation determines the flow velocity along section AB, and the second along section

BC. Each variable used in these calculations was extracted from the municipality mass movement study. Flow travel distances were obtained according to Eq, (54).

$$d = \left(1 - \frac{\tan\phi}{\tan\beta}\right) \frac{h'}{\tan\phi_p} \quad (54)$$

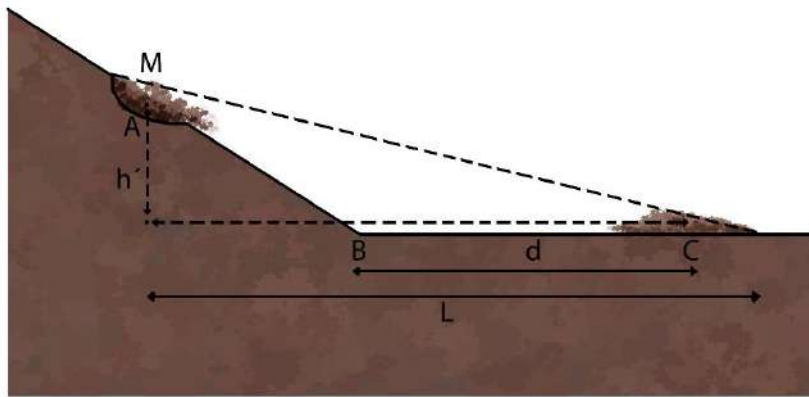
$$V_{BC} = \sqrt{V_B^2 - 2g \tan\phi_p d} \quad (55)$$

Where: β is the terrain slope. ϕ is the internal friction angle on the slope (A-B). ϕ_p is the internal friction angle on the flat zone (B-C). The profiles analysed to obtain the intensity of the mass movement, in terms of velocity, height and travel distances of the landslide, are shown in Figure 49.

With the above information, it is possible to quantify the hazard magnitude in terms of momentum flux ($h v^2$) and height (h) at the time of possible impact for each building.

Figure 48

Movement of a Block on a Curved Surface with Constant Frictional Resistance.



The next step is to quantify the buildings' fragility (URM and CM buildings); due to this research, structural fragility curves are obtained, considering two phenomena: the probability of damage by direct impact on the structure or the possibility of permanent ground deformation.

Thresholds have been calculated for different damage states with their uncertainty, considering the intensity in terms of height (h) or momentum flux ($h v^2$). Physical fragility is analysed according to the number of storeys and wall thickness.

Given the phenomenon's intensity and the building's fragility, it is possible to find the probability of being in a specific damage state (slight, moderate, extensive, and complete). Finally, the loss risk is calculated by relating the intensity and the structures' damage state as specified in Eq. (53). This analysis is georeferenced in Figures 50 and 51.

1 **Figure 49**

2 *Profiles for the Study of Physical Vulnerability and Risk Due to Mass Movements, Vereda Pomona Neighbourhood.*

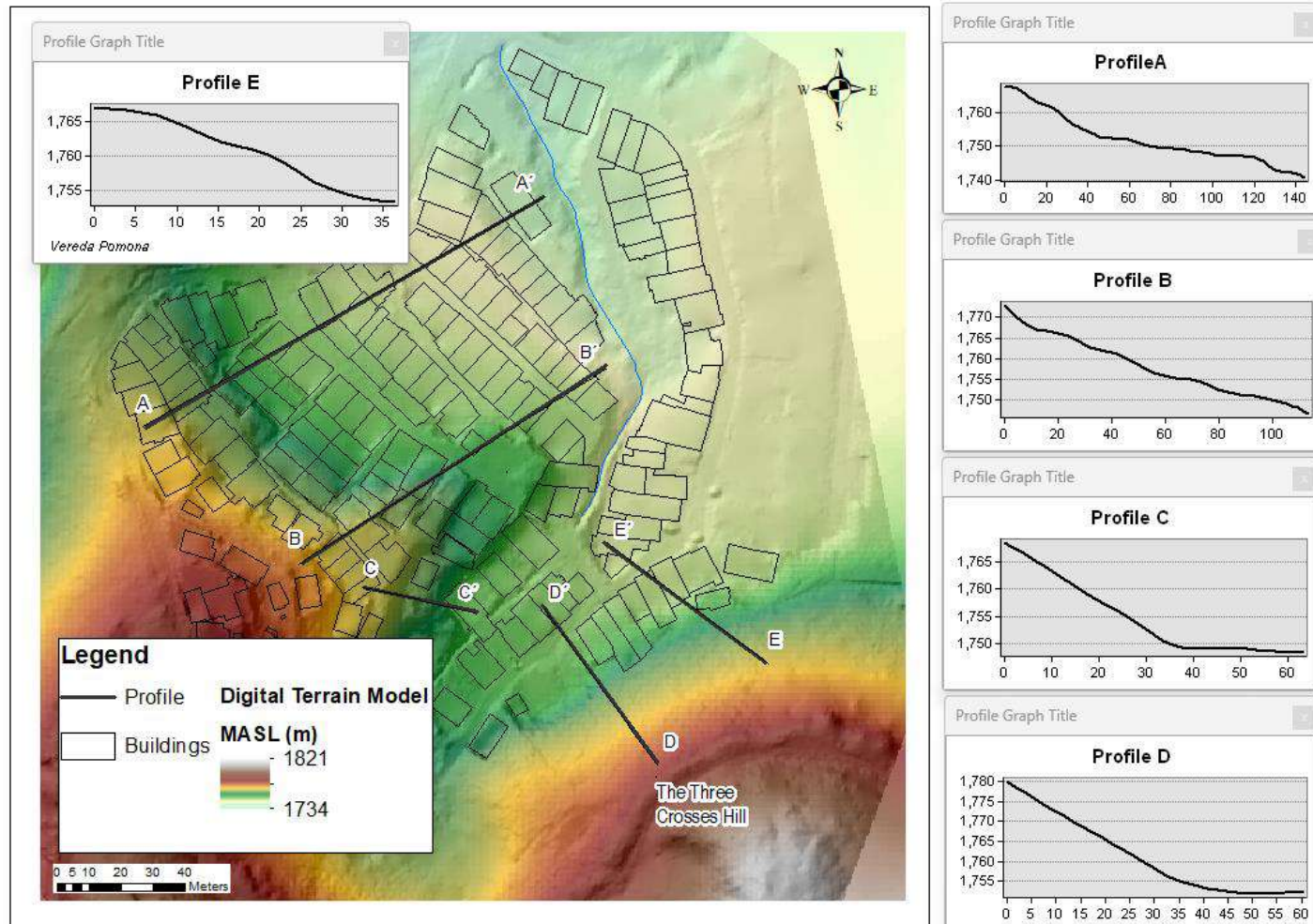


Figure 50

Total Physical Losses Percentage Due to Mass Movements, Vereda Pomona Neighbourhood.

