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MECHANIC-BASED PROCEDURE FOR THE DAMAGE MECHANISM EVALUATION OF HISTORIC MASONRY STRUCTURES

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ABSTRACT

Cultural Heritage (CH) assets are a distinctive and valuable fragment of many areas worldwide, often representing a very important component of individual and collective identity. CH assets encompass not only individual buildings but also historic urban centres which are commonly formed by clusters of single units, undergoing to continuous transformations overt time. Although these additions tend to maintain the original shape, they very often modify the global structural behavior of the compounds. Furthermore, owing to their low-engineered construction features, CH assets are also perceived to be highly vulnerable to different natural hazards. To date, much research effort has been devoted to thoroughly understand the mechanism of damage that CH assets undergo in case of earthquakes. However, there is still a compelling need to develop quantitative methods able to assess the structural vulnerability of these buildings against different co-occurring perils, such as flooding or high wind speeds. Knowledge would be of great important to advance current mitigation measures for the retrofit of CH assets in multi-hazard prone countries, enhancing the decision-making process of their preservation to future generations.

This paper presents a mechanic-based procedure for the collapse load evaluation of a specific category of CH assets, namely historical masonry structures (HMS), undergoing to earthquake, flood and wind.

The procedure has been currently applied to a single wall, aiming to be extended to the whole façade element, considering variation in geometry, layout and material characteristics.

Keywords: Multi-Hazard; Quantitative Analytical Procedure; Cultural Heritage Assets, Failure Mechanism Identification

1 MULTI RISK FRAMEWORK FOR HISTORICAL MASONRY STRUCTURES

Historic masonry structures (HMS) are the most ubiquitous building typologies and a very representative urban fragment of many cities around the world. They are recognized to possess cultural value and universal significance, thus playing a key role in determining the 'sense of community' and the 'social identity' of many population worldwide. However, due to the complexities and uncertainties related to material properties, aging effects and variety of craftsmanship, HMS are also inevitably vulnerable to destructive natural events.

Even though life safety represents the first and foremost objective when pursuing hazard assessment of buildings, it is becoming of increasing importance to advance methods and techniques to assess the vulnerability of assets of historic value which can be irreversibly lost due to the impact of natural disasters.

While an advanced and detailed literature covering the topic of seismic vulnerability assessment of non-

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engineered structural typologies has been already developed (D'Ayala D. 2013), work on the field of flood and wind vulnerability is notoriously more sparse, hindering the definition of a harmonized method to carry the analysis in a commensurate manner across these hazards and to rank the results on the basis of a single common scale.
Another fundamental aspect to consider when dealing with masonry structures relates to the definition of the most appropriate mechanical model to describe the failure pattern developing as a consequence of the application of a given horizontal loading profile. This becomes even more arduous when dealing with perils of different nature which are, consequently, characterized by different profile shapes, while also aiming to control the number of assumptions made.
In this study, a mechanic-based procedure to determine the mechanism of failure of unreinforced masonry structures undergoing to seismic, flood and wind loadings at large territorial scale is presented. The procedure is based on the plastic analysis principles and aims to establish a rather simple but rigorous procedure that can be employed to assess large population of buildings with limited computational efforts.
The study is carried out at single façade scale. The masonry building is in fact disassembled into single-wall elements to which each of the three loading profiles is applied. A feasible crack pattern is hypothesized, which must comply with geometry, layout and boundary conditions of the wall. The final aim is to establish, among the loadings considered, the one causing the greatest extent of damage while also mobilizing the greatest wall portion in the mechanism.
The paper begins with an overview of available methods for vulnerability assessment of HMS subjected to individual hazards and in particular to earthquake, flood and wind.
The following section is dedicated to the review of available mechanical models and procedures developed to date, to predict the ultimate load capacity of URM walls subjected to horizontal loadings. To follow, the detailed steps of the mechanic-based procedure proposed to evaluate the failure mechanisms of HMS subjected to earthquake, flood and wind loadings are given in Section 4.
The last section of this work presents an application of the method to a chosen case study. In addition, an analysis of the main global and local geometric aspects that affect the obtained results is provided.

