COMPDYN 2025
10th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Rhodes Island, Greece, 15-18 June 2025

ACCELERATION, JERK AND MASONRY DISAGGREGATION: SEISMIC EFFECTS ON EXISTING MASONRY BUILDINGS

Massimo Mariani[1] and Francesco Pugi[2]

[1] Studio Ricerche Applicate
06123 Perugia, Italy
ricercheapplicate@libero.it

[2] Aedes Software: Ricerca e Sviluppo
56028 San Miniato, Pisa, Italy
francesco.pugi@aedes.it

Abstract

Masonry buildings constitute a significant portion of the existing building heritage. Observation of damage from seismic events reveals a hierarchy of collapse mechanisms, ranked by vulnerability: masonry disaggregation, connection failures, rigid body mechanisms, exceedance of elasto-plastic capacity.
Masonry disaggregation is a brittle failure mechanism typical of low-quality masonry subjected to high structural accelerations. Despite its critical role, it is generally overlooked in seismic risk assessments and retrofitting projects.
This work highlights and characterizes the close relationship between masonry quality, disaggregation and high-frequency seismic content (Jerk). The tendency toward disaggregation is not solely an intrinsic property of the masonry's texture; it is also triggered and enhanced by vertical vibration dynamics. By analyzing Jerk intensity and frequency, and considering its effects on masonry quality, disaggregation can be effectively incorporated in computational analyses, allowing for more realistic seismic risk indicators. Studies conducted in this field show that upgrading low-quality masonry is a key element in any structural strengthening intervention.

Keywords: masonry disaggregation in existing building; high-frequency seismic content (Jerk); masonry quality index; dynamic of vertical vibration; relationship between Jerk, disaggregation and masonry quality.

1 INTRODUCTION

The recent seismic events in Central Italy have confirmed that masonry buildings respond differently depending on the quality of their constituent elements: many constructions made with poor-quality masonry experienced total or partial collapses due to disaggregation. As a result of disaggregation, cohesion is lost because the mortar is "pulverized"; the masonry components separate from each other, and the wall degrades into a chaotic mass of stone elements (Figure 1) [1]. Conversely, structures built with well-designed and properly constructed masonry, or adequately strengthened (as in the case of the historical center of Norcia), generally demonstrated good seismic performance.

Figure 1. Disaggregative effects caused by the seismic events in Central Italy, 2016 [1].
Top: Pescara del Tronto. Bottom: Corso Umberto I in Amatrice before the event (left) and after the event (right).

The Italian Technical Standards [2,3] clearly highlight that, in the context of seismic vulnerability assessments of existing masonry buildings, the phenomenon of masonry disaggregation must be taken into account. These assessments must primarily consider local failure mechanisms, whit verification of possible disaggregation; subsequently, the global behavior of the masonry structure is analyzed. This establishes a hierarchy of mechanisms corresponding to structural damage induced by increasing seismic actions:

- disaggregation, typical of poor-quality masonry, especially in historical constructions;
- collapse mechanisms due to overturning of rigid bodies;
- global failure mechanisms related to the resistance capacity of masonry walls.

This hierarchy of mechanisms is explicitly described in key references concerning the evaluation criteria of seismic capacity in existing masonry buildings [4,5,6,7].

Therefore, structural models must account, especially in the case of historical masonry, for issues related to disaggregation: for walls vulnerable to such phenomena, a macroelement structural behavior cannot be identified, as the structure tends to decompose under cyclic seismic actions, and analytical models based on resistance and deformability parameters lose their significance. For this reason, disaggregative behavior precedes other resistant mechanisms in the hierarchy.

2 INFLUENCE OF MASONRY QUALITY INDEX ON DISAGGREGATION

The mechanism of masonry disaggregation and, more broadly, the structural behavior of existing masonry buildings, are closely linked to the quality of the load-bearing walls, particularly masonry typology and texture. An appropriate masonry quality indicator, based on the inspection of masonry construction, can effectively identify the mechanical properties of the masonry ([3], §C8.5.3.1).

The Masonry Quality Index (MQI) method, aimed at assessing the mechanical quality of masonry [5,6], is based on a visual inspection of facings and cross-sections of masonry panels, with the goal of verifying adherence to good traditional construction practices. Such inspections yield numerical indices that correlate strongly with both the most significant mechanical parameters of the masonry examined and the expected structural responses.

Furthermore, MQI allows for the assessment of a wall's tendency towards disaggregation under seismic action: an essential consideration for historical masonry, particularly when the masonry exhibits mediocre quality and weak mortar cohesion.

In [5], it is noted how the Out-of-Plane Masonry Quality Index (MQI_o) encapsulates adherence or not to traditional construction practices intended to achieve monolithic behavior.

The authors agree that “to determine a possible threshold value for MQI_o indicating a masonry typology’s propensity for disaggregation, the masonry types identified by Italian technical standards have been analyzed, distinguishing conditions that frequently resulted in masonry disaggregation in recent earthquakes from those associated instead with local/global collapse mechanisms without disaggregation” [5].

Table C8.5.I of the Italian standards [3] provides baseline mechanical parameters corresponding to standardized conditions (proper masonry texture, disconnected wall facings, lime mortar with modest properties, absence of bonding courses, absence of strengthening interventions). Starting from these baseline values, different scenarios can be assessed using correction factors defined in another table (Table C8.5.II).

Amplification parameters may be applied to each masonry typology under the following conditions: good mortar quality, presence of bonding courses (or horizontal tie courses), and systematic presence of transverse elements connecting wall facings. Conversely, cases with particularly poor mortar quality or, in brick masonry, excessively large mortar joints can also be considered.

In [5], referencing masonry typologies proposed by the Italian standards and leveraging post-earthquake experiences from several Italian seismic events, it is recommended to use an $MQI_o=4$ as threshold value for ordinary buildings located in medium-to-high seismic hazard areas. The authors consider this value appropriate for vulnerability assessments. This threshold is also supported by other studies examining the influence of masonry texture on structural behavior [7]. MQI_o values equal to or below 4 indicate potential susceptibility to disaggregation phenomena.

Figure 2 presents the MQI_o values for each masonry typology, distinguished by regular/irregular texture, presence/absence of transverse interlocking, and presence/absence of very poor-quality mortar. The figure highlights the threshold $MQI_o = 4$ with a red horizontal line.

Low-quality masonry typologies (irregular masonry without interlocking, or regular masonry without interlocking and with poor-quality mortar) typically fall below this threshold line. As stated by Borri and De Maria, “values below 4 arise from significant deviations from traditional construction practices,” whereas "both irregular and regular masonry typologies that incorporate transverse interlocking and adequate mortar quality generally have MQI_o values exceeding 4” [6].

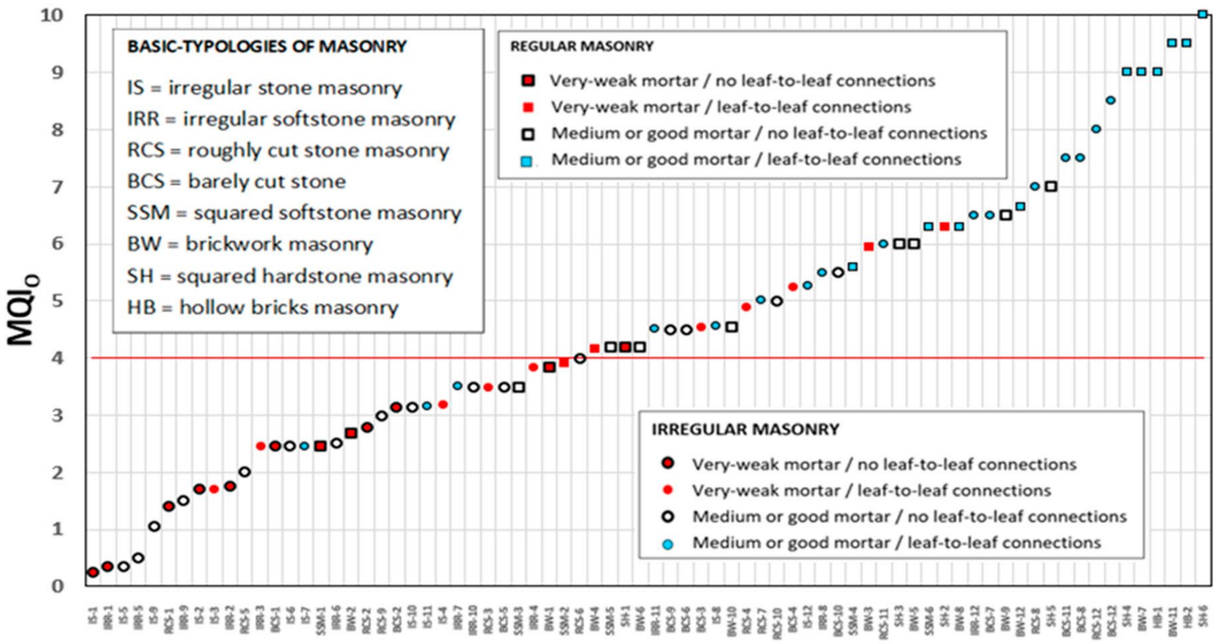


Figure 2. MQI_O values for the masonry typologies suggested by the Italian standards [5]

The poor quality identified by $MQI_o \leq 4$ is a necessary condition for disaggregation, but for the phenomenon to occur, a sufficiently high seismic acceleration intensity is also required [6]. Denoting by a_D the acceleration threshold needed to trigger disaggregation, this threshold will depend on masonry quality, specifically on the out-of-plane MQI value. Each masonry typology present within a building is thus characterized by a corresponding value of a_D, defined by the function: $a_D = a_D(MQI_o)$.

Another condition that can enhance disaggregation is the wall being external: less restraint provided by adjacent structures, or greater freedom of movement, facilitates the detachment of materials. This condition can be accounted for in structural models by applying disaggregation verifications specifically to external walls.

Starting from ground values indicated by PGA, the horizontal acceleration undergoes amplification along the building's elevation. The seismic acceleration acting on a structural element depends on the element’s height (for instance, the base height of a masonry wall, corresponding to the floor level where it is located) and on the dynamic characteristics of the building. By means of the formulation of floor spectra (§C7.2.3 [3]), it is possible to define a spectral acceleration at height z, affecting a given wall based on its position, the dynamic properties of the structure, and the site location:

$$a_z(z) = S_e(T_1, \xi) \cdot \gamma_1 \cdot \psi_1(z) \cdot \sqrt{1 + 0.0004\xi^2} \tag{1.1}$$

where:

T_1 is the fundamental vibration period of the structure. T_1 can be estimated using the relation (C7.3.2 in [3]): $T_1 = C_1 H^{3/4}$, where C_1=0.050 for masonry buildings and H is the building height in meters. Alternatively, T_1 can be determined through a modal analysis.

$S_e(T_1, \xi)$ is the elastic response spectrum evaluated for the period T_1 and viscous damping ratio ξ (5%).

γ_1 is the modal participation factor of the fundamental vibration mode, which can be assumed as: [3N/(2N+1)] where N is the number of floors of the building (C7.2.10 in [3]).

$\psi_1(z)$ is the value of the fundamental mode shape at height z, assumed equal to z/H.

The spectral acceleration a_z must not be lower than the ground acceleration PGA.

Local safety verification for a poor-quality masonry wall prone to possible disaggregation must therefore consider the structural acceleration affecting the wall based on its elevation and the dynamic properties of the building. The position in elevation is indeed a relevant factor regarding possible disaggregation: seismic damage has shown numerous cases, documented in the references, where disaggregation occurred predominantly in walls located on upper floors, where acceleration reaches higher values while simultaneously the stress levels in masonry are lower compared to lower floors.

Considering the above, verification against disaggregation can be performed following the methods listed below, currently implemented in professional software designed for the analysis of existing masonry buildings [8]:

- Each wall is characterized based on its propensity toward disaggregation: the Out-of-Plane Masonry Quality Index (MQI_o) is calculated, and if $MQI_o \leq 4$, possible disaggregation must be considered in the safety evaluation.
- For each wall with $MQI_o \leq 4$, the structural acceleration at the wall's base is evaluated through the floor response spectrum, considering site seismic data, the wall's position along the building's elevation, and the building's dynamic properties.
- If the structural acceleration exceeds the threshold acceleration a_D, specified as input based on the masonry typology constituting the wall, the disaggregation verification is not satisfied. By defining a_D as "capacity" and the structural acceleration affecting the wall as "demand," and considering that both accelerations can be related to ground-level PGA, it is possible to define a safety coefficient as the ratio between capacity and demand in terms of PGA. This coefficient coincides with the seismic risk index ζ_E, which assumes a specific value for each masonry pier. The minimum value among all identifies the building's overall capacity against disaggregation. This mechanism is then inserted into the hierarchy of structural behaviors within the comprehensive results concerning the building's safety verification.

Based on the illustrated methodology, for a masonry wall, the tendency towards disaggregation (D) is defined as a function of its Out-of-Plane Masonry Quality Index (MQI_o). A lower MQI_o value indicates a higher likelihood of disaggregation. The function linking MQI_o to disaggregation incorporates the threshold acceleration a_D and the set of geometric and seismic parameters (P) that characterize the wall's position within the building, as well as the spectral acceleration at its base, depending on the site's seismic zone and the dynamic properties of the building.

$$D = f(MQI_o; a_D; P), \quad with\ a_D = a_D(MQI_o) \tag{1.2}$$

If $MQI_o > 4$, it is assumed that the disaggregation mechanism cannot be activated. Below 4, the lower the value of MQI_o the higher the propensity for disaggregation.

Based on the above, the methodology is summarized schematically in Figure 3.

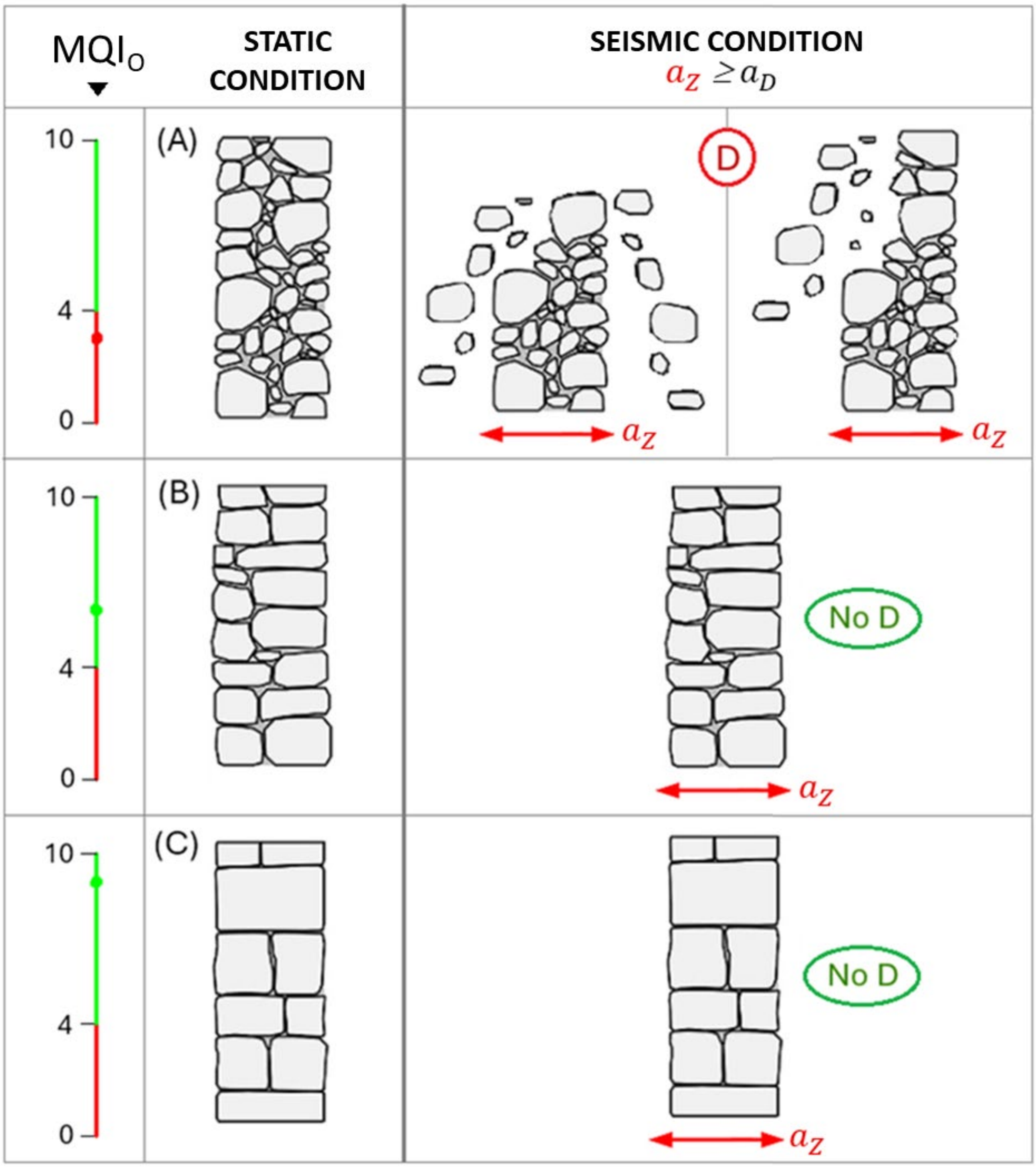


Figure 3. Out-of-plane Masonry Quality Index and disaggregation mechanism: $D = f(MQI_O; a_D; P)$

In Figure 3, three types of stone masonry from those listed in the Italian Standards have been selected. They are distinguished by the constitutive material and the masonry texture, and are organized in order of increasing quality:

(A) Rubble stone masonry, with $MQI_o \leq 4$;
(B) Roughly cut stone masonry, with $MQI_o > 4$;
(C) Well-organized squared hardstone masonry, with $MQI_o >> 4$.

A seismic event intense enough to produce horizontal acceleration at the base of the wall exceeding the disaggregation threshold will trigger the disaggregation mechanism in type (A) masonry. In contrast, masonry types (B) and (C) are not susceptible to this type of failure. The distinction between types (B) and (C) is functional to the developments discussed in the following sections of this work (see Figure 12).