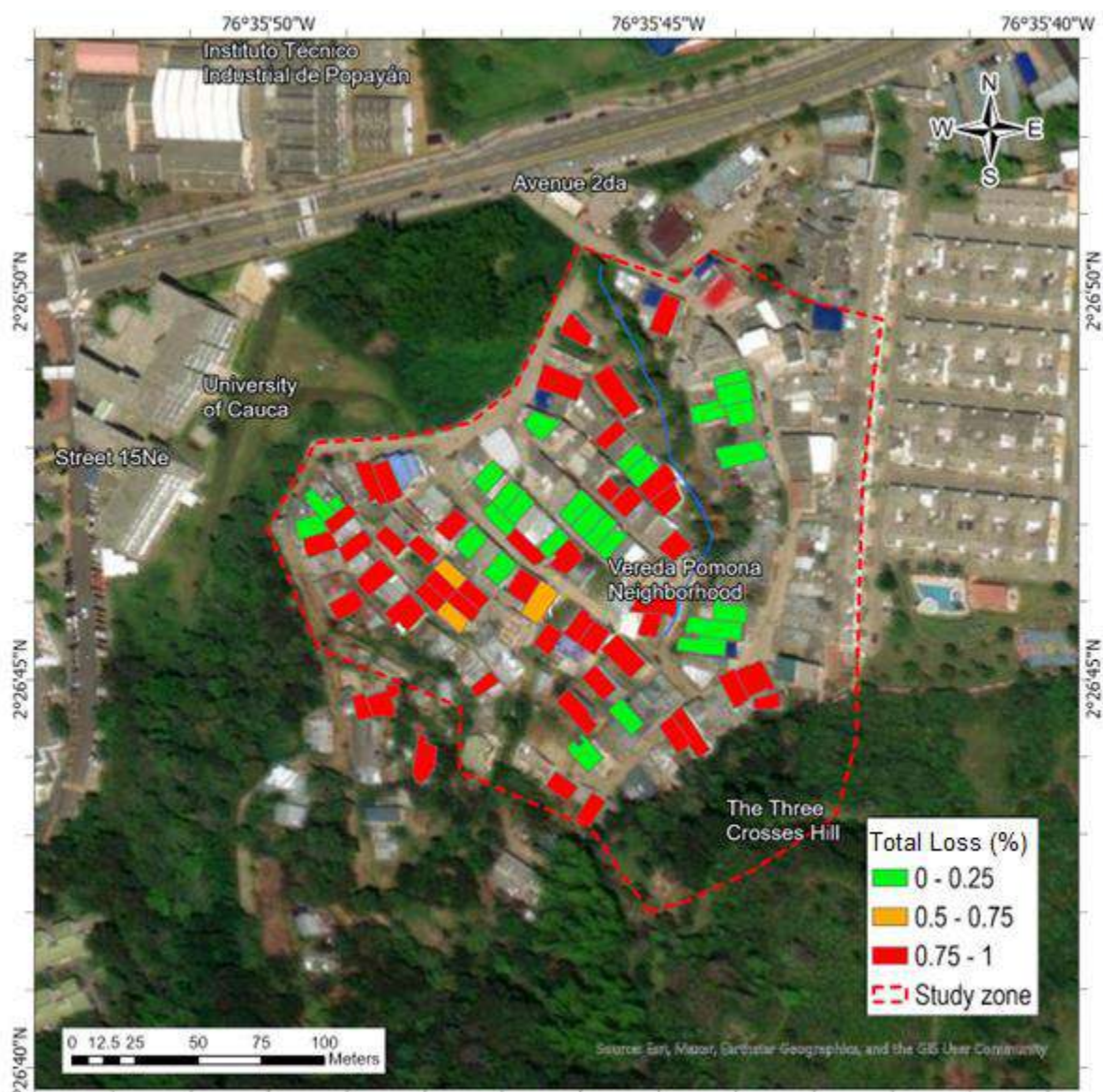
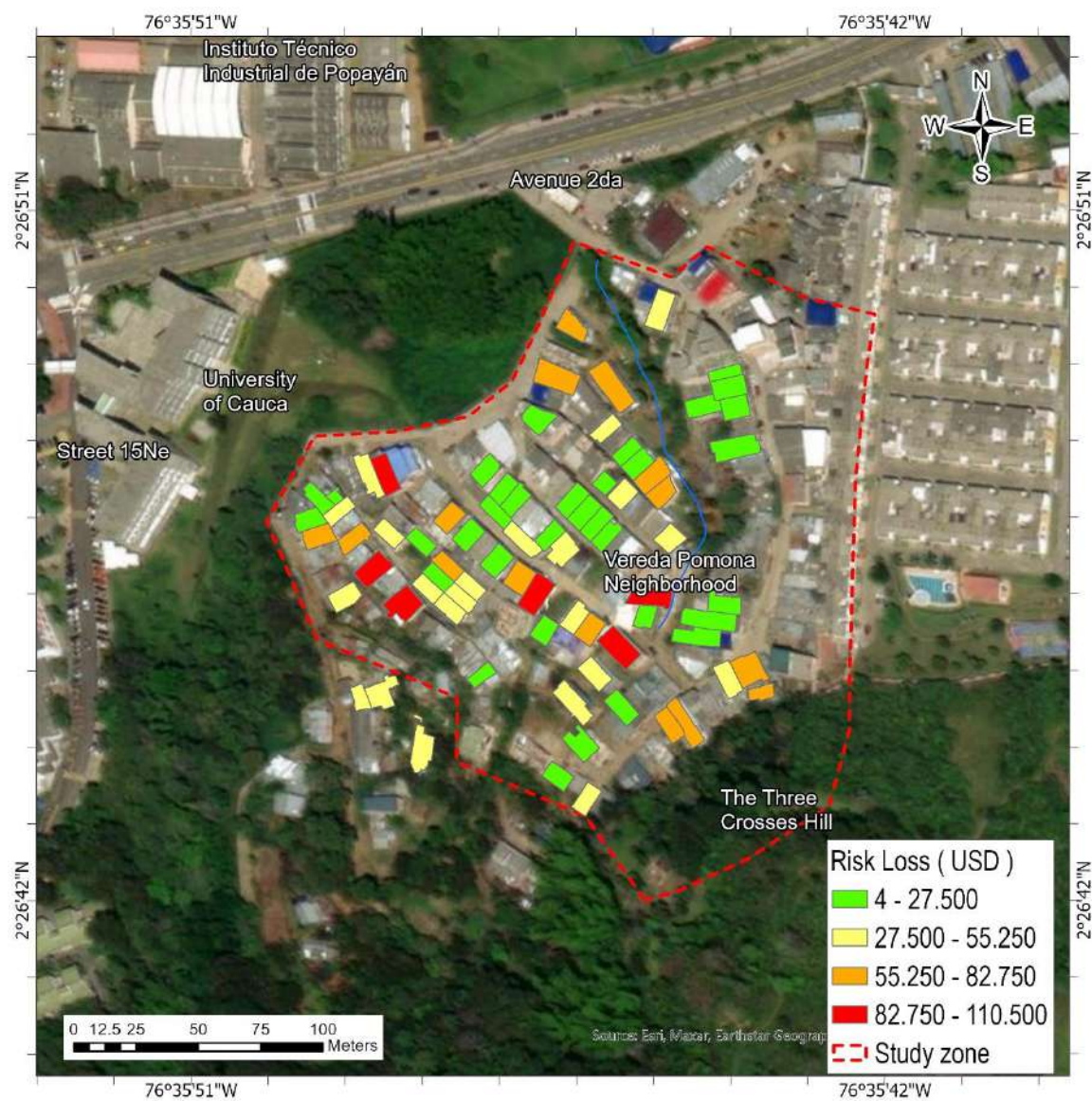


Figure 51

Total Economic Loss Given a Specific Event by Mass Movements, Vereda Pomona Neighbourhood.



5.3. Remarks

To calculate the risk loss, it is possible to relate the intensity to the physical vulnerability for different damage states (light, moderate, extensive, complete) using the buildings' fragility curves due to mass movements. This physical and probabilistic procedure has not yet been addressed in the methodological guidelines because the nature of the methods used (qualitative, semi-quantitative, empirical) does not allow it.

For the calculation of total losses (%), the loss percentages for each damage state shown in Table 30, were used, as they are physically related to each damage state. These factors are similar to the damage loss values in the Colombian Methodological Guide for Hazard, Vulnerability and Risk Studies in Table 6-1.

Loss percentages for each damage state and the building's replacement cost are sensitive variables in the risk calculation that need to be studied from different perspectives, e.g., community, government, and insurance.

Calculating the loss risk allows the community, government agencies, and insurance companies to make more reliable decisions. This is vital for land-use planning, which determines a region's present and future.

6. Conclusions

Literature on structural vulnerability due to mass movements in cities and towns was reviewed. These studies recognise the need to quantify vulnerability using fragility curves. It is estimated, especially in underdeveloped countries, that at least 70% of their buildings are masonry (URM and CM structures), so calculating damage state thresholds in these typologies using a clear and computationally low-cost methodology is essential for risk reduction.

The global structural vulnerability due to mass movements is quantified by considering the shear damage mechanisms of URM and CM buildings. An efficient procedure is presented for calculating damage state thresholds and their uncertainties. This is useful in analysing vulnerability, balancing computational precision, time cost, and easy implementation. The methodology includes the probability of damage due to the direct impact of the mass movement against buildings and probable permanent ground deformations, which will allow a better understanding of the phenomenon and the possible combination of probabilities.

Thresholds due to mass movements are provided for light, moderate, extensive, and complete damage states in URM and CM buildings of one to four storeys for two typical wall thicknesses (12 and 20 cm). Fragility curves are provided as a function of momentum flux and mass movements height.

Field data indicating intensity parameters and buildings' damage states to validate vulnerability assessment procedures are scarce. However, a database of mass movements in South Korea and field data from an event in Colombia fit well with the fragility curves given by the model.

The given methodology for clay brick masonry buildings contributes to a better understanding of physical vulnerability against mass movements events. The procedure is

computationally efficient, easy to implement, and adaptable to multi-hazard analysis, e.g., earthquakes, permanent ground failures, Natech events, etc. This can effectively help reducing risks in less developed areas in an uncertain climate change environment.

A structural fragility model is presented, which allows us to obtain a generic risk model due to mass movement phenomena. Risk losses are determined for a Popayán, Cauca, Colombia neighbourhood as a practical exercise. The results of this research are expected to be innovative, helpful to the engineering community, and beneficial to society.

7. Future Research Topics

The following main topics are mentioned:

- ✓ Infrastructure's physical vulnerability assessment (other types of buildings, roads, bridges, viaducts, lifelines, etc.) against mass movements using physical and probabilistic models.
- ✓ Dynamic contact tridimensional models (mass movements/structures) where the impact force varies with time (time history models).
- ✓ A machine learning-based analysis for predicting fragility curve parameters of buildings against mass movements.
- ✓ Vulnerability and risk assessment based on multi-hazard analysis (mass movements, floods, earthquakes, Natech events, etc.).
- ✓ Investigations (compilations) of mass movement events than providing information on hazard intensity (depth, velocity and momentum flow), exposure zones of structures, classification of structures, damage status (including foundation failure), physical and mechanical characteristics of structures, foundation soil, etc.
- ✓ Study loss per cent corresponding to each damage state and infrastructures' replacement cost from different perspectives (community, government agencies, insurance companies, etc.).
- ✓ Minimum considerations in designing and constructing retention structures to prevent and reduce the risk of mass movements, adapted to the Andean countries.
- ✓ Conduct social vulnerability studies, especially in underdeveloped countries, to assess vulnerability to environmental hazards and deepen governmental, social, academic, and research efforts to formulate timely risk reduction plans.