2. AVAILABLE METHODS TO ASSESS MULTI-HAZARD VULNERABILITY OF HMS

2.1 Seismic Vulnerability Assessment

Guidance on suitable approaches for analytical seismic vulnerability assessment of CH assets and heritage structures is provided by DPCM (2008). These approaches rely on medium quality of the data and apply relatively simple analytical methods, based on a modest number of global and local geometric parameters to describe the behavior of the wall. More specifically, a mathematical model of index buildings, which is representative of the building typologies considered, is defined and the response of such model to a given expected level of shaking intensities is computed.
Specifically tailored to assess the seismic vulnerability of URM heritage buildings at large territorial scale, the FaMIVE method ('Failure Mechanisms Identification and Vulnerability Evaluation') developed by D. D'Ayala and E. Speranza (2003) has been developed in the last 15 years, evolving from a procedure to identify possible collapse mechanisms for masonry facades characterized by different levels of lateral constraints and to determine their lateral acceleration capacity to a method for the derivation of capacity curves (D'Ayala D. 2005) and computation of fragility functions for populations of buildings of similar typologies and/or collapse modes (D'Ayala D. 2013).
The seismic vulnerability of masonry buildings can also be estimated by means of more sophisticated Capacity Spectrum Based Methods based on FE models. However, the an-isotropic nature of masonry, the intrinsic geometrical complexities and the computational efforts required to process the analyses, yield to the consideration that similar approaches are not entirely suitable for the scope of vulnerability assessment at large territorial scale.

2.2 Flood Vulnerability Assessment

The increase in flood events due to climate change has risen the awareness of the importance to develop appropriate flood risk assessment techniques suitable for the evaluation of damage and loss incurred to historic buildings located in flood-prone areas. However, to date the limited work available in literature is still very qualitative in nature and primarily focused on the assessment of general losses rather than on the more specific aspects of structural damage caused by the event (D'Ayala et al, 2006).

Flood damage to buildings and contents depends on several variables in relation to the flood events. The different types of flood actions are usually classified according to the relevance and predictability of their occurrence (Holicky, M., & Sykora, M. (2010), Mebarki, A. et al (2012).

In terms of available methods to assess the flood damage to buildings, semi-empirical procedures require parameters such as inundation level, flow velocity, flood duration and building characteristics. Empirical approaches usually require large dataset of buildings to account for different construction materials. The building types are allocated to different flood vulnerability classes and their observed damage is ranked based on qualitative parameters. However, the damage extent is considerably affected by inherent uncertainty, thus implying that such tools are best suited for damage prediction of large building stocks in urban areas (Schwarz, J., Maiwald, H., 2008). The procedure proposed by Stephenson & D'Ayala (2014) specifically focused on historic buildings, entails the assessment of flood vulnerability of historic buildings by means of vulnerability indices (VI_f) defined as a result of a scoring process of specific set of vulnerability parameters, which are proved to be relevant to the building typology considered. In particular, each indicator is designed to be tailored for different geographic and cultural settings, providing with a qualitative estimate of the vulnerability of buildings in presence of water.

2.3 Wind Vulnerability Assessment

To date, the most complete review of methods to assess the building vulnerability to extreme wind, in particular to hurricanes, is provided by Pita et al, (2014). According to the literature, there are five different types of general approaches that can be adopted to carry out the building vulnerability estimation to wind hazard. However, none of the available is specifically tailored to deal with the specific aspects of HMS.

Methods based on exclusive regression of building past-loss data (1) have been widely employed during past years; however, these proved to be highly dependent on the most representative type of constructions of the area investigated, thus affecting the reliability of vulnerability functions (Pinelli et al. 2004). The enhanced data models (2), adopted to complement the past-loss data models with engineering and meteorology expert knowledge, proved to be a better fit in cases where the lack of information of past data represented a limitation. Although more subjective in nature, methods based on expert opinion appraisal, else defined as heuristic approaches (3), were a useful alternative when past-loss data where not sufficiently or enough detailed to develop meaningful vulnerability curves. Following the very similar concept developed for seismic vulnerability assessment, Wehner et al. (2010) adopted a procedure to develop an extensive set of heuristic vulnerability curves for Australian building stocks subjected to extreme wind speed. Component-based methods (4) were developed as a more realistic alternative to enhance data models by assessing the vulnerability within an engineering framework complemented with expert opinion and based upon estimations of physical damage and expert opinions. The two innovations introduced through this fourth category of methods were a) the total damage experienced by the building is treated as an aggregation of single damages to the main building components; b) the assessment of the interior and exterior damage of a building is treated and assessed separately. Lastly, simulation vulnerability models (5) enhanced the physical models by providing with a more precise estimation of the wind load effects on the buildings, thus with a more realistic assessment that accounts for the impact of debris-induced damage on structural components. In addition, by means of Monte-Carlo simulation, these models were also able to assess the variability of the results obtained when different components properties were considered (Walker, G. R. 2011).