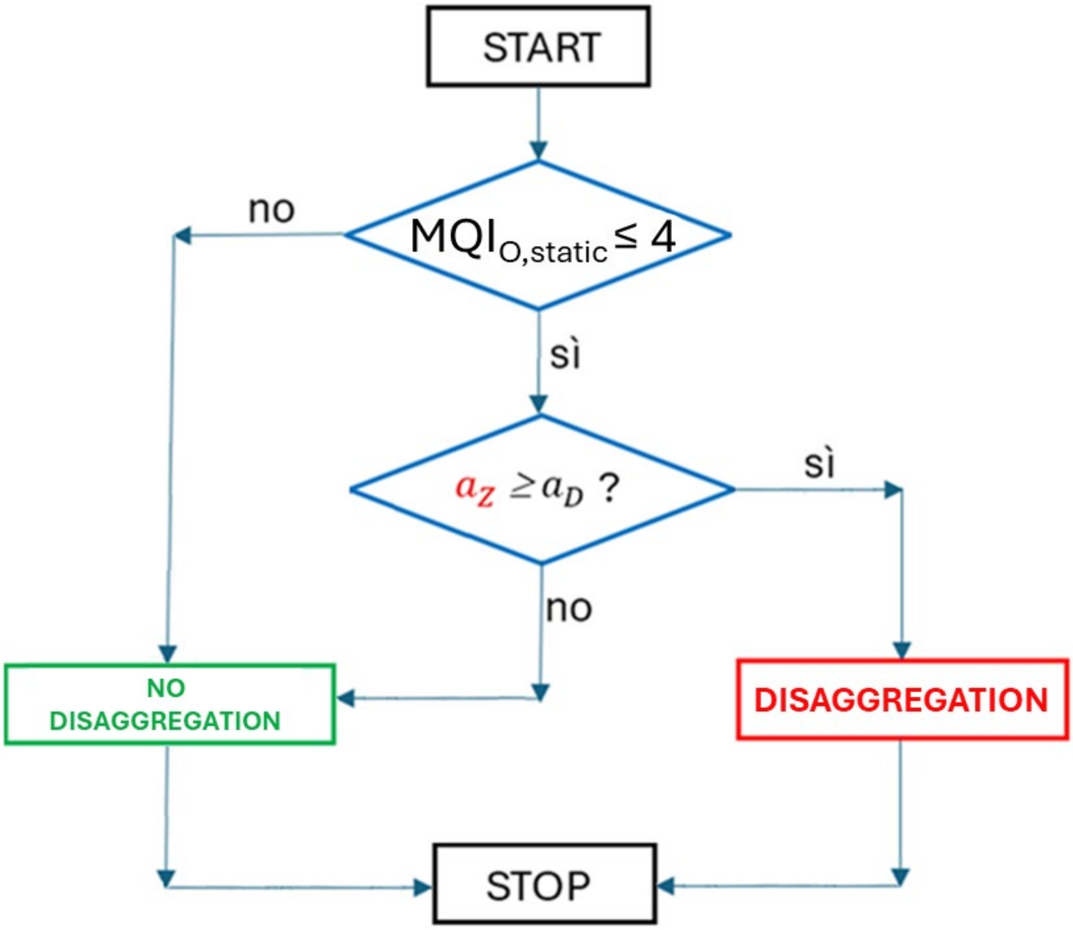


Figure 4. Process for assessing the disaggregation tendency of a masonry wall under seismic action.

3 INFLUENCE OF SEISMIC JERK ON MASONRY DISAGGREGATION

As outlined above, the tendency of masonry to disaggregate is considered an intrinsic property of the material. For any given wall within a building located in given a seismic zone, a seismic risk index for disaggregation can be defined based on the type of masonry and its key structural characteristics.

However, masonry quality can evolve during a seismic event. As quality degrades over time, the tendency to disaggregation may increase accordingly, following the relationship between disaggregation and Masonry Quality Index.

In 2007, Meyer et al. investigated seismic damage in existing masonry buildings associated with disaggregation mechanisms, with a particular focus on poorly constructed masonry using very weak mortar [9]. The authors found that high-frequency components of seismic motion, as they travel up the height of a masonry building, can trigger disaggregative mechanisms that are largely underestimated. These high-frequency waves can produce small vertical vibrations between masonry units, eventually causing irreversible relative displacements that lead to structural failure.

The study also found that in walls with an inner core composed of loose or poorly bonded material, the outward thrust tends to increase as the core begins to exhibit fluid-like behavior. Static and dynamic experimental tests, backed by numerical modeling, confirmed the potentially destructive effects of high-frequency seismic waves on unreinforced masonry structures.

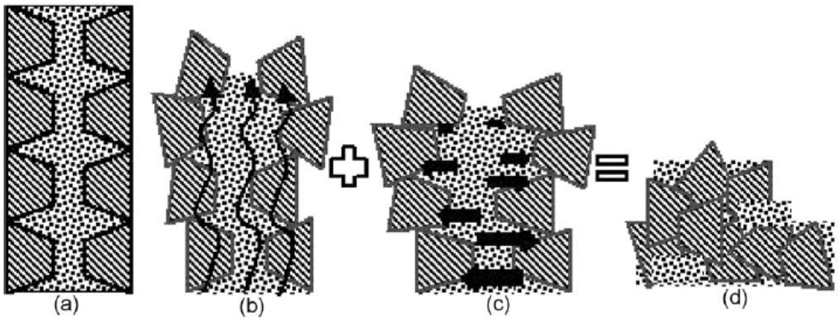


Figure 5. Collapse mechanism triggered by high-frequency seismic waves [9]

The evolution of the disaggregation process is illustrated in Figure 5: (a) in a two-leaf masonry wall with a core of loose material; (b) seismic vibrations cause the stone elements to shift; (c) the core begins to compact and take on fluid-like behavior, increasing internal lateral pressure; (d) eventually, the wall collapses.

Through experimental testing on unreinforced two-leaf masonry walls with internal core, placed on shaking tables subjected to both vertical and horizontal excitation, Meyer et al. observed disaggregation-type damage occurring at frequencies between 10 Hz and 20 Hz and horizontal accelerations ranging from 0.150 g to 0.450 g[1]. The failure threshold was found to depend on the presence of through-stones (diatones), which enhance masonry quality and raise the level of acceleration needed to cause collapse. The disaggregation mechanism results from the combined effect of horizontal and vertical accelerations, with high-frequency vertical vibrations playing a critical role.

Figure 6 shows on the right the experimental results obtained by Meyer et al. on two-leaf stone masonry walls with a loose internal core. The walls are categorized based on the presence of through-stones: (a) absent, (b) occasionally present, and (c) widely present.

As illustrated in the figure, each of the three wall types can be associated with a corresponding Masonry Quality Index, considering roughly cut stone masonry, very-weak mortar, and varying the level of connection between the leaves.

This results in three pairs of values (MQI_o, a_D): (1.40, 0.19g), (2.45, 0.32g), (3.50, 0.45g). As the MQI_o increases from 1.40 to 3.50, the acceleration threshold for disaggregation a_D rises from 0.19g to 0.45g.

As previously noted, $MQI_o \leq 4$ is a necessary condition for disaggregation to occur; however, the phenomenon also requires sufficiently high structural acceleration in order to be triggered [6]. The experimental tests presented, conducted on masonry types susceptible to disaggregation, provide a useful reference for defining a_D a function of MQI_o. The relationship (MQI_o, a_D) identified above can be reasonably extended to other masonry typologies prone to disaggregation, by assigning a_D = 0.15 g for MQI_o = 0, and a_D = 0.45 g for MQI_o = 4. This leads to the definition of a linear function that establishes a one-to-one relationship between out-of-plane Masonry Quality Index values and the corresponding acceleration threshold for disaggregation:

[1] When applied to a real building, the accelerations at the base of the test walls can be interpreted as spectral accelerations acting at the base of actual walls within the structure

$$a_D = 0.150 \cdot \left(\frac{MQI_o}{2} + 1\right) \tag{2.1}$$

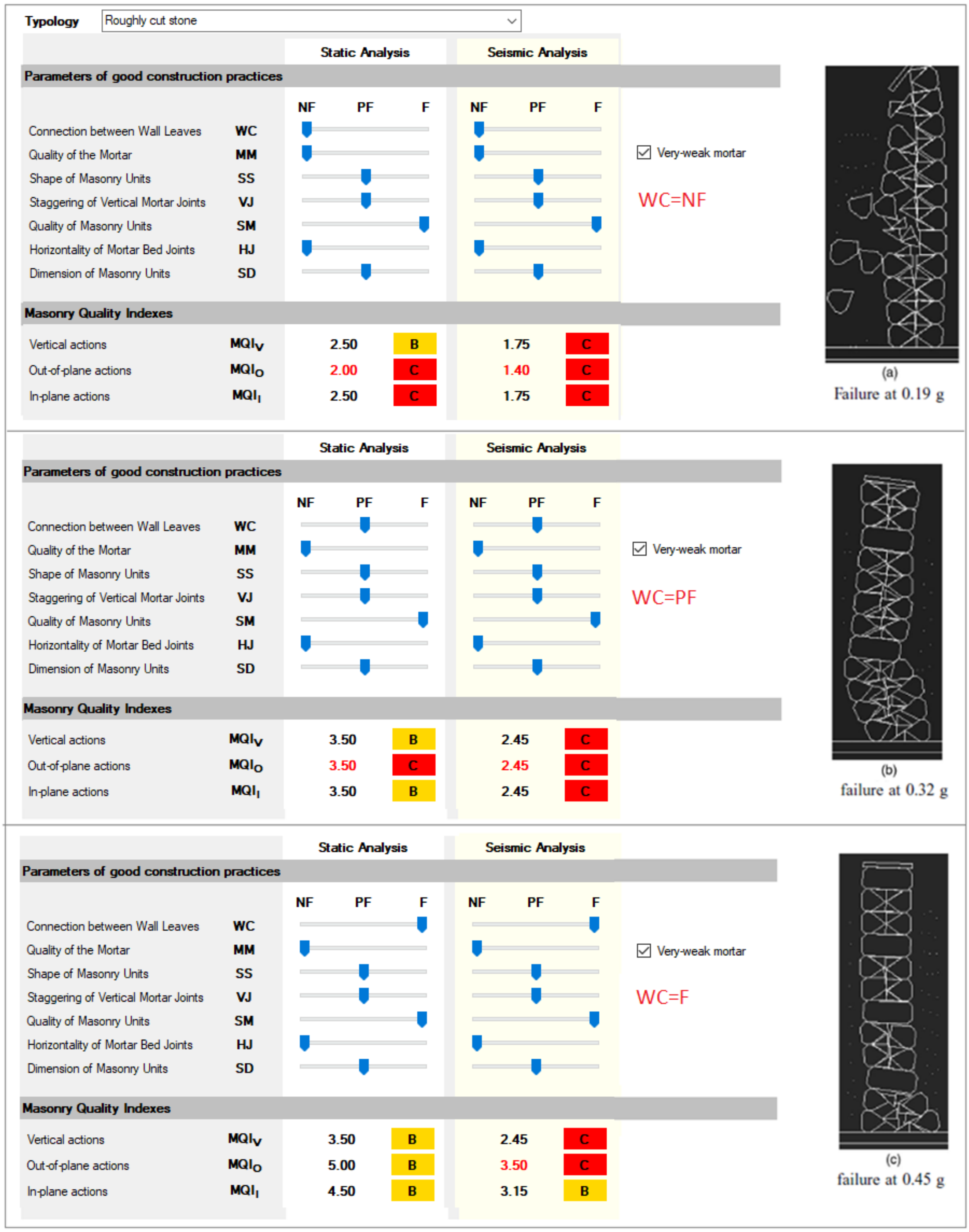


Figure 6. Correspondence between MQIₒ and the acceleration threshold for disaggregation (a_D), based on the presence or absence of through-stones.

The relationship between a_D and MQIₒ, derived from tests that account for high-frequency seismic content, serves as a reference for assessing disaggregation mechanisms even when Jerk intensity is not explicitly considered (see Figure 4). This is justified by the fact that high-intensity earthquakes generally exhibit a significant high-frequency content.

However, the authors believe that the current methodology for assessing vulnerability to disaggregation should be further developed by explicitly incorporating the role of seismic Jerk.

The key assumption is that stronger high-frequency content amplifies disaggregation mechanisms: the shifting of stone elements and the loss of mortar cohesion progressively degrade the quality of masonry during the seismic event, resulting in a lower acceleration threshold a_D. Jerk generates impulsive forces that become increasingly damaging as both their intensity and frequency rise. This perspective allows for the identification of masonry walls that, under normal (non-seismic) conditions, are not prone to disaggregation ($MQI_o > 4$), but that, due to seismic action, experience quality degradation that brings MQI_o below the critical threshold of 4, thereby making them susceptible to disaggregation when subjected to high acceleration.

This provides a plausible explanation for the large number of collapses due to masonry disaggregation observed during the most significant seismic events (those characterized by both high intensity and long duration). Not all these failures can be attributed to the lowest quality of masonry (e.g. rubble stone masonry, missing or very-weak mortar, loose internal core, absence of through-stones). In fact, Figure 2 shows that, even under non-seismic conditions, several masonry typologies have MQI_o values below 4, such as irregular softstone masonry, roughly cut stone masonry, solid bricks masonry with large mortar joints. For this reason, it is reasonable to assume that even masonry types not initially classified as low or poor quality can deteriorate during a seismic event due to mortar disintegration and the shifting of stone elements.

Taking this approach enables more effective strengthening strategies by targeting not only masonry clearly vulnerable to disaggregation (i.e., with $MQI_o \leq 4$ under static conditions), but also masonry whose quality may deteriorate during a seismic event, potentially leading to unexpected brittle collapse.

Figure 7 shows examples of rubble stone masonry and roughly cut stone masonry [4]. Rubble stone masonry is almost always characterized by $MQI_o \leq 4$, while roughly cut stone masonry is more likely to have $MQI_o > 4$. However, during a high-frequency seismic event, even this type of masonry may experience quality degradation, reducing MQI_o and making disaggregation possible, despite the original configuration not indicating such a risk.

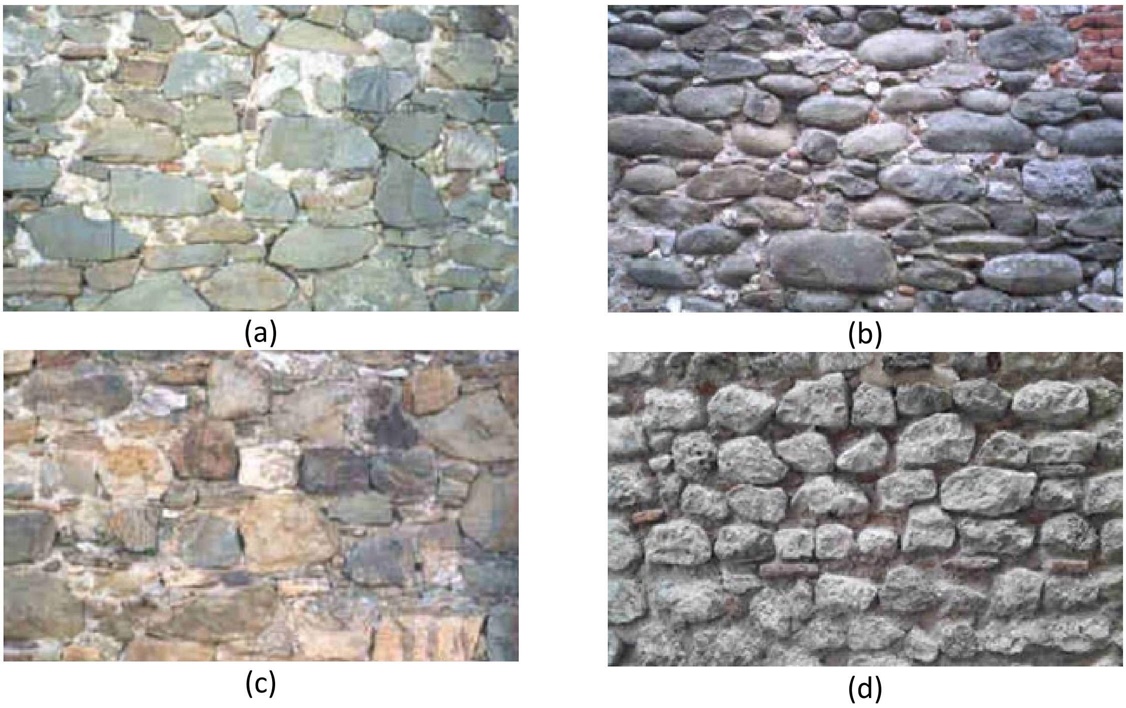


Figure 7. (a, b) Rubble stone masonry: sharp edges and round edges.
(c, d) Roughly cut stone masonry

The concepts related to high-frequency content can be reframed in light of recent research on Jerk, a seismic parameter defined as the derivative of acceleration, which captures the high-frequency content of seismic excitation [10, 11].

In 2005, Tong et al. [10] investigated seismic Jerk, aiming to better understand its amplitude, frequency, and duration. The study was based on data recorded during the Chi-Chi earthquake in Taiwan (M_W 7.6, at 17:47 on September 20, 1999, followed by an aftershock of M_W 6.2 at 00:14 on September 22, 1999). Since Jerk sensors were not yet available (these instruments were only developed and implemented in later years) Jerk data were obtained numerically from recorded acceleration signals. The peak amplitude of Jerk is referred to as **PGJ**, analogous in meaning to the peak ground acceleration (**PGA**). Like acceleration, Jerk is a spatial parameter and can be characterized along the three reference directions: the two horizontal components (NS, EW) and the vertical (Vert. or UD).