8. Appendix

Appendix A

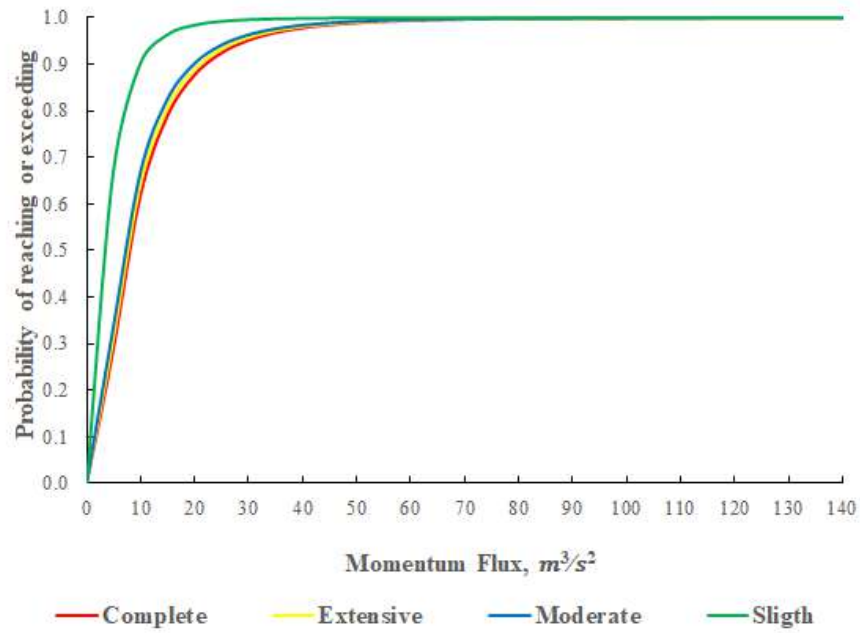


Figure A1. Fragility curve for URM buildings 1 storey. Wall thickness average 12 cm. Momentum flux intensity

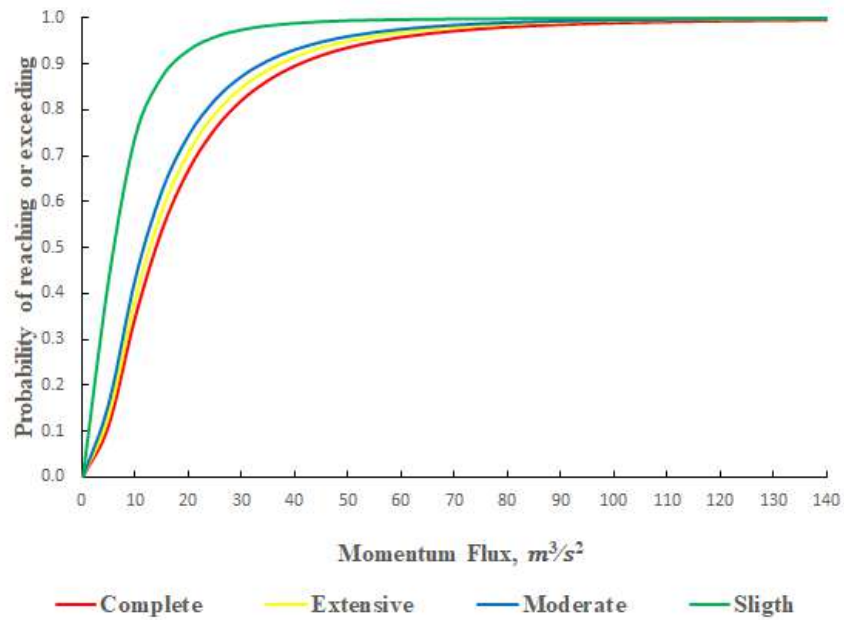


Figure A2. Fragility curve for URM buildings 1 storey. Wall thickness average 20 cm. Momentum flux intensity

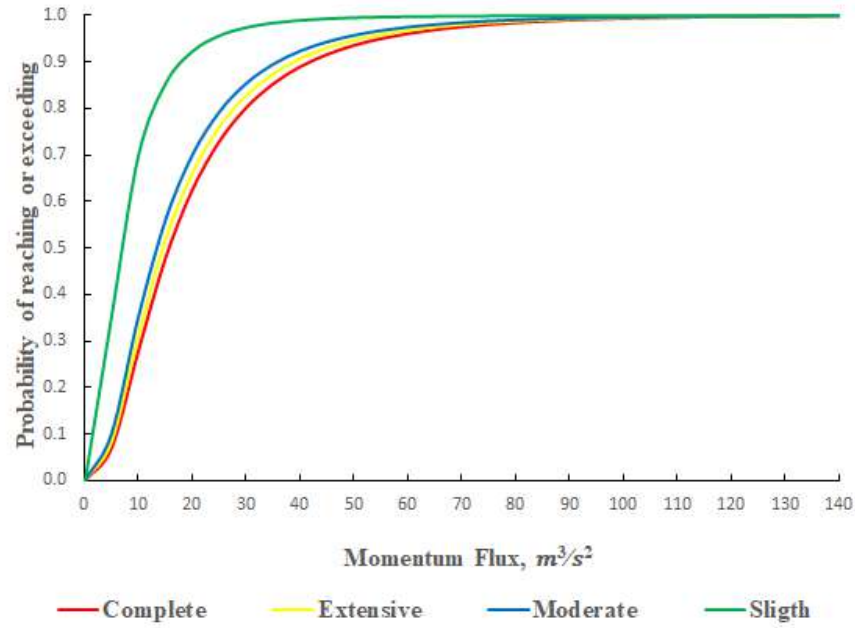


Figure A3. Fragility curve for URM buildings 2 storeys. Wall thickness average 12 cm. Momentum flux intensity

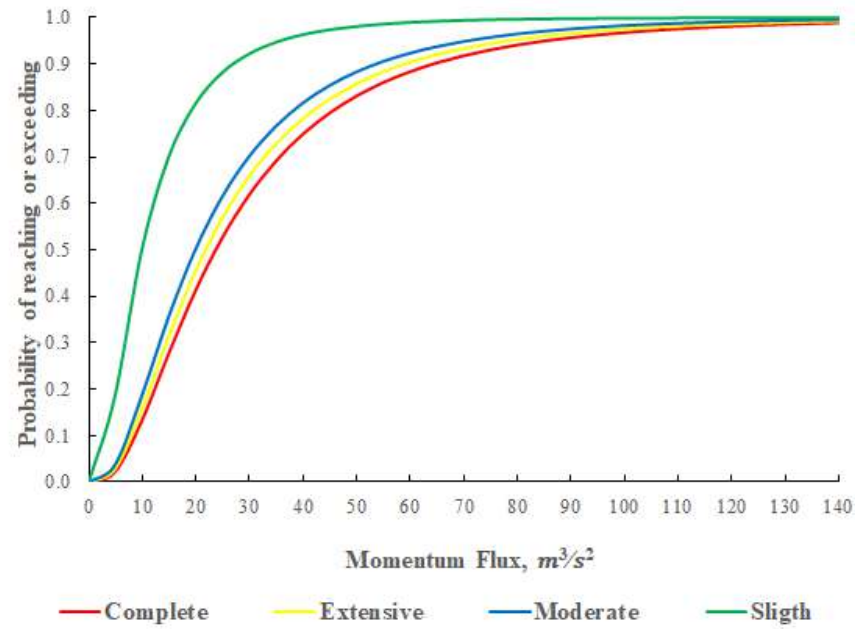


Figure A4. Fragility curve for URM buildings 2 storeys. Wall thickness average 20 cm. Momentum flux intensity

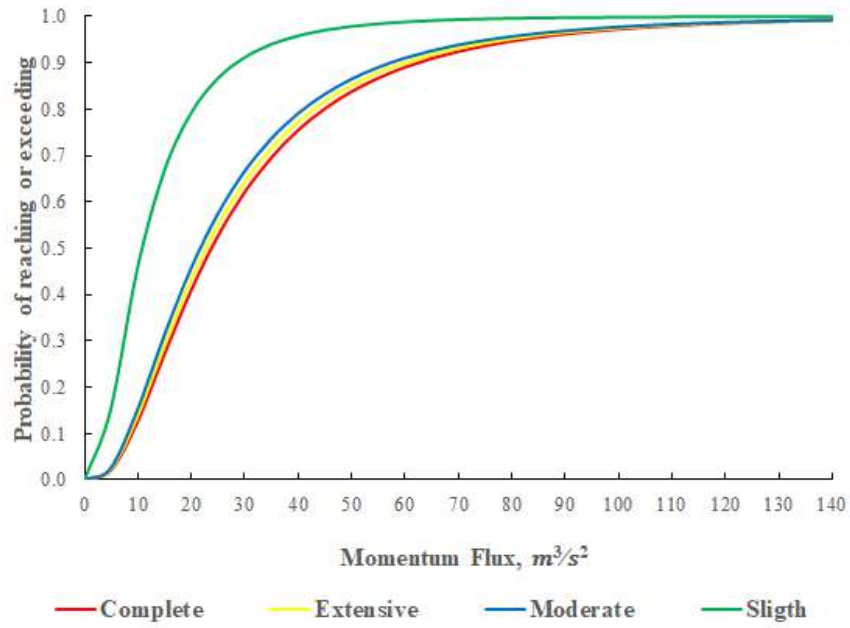


Figure A5. Fragility curve for URM buildings 3 storeys. Wall thickness average 12 cm. Momentum flux intensity

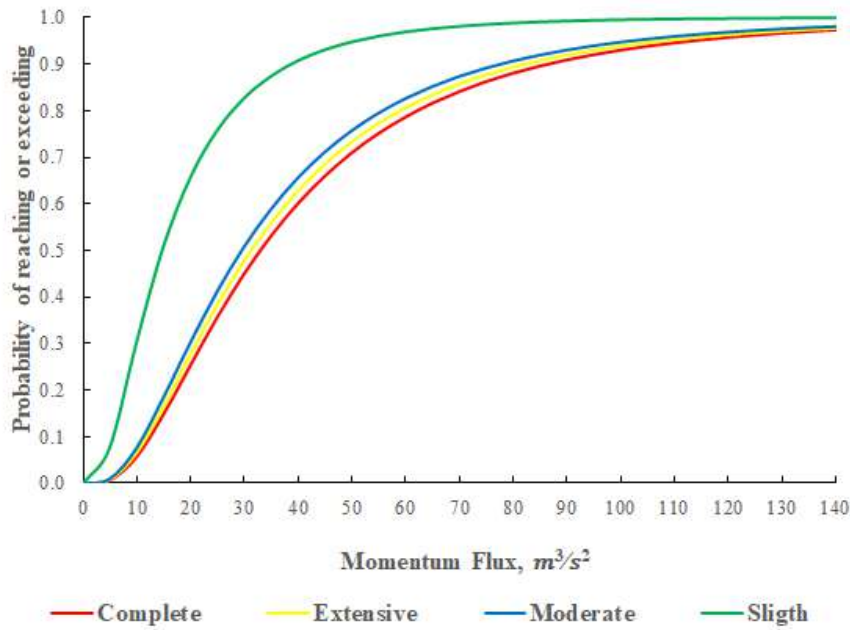


Figure A6. Fragility curve for URM buildings 3 storeys. Wall thickness average 20 cm. Momentum flux intensity