3. AVAILABLE METHODS TO PREDICT THE ULTIMATE LOAD CAPACITY OF URM

Unreinforced masonry structures subjected to lateral loadings experience a combination of two different types of response: in-plane and out-of-plane. While many research efforts have been devoted to assessing the wall in-plane behavior, significant to understand how the building transfers the lateral

forces to the foundations, past and recent studies have demonstrated that out-of-plane actions are the most damaging for the structure, often leading to complete collapse (Sorrentino et al. 2017).
The response of URM to lateral actions is attributed to the ultimate strength capacity, and most of the state-of-the-art methodologies that aim at determining this key-factor that control this response are based on plastic analysis principles.
The pioneering work of Heyman (1997) who first promoted the use of limit state analysis based on plasticity theory to solve the problems of statics in masonry, must be considered the milestone for all the following studies. To be applied to a quasi-brittle material such as masonry, the three main assumptions to consider are: 1) masonry possesses zero tensile strength, 2) masonry possesses infinite compressive strength, 3) sliding failure is not permitted.
The first step of limit state analysis is the definition of a plausible pattern of yield lines, which establishes a potentially plausible collapse mechanism. The mechanism must be consistent with the given restrain conditions, applied load, geometry and layout of the wall. However, the definition of the correct crack pattern is not always univocal. Among all the possible crack patterns that can be postulated, the 'correct' pattern is the one requiring the least work for its formation (Lovegrove, R. 1988). Bending and twisting moments are uniformly distributed along each line, which separates the wall into rigid zones behaving elastically and remaining plane during the collapse condition.
There a number of various adaptation of limit state analysis available in literature, whose main aim is to find the ultimate load capacity of URM walls.
After having been introduced for the first time by Johansen (1962), for the design of RC slabs, the Yield Line (YL) Theory was adopted by the British Code and Eurocode 6 (Standard, B. 2005). According to the latter, the lateral resistance of masonry walls depends on the orthogonal strength ratio (μ), defined as the ratio between the two flexural strengths in the two orthogonal directions (parallel and perpendicular to the bed joints). Additionally, it is assumed that the two moment capacities in the two directions are reached simultaneously and the Young Modulus (E) is the same in both the directions of the wall. This latter assumption was discussed by Sinha (1978) and then changed as part of a new modified approach defined as Fracture Line Theory (FLT). Such method was based on a specific set of assumptions, namely: 1) all the deformations take place along the fracture lines only, 2) each individual wall portion rotate as a rigid body, 3) the load distribution takes into consideration the value of stiffness in both directions, 4) the fracture lines develop when the strengths are reached simultaneously in both the directions of the wall. Apart from the mentioned assumptions, the most remarkable difference relates to the definition of the μ parameters, as the reciprocal of the μ factor included in Eurocode 6, as shown in Equation 1

$$\mu_{BS} = 1/\mu_{FLT} \tag{1}$$

Additionally, according to the FLT, although independent, the internal moments in horizontal and vertical directions are linked by the μ factor.
In an effort to improve the design method based on YL theory, Lawrence, S., & Marshall, R. (2000) introduced a revised approach, which was later on included in AS3700-2001.
Differently from YL theory, which bases the crack angle on a minimum energy approach, this new method is based on the assumption that the diagonal crack angle depends on the geometry and the layout of the masonry units constituting the wall. More importantly and for the first time, this method identifies the torsional behavior of the bed joints as one of the most relevant parameters to consider in the determination of the out-of-plane load resistance of the wall.
In this context, Griffith et al (2005) developed a procedure to compute the ultimate vertical, horizontal and diagonal moments that fits the YL approach in a more thorough physical manner than what suggested in EC6. According to this method, the overall diagonal moment capacity is given as the sum of the contributing flexural and torsional moments of the bed joints. By contrast, all the contribution coming from the perpendicular joints are neglected, as they tend to crack in early stages, therefore they are considered not to contribute in the definition of the overall wall moment capacity.
Ultimately, the diagonal moment capacity depends on the contributions given by the flexural tensile strength of masonry perpendicular to bed joints, the compressive stress given by the wall self-weight, the wall thickness, the height of the masonry unit and the contribution of an additional factor accounting for the interaction between torsion and flexure (Vaculik, J., 2012).

4. THE MECHANIC-BASED PROCEDURE PROPOSED FOR THE FAILURE MECHANISMS IDENTIFICATION OF HMS

The mechanic-based procedure proposed in this study for the identification of the failure mechanisms of HMS subjected to earthquake, flood and wind loadings belongs to a wider framework, summarized in Figure 1.