The methodology developed by Tong et al. is applicable to any seismic event where Jerk was not directly recorded, and it has been adopted by the authors of this study to analyze data from major recent earthquakes in Italy. Eight significant seismic events that occurred in the Italian territory were considered: Central Italy (30.10.2016), Accumoli (24.08.2016), Emilia (29.05.2012), L'Aquila (06.04.2009), Umbria-Marche (26.09.1997), Irpinia (23.11.1980), Valnerina (19.09.1979), and Friuli (06.05.1976). Accelerometric records for these events are available in the databases of INGV, the Italian National Institute of Geophysics and Volcanology [11].

A statistical study was then carried out on these events, analyzing a total of 447 records, with the aim of determining both the correlation between acceleration and Jerk, and the characteristic frequencies through Fourier spectrum analysis [11].

The PGA–PGJ correlations for the horizontal and vertical components are reported separately in [11]; Figure 8 shows the correlation for the vertical component.

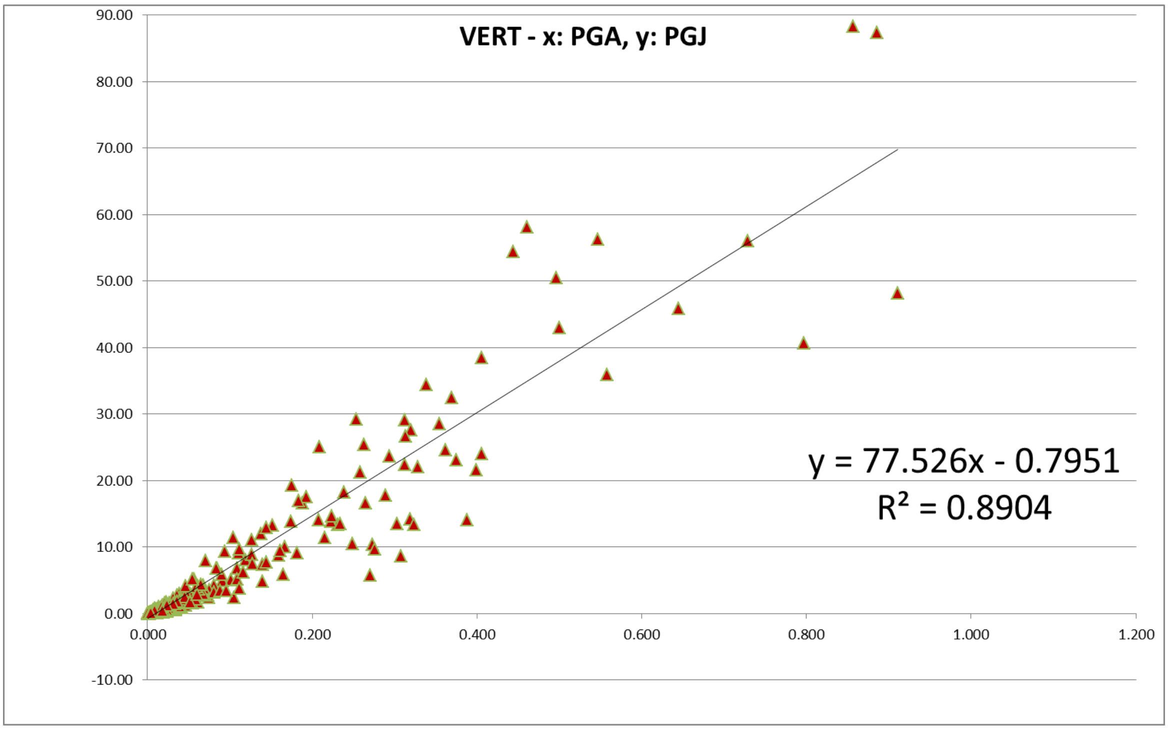
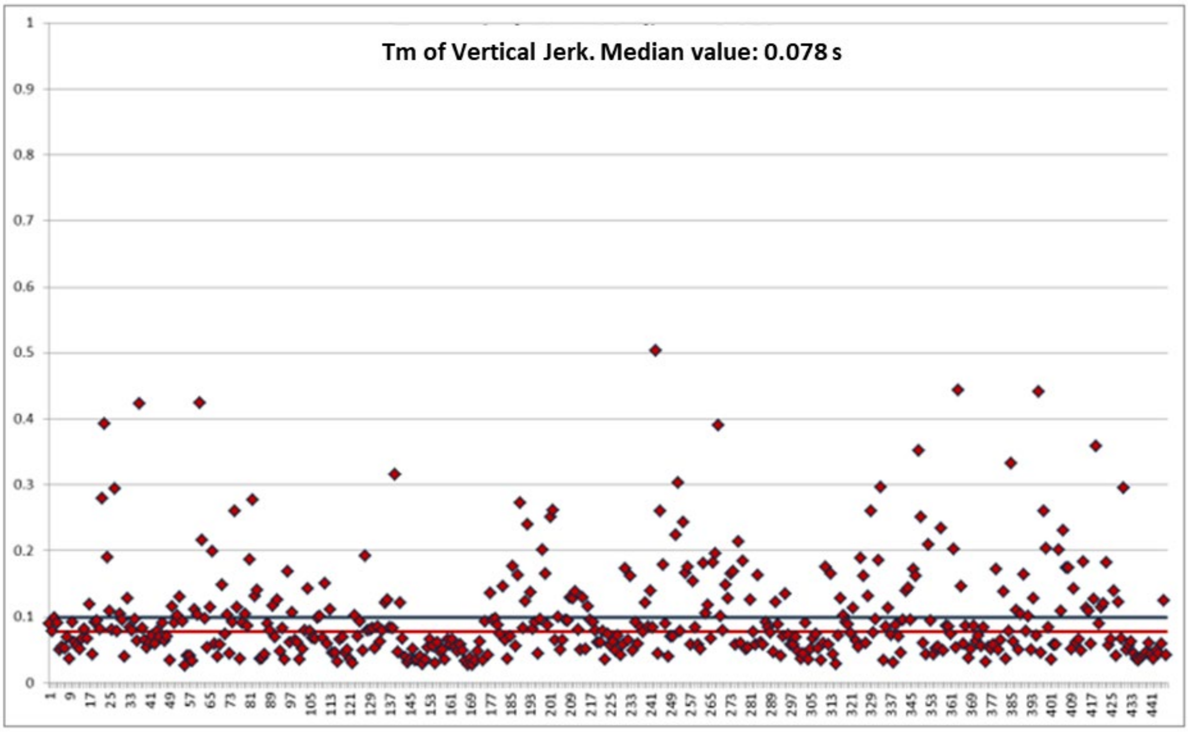


Figure 8. PGA–PGJ correlation for the vertical component.



Class	Frequency	
0.025	0.00%	0
0.05	21.48%	96
0.075	27.52%	123
0.1	19.69%	88
0.125	8.50%	38
0.15	6.71%	30
0.175	4.25%	19
0.2	3.58%	16
0.225	1.79%	8
0.25	0.89%	4
0.275	1.79%	8
0.3	1.12%	5
Other	2.68%	12

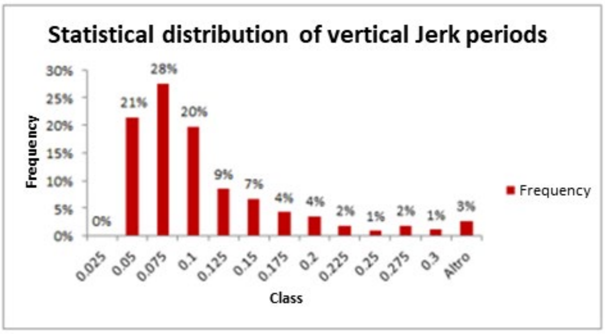


Figure 9. Statistical analysis of vertical Jerk periods [11]

Tong et al. [10] identified 10 g/s as the threshold for very strong Jerk values[2]. They analyzed both horizontal and vertical Jerk, so the threshold values can generally be associated with the spatial Jerk vector, including the vertical component.

Haoxiang He et al. [13] developed horizontal Jerk response spectra for various sites. For example, for natural periods around 0.05–0.1 seconds, the design spectrum for Jerk on a type A soil shows an amplification of approximately 2.5 (Figure 10).

In addition, to quantify the effect of Jerk at a given height within a building, a floor response spectrum for Jerk must be considered. According to Tong, it is reasonable to assume that the amplification of horizontal Jerk with elevation is similar to that of acceleration.

[2] The level of damage also depends on the duration of strong Jerk. However, this parameter is not considered in the present work because, although it can be characterized for existing records, it cannot be conceptually extended to future events and is therefore not suitable for use in preventive design. In general, “high-intensity earthquakes” should be understood as events characterized not only by high acceleration and Jerk, but also by a duration sufficient to cause significant damage.

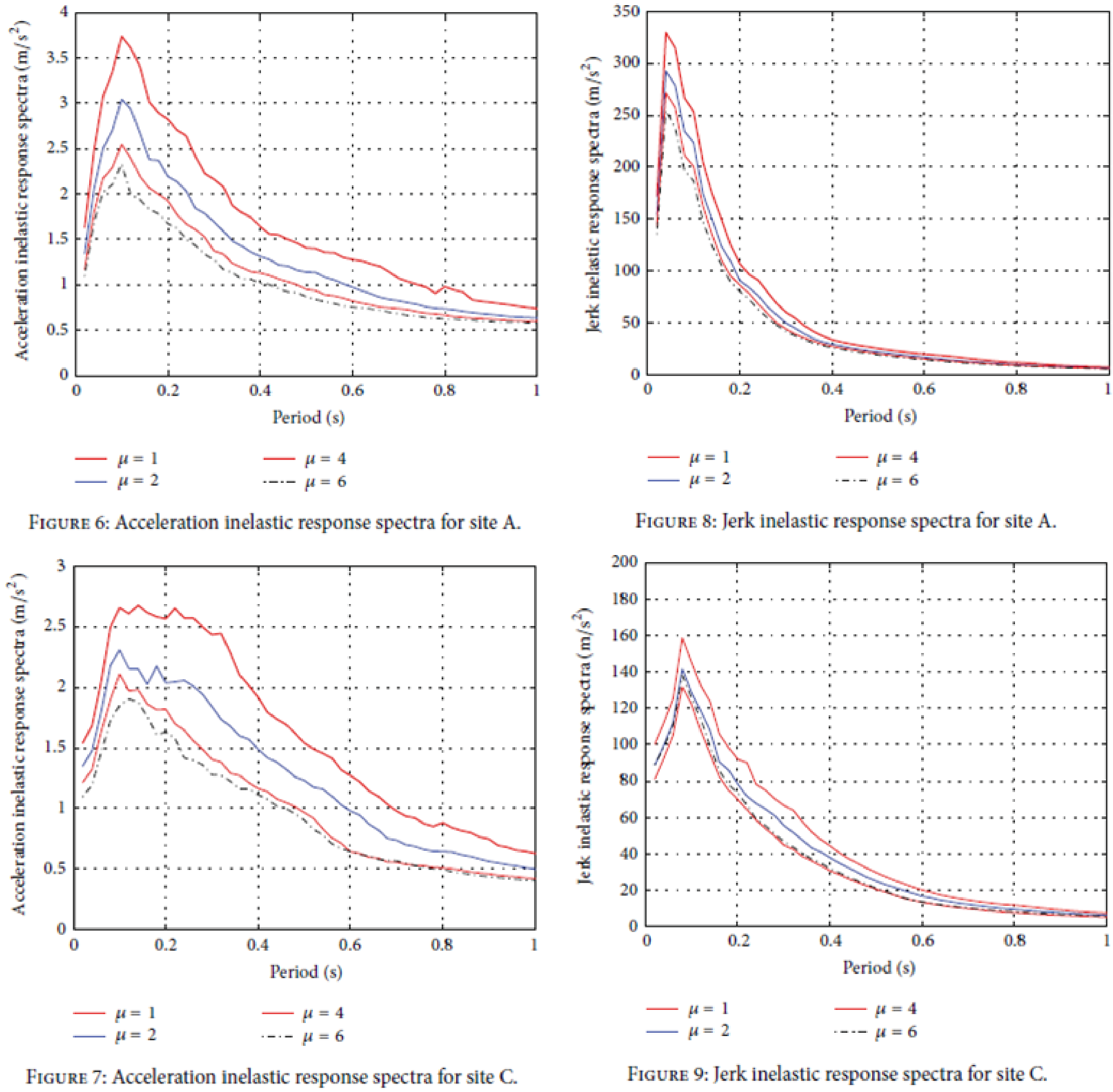


Figure 10. Inelastic response spectra in terms of horizontal acceleration (m/s^2) and Jerk (m/s^3) [13]

Some considerations are needed regarding the role of horizontal Jerk in the global analysis of buildings. Masonry structures of medium/low quality are typically characterized by low moduli of elasticity and, as a result, horizontal fundamental periods around 0.2–0.3 seconds (corresponding to frequencies between 5 Hz and 3 Hz). These fall within a spectral range where Jerk is generally not amplified and may even be attenuated relative to PGJ. Conversely, masonry types that are stiffer in response to horizontal motion (e.g., solid brick masonry with cement mortar) are more likely to fall within the spectral region where horizontal Jerk is amplified. However, such masonry types are either not susceptible or only marginally susceptible to disaggregation.

Unlike in the horizontal direction, the vertical fundamental period of masonry structures (even those built with low-quality materials) is always very short due to their high axial stiffness. Although vertical Jerk spectra are not currently available (not provided in [13]), it is reasonable to assume that the structure does not act as an attenuating filter. This is because the frequency content of vertical Jerk tends to be high, typically between 10 Hz and 20 Hz, and is of the same order of magnitude as the fundamental vertical frequency of stiff structures such as existing masonry buildings. As a result, the propagation of Jerk from the ground up through the building's height may lead to resonance phenomena, a topic previously investigated by the Authors in [11]. Based on current knowledge, the Authors believe that a sufficiently robust working assumption is to refer to the vertical peak ground Jerk (**PGJ**$_V$) as the reference value, and to account for the possibility that masonry's high vertical stiffness may amplify its effects through resonance.

For design applications, the vertical peak ground acceleration $PGA_V = a_{gV} \cdot S$ (where, according to the Italian Standards, a_{gV} is equal to the horizontal acceleration a_{gH} and S is equal to the topographic coefficient S_T [2]) can be directly used to derive the vertical peak ground Jerk PGJ_V through the statistical relationship given in Figure 8.

$$PGJ_V = 77.526\, PGA_V - 0.795 \tag{2.2}$$

In the building's walls, the vertical Jerk peak is amplified by resonance effects, as previously discussed. The structural vertical Jerk, denoted J_{vs}, is defined as the product of the vertical peak ground Jerk PGJ_v and the resonance amplification factor C_{ampl}:

$$J_{VS} = PGJ_V \cdot C_{ampl} \tag{2.3}$$

Equation (2.3) provides, for the design earthquake, the Jerk value that can be compared with the structural damage threshold J_{VSd}. The resonance amplification factor can be calculated using the following expression [11]:

$$C_{ampl} = \frac{1}{\sqrt{\left(1-\frac{T_1^2}{T^2}\right)^2 + 4\,\xi_{eq}^2 \frac{T_1^2}{T^2}}} \tag{2.4}$$

where:
T_1 is the fundamental period of the structure in the vertical direction;
T is the period of the forcing action, that is the representative period of the vertical Jerk;
ξ_{eq} is the equivalent damping ratio.

When the brittle disaggregation mechanism prevails over other behaviors in the hierarchy of mechanisms, failures due to the overturning of rigid bodies or insufficient strength have either not yet occurred or remain limited. In this case, the vertical natural period T_1 can be evaluated using the unreduced mechanical properties of the materials, and the damping ratio ξ_{eq} can reasonably be assumed to be 5%, as typically adopted for elastic response spectra.

A higher damping ratio might more accurately reflect the dissipative phenomena associated with how vibrations propagate within the building. This behavior depends on the filtering effect exerted by the structure itself, beginning with soil–foundation interaction and extending through the dynamics of the load-bearing system. However, in the absence of more precise data, assuming a 5% damping ratio offers a conservative basis for design.

The amplification factor C_{ampl}, calculated using Equation (2.4), depends on the mean period T representative of vertical Jerk. The analysis presented in Figure 9 highlights the most statistically significant mean periods (each with a frequency of occurrence above 4%), which are listed in Table 1. Together, these periods represent 93% of the 447 Italian seismic records analyzed.

Vertical Jerk Period T (s)	Vertical Jerk Frequency $f = 1/T$ (Hz)	Statistical Frequency
0.050	20.00	21%
0.075	13.33	28%
0.100	10.00	20%
0.125	8.00	9%
0.150	6.67	7%
0.175	5.71	4%
0.200	5.00	4%

Table 1. Representative Vertical Jerk Periods and Statistical Frequency

Figure 11 shows the following curves:

- in blue, red, and green: the amplification factor curves corresponding to the most significant representative vertical Jerk periods: 0.050 s, 0.075 s, and 0.100 s (for simplicity, curves for the remaining period values listed in Table I are omitted);
- in grey: the average of all curves corresponding to the periods in Table I, each weighted by its statistical frequency;
- in black (dashed line): the trend line (a third-degree polynomial), representing the optimized weighted average. This curve is adopted as the reference for calculating the amplification factor C_{ampl} as a function of the structure's vertical fundamental period T_1.