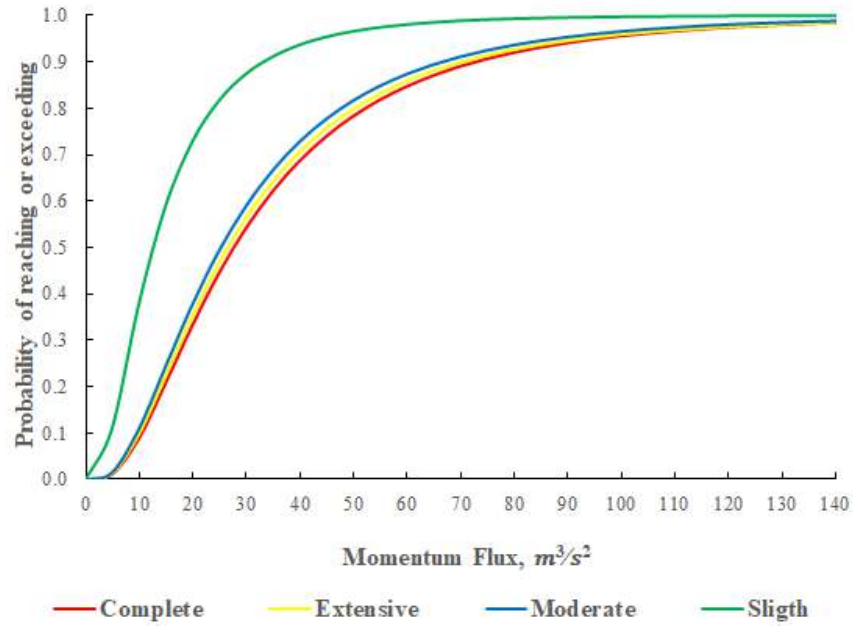


Figure A7. Fragility curve for URM buildings 4 storeys. Wall thickness average 12 cm. Momentum flux intensity

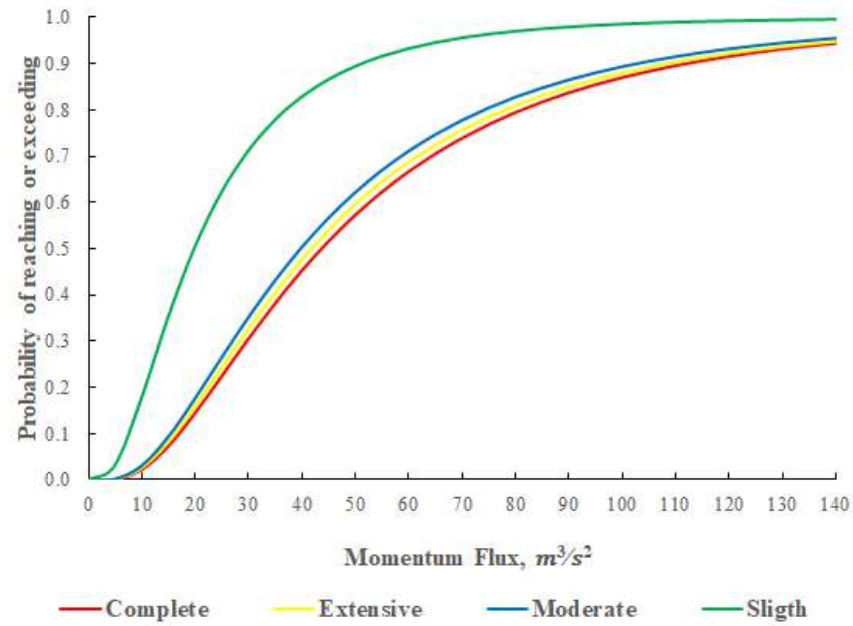


Figure A8. Fragility curve for URM buildings 4 storeys. Wall thickness average 20 cm. Momentum flux intensity

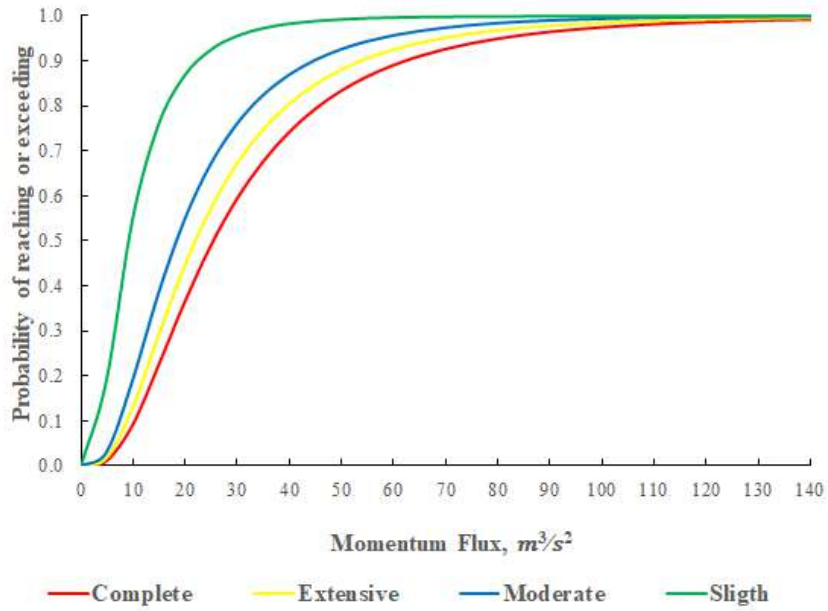


Figure A9. Fragility curve for CM buildings 1 storey. Wall thickness average 12 cm. Momentum flux intensity

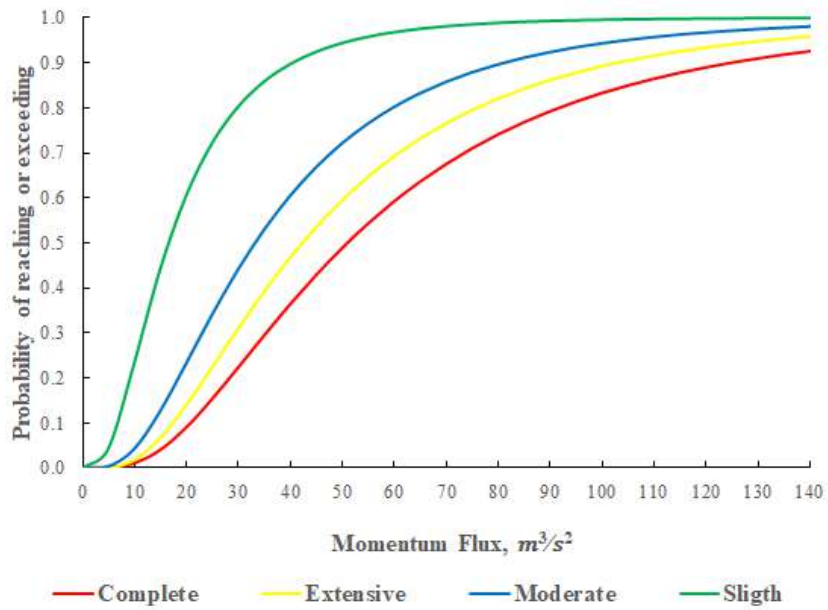


Figure A10. Fragility curve for CM buildings 1 storey. Wall thickness average 20 cm. Momentum flux intensity

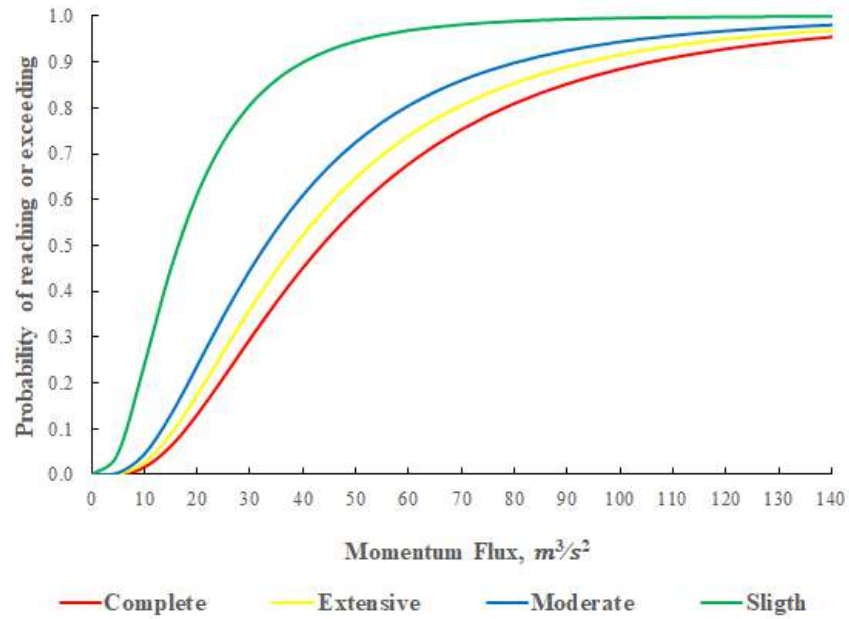


Figure A11. Fragility curve for CM buildings 2 storeys. Wall thickness average 12 cm. Momentum flux intensity

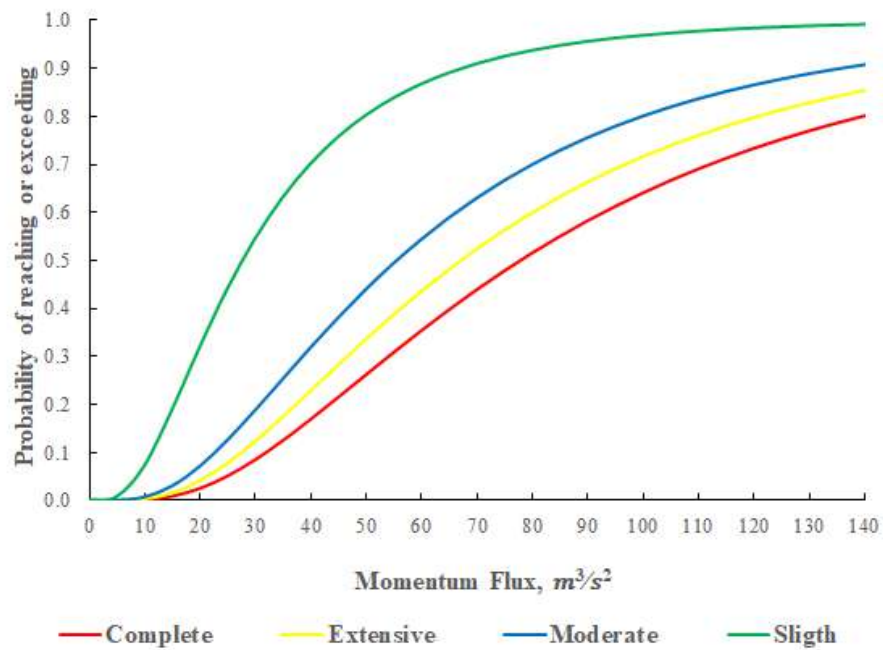


Figure A12. Fragility curve for CM buildings 2 storeys. Wall thickness average 20 cm. Momentum flux intensity