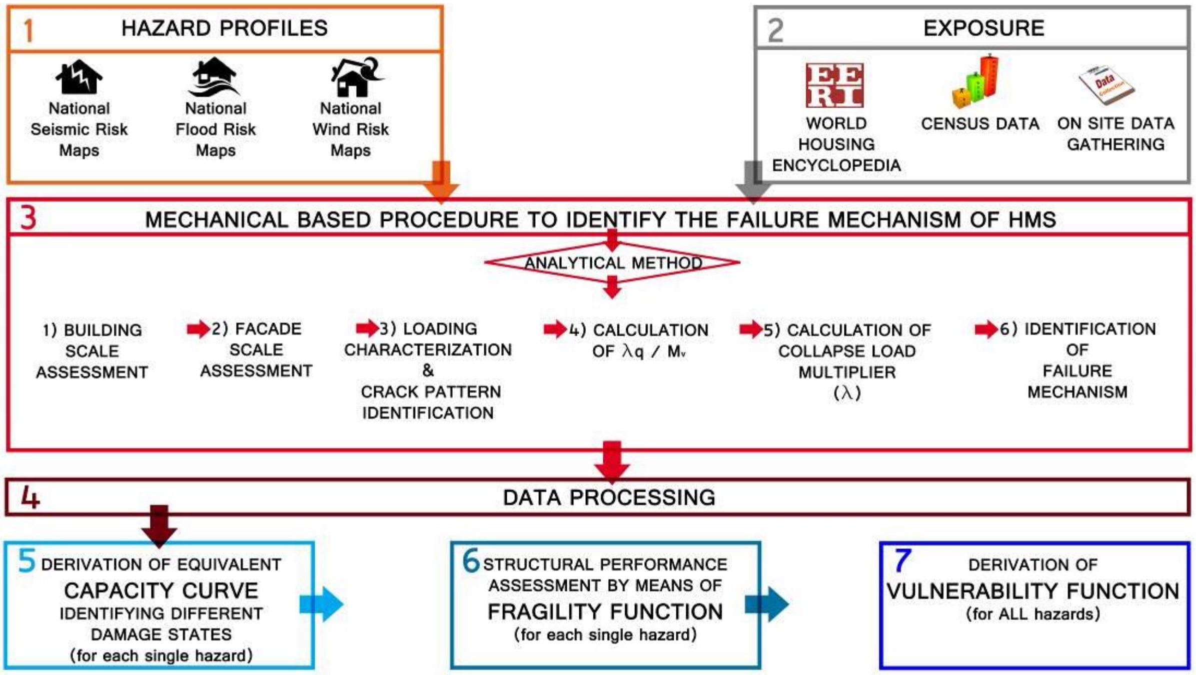


Figure 1 General framework for multi-hazard assessment of HMS

The assessment begins with the identification of the hazard sources within a given region of interest and the evaluation of each individual hazard independently, to identify the spatial distribution of effects, the characteristic intensities of each hazard and their occurrence probabilities (1). In parallel, the exposure assessment is carried out, aiming to identify the type of assets at risk within the designated area of interest. This step represents the contextualization of the risk assessment procedure (2).

The third step is to choose a suitable method to assess the vulnerability of the given asset to the specific hazards (3). The method is based on limit state principles, thus aiming to define the maximum load that the wall can sustain (i.e. 'load limit'). The main outcome of this analysis is the definition of the collapse load multiplier λ, which determines the most damaging loading profile among the ones applied to the wall. The following step is the data processing (4), which focuses on the establishment of the most suitable analytical tool to use to process and store the analytical routines to define the failure mechanisms of masonry walls for different loading conditions.

Directly related to the definition of the λ factor is the concept of capacity curve, which is the most representative bilinear relationship to represent the capacity of a wall against applied external forces.

To be determined, there are three fundamental parameters to consider, namely the lateral effective stiffness of the wall (which depends on geometry, boundary conditions, additional forces coming from horizontal and vertical structural elements acting on the wall and the actions coming from return walls involved in the mechanism), the nominal yield displacement (which, in a 'brittle' material such as masonry can be considered as the point in which the first crack forms in the wall, thus changing the initial stiffness) and the ultimate displacement at the top of the wall.

Additionally, to assess the performance levels of a building (or of a single structural element, i.e. façade) it is necessary to establish damage states, with the aim of defining the condition of the assessed element for increasing intensity levels.

To conclude the structural performance assessment, fragility functions must be derived, which indicate the conditional distribution of structural response for given IM levels.

This assessment is carried out independently for each of the hazards considered, since they are characterized by independent hazard intensity measure (IMs).
Once the single-type fragility functions are obtained, it is possible to define the vulnerability of the structural element through the vulnerability functions, which define the probability of having losses, given a level of IM.
The focus of this paper is only on the mechanical-based procedure for the damage mechanism evaluation. A more detailed description of the steps is given Figure 2