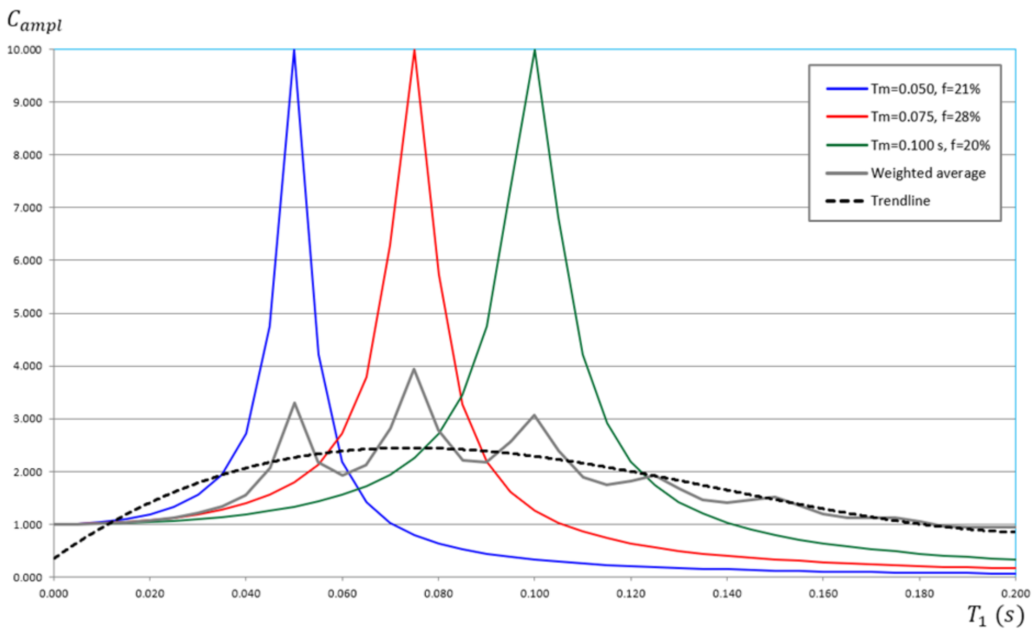


Figure 11. Vertical Jerk amplification factor C_{ampl} as a function of the structure's vertical fundamental period T_1.

For the design purposes of this study, C_{ampl} is assumed to reach its maximum value of 2.5 within the period range of 0.040 to 0.120 seconds. Its variation with respect to the structure's fundamental period is defined by the following expression:

$$C_{ampl} = \begin{cases} 1.0 \; per \; T_{1Z} < 0.020 \; s \\ 1.5 \; per \; 0.020 \leq T_{1Z} \leq 0.040 \; s \\ 2.5 \; per \; 0.040 \leq T_{1Z} \leq 0.120 \; s \\ 1.5 \; per \; 0.120 \leq T_{1Z} \leq 0.160 \; s \\ 1.0 \; per \; T_{1Z} > 0.160 \; s \end{cases} \tag{2.5}$$

The relationship between disaggregation, the Masonry Quality Index, and Jerk is illustrated in Figure 12, with reference to the three distinct masonry types previously introduced in Figure 3[3]. In Figure 3, the three reference masonry types are classified according to MQI_o, considered as an intrinsic property of the material and masonry texture under non-seismic (static) conditions. In Figure 12, the Masonry Quality Index is re-evaluated during the seismic event, reflecting the deterioration caused by the high-frequency content of the seismic excitation (Jerk). This results in a noticeable drop in the index compared to the static state, leading to a greater tendency of the masonry types to undergo disaggregation mechanisms (most notably in type B, roughly cut stone masonry).

The variation in the Masonry Quality Index is expressed by the following equation:

$$\Delta\, MQI_o = \left(MQI_{o,Static} - MQI_{o,Seismic}\right) \tag{2.6}$$

where:

$$MQI_{o,Seismic} = f\left(MQI_{o,Static};\, J_{VS};\, J_{VSd}\right) \tag{2.7}$$

In Figure 12, the downward arrows along the MQI scale indicate the variation in the index. The connection between disaggregation and the seismic Masonry Quality Index, as influenced by Jerk, is summarized by the following function:

$$D = f\left(MQI_{o,Seismic};\, a_D; P\right), \text{with } a_D = a_D\left(MQI_{o,Seismic}\right) \tag{2.8}$$

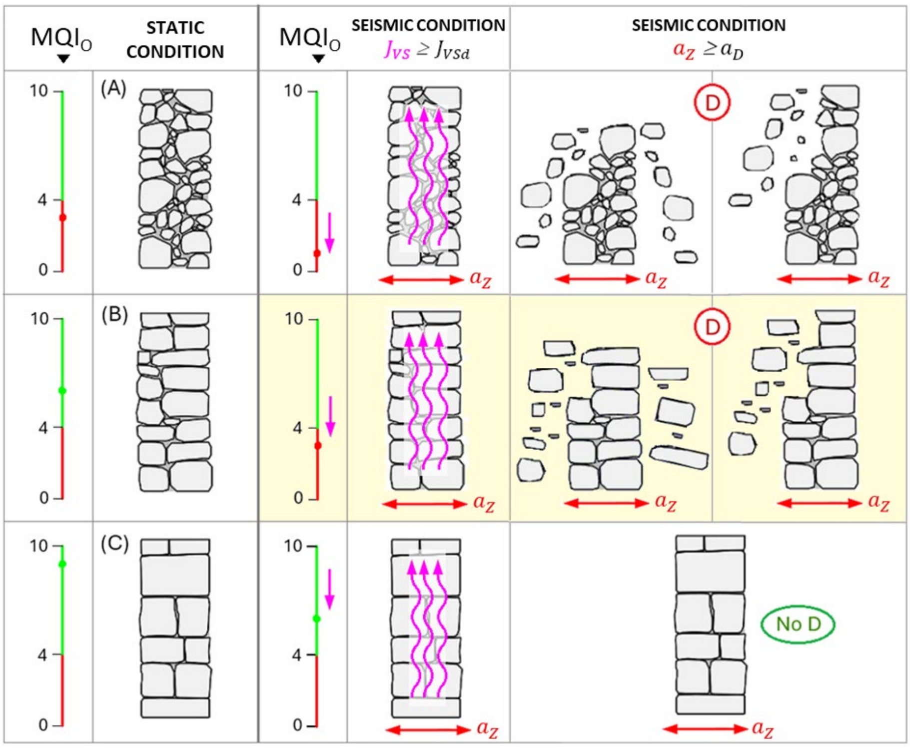


Figure 12. Out-of-plane Masonry Quality Index, modified during the seismic event due to Jerk, and associated disaggregation mechanism: $D = f\left(MQI_{o,Seismic};\, a_D; P\right)$

[3] The connection between Jerk and disaggregation is also supported by other studies. According to Xueshan et al. [14], the propagation of vibrational Jerk waves is directly linked to stress concentrations and localized damage, which, in homogeneous materials, are triggered by the breaking of molecular bonds. In the case of masonry, it is natural to extend this concept to the macroscopic scale, where disaggregation results from the loss of bonding between stone units due to mortar degradation (and, more rarely, from brittle failure of the masonry elements).

The flowchart shown in Figure 13 (to be compared with Figure 4) illustrates the process for evaluating the disaggregation tendency of a masonry wall subjected to seismic loading, taking into account the effect of vertical Jerk.

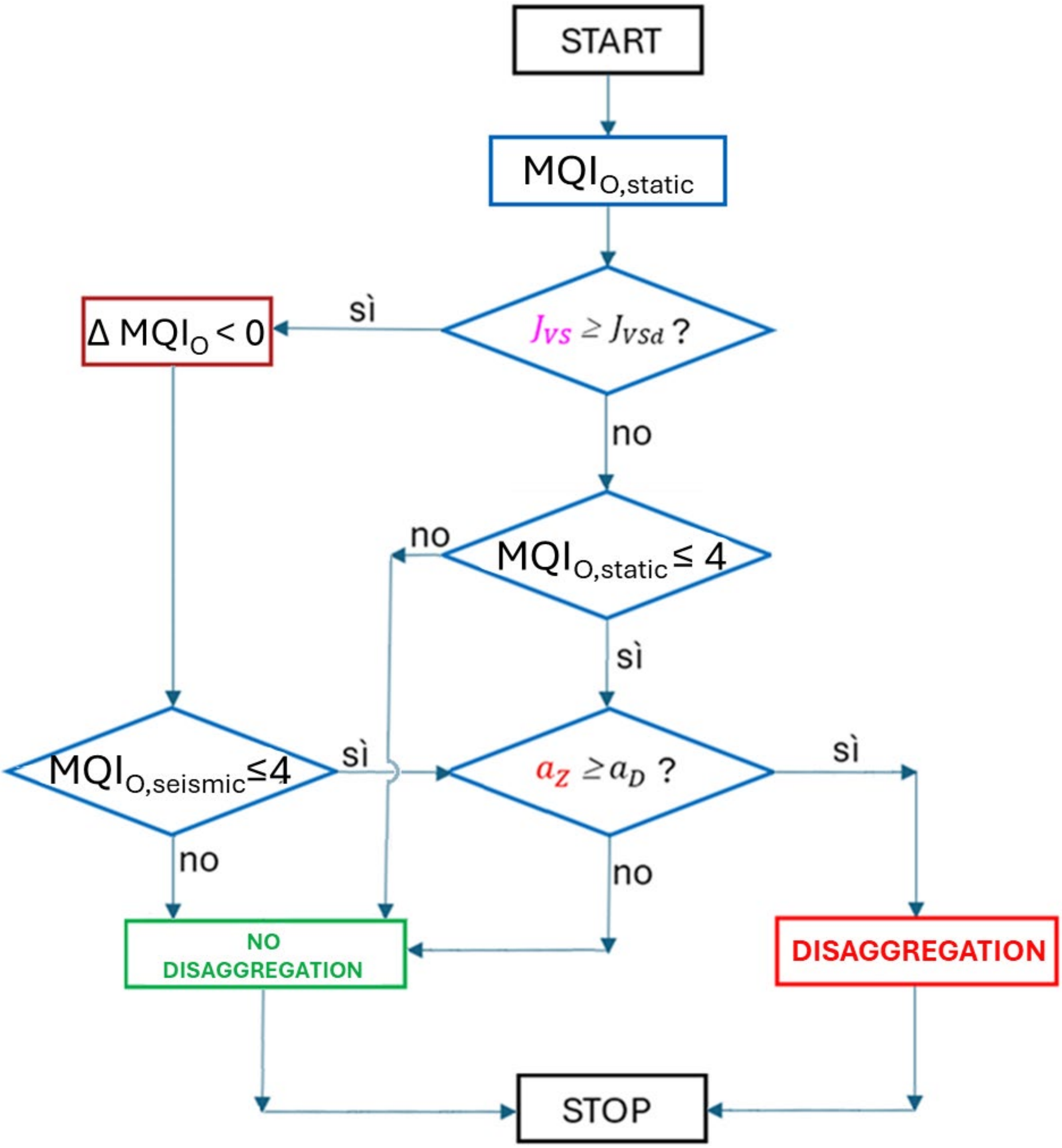


Figure 13. Process for assessing the disaggregation tendency of a masonry wall under seismic loading, accounting for the effect of vertical Jerk.

The process for evaluating disaggregation tendency is based on the definition of the following key points:

1) the value of the acceleration threshold a_D that triggers disaggregation;
2) the structural vertical Jerk threshold J_{VSd}, beyond which masonry damage may occur;
3) the criterion for reducing masonry quality when the structural Jerk J_{vs} exceeds the damage threshold J_{VSd}.

The acceleration threshold a_D has already been discussed.

Turning now to the Jerk threshold: as previously noted, a value of 10 g/s has been identified as a level of strong Jerk capable of inducing structural damage [10]. Based on the relationship between PGA and PGJ (Equation 2.2), a PGJ greater than 10 g/s corresponds to a PGA exceeding 0.139 g. When this damage threshold is interpreted as the disaggregation threshold (failure mechanism associated with masonry types having $MQI_o < 4$) it aligns well with the acceleration threshold a_D = 0.150 g, which corresponds to the lowest MQI_o values.

The Authors therefore consider it reasonable to assume that accelerations below 0.150 g are insufficient to trigger disaggregation, and that vertical Jerk levels below 10 g/s are unlikely to generate vibrations strong enough to significantly degrade masonry quality.

As a reference, Table 2 reports the PGA_H and PGJ_V values extracted from the set of 447 records analyzed in [11], which formed the basis for the correlation shown in Figure 8, where PGJ_V exceeds 10 g/s. In Table II, blue indicates values between 10 and 15 g/s, red indicates values between 15 and 20 g/s, and magenta indicates values greater than 20 g/s. All events include one or more records with Jerk values exceeding 10 g/s. For the older events, a smaller number of records are available.

Looking at the more recent earthquakes (those from 2009 onward) all events include records with high Jerk values. The Accumoli earthquake of August 24, 2016, for example, caused widespread disaggregation of buildings in Amatrice (Figure 1). At station IT-AMT, the recorded values were: PGA_H = 0.868 g and PGJ_V = 21.53 g/s. The combination of a very high Jerk peak (over 21 g/s) and a strong horizontal acceleration (0.868 g) created highly favorable conditions for disaggregation, which was clearly observed in numerous masonry walls[4].

It's worth noting that the Norcia earthquake of October 30, 2016, despite also registering high Jerk (peak 23.14 g/s) and horizontal acceleration (peak 0.486 g), did not cause the same level of disaggregation in residential buildings as seen in Amatrice. This is particularly notable given that Norcia had already been heavily impacted by the earlier events of August 24 and October 26. One key difference is that buildings in Norcia were constructed with thick masonry walls, following well-established traditional construction practices, and many had been strengthened with reinforced mortar coating during seismic retrofitting campaigns in the 1980s and 1990s.

Seismic damage evidence shows that disaggregation is the result of a combined effect of horizontal acceleration, vertical Jerk, the type of masonry, and key building characteristics, such as geometry, boundary conditions, applied loads, and the connections between walls and between walls and floors.

[4] During a seismic event, the peaks of vertical Jerk and horizontal acceleration do not occur at the same moment; however, their combined effect does not rely on perfect timing. Through its vibrations, Jerk delivers continuous impulsive shocks that weaken the material, making it more brittle, while acceleration, through its inertial motion, amplifies instability and contributes to the expulsion of material. Disaggregation is therefore the result of a combination of impulsive and inertial effects.

Event	Station			PGA$_{EW}$ (g)	PGA$_{NS}$ (g)	PGJ$_{V}$ >= 10 g/s
1.Friuli	06051976	IT	TLM1	0.316	0.352	**10.31**
2.Valnerina	19091979	IT	CSC	0.211	0.157	**16.63**
3.Irpinia	23111980	IT	STR	0.320	0.225	**13.35**
4.Umbria-Marche	26091997	IT	NCR	0.423	0.502	**24.00**
5.L'Aquila	06042009	IT	AQA	0.403	0.442	**54.42**
5.L'Aquila	06042009	IT	AQV	0.657	0.546	**50.54**
5.L'Aquila	06042009	IT	AQK	0.330	0.354	**24.60**
5.L'Aquila	06042009	MN	AQU	0.260	0.308	**22.36**
5.L'Aquila	06042009	IT	AQG	0.446	0.489	**18.24**
6.Emilia	29052012	IT	MRN	0.223	0.294	**88.28**
6.Emilia	29052012	TV	MIR02	0.217	0.237	**58.08**
6.Emilia	29052012	BA	MIRE	0.177	0.271	**56.04**
6.Emilia	29052012	BA	MIRH	0.150	0.270	**42.96**
6.Emilia	29052012	TV	MIR03	0.208	0.327	**38.44**
6.Emilia	29052012	TV	MIR01	0.419	0.380	**32.43**
6.Emilia	29052012	IT	SAN0	0.174	0.221	**29.04**
6.Emilia	29052012	IV	T0800	0.337	0.254	**27.60**
6.Emilia	29052012	TV	MIR08	0.223	0.248	**26.64**
6.Emilia	29052012	TV	MIR04	0.400	0.307	**25.35**
6.Emilia	29052012	IV	T0818	0.243	0.279	**25.05**
6.Emilia	29052012	IV	T0814	0.444	0.505	**21.23**
6.Emilia	29052012	IV	T0813	0.368	0.338	**19.29**
6.Emilia	29052012	IT	FIN0	0.212	0.239	**17.50**
6.Emilia	29052012	IV	T0802	0.264	0.296	**13.83**
6.Emilia	29052012	TV	MIR05	0.177	0.275	**13.31**
6.Emilia	29052012	IT	SMS0	0.178	0.179	**11.36**
6.Emilia	29052012	IV	T0811	0.193	0.206	**11.01**
7.Accumuli	24082016	IT	AMT	0.868	0.376	**21.53**
7.Accumuli	24082016	IT	NRC	0.360	0.374	**11.43**
7.Accumuli	24082016	IT	NOR	0.202	0.180	**10.51**
8.Central Italy	30102016	IV	T1213	0.795	0.867	**87.33**
8.Central Italy	30102016	IT	CNE	0.476	0.294	**56.27**
8.Central Italy	30102016	IT	FCC	0.950	0.860	**48.14**
8.Central Italy	30102016	IV	T1214	0.605	0.421	**45.91**
8.Central Italy	30102016	IT	CLO	0.427	0.583	**40.70**
8.Central Italy	30102016	IT	ACC	0.434	0.392	**35.88**
8.Central Italy	30102016	IT	MCV	0.292	0.359	**34.46**
8.Central Italy	30102016	IT	ACT	0.279	0.400	**29.21**
8.Central Italy	30102016	IV	T1244	0.286	0.192	**28.57**
8.Central Italy	30102016	IV	T1299	0.454	0.445	**23.71**
8.Central Italy	30102016	IT	NRC	0.486	0.372	**23.14**
8.Central Italy	30102016	3A	MZ08	0.537	0.436	**22.09**
8.Central Italy	30102016	IT	NOR	0.312	0.294	**17.77**
8.Central Italy	30102016	IT	PRE	0.250	0.311	**16.99**
8.Central Italy	30102016	3A	MZ24	1.021	0.762	**16.65**
8.Central Italy	30102016	IT	FOC	0.380	0.342	**14.62**
8.Central Italy	30102016	3A	MZ102	0.372	0.405	**14.18**
8.Central Italy	30102016	3A	MZ04	0.646	0.809	**14.12**
8.Central Italy	30102016	3A	MZ19	0.363	0.403	**14.01**
8.Central Italy	30102016	3A	MZ29	0.689	0.413	**13.78**
8.Central Italy	30102016	3A	MZ30	0.462	0.484	**13.52**
8.Central Italy	30102016	IV	T1201	0.346	0.483	**13.51**
8.Central Italy	30102016	IT	AMT	0.532	0.401	**13.43**
8.Central Italy	30102016	IV	T1220	0.239	0.257	**12.87**
8.Central Italy	30102016	IT	CIT	0.326	0.213	**11.94**

Table 2. Peak ground vertical Jerk values exceeding 10 g/s in recent Italian seismic events and corresponding peak ground horizontal accelerations (sorted by event and by increasing PGJ)

Building on the analysis of the key factors in assessing disaggregation tendency, the criterion for modeling Jerk-induced damage involves identifying which aspects of masonry quality are most affected by Jerk, ultimately leading to degradation. This is necessarily a qualitative approach, similar to the assessments used in the MQI, and can reasonably be framed around key construction practice parameters known to influence disaggregation [4,5]:

MM Quality of the mortar/interaction between masonry units;
WC Level of connection between adjacent wall leaves;
HJ Horizontality of mortar bed joints.