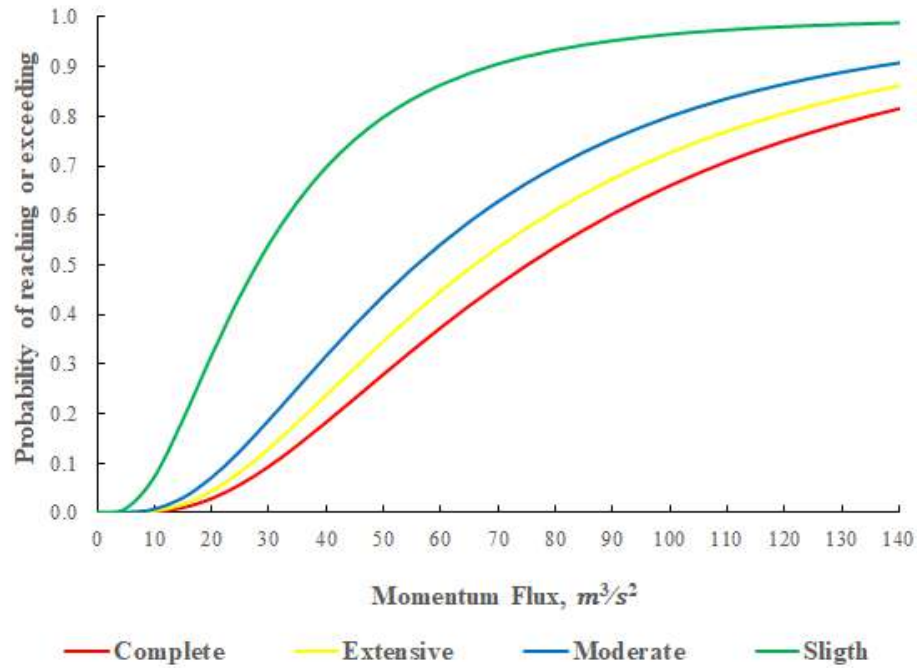


Figure A13. Fragility curve for CM buildings 3 storeys. Wall thickness average 12 cm. Momentum flux intensity

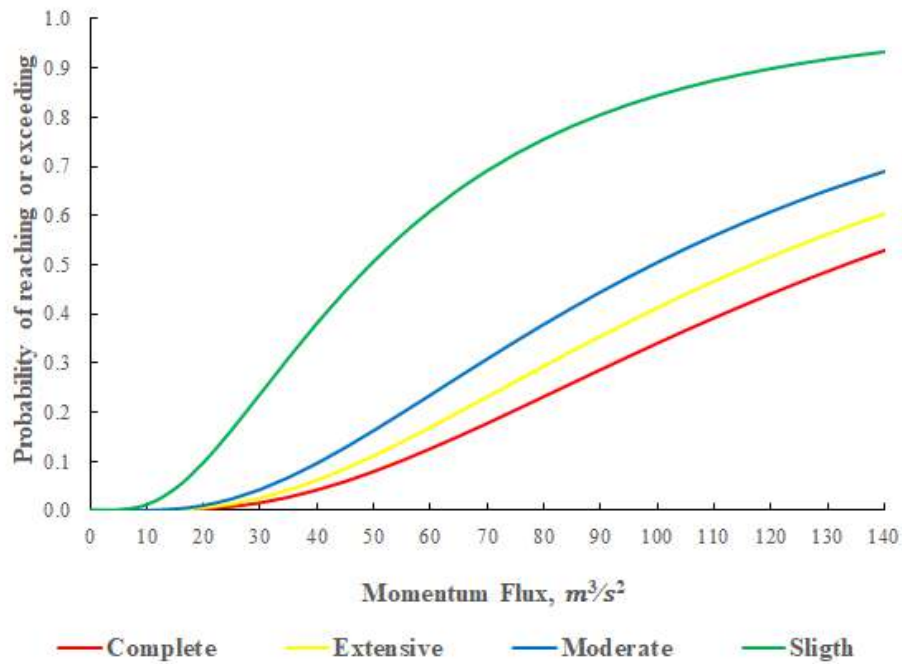


Figure A14. Fragility curve for CM buildings 3 storeys. Wall thickness average 20 cm. Momentum flux intensity

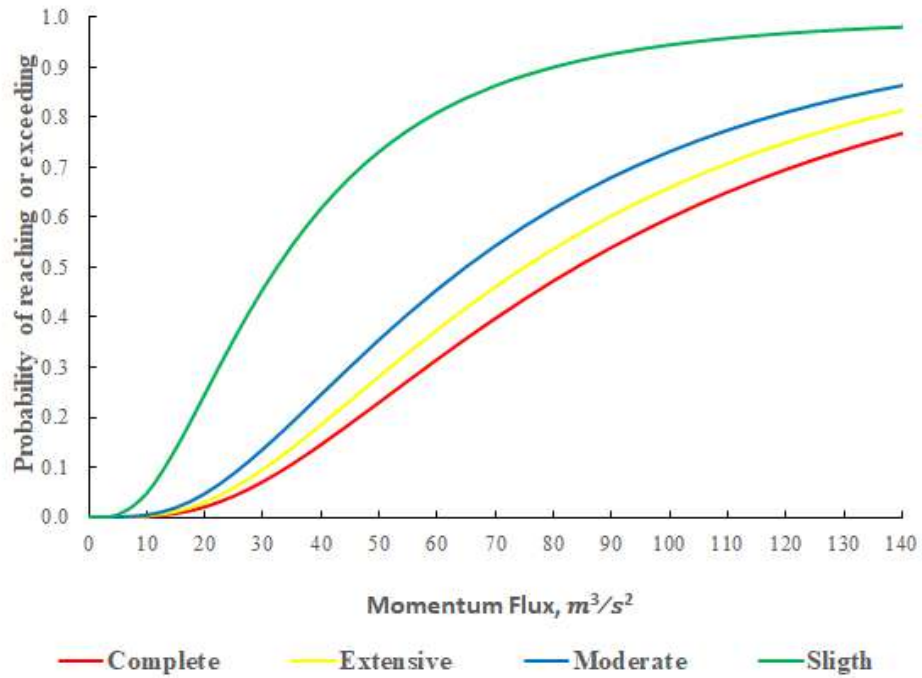


Figure A15. Fragility curve for CM buildings 4 storeys. Wall thickness average 12 cm. Momentum flux intensity

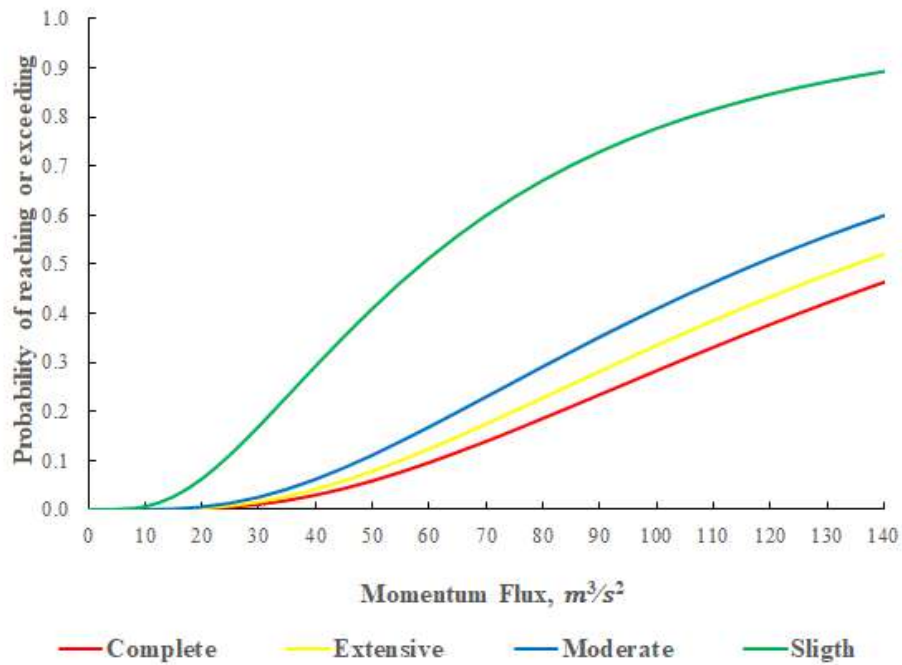


Figure A16. Fragility curve for CM buildings 4 storeys. Wall thickness average 20 cm. Momentum flux intensity