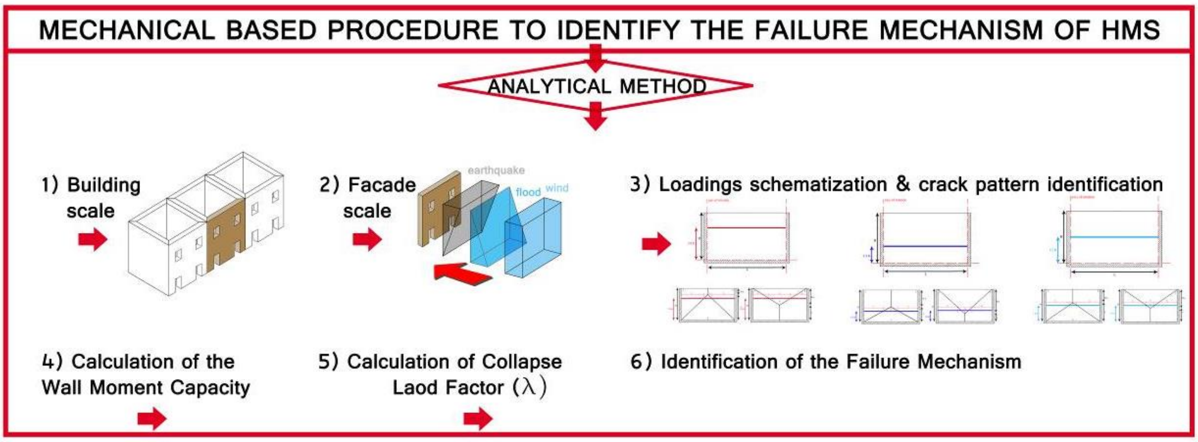


Figure 2 Mechanical Based Procedure to Identify the Failure Mechanism of HMS

The procedure is designed to carry the analysis going from the whole building to the single facade scale. The three loading profiles (i.e. earthquake, flood and wind actions) are schematically applied as 'simplified' knife edge loads (KEL), with point of application inferred from the specific loading shape. In accordance to geometry, layout, boundary conditions and position of KEL, a crack pattern is assumed. Each individual assessment aims to find the wall moment capacity, defined as the ratio of external work (W_e) and internal work (W_i).
More specifically, the W_e is given as the sum of the product between all the resultants of the three loading profiles considered and the consequent linear displacement, integrated along the wall surface.
The W_i is given as a product of the moment capacity per unit length, the length of the crack and the virtual rotation (i.e. the rotation undertaken by the crack when subjected to the given loading profile).
Furthermore, it must be considered that the moment in vertical direction M_h is defined as a function of one in horizontal direction M_v scaled by the factor μ, as shown in Equation 2:

$$M_h = \mu\ M_v \tag{2}$$

A similar approach has been previously adopted in EC6 (Standard, B. 2005).
Once the internal and external contributions are defined and equated (as shown in Equation 3), it is possible to obtain the $\lambda q\ /\ M_v$ ratio (Equation 5).

$$W_e = W_i \tag{3}$$

$$\lambda q\ K_1 = M_v\ K_2 \tag{4}$$

$$\lambda q\ /\ M_v = K_2\ /\ K_1 \tag{5}$$

More specifically, the ratio shown in Equation 5 is a function of both the value of λq (the value of collapse load comprehensive of the distance between the fixed position of the KEL and the position of maximum displacement x) and the wall moment capacity in horizontal direction M_v. The contribution of M_h is considered within K_2. Equating internal and external works corresponds to equating the wall moment capacity M_v (i.e. the capacity of bearing a certain loading applied defined by means of other parameters, all indicated by means of K_1) to the ultimate load λq which causes collapse (defined as a product of other parameters, all indicated by K_2). Equation 5 shows the fully parametric nature of the the ratio λq / M_v. The collapse load multiplier represents the minimum load factor determining the equilibrium condition of internal resistance and external loading applied. Following relations outline the main steps to determine the multiplier:

$$M_v = (f_{mt} + f_{sw})\ Z = K_3 \tag{6}$$

$$\lambda q = (K_2 / K_1)\ K_3 \tag{7}$$

$$q = f\ (x,y,z) = K_4 \tag{8}$$

$$\lambda = (K_2 / K_1)\ (K_3 / K_4) \tag{9}$$

Equation 6 shows the components considered for the determination of the wall moment capacity along the horizontal direction, and these are given as the product of the flexural tensile strength of masonry and the compressive stress due to self-weight multiplied by the wall section modulus. Since all the components in Equation 5 can vary according to the geometry and the material properties of the wall considered, this Equation is fully parametric. Once implemented in Equation 7, the value of M_v allows to define λq. If the loading conditions can be specified (i.e. the seismicity of the investigated location, the characteristic value of hydrodynamic water pressure, the wind speed, as shown in Equation 8), then the value of q becomes a ‘known’ parameter of the Equation 9, allowing the definition of the specific multiplier λ that yields the wall to collapse.

More specifically, λ is larger/equal to the safety factor, thus being the smallest of all the kinematically admissible solutions. The smallest λ factor involving the greatest portion of wall in the failure mechanism assumed determines which peril is the most damaging and, consequently, which must be prioritized.