The disaggregating effect on mortar is the primary consequence of vertical vibrations, accompanied by the displacement of stone elements, which further weakens any existing transverse connections and disrupts the horizontality of the masonry courses, if originally present.

The Masonry Quality Index, based on visual inspection of wall facings and cross-sections, is determined by evaluating compliance with good construction practices. Each parameter is assessed using the following ratings [5]:

F Fulfilled;
PF Partially Fulfilled;
NF Not Fulfilled.

Therefore, three quality classes are defined for each parameter. For mortar, however, based on the guidelines provided in the Italian Standards [3], an additional class is included to account for very-weak mortar. This class represents the lowest level in the classification, resulting in a total of four quality classes for mortar.

The detrimental effect of Jerk on masonry quality can be modeled as a reduction in the quality class, applied separately to each of the three reference parameters. The extent of this reduction increases with higher Jerk values. The approach proposed in this study is outlined in Table 3. To produce a noticeable impact on WC (transversal connection) and HJ (horizontality of mortar joints), the Jerk must be significantly higher than what is required to degrade mortar quality. Additionally, it is useful to differentiate between masonry types:

- Irregular masonry, made with unshaped stones (rubble stone masonry);
- Roughly cut stone masonry;
- Regularly coursed masonry, which may also be affected by disaggregation, especially in cases with thick mortar joints.

Moreover, very high values of vertical Jerk are associated with a reduction of the out-of-plane Masonry Quality Index to its minimum (zero). In this condition, all construction practice parameters are considered no longer fulfilled: the masonry is assumed to be severely damaged and equated to the poorest quality condition. The corresponding acceleration threshold for the onset of disaggregation is therefore set at 0.150 g, which is the value associated with $MQI_o = 0$, according to the formulation previously described.

<table>
<tr><th></th><th>$10 \le J_{VS} < 20$</th><th>$20 \le J_{VS} < 30$</th><th>$30 \le J_{VS} < 40$</th><th>$40 \le J_{VS} < 50$</th><th>$50 \le J_{VS} < 60$</th><th>$J_{VS} \ge 60$</th></tr>
<tr><td colspan="7">Rubble stone masonry</td></tr>
<tr><td>MM</td><td>-1</td><td>-2</td><td>-3</td><td colspan="3" rowspan="3">$MQI_o = 0$
$a_D = 0.150$ g</td></tr>
<tr><td>WC</td><td>0</td><td>-1</td><td>-2</td></tr>
<tr><td>HJ</td><td>0</td><td>-1</td><td>-2</td></tr>
<tr><td colspan="7">Roughly cut stone masonry</td></tr>
<tr><td>MM</td><td>0</td><td>-1</td><td>-2</td><td>-3</td><td colspan="2" rowspan="3">$MQI_o = 0$
$a_D = 0.150$ g</td></tr>
<tr><td>PD</td><td>0</td><td>0</td><td>-1</td><td>-2</td></tr>
<tr><td>HJ</td><td>0</td><td>0</td><td>-1</td><td>-2</td></tr>
<tr><td colspan="7">Regularly coursed masonry</td></tr>
<tr><td>MM</td><td>0</td><td>0</td><td>-1</td><td>-2</td><td>-3</td><td rowspan="3">$MQI_o = 0$
$a_D = 0.150$ g</td></tr>
<tr><td>WC</td><td>0</td><td>0</td><td>0</td><td>-1</td><td>-2</td></tr>
<tr><td>HJ</td><td>0</td><td>0</td><td>0</td><td>-1</td><td>-2</td></tr>
</table>

Table 3. Proposed downgrading of construction practice parameters due to the effect of vertical Jerk (J_{VS} in g/s).

The damage threshold values for structural vertical Jerk ($J_{vs}d$) are defined in Table 3 using intervals of 10 g/s, starting from 10 g/s up to 60 g/s and beyond. A more robust definition of the downgrading criteria will be possible through experimental campaigns and model updating procedures. In particular, when the original structural properties of a group of buildings are known and those buildings are subsequently damaged by a real seismic event, model updating based on observed damage can be used to refine both the analytical models and the assessment procedures. This, in turn, allows for a more precise identification of damage thresholds. Essentially, the model is recalibrated so that its output corresponds to the actual damage caused by the recorded seismic event, using parameters derived from ground motion data.

Given the current state of knowledge, this study proposes a methodological framework that adopts a plausible approach, consistent with the referenced literature, for both the damage threshold parameters (a_D and J_{VSd}) and the downgrading criteria outlined in Table 3. While further research is still needed, this approach already allows designers to account for the potential impact of high-frequency seismic effects during the design process, helping to enhance the safety of existing masonry structures.

In the following section, the proposed methodology is applied to a case study in which the disaggregation tendency is assessed both with and without considering the effects of Jerk[5].

[5] The negative effects of Jerk impact not only the internal composition of the masonry material, that is, the mechanical properties of the load-bearing walls, but also the structural connections. As a result, Jerk-induced damage can also occur in masonry structures made with good-quality materials (such as solid brick or well-textured, regularly coursed masonry) if the structural connections are inadequate or undersized. This study focuses specifically on Jerk-related damage affecting the internal composition of the masonry material and its consequences on the dynamic behavior of the structural system. Further insights into Jerk-induced damage at structural nodes and connections can be found in other publications by the Authors [15]. More broadly, Jerk may affect the internal structure of masonry not only depending on the intensity but also the duration of the seismic event. Even when damage is not immediately visible, it can weaken the material, increasing its vulnerability to future earthquakes. This highlights the importance of studying the concept of “damage memory”, that is, the degradation accumulated in structural materials due to past seismic events and its compounding effects in the case of repeated shocks. Historical analysis, a key step in assessing the structural properties of existing buildings, should take into account the potential impact of previous earthquakes on masonry quality, even when damage is not readily apparent. In such cases, appropriate investigative techniques can be used to detect hidden cracks or internal deterioration. This is a topic referenced in key studies [1], but one that, as of now, still requires further development in the field of research.

The following example refers to a three-story residential building, representative of the typologies that exhibited disaggregation after major seismic events.

Figures 14 and 15 show photographs documenting the damage observed in Amatrice (2016) and Friuli (1976); similar images can be found in photographic archives from other earthquakes. It is worth noting that disaggregation tends to spare taller masonry structures, such as towers, suggesting a possible connection between the absence of disaggregation in towers and the effects of Jerk on masonry quality.

Figure 14. Seismic damage in Amatrice (2016).
Masonry towers that were unaffected by the widespread disaggregation are highlighted.

Figure 15. Seismic damage in Friuli (1976). A bell tower unaffected by disaggregation is highlighted.

Towers are generally built with high-quality masonry (e.g. squared stone blocks) and typically show little tendency towards disaggregation. During a seismic event, the reduction in quality caused by Jerk may not be sufficient to trigger disaggregation in such structures.

As previously noted (see Figure 11), the amplification coefficient for Jerk decreases for vertical natural periods greater than 0.070 s. The taller the building, the higher its natural period, and the lower the Jerk amplification. Therefore, it is believed that in taller structures, Jerk is less likely to degrade masonry quality to the point of initiating disaggregation.

4 CASE STUDY: ANALYSIS OF DISAGGREGATION MECHANISMS

The evaluation of disaggregation mechanisms is performed on an existing masonry building, hypothetically located in two different seismic zones affected by significant events: Fivizzano (MS) and Amatrice (RI), which has a higher level of seismicity.

The building has three stories, with the ground floor originally built in an initial construction phase, followed by subsequent additions both laterally and in elevation. The original masonry, located in the central part of the ground floor, is made of roughly cut stones. The upper floors were also constructed using roughly cut stones, though of lower quality due to the absence of through-stones. The lateral extension is made of solid bricks and lime mortar, with joints thicker than 13 mm (Figure 16). Each of these masonry types could, under certain conditions, be impacted by seismic vibrations[6].

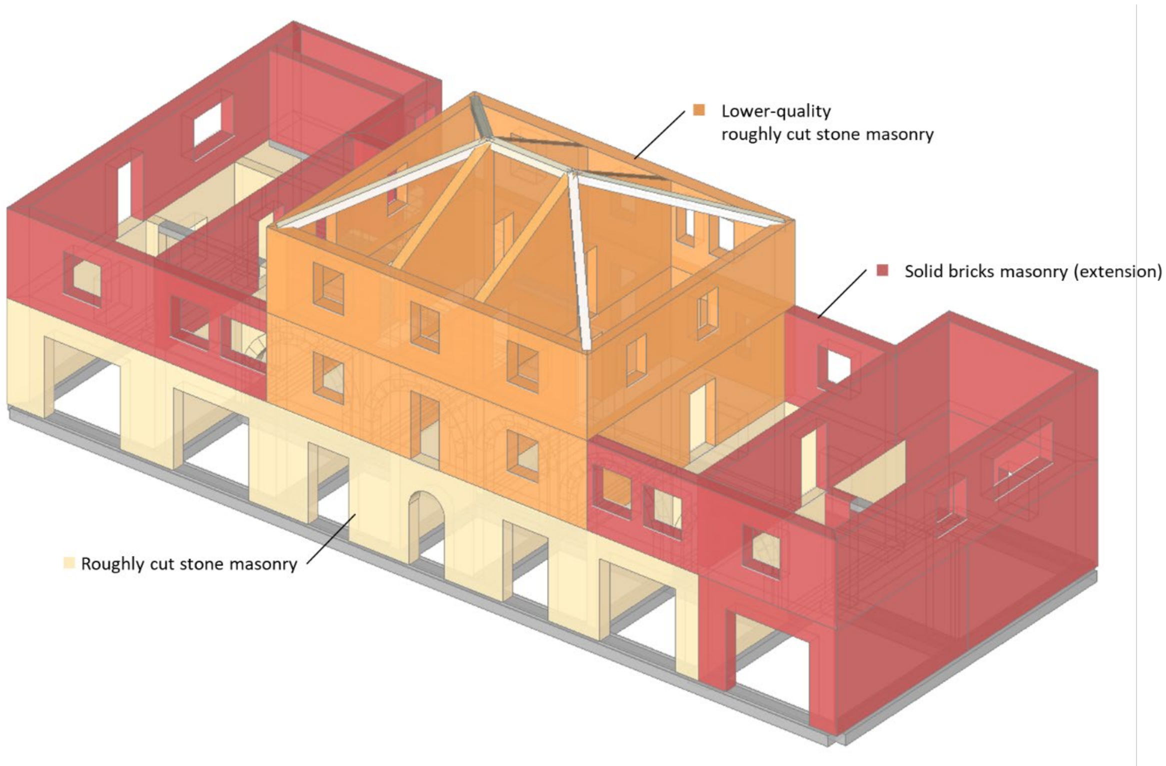


Figure 16. Case study building and its constituent masonry materials.

[6] In the context of this study, it was considered appropriate not to use an example featuring poor-quality rubble stone masonry, which is inherently prone to disaggregation. Instead, the aim is to show how even masonry of modest quality, but reasonably organized, can be affected by high-frequency seismic content through the degradation of its quality during the event.

The assessment must evaluate both the vulnerability of the current state and a proposed strengthening intervention, with the requirement that the seismic risk index increases by at least $\Delta\zeta_E \geq 0.10$. Additionally, the client requests a higher level of safety with respect to disaggregation: the proposed intervention must ensure compliance with disaggregation safety requirements, aiming for a seismic risk index of $\zeta_E \geq 0.80$ for this specific failure mechanism[7].

The analysis of the building, including the safety verification against disaggregation, is carried out through the following modeling configurations:

(A.1), (A.2) Current state, without considering Jerk effects, for Fivizzano and Amatrice, respectively.

(A.3), (A.4) Current state, with Jerk effects considered, for Fivizzano and Amatrice, respectively.

(A.5) Strengthened state, with consolidation intervention, focused on the Amatrice case.

For all models, the disaggregation check is performed alongside kinematic and pushover analyses, following the established hierarchy of collapse mechanisms. The pushover analysis includes the effects of vertical seismic acceleration, using the methodology developed by the Authors [16].

As previously noted, among the construction practice parameters considered in the MQI, those affected by disaggregation are:

MM Quality of the mortar/interaction between masonry units;
WC Level of connection between adjacent wall leaves;
HJ Horizontality of mortar bed joints.

First, the mechanical parameters are evaluated using the MQI, without taking into account the effects of Jerk.

Figure 17. MQI for the roughly cut stone masonry at the ground floor.

[7] The seismic risk index ζ_E is defined as the ratio between capacity and demand in terms of PGA. A value of $\zeta_E \geq 1.00$ indicates full compliance. However, Italian standards allow a lower target for existing buildings in their current state, accepting $\zeta_E \geq 0.80$ as an adequate performance level [3]. In the case study building, this target level of $\zeta_E \geq 0.80$ is specifically required for disaggregation.

Figure 17 shows an MQI$_o$ value of 4.50 > 4: walls made of this type of roughly cut stone masonry are not initially prone to disaggregation. However, they may become vulnerable if the masonry quality deteriorates due to Jerk effects, as will be discussed later.

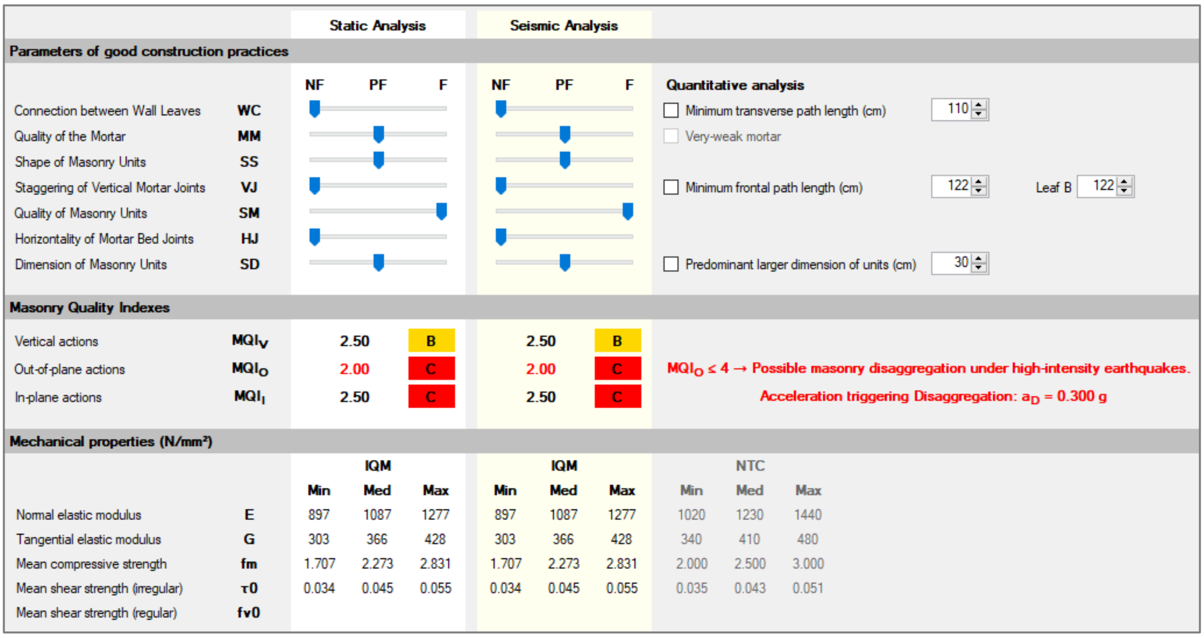


Figure 18. MQI for the lower-quality roughly cut stone masonry on the upper floors.

Figure 18 shows an MQI$_o$ value of 2.00, which is below the threshold of 4. This indicates that walls made of lower-quality roughly cut stone masonry must be checked for disaggregation under high-intensity seismic acceleration, even without considering Jerk effects. Specifically, the threshold acceleration for triggering disaggregation (a_D) is calculated using Equation (2.1) and is equal to 0.300 g for this masonry type.

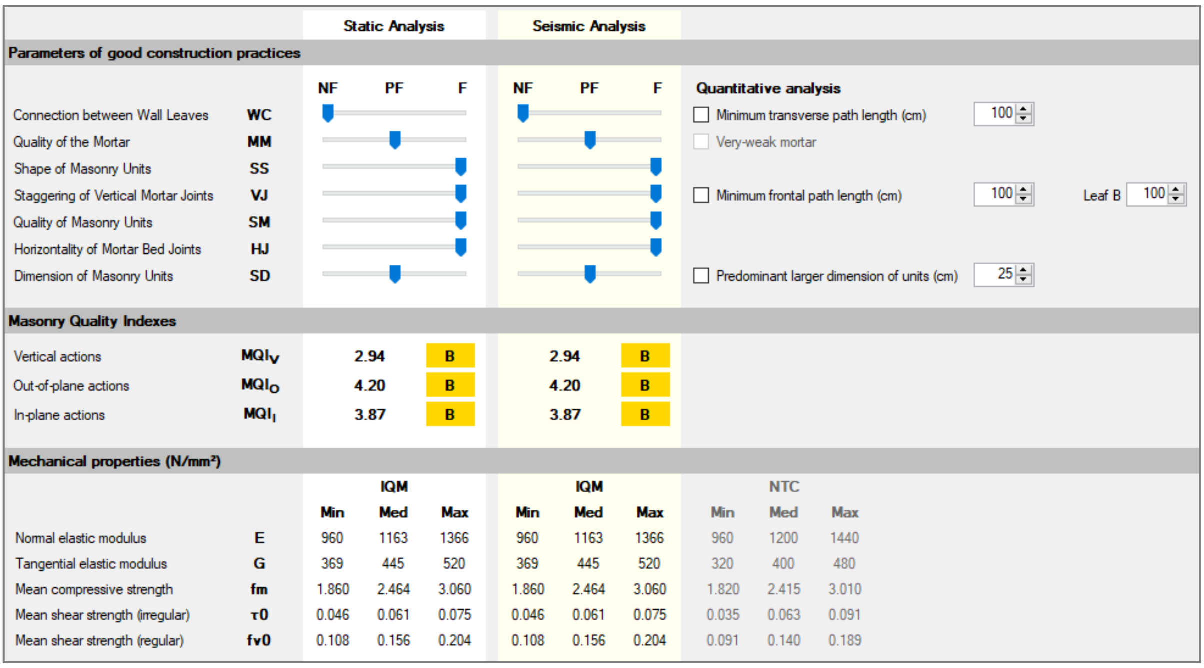


Figure 19. MQI for the solid brick masonry with wide mortar joints (> 13 mm) used in the extension of the building.