Appendix B

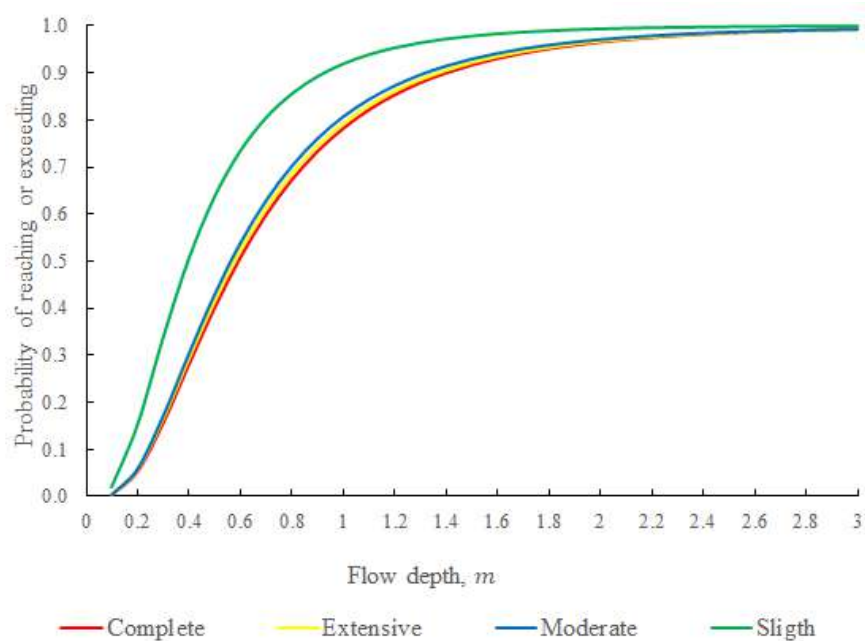


Figure B1. Fragility curve for URM buildings 1 storey. Wall thickness averages 12 cm. Flow depth intensity

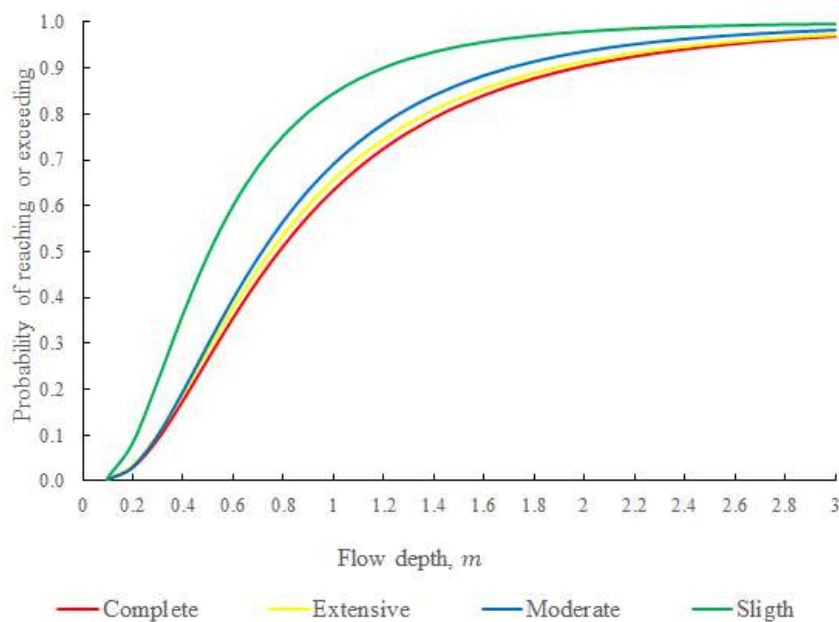


Figure B2. Fragility curve for URM buildings 1 storey. Wall thickness averages 20 cm. Flow depth intensity

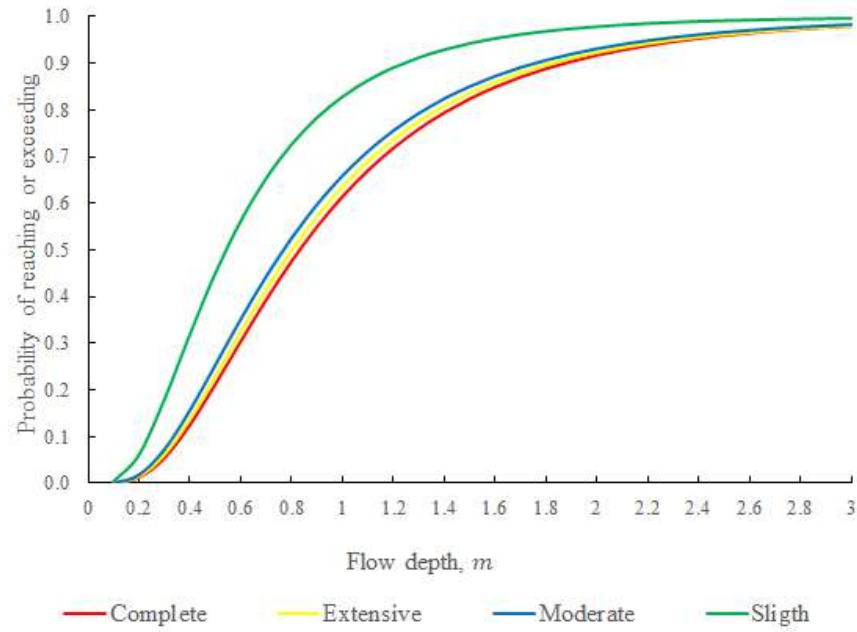


Figure B3. Fragility curve for URM buildings 2 storeys. Wall thickness averages 12 cm. Flow depth intensity

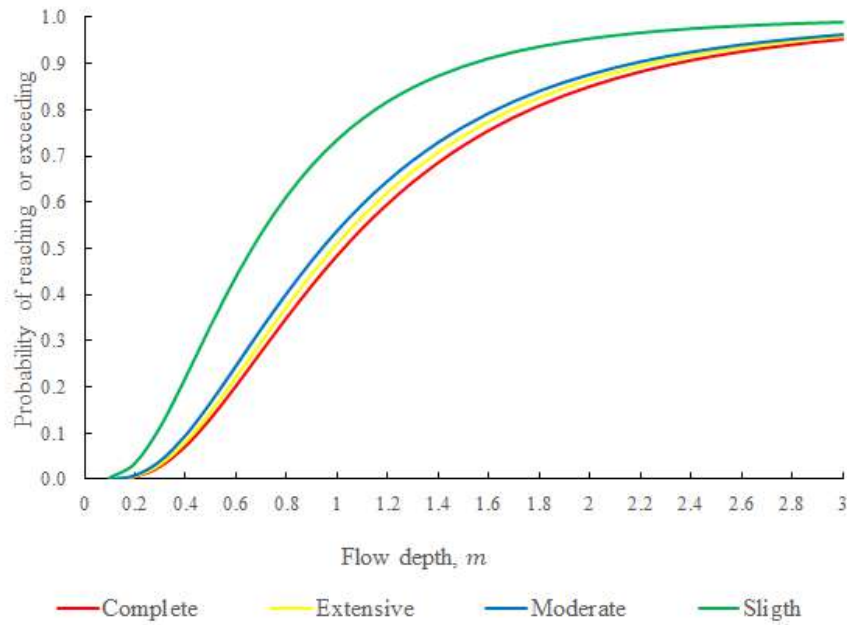


Figure B4. Fragility curve for URM buildings 2 storeys. Wall thickness averages 20 cm. Flow depth intensity

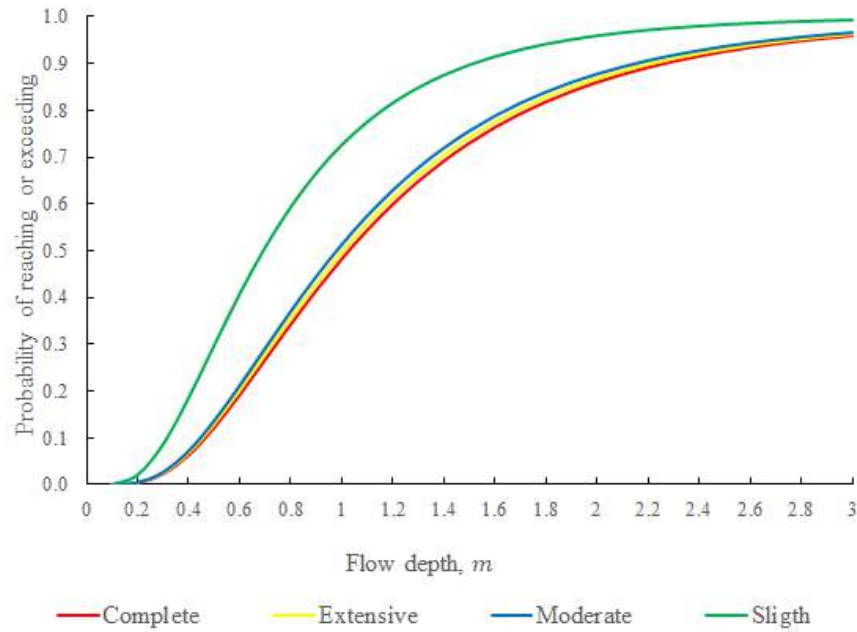


Figure B5. Fragility curve for URM buildings 3 storeys. Wall thickness averages 12 cm. Flow depth intensity

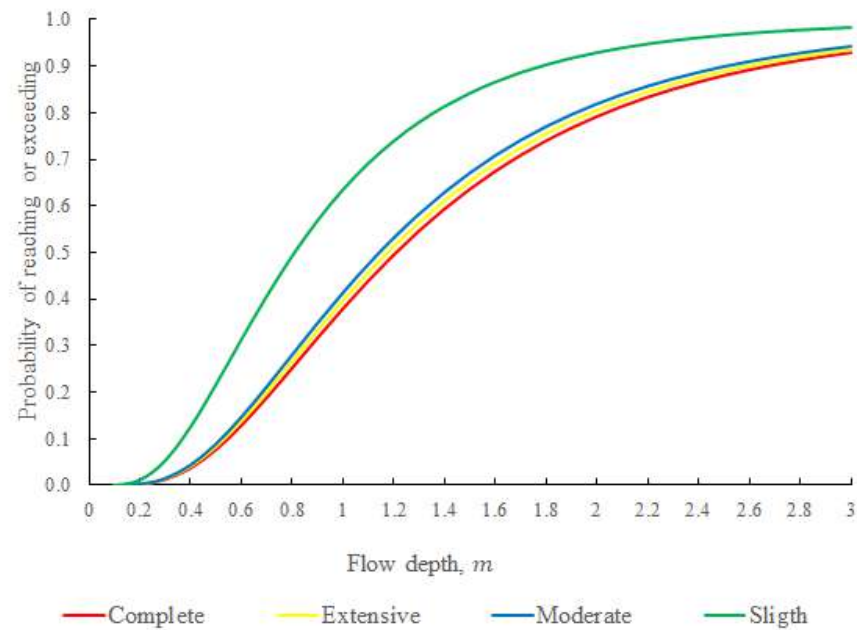


Figure B6. Fragility curve for URM buildings 3 storeys. Wall thickness averages 20 cm. Flow depth intensity