5. APPLICATION OF THE OUTLINED MECHANICAL PROCEDURE

For the present study the wall chosen has no openings, no additional overburden weights coming from horizontal structures (floors and roof) except for its own weight. The geometric parameters and material properties are summarized in Table 1:

Table 1. Geometric and material characteristics of the wall

Wall Parameter	**Unit**
Wall Height [H]	4m
Wall Length [L]	4m
Wall Thickness [t]	0.200 m
Brick Unit Height	0.100 m
Brick Unit Length	0.250 m
Mortar Joint Thickness	0.005 m
Flexural Tensile Strength	0.3 MPa

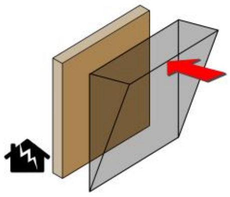
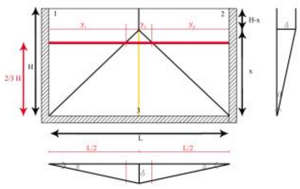
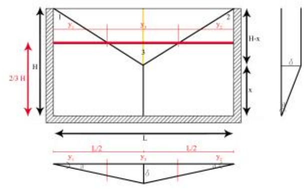
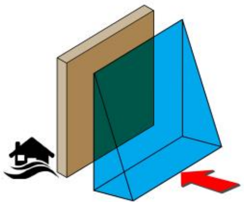
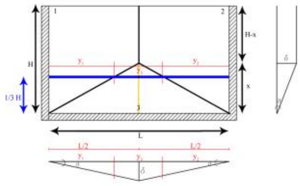
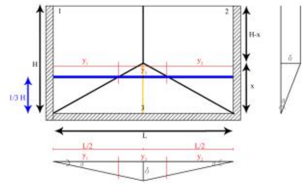
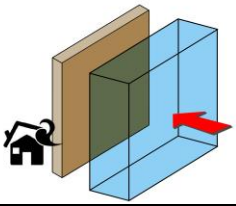
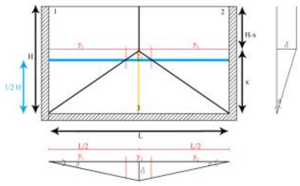
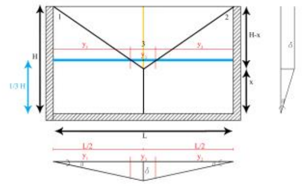
In terms of boundary conditions, the wall is assumed to be simply supported on three edges (bottom and laterals) with fourth free top edge. The loading profiles with point of application and main mathematical formulation are reported in Table 2. All the equations shown in the Table contain the λ factor, which represents the lowest multiplier causing collapse.

Table 2. Loading profiles: seismic, flood and wind

Wall Parameter	**Unit**	**Mathematical formulation**
$q_{seismic}$	2/3 H	$q_{seimsic} = \lambda(g\ H\ t\ \rho_{brick})$
q_{flood}	1/3 H	$q_{flood} = \lambda(1/2\ g\ H^2\ \rho_{water})$
q_{wind}	1/2 H	$q_{wind} = \lambda(1/2\ V^2\ H\ C\ \rho_{air})$

The failure patterns for each of the three hazards considered are shown in Table 3

Table 3. Failure patterns identification: earthquake, flood and wind KEL

Loading Profile	**Failure Mechanism 1**	**Failure Mechanism 2**
		
		
		

5.1 Results and Discussion

The comparison between the λ curves, for both Failure Mechanism 1 and 2 is presented in Figure 3.

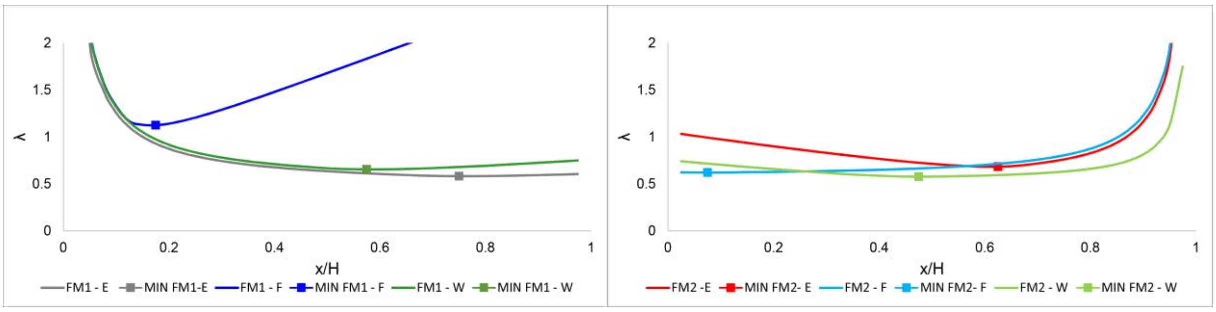


Figure 3 Comparison between λ curves for seismic, flood and wind loading, FM1 (*left*) and FM2 (*right*)

The aim of the two graphs presented in Figure 3 is to show the peril that causes the failure condition corresponding to the smallest λ multiplier, while also involving the greatest wall portion.
To this end, the seismic intensity of peak ground acceleration, the height of the water and the wind speed have been set in such a way to develop the same magnitude of applied load on the wall considered.