Figure 19 shows an MQI_o value of 4.20, which is above the threshold of 4. This indicates that walls made of this type of solid brick masonry are not initially prone to disaggregation. However, they could become vulnerable if the masonry quality deteriorates due to Jerk effects, as will be discussed later.

Model A.1 (site: Fivizzano, Jerk effects not considered)
The building has a topographic coefficient S_T = 1.00. For the horizontal seismic component: $PGA_H = a_g S_S S_T = 0.200 \cdot 1.411 \cdot 1.00 = 0.282$ g.
Based on the MQI_o values for the different masonry types, the disaggregation check must be performed on the lower-quality roughly cut stone masonry walls located on the first and second floors. The acceleration at the base of these walls is evaluated using the floor response spectrum. For the walls on the top floor, with a base elevation of z = 7.100 m, the spectral acceleration is given by:

$$a_z(7.1) = S_e(T_1, \xi) \cdot \gamma_1 \cdot \psi_1(z) \cdot \sqrt{1 + 0.0004\xi^2} \tag{3.1}$$

where:

T_1 is the fundamental vibration period of the building, which is estimated here using the formula $T_1 = C_1 H^{3/4} = 0.05 \cdot (10.2)^{3/4} = 0.285\ s$ (being H = 10.2 m the total height of the building). Since $T_B = 0.149\ s$ and $T_C = 0.447\ s$: $T_B \leq T_1 < T_C$;

$S_e(T_1, \xi)$ is the elastic response spectrum evaluated for the period T_1 and viscous damping $\xi = 5\%$: $S_e(T_1, \xi) = a_g \cdot S \cdot \eta \cdot F_O = 0.200 \cdot 1.411 \cdot 1.0 \cdot 2.412 = 0.680\ g$;

γ_1 is the modal participation coefficient of the fundamental vibration mode, assumed to be $\gamma_1 = \frac{3N}{2N+1} = 1.286$ (where N = 3 is the number of floors in the building);

$\psi_1(z)$ is the value of the fundamental modal shape at height z given by: $\frac{z}{H} = \frac{7.1}{10.2} = 0.696$.

The masonry piers on the top floor are subjected to a spectral acceleration at the base equal to:

$$a_z(7.1) = 0.680 \cdot 1.286 \cdot 0.696 \cdot \sqrt{1 + 0.0004 \cdot 5^2} = 0.612g \tag{3.2}$$

The demand in terms of $PGA = a_g \cdot S = 0.282\ g$ leads to a spectral acceleration of 0.612 g. The disaggregation activation threshold, considered as the "capacity", for the lower-quality roughly cut stone masonry is set at a_D = 0.300 g.

The results of the disaggregation verification are shown in Figure 20. The masonry piers in dark green are not prone to disaggregation, as their MQI_o is greater than 4. The other piers undergo verification in terms of spectral acceleration: those in light green satisfy the verification, while those in red do not. The image also provides detailed results for two representative piers: pier #2 on the first floor, where the verification is satisfied, and pier #3 on the top floor, where the verification has failed.

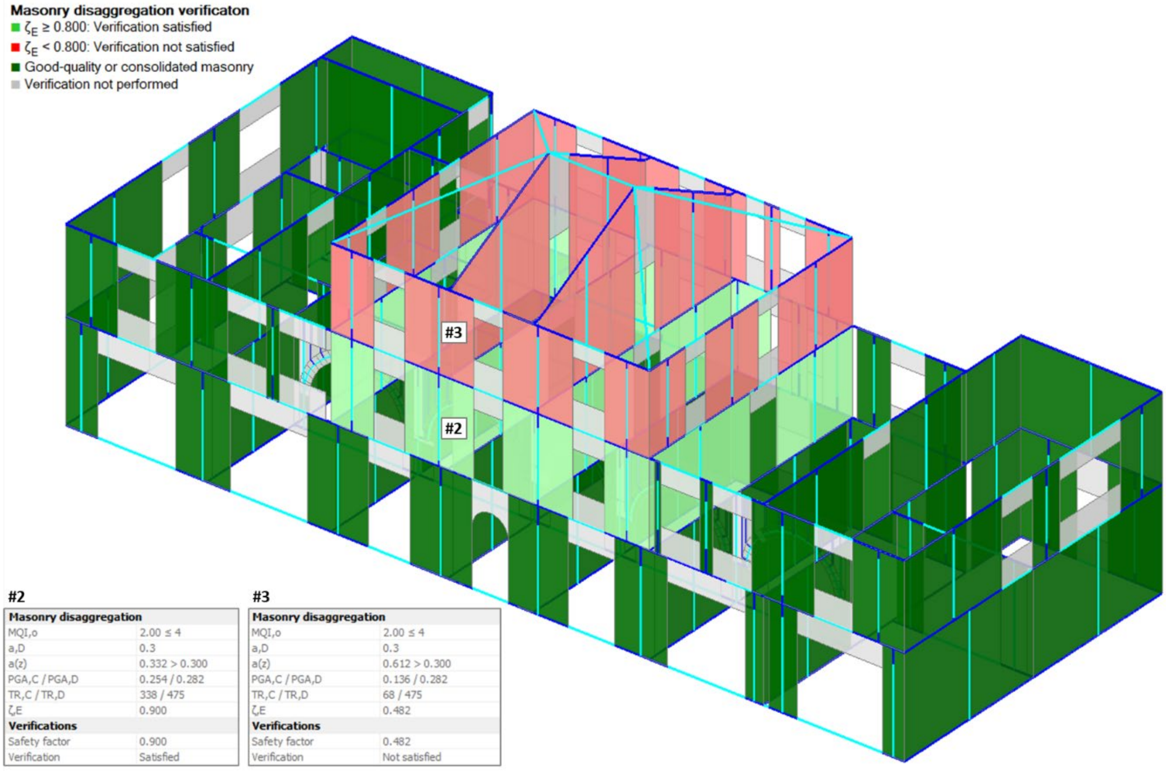


Figure 20. Model (A.1): Disaggregation verification

The summary of the results obtained from all the analyses (omitting, for simplicity, the details of kinematic and pushover analyses) is presented in Table 4. The complete analysis evaluates all potential failure mechanisms by applying the hierarchy of mechanisms, which - as previously noted - is essential for assessing the seismic performance of existing masonry buildings [4,5,6,7].

Masonry disaggregation reduces the seismic risk index from 0.585, obtained through collapse mechanism analysis, to 0.482. This means that neglecting the disaggregation mechanism would lead to an overestimation of the building's capacity in its current state. Disaggregation lowers the seismic risk index by 17.6% and occurs before both local mechanisms (kinematic) and global mechanisms (pushover), effectively preventing them from manifesting.

SLV: Masonry disaggregation	**0.482**
SLV: Local mechanisms	**0.585**
SLV: Pushover	**0.752**

Table 4. Model (A.1): summary of results with hierarchy of mechanisms

Model A.2 (site: Amatrice, Jerk effects not considered)
Compared to the previous model, the structural data remains the same, but the seismic parameters have been updated. For the horizontal seismic component:
$PGA_H = a_g\, S_S\, S_T = 0.225 \cdot 1.333 \cdot 1.00 = 0.345$ g.
Following the same procedure used for Model (A.1), the results shown in Figure 21 and Table 5 are obtained.

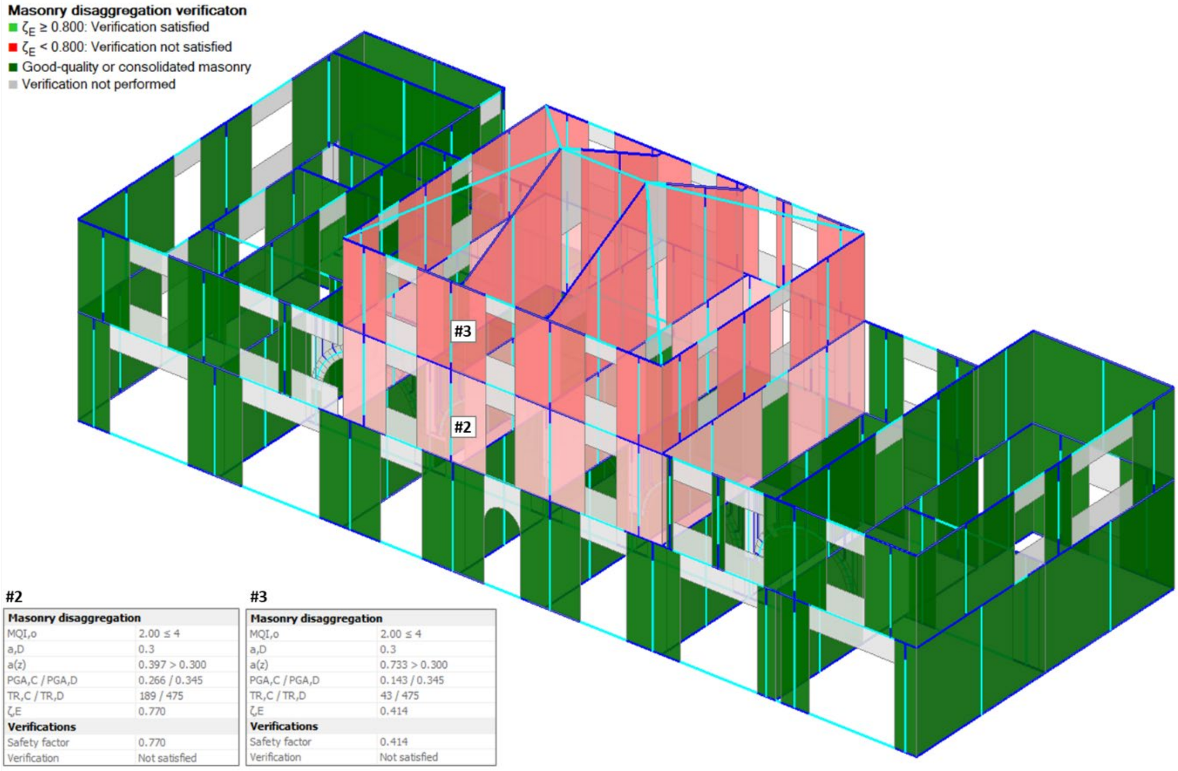


Figure 21. Model (A.2): Disaggregation verification

SLV: Masonry disaggregation	**0.414**
SLV: Local mechanisms	**0.478**
SLV: Pushover	**0.626**

Table 5. Model (A.2): summary of results with hierarchy of mechanisms

The comparison between Models (A.2) and (A.1) highlights the effects of increased seismic hazard: the same building, relocated from Fivizzano to Amatrice, shows disaggregation affecting both upper floors. As with Model (A.1), neglecting the disaggregation mechanism would lead to an overestimation of the building's capacity in its current condition. Disaggregation reduces ζ_E from 0.478 to 0.414 (−13.4%) and occurs before both local (kinematic) and global (pushover) failure mechanisms, effectively making them irrelevant.

Model A.3 (site: Fivizzano, Jerk effects are considered)
For the vertical seismic component, S_S is assumed to be equal to 1.0. Therefore:
$PGA_V = a_{gV} S_S S_T = 0.200$ g, since, according to Italian standards [2], $a_{gV} = a_g$.
Applying Equation (2.2), the value of peak ground Jerk is obtained as:
$PGJ_V = 77.526 \cdot PGA_V - 0.795 = 14.71$ g/s $\Rightarrow 10 < PGJ_V < 15$ g/s.
To obtain the design structural Jerk value, potential resonance must be taken into account. The building's fundamental vertical period, calculated in Model (A.1), is $T_{1Z} = 0.098$ **s**, and according to Equation (2.5), the amplification coefficient is 2.5, from which:

$$J_{VS} = PGJ_V \cdot C_{ampl} = 14.71 \cdot 2.5 = 36.78\ g/s \Rightarrow 30 \leq J_{VS} < 40\ g/s \tag{3.3}$$

According to Table 2, the roughly cut stone masonry shows a two-class reduction for the MM parameter, and a one-class reduction for both WC and HJ. The calculation of the seismic

MQI_o is shown in Figures 22 and 23, respectively for the masonry at the ground floor and for the lower-quality masonry at the upper levels.

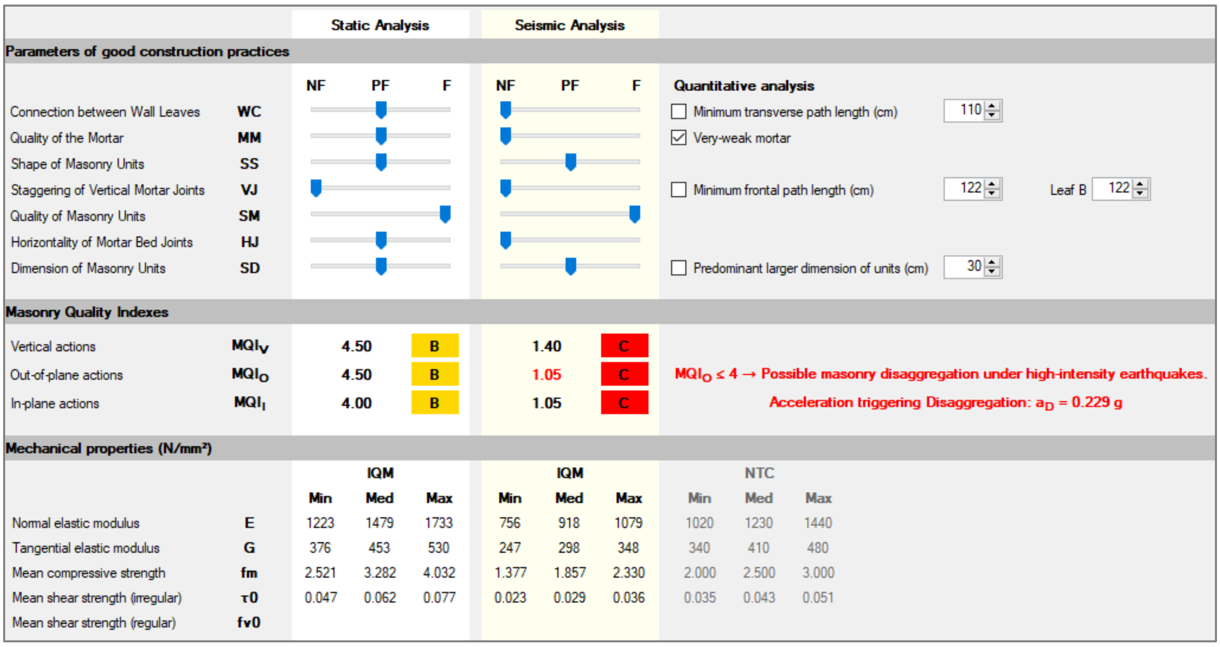


Figure 22. MQI for the roughly cut stone masonry at the ground floor, considering Jerk effects (Fivizzano)

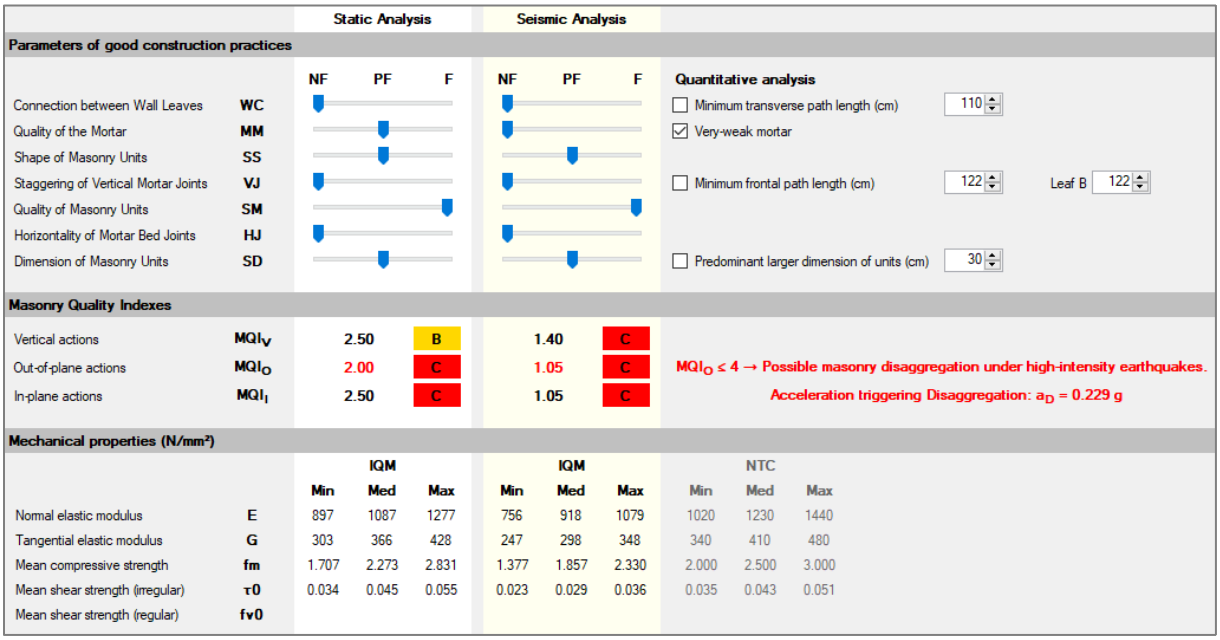


Figure 23. MQI for the lower-quality roughly cut stone masonry on the upper floors, considering Jerk effects (Fivizzano)

Both types of roughly cut stone masonry present in the building are downgraded to the lowest possible MQI_o values, since each parameter can only degrade to its minimum quality class. For example, in the higher-quality masonry (Figure 22), the WC parameter, originally rated as PF, drops one level to NF. In the lower-quality roughly cut stone masonry (Figure 23), WC was already at NF, therefore cannot degrade further, resulting in the same final classification. In both cases, WC ends up at NF, and the overall MQI_o for both masonry types is equalized to

1.05, which, according to Equation (2.1), corresponds to a disaggregation threshold a_D = 0.229 g.