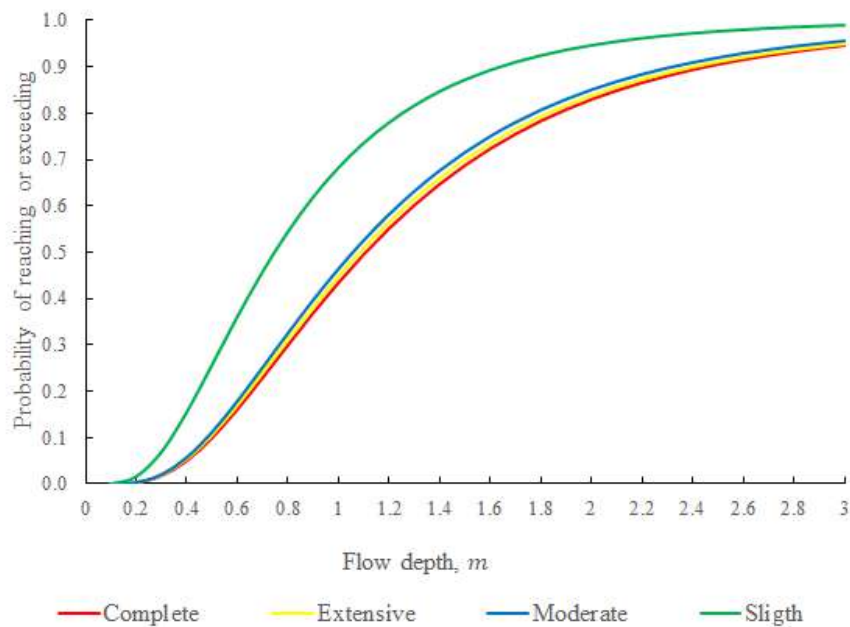


Figure B7. Fragility curve for URM buildings 4 storeys. Wall thickness averages 12 cm. Flow depth intensity

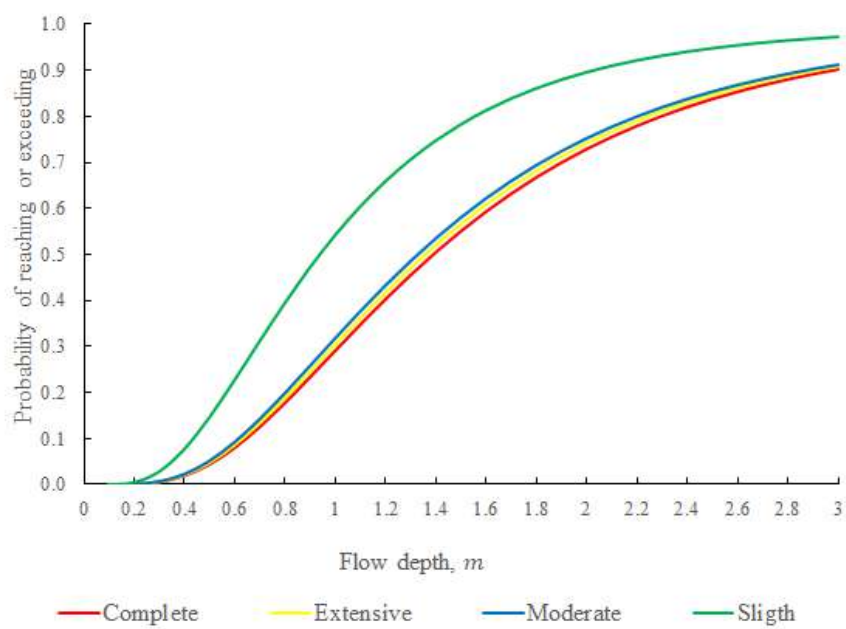


Figure B8. Fragility curve for URM buildings 4 storeys. Wall thickness averages 20 cm. Flow depth intensity

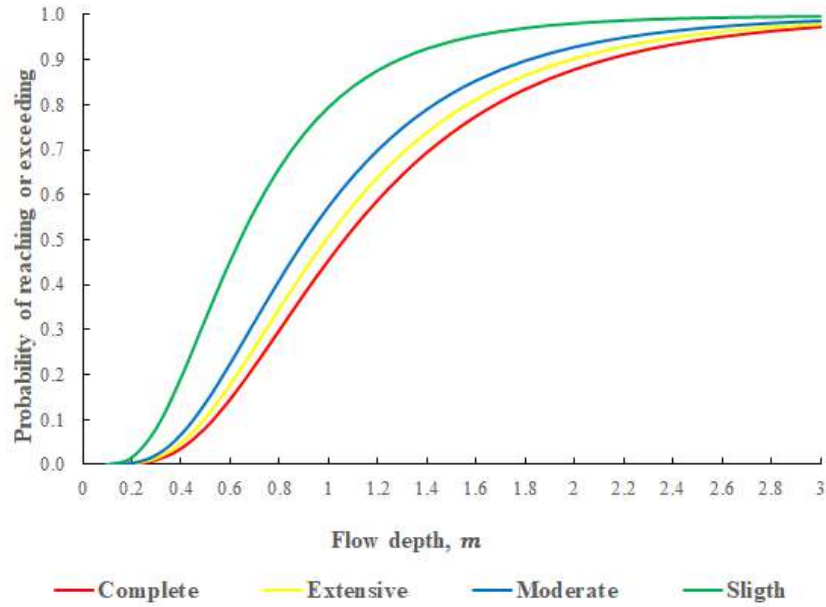


Figure B9. Fragility curve for CM buildings 1 storey. Wall thickness averages 12 cm. Flow depth intensity

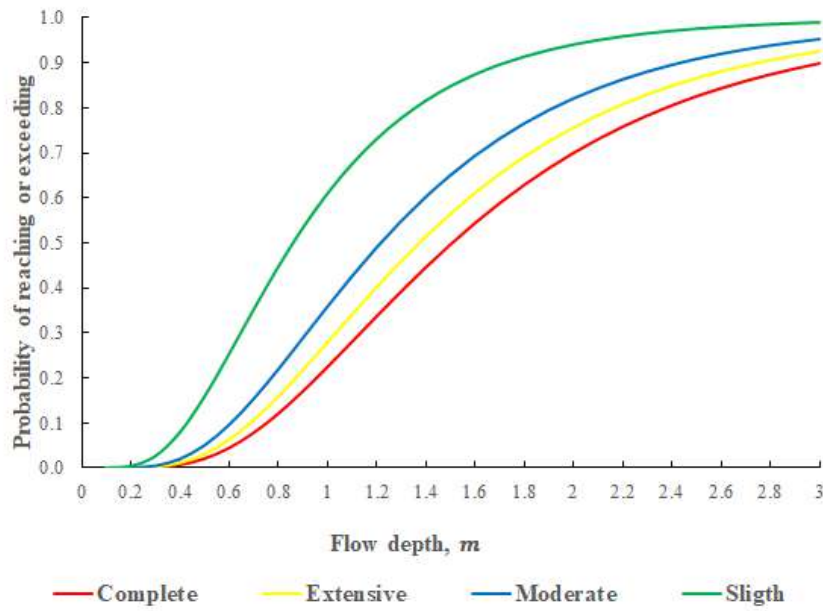


Figure B10. Fragility curve for CM buildings 1 storey. Wall thickness average 20 cm. Flow depth intensity

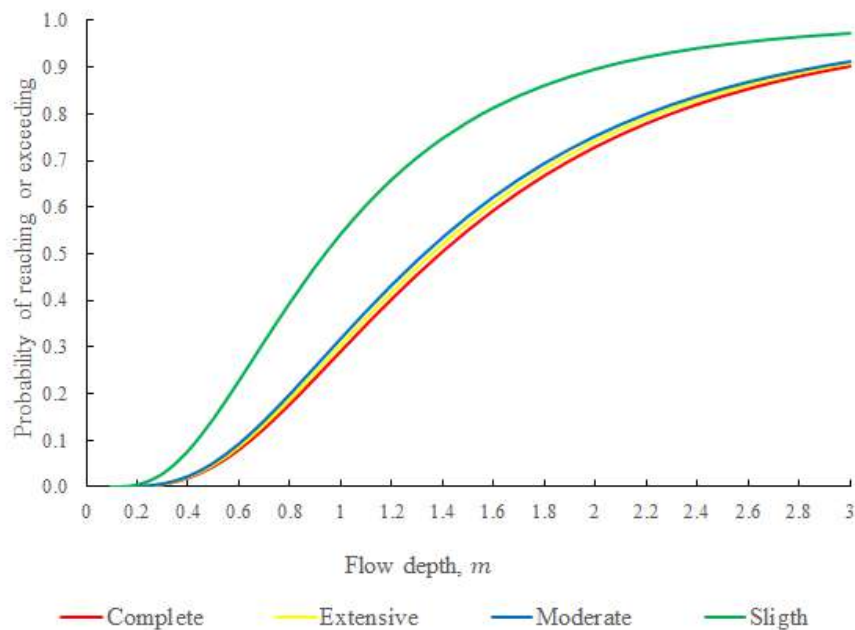


Figure B11. Fragility curve for CM buildings 2 storeys. Wall thickness averages 12 cm. Flow depth intensity