More in general, the graphs show that the maximum displacement of the wall is always reached after the point of application of the KEL, thus implying that, regardless the simplified assumption related to the loadings, the procedure can provide with good estimations.
A summary of the values obtained for both FM1 and FM2 is given in Table 4.

Table 4. Comparison between FM1 and FM2 for seismic, flood and wind loadings

Failure Mechanism	**Min λ**	**x/H**
$FM1_{seismic}$	0.582	0.750
$FM1_{flood}$	1.124	0.175
$FM1_{wind}$	0.654	0.575
$FM2_{seismic}$	0.684	0.625
$FM2_{flood}$	0.621	0.075
$FM2_{wind}$	0.577	0.475

According to the summary of FM1, the wall has minimum resistance to the seismic loading, while, when looking at FM2, the flood loading is the one causing the crack pattern to form prior to the other perils considered, while also involving the greatest extent of wall.

5.2 Sensitivity Analysis

To evaluate which of the parameters implemented in the analytical procedure proposed plays the most influential role in determining the λ multiplier, a sensitivity analysis has been conducted. To this end, the parameters listed in Table 1 have been varied across the ranges presented in Table 5.

Table 5. Global, local and material properties range for sensitivity analysis

Wall Parameter	**Unit**
Wall Height [H]	2m - 4m
Wall Length [L]	2m - 4m
Wall Thickness [t]	0.100 m - 0.250m
Brick Unit Height	0.050m - 0.150m
Brick Unit Length	0.100 m - 0.250m
Flexural Tensile Strength	0.3 MPa - 0.6 MPa

The global geometric parameters are varied within a range of dimensions indicating the most commonly observed residential masonry buildings facades' dimensions.
Local geometric aspect ratios of masonry units and the range chosen for the flexural tensile strength are instead taken from experimental data carried out by Schubertl (1994) and later by Vaculik (2012).

Figure 4 show the variation of λ for different wall aspect ratios. More specifically, Figure 4 *left* shows the case of constant wall height, while Figure 4 *right* shows the case of constant wall length.
In both Figures, the effects of H/L variation can be particularly evident through the observable shift of the minimum λ value. When decreasing the H/L (Figure 4 *left*) the point of maximum displacement tends to 1, highlighting that wider walls tend to reach the position of maximum displacement towards the top of the wall, thus involving the whole wall height. On the other hand, in the case of very tall walls with constant length (i.e. L=4 m), the point of maximum displacement is reached around mid-height, thus implying that after the diagonal cracks reach that point, the vertical crack line will form: more specifically, the taller the wall, the longer the vertical crack, thus implying a commonly defined 'one-

way slab' behavior.

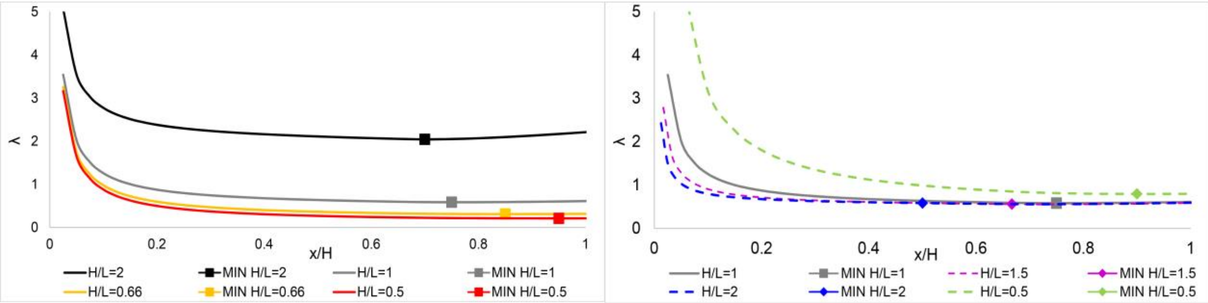


Figure 4 λ factor versus x/H for different H/L ratio - FM1 – Seismic Load

Figure 5 shows the influence of the local aspect ratio on the determination of λ values.