As for the solid brick masonry, Table 3 indicates that at Jvs = 36.78 g/s, there is a one-class reduction in the MM parameter, with no additional effects. Figure 24 shows the calculation of the MQI for solid bricks masonry, accounting for the degradation of the MM parameter. This results in a drop in MQI_o from 4.20 to 3.85, falling below the threshold of 4. As a consequence, due to the effect of Jerk, the solid bricks masonry becomes subject to potential disaggregation. In the corresponding verification, the disaggregation threshold acceleration to be considered is a_D = 0.439 g.

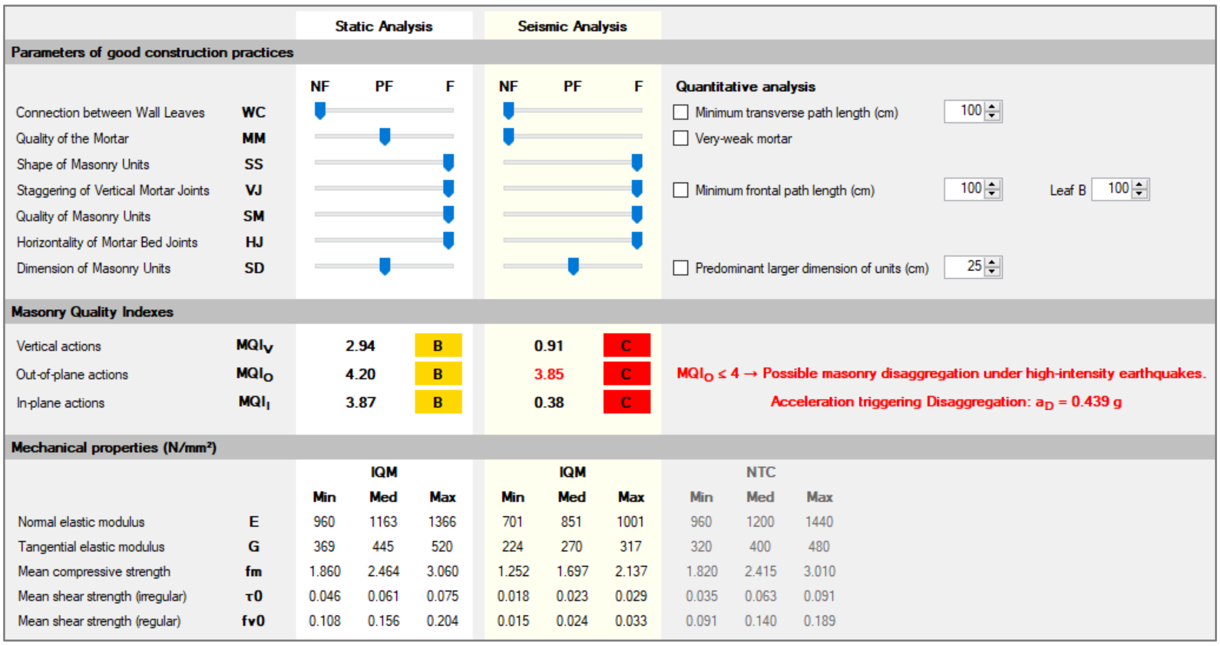


Figure 24. MQI for the solid brick masonry with wide mortar joints (> 13 mm) used in the extension of the building, considering the Jerk effects (Fivizzano)

Following a similar procedure to the one previously described, the result of the disaggregation verification is shown in Figure 25. Comparing the disaggregation verification results that account for Jerk effects - Model (A.3), shown in Figure 25 - with those of Model (A.1) in Figure 20, where Jerk is not considered, the following differences emerge:

- All walls, including those made of solid bricks, now exhibit reduced masonry quality, making them susceptible to disaggregation. No wall can be excluded a priori from this failure mechanism.
- Unlike the case without Jerk, the roughly cut stone walls at the intermediate level (first floor) now fail the disaggregation verification. As a result, the disaggregation mechanism extends from the top floor down to the level below.

The figure also shows detailed verification results for four representative piers: pier #1 on the ground floor, where verification is satisfied, pier #2 on the first floor, where verification is not satisfied, pier #3 on the top floor, where verification is not satisfied, pier #4 on the first floor (in the extension area), where verification is satisfied.

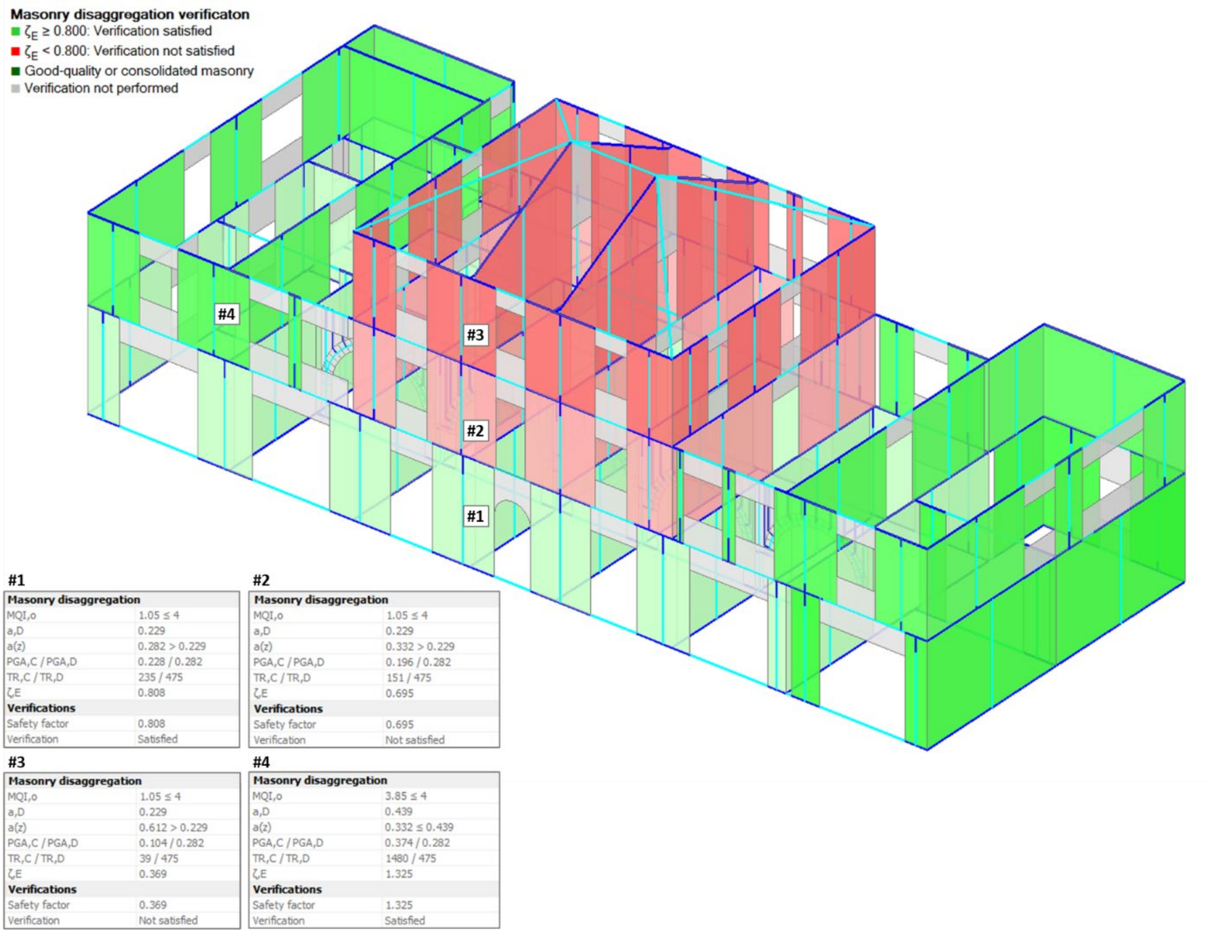


Figure 25. Model (A.3): Disaggregation verification considering Jerk effects (Fivizzano)

In summary, the effects of Jerk on the disaggregation verification are illustrated in Figure 26. The image highlights the increasing relevance of disaggregation mechanisms when moving from Model (A.1) to Model (A.3).

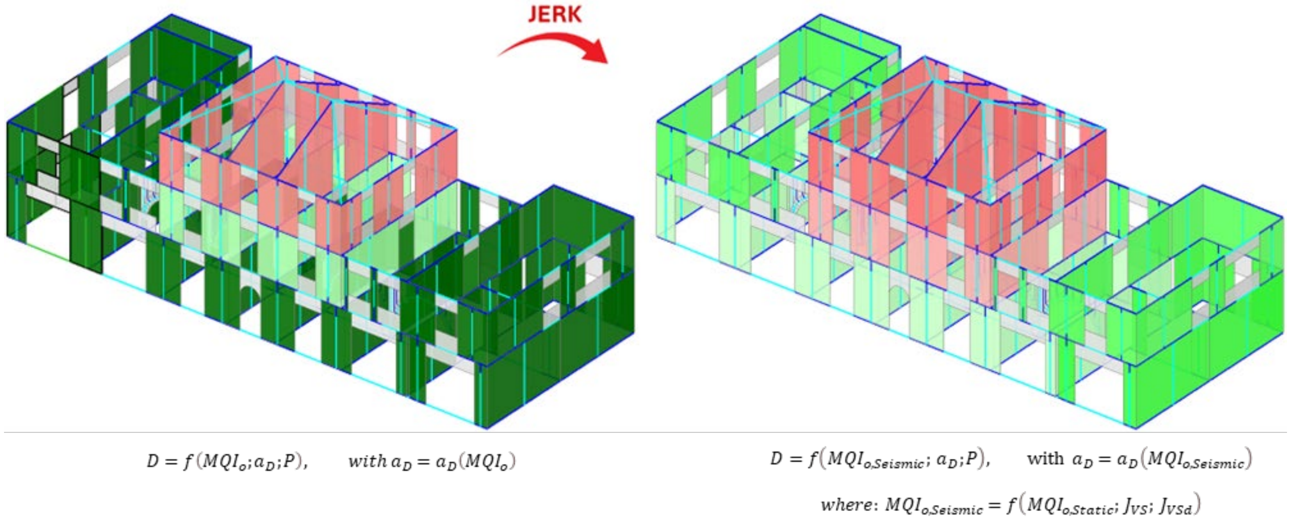


Figure 26. Effects of Jerk on disaggregation verification of the case study building located in Fivizzano

Table 6 shows that when the effects of Jerk are taken into account, masonry disaggregation reduces the seismic risk index from 0.585 (as obtained from collapse mechanism analysis) to 0.369 (compared to 0.482 when Jerk is not considered). This means that disaggregation, when influenced by Jerk, lowers the seismic risk index by 36.9%, instead of the 17.6% reduction observed without considering Jerk effects.

SLV: Masonry disaggregation	**0.369**
SLV: Local mechanisms	**0.585**
SLV: Pushover	**0.617**

Table 6. Model (A.3): summary of results with hierarchy of mechanisms, considering Jerk effects on disaggregation (Fivizzano)

Model A.4 (site: Amatrice, Jerk effects are considered)
For the vertical seismic component, S_S is assumed to be equal to 1.0. Therefore:
$PGA_V = a_{gV}\ S_S\ S_T = 0.259$ g, since, according to Italian standards [2], $a_{gV} = a_g$.
Applying Equation (2.2), the value of peak ground Jerk is obtained as:
$PGJ_V = 77.526 \cdot PGA_V - 0.795 = 19.28$ g/s $\Rightarrow 15 < PGJ_V < 20$ g/s.
To obtain the design structural Jerk value, potential resonance must be taken into account. The building's fundamental vertical period, calculated in Model (A.2), same as in Model (A.1), is $T_{1Z} = 0.098$ **s**, and according to Equation (2.5), the amplification coefficient is 2.5, from which:

$$J_{VS} = PGJ_V \cdot C_{ampl} = 19.28 \cdot 2.5 = 48.20\ g/s \Rightarrow 40 \leq J_{VS} < 50\ g/s \qquad (3.3)$$

According to Table 2, the roughly cut stone masonry shows a three-class reduction for the MM parameter, and a two-class reduction for both WC and HJ. The calculation of the seismic MQI_o is shown in Figures 27 and 28, respectively for the masonry at the ground floor and for the lower-quality masonry at the upper levels.

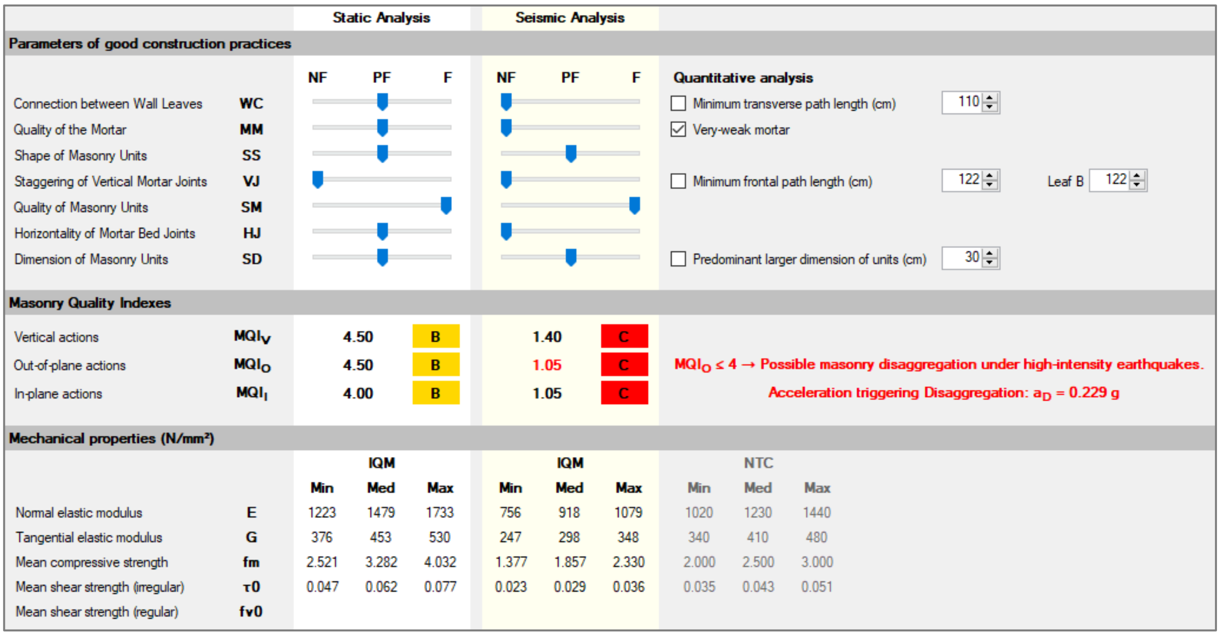


Figure 27. MQI for the roughly cut stone masonry at the ground floor, considering Jerk effects (Amatrice)

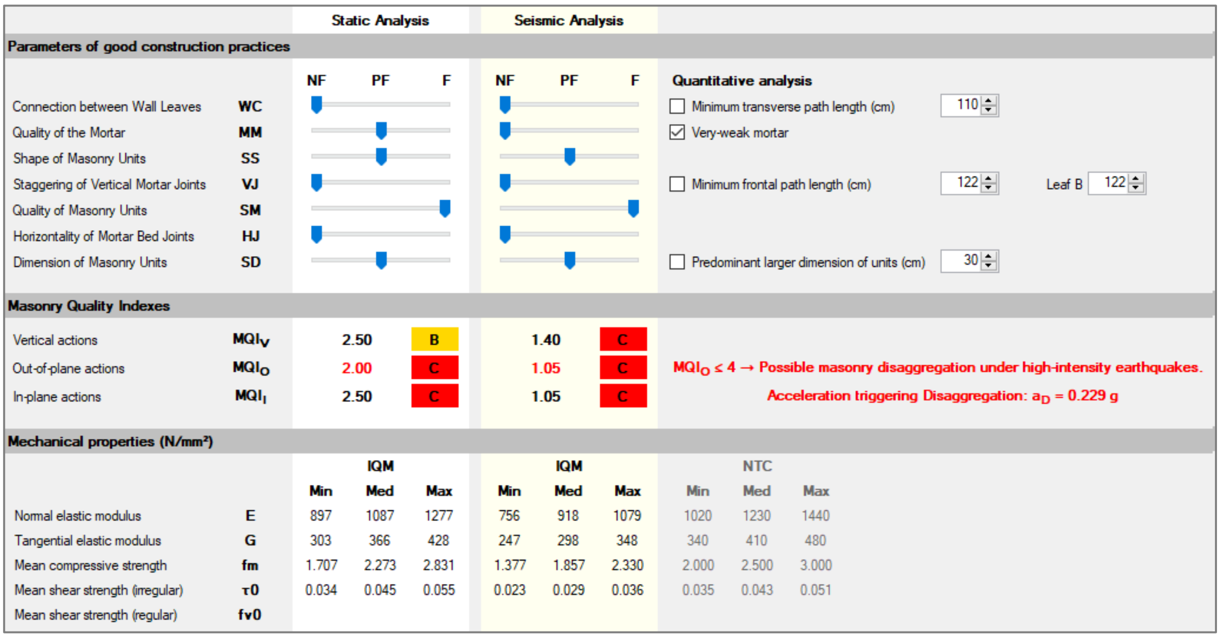


Figure 28. MQI for the lower-quality roughly cut stone masonry on the upper floors, considering Jerk effects (Amatrice)

As previously noted, the downgrading of quality is limited by the lowest allowable values of MQI_o, which results in the same values observed for the Fivizzano site (see Figure 27 vs. Figure 22, and Figure 28 vs. Figure 23).