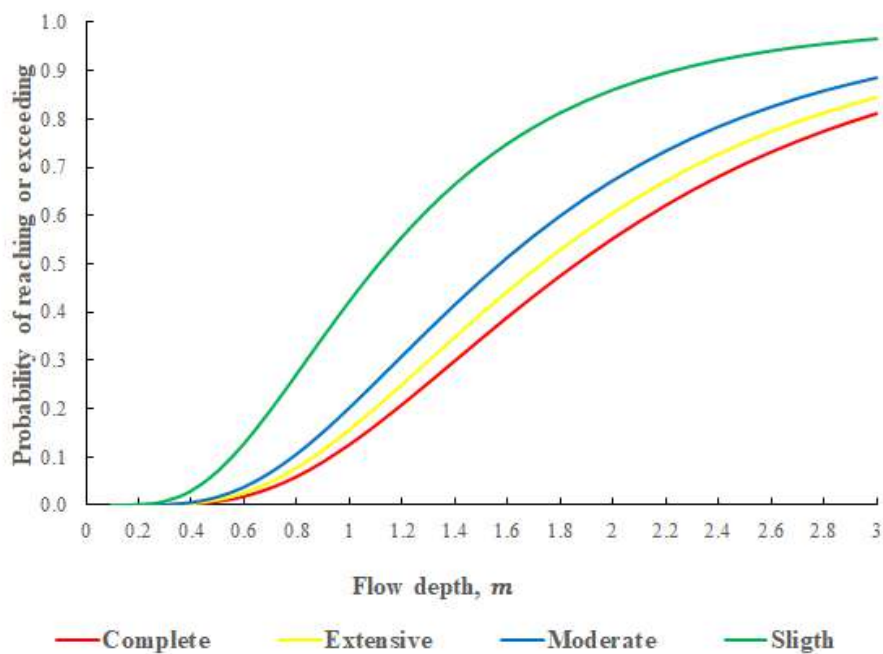


Figure B12. Fragility curve for CM buildings 2 storeys. Wall thickness averages 20 cm. Flow depth intensity

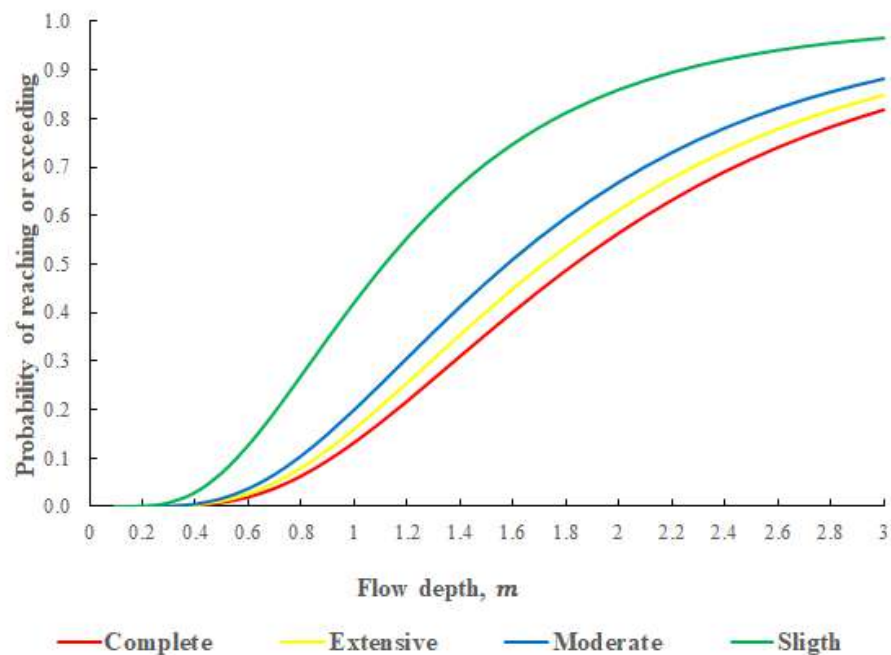


Figure B13. Fragility curve for CM buildings 3 storeys. Wall thickness average 12 cm. Flow depth intensity

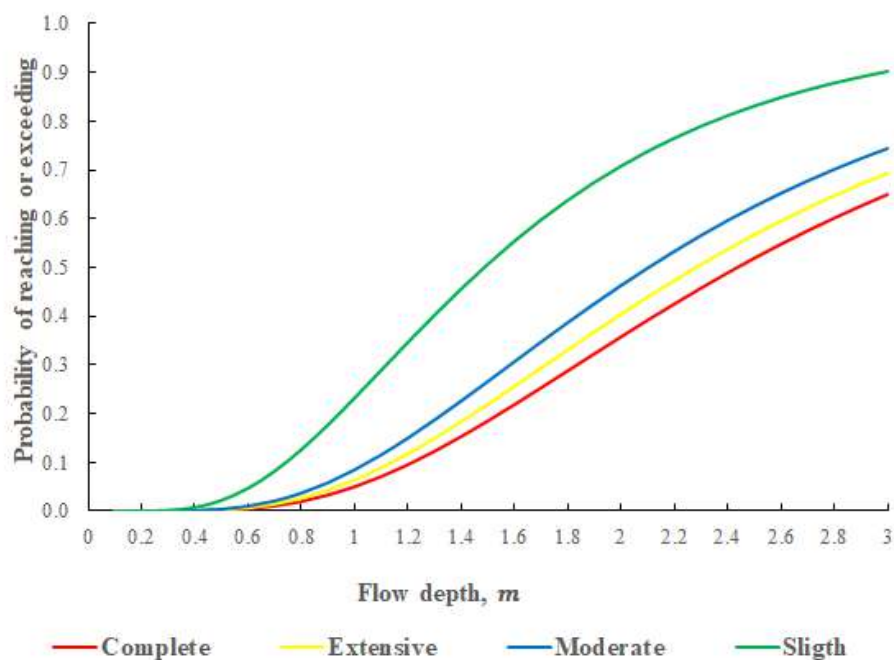


Figure B14. Fragility curve for CM buildings 3 storeys. Wall thickness averages 20 cm. Flow depth intensity

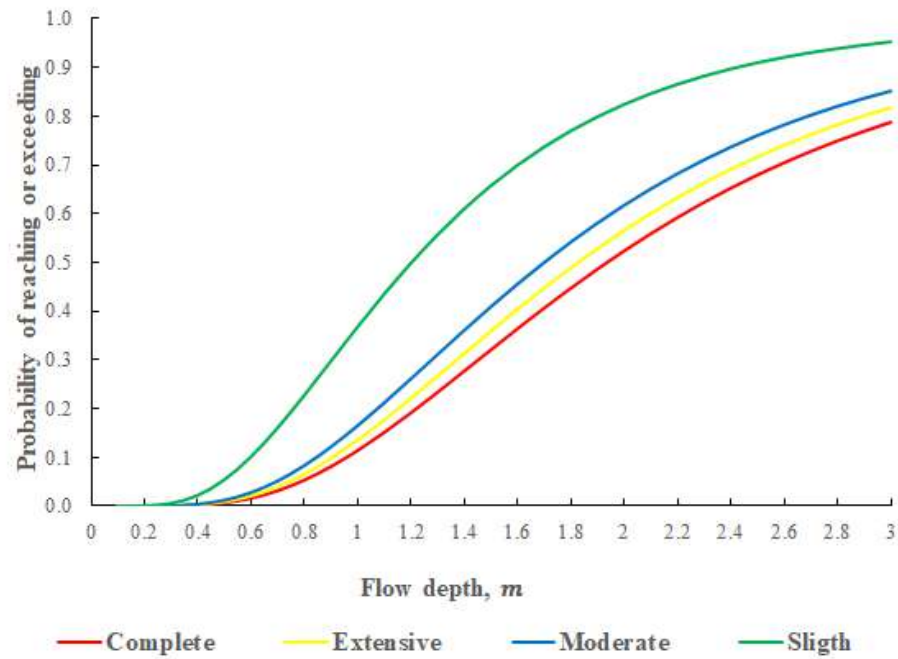


Figure B15. Fragility curve for CM buildings 4 storeys. Wall thickness averages 12 cm. Flow depth intensity

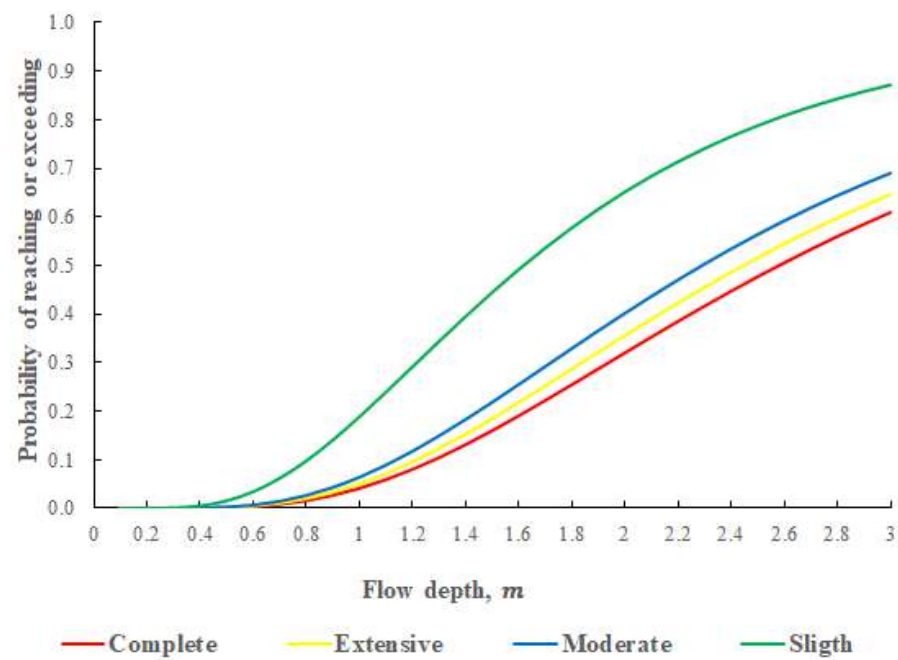


Figure B16. Fragility curve for CM buildings 4 storeys. Wall thickness averages 20 cm. Flow depth intensity

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