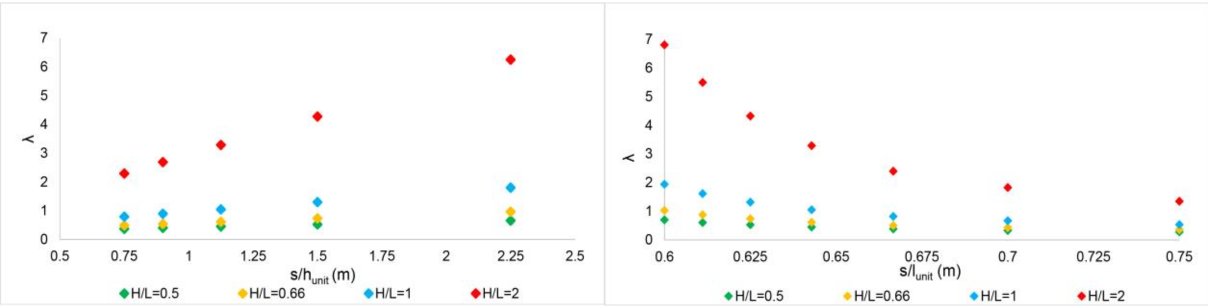


Figure 5 *Left* λ factor versus s/h_{unit}. *Right* λ factor versus s/l_{unit}

More specifically, Figure 5 *left* shows the variation in λ against s/h_{unit} (i.e. staggering ratio over height of the masonry unit), for different H/L ratios. Conditional to the attainment of the results shown in the graph is the fact that the l_{unit} is constant and equal to 0.175 m. The graph shows that an increment of H/L ratio corresponds to an increment in terms of min λ values. Since the number of masonry brick courses within taller walls is greater than the one within shorter ones, when increasing the h/H the lateral load required to reach the collapse condition (i.e. λ value) increases proportionally.
Figure 5 *right* shows the influence of the masonry unit length (l_{unit}) on the determination of λ factor. It is very important to note that currently, the proposed procedure calculates the staggering ratio as the semi total length of the masonry unit, therefore the s/ l_{unit} ratio is continuously re-adjusted according to the range of masonry lengths defined. Greater staggering ratios directly influence the μ parameter, causing an increase in wall moment capacity and, consequently, a decrease in λ values. This helps explaining the trend followed by the curves presented in Figure 5 *right.*

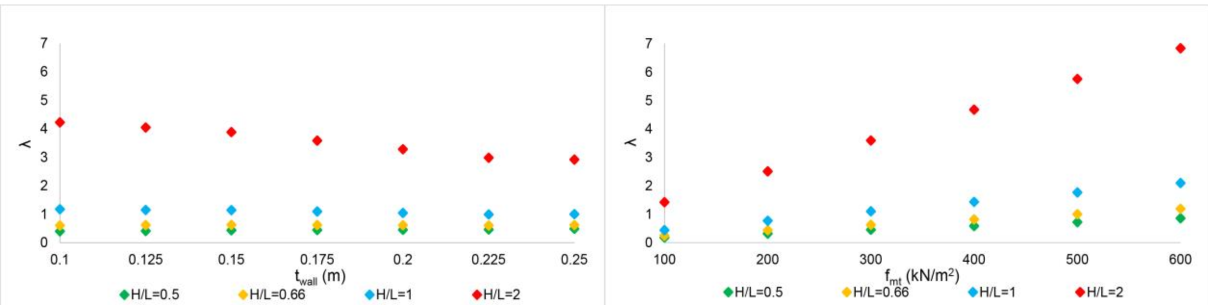


Figure 6 *Left* λ factor versus t_{wall}. *Right* λ factor versus f_{mt}

Figure 6 *left* shows the variation in λ against the thickness of the wall. Increasing the wall thickness causes an increase in wall's weight, which proportionally affects the μ factor and hence the wall moment capacity. This exhaustively explains the decreasing trend of the curves shown in the graph. However, along with this decrease, it is also possible to note a shift in terms of x/H position (i.e. position of maximum displacement). More specifically, thicker walls tend to develop much wider cracks in comparison to more slender wall sections. When looking at the case of 0.100 m wall thickness, the maximum displacement position is 0.75 x/H, while in the case of 0.250 m wall thickness, this position shifts to 0.9 x/H.

To conclude the section of sensitivity analysis, Figure 6 *right* shows the extent of variation in terms of λ multiplier when varying the flexural tensile strength f_{mt} across different H/L wall ratios. More specifically, the height of the wall is constant, while the length of the panel varies.

The increasing trend of the curves is associated to physical reasons: in fact, walls characterized by stronger bonds between constituent masonry units require a greater applied load to reach collapse.

This aspect is further emphasized when considering different walls' aspect ratios: since the flexural tensile strength is a material property linked to the mutual interaction between masonry unit's bonds, greater H/L ratios imply an increasing number of masonry units, therefore resulting in a greater extent of variation in term of minimum λ multiplier required to reach collapse.

6. CONCLUSION

The paper presents a mechanical-based procedure to identify the most probable failure mechanism of HMS when subjected to horizontal forces of different physical nature, such as earthquake, flood and wind loadings. The procedure follows the main concepts of the limit state analysis and aims to determine in a simple but rigorous manner, the lowest collapse multiplier λ associated to all possible failure mechanisms which can be assumed, according to boundary conditions, geometry and layout of the analyzed structural element (wall/façade). The procedure aims to become an expeditious tool for the assessment of large population of buildings at territorial scale. Ultimately, the procedure can also be extended to assess other natural hazards (i.e. landslides, ash flows) which can be represented as horizontal loadings characterized by a specific loading profile that can be 'simplified' as a knife edge load action along the wall contact surface. This statement, however, requires further investigation, as the procedure itself is still under revision.

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