For the solid brick masonry, Table 3 indicates that with a structural vertical Jerk value of $J_{VS} = 48.20$ g/s, there is a two-class reduction for the MM parameter and a one-class reduction for both WC and HJ. Figure 29 shows the recalculated MQI reflecting this degradation across all three parameters. The result is a drop in MQI_o from 4.20 to 2.20, falling below the disaggregation threshold of 4. Compared to the Fivizzano case (Figure 24), this reduction is noticeably more severe (down to 2.20 instead of 3.85), reflecting a greater deterioration in mechanical quality. As a consequence, the solid brick masonry becomes vulnerable to disaggregation due to Jerk effects, and the corresponding verification must now consider a lower activation threshold acceleration of a_D = 0.315 g, significantly reduced from the 0.439 g value found for Fivizzano.

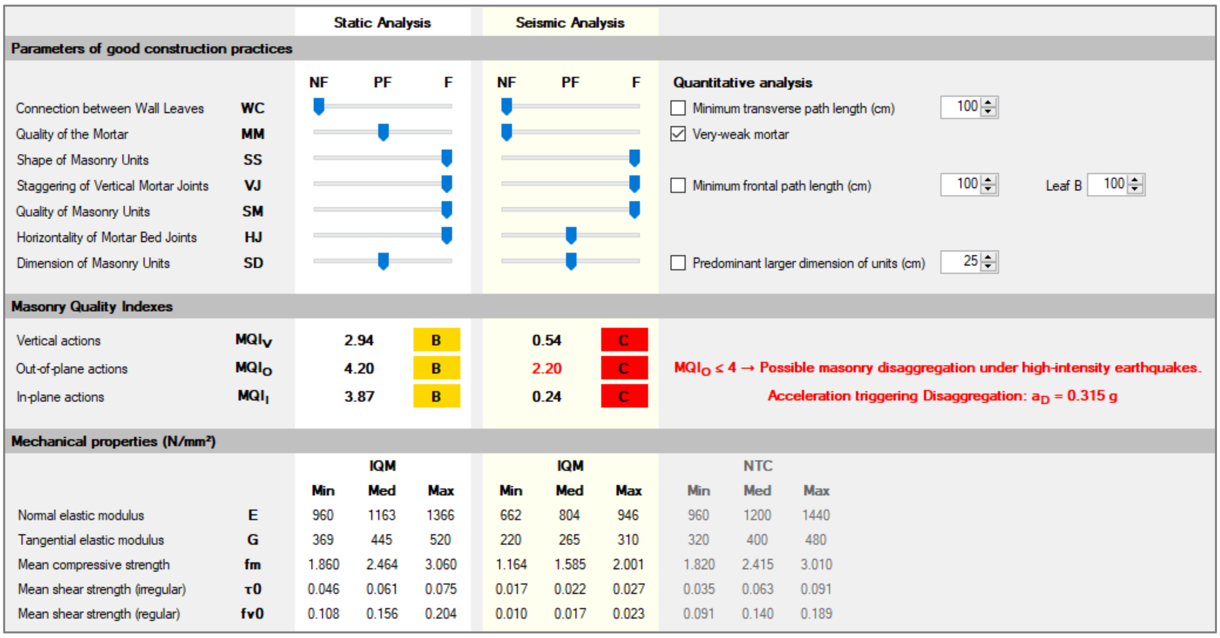


Figure 29. MQI for the solid brick masonry with wide mortar joints (> 13 mm) used in the extension of the building, considering the Jerk effects (Amatrice)

Following the same approach as outlined earlier, the disaggregation verification results are shown in Figure 30. All the walls made of roughly cut stone masonry are found to be subject to disaggregation, while the solid brick masonry walls, although potentially vulnerable, theoretically pass the verification. Figure 30 provides detailed disaggregation verification results for five representative walls: Pier #1 on the ground floor, pier #2 on the first floor and pier #3 on the top floor all fail the verification. Pier #4 on the first floor (in the extension area) theoretically passes the verification but is nevertheless compromised due to the collapse of the supporting walls below (see Figure 31). Pier #5 on the ground floor also fails the verification.

However, this outcome remains theoretical. In a real seismic event, the disaggregation of a lower wall typically results in the collapse of the masonry built above it. Therefore, the more realistic scenario is depicted in Figure 31, which should be considered representative of Model (A.4).

Comparing the disaggregation verification results that include the effects of Jerk (Model A.4, shown in Figure 31b) with those from Model A.2 (Figure 21), where Jerk effects are not considered, the following differences can be observed:

- All walls, including those made of solid brick, now exhibit reduced masonry quality under seismic action, making them susceptible to disaggregation. No wall can be excluded a priori from this potential failure mechanism.
- Unlike the case without Jerk, all roughly cut stone walls on the ground floor now fail the disaggregation verification. As a result, disaggregation extends throughout the entire height of the building and affects even the extension wing, where the brick masonry walls on the first floor, though not theoretically disaggregated on their own, collapse due to the failure of the underlying roughly cut stone masonry.

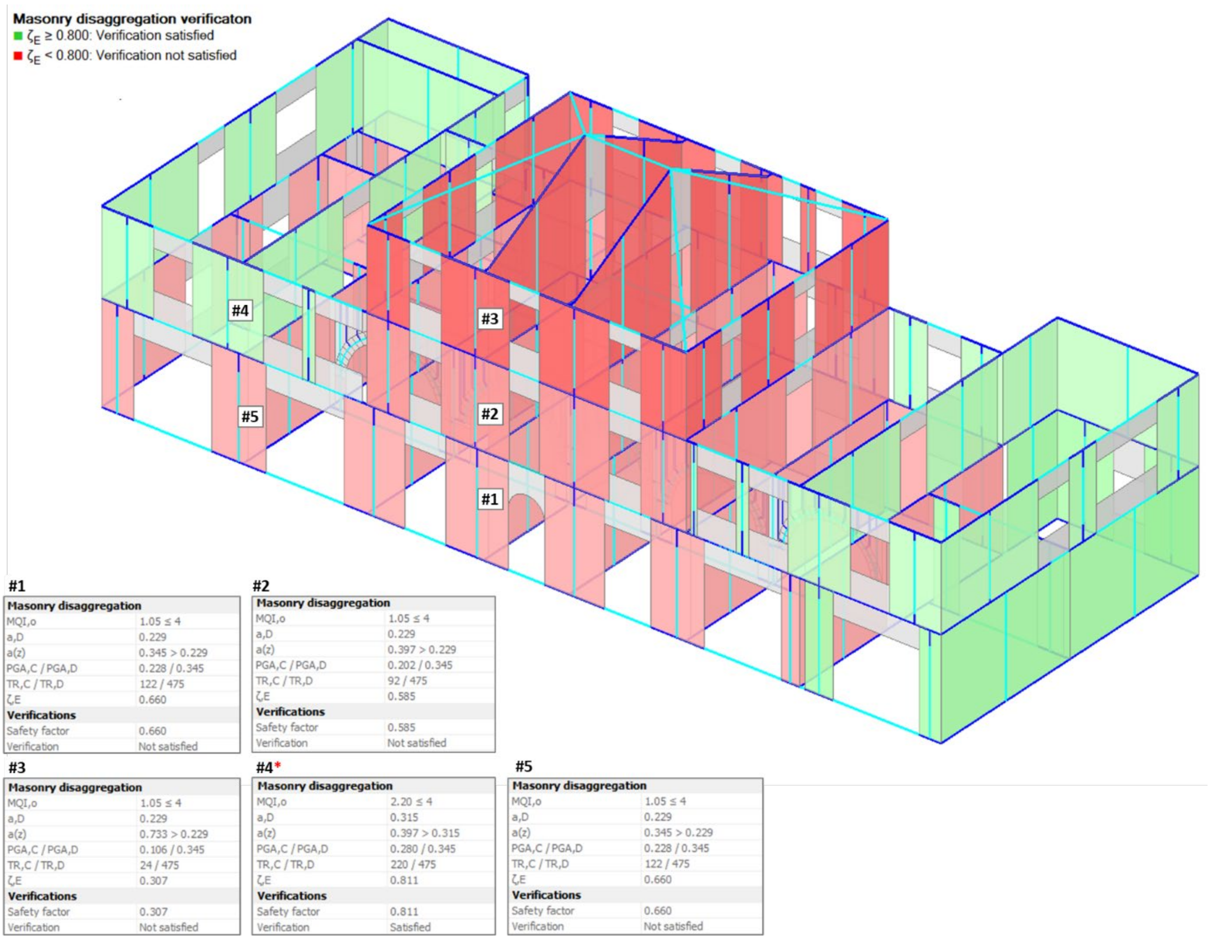


Figure 30. Model (A.4): Theoretical disaggregation verification considering Jerk effects (Amatrice)

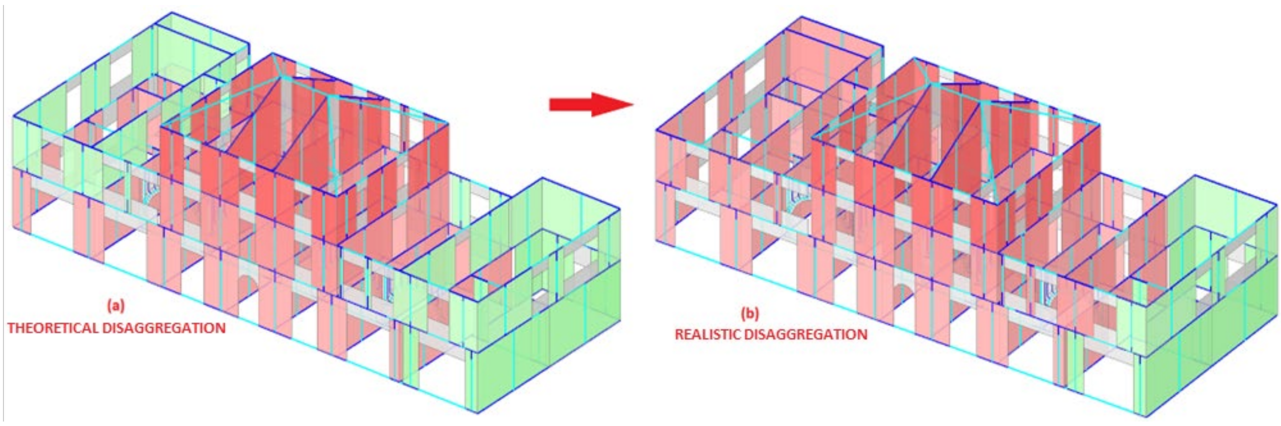


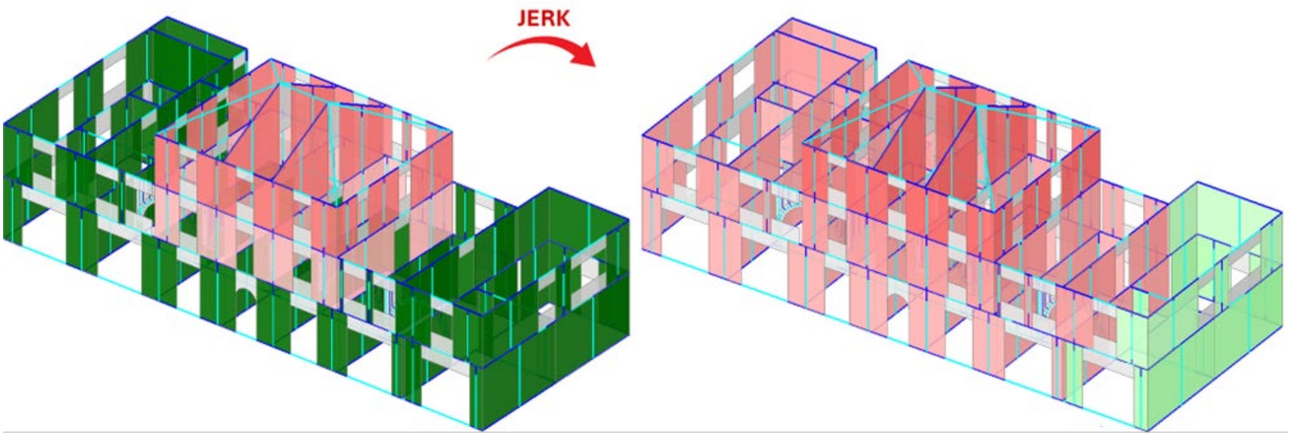
Figure 31. Model (A.4): Realistic disaggregation verification considering Jerk effects (Amatrice)

Table 7 summarizes the results of all analyses.

SLV: Masonry disaggregation	**0.307**
SLV: Local mechanisms	**0.478**
SLV: Pushover	**0.496**

Table 7. Model (A.4): summary of results with hierarchy of mechanisms, considering Jerk effects on disaggregation (Amatrice)

In summary, the effects of Jerk on disaggregation verification are illustrated in Figure 32, which highlights the increasing relevance of disaggregation mechanisms when moving from Model (A.2) to Model (A.4).



$D = f(MQI_o; a_D; P), \quad with\ a_D = a_D(MQI_o)$

$D = f(MQI_{o,Seismic}; a_D; P), \quad \text{with } a_D = a_D(MQI_{o,Seismic})$

$where: MQI_{o,Seismic} = f(MQI_{o,Static}; J_{VS}; J_{VSd})$

Figure 32. Effects of Jerk on disaggregation verification of the case study building located in Amatrice

Table 7 shows that when the effects of Jerk are considered, masonry disaggregation reduces the seismic risk index from 0.478 (obtained through kinematic analysis) to 0.307, as opposed to 0.414 when Jerk is not taken into account. The disaggregation mechanism, when influenced by Jerk, leads to a 35.8% reduction in the seismic risk index, compared to a 13.4% reduction without considering Jerk.

By analyzing the case study building, whether located in Fivizzano or Amatrice, it was observed that the most critical failure mechanism is disaggregation. Due to the effects of Jerk, this mechanism affects large portions of the structure, ultimately leading to a near-global failure in the case of the Amatrice site.

For the other structural behaviors (local mechanisms and pushover), the corresponding results lose their physical significance once disaggregation occurs. However, they remain important as they highlight how, by neglecting disaggregation, the seismic capacity of the building can be significantly overestimated.

Once the assessment of the current state has been completed, a retrofit strategy must be defined to meet the following requirements:

- an increase of the seismic risk index by $\Delta\zeta_E \geq 0.10$;
- a safety level corresponding to full compliance with respect to disaggregation.

The retrofit configuration must therefore reduce vulnerability by mitigating the disaggregation mechanism and enabling the structure to develop its capacity through both local (kinematic) and global (pushover) behaviors.

The proposed retrofit is developed with reference to the building located in Amatrice, represented by Model (A.2). The intervention aims to eliminate the risk of disaggregation by adequately strengthening the walls. With that in mind, the reference seismic risk indexes correspond to the results presented in Table 5, where, apart from disaggregation, ζ_E is equal to 0.478 for local mechanisms (kinematics) and 0.626 for the global behavior (assessed through pushover analysis).

Regarding disaggregation, the following approaches can be adopted:

a) Enhancing the quality of the masonry, for instance by restoring the internal mortar and introducing through-stones;
b) Implementing strengthening measures that prevent potential disaggregation movements, such as reinforced repointing of mortar joints or the application of reinforced mortar coatings.

The masonry quality defined by the MQI refers to the current condition of the walls, taking into account factors such as the performance of the mortar, the presence of through-stones, transverse interlocking, and the presence of regular courses (see Figure 2). When the masonry is strengthened through injection, reinforced mortar coating, or reinforced repointing, corresponding MQI values are not defined. In the context of retrofit design, the MQI serves to provide reference mechanical parameters for the existing masonry, which are then enhanced based on the type of intervention using the corrective coefficients provided in the Italian technical standards [3].

In general, disaggregation mechanisms can be effectively prevented and eliminated in advance through a well-designed strengthening intervention applied to all walls that, in their current state, are potentially prone to disaggregation. These walls should be identified based on their original masonry quality and the effects of seismic Jerk, following the methodology outlined in this study. In the retrofit configuration, the out-of-plane quality index (MQI_o) is no longer defined, and corresponding checks are not required—the disaggregation verification is implicitly considered fulfilled. The strengthening techniques mentioned in points (a) and (b) are effective in all cases where MQI_o is less than or equal to 4, whether due to poor masonry quality under static conditions or to initially moderate quality that deteriorates due to Jerk during a seismic event. In either case, the impact of Jerk is rendered negligible[8].

Model (A.5) is derived from the as-is condition of Model (A.2), incorporating reinforced repointing as the chosen strengthening technique. Based on the vulnerability assessment results (see Figure 32), the intervention is applied to the entire load-bearing structure. For the three masonry types present in the building, the improvement in mechanical parameters achieved through the corrective factors recommended by the Italian Standards [3] is shown in Figure 33.

[8] This does not mean that the repeated impact during a seismic event ceases to negatively affect the material. However, in the case of regenerated mortar through injections or reinforced mortar coating applied to only one face, additional high-intensity events would be required to cause significant deterioration of the masonry and, through a process of "damage memory" [1], potentially return it to a state where disaggregation becomes possible again. A more durable strengthening solution is achieved with reinforced plaster applied to both faces of the wall, combined with transverse connections. In this configuration, even if progressive degradation occurs in the confined masonry core due to high-frequency vibrations, the transverse ties ensure that the two reinforced mortar coating layers act together as a single unit. This reduces the influence of internal cracking, effectively confining the wall and preventing disaggregation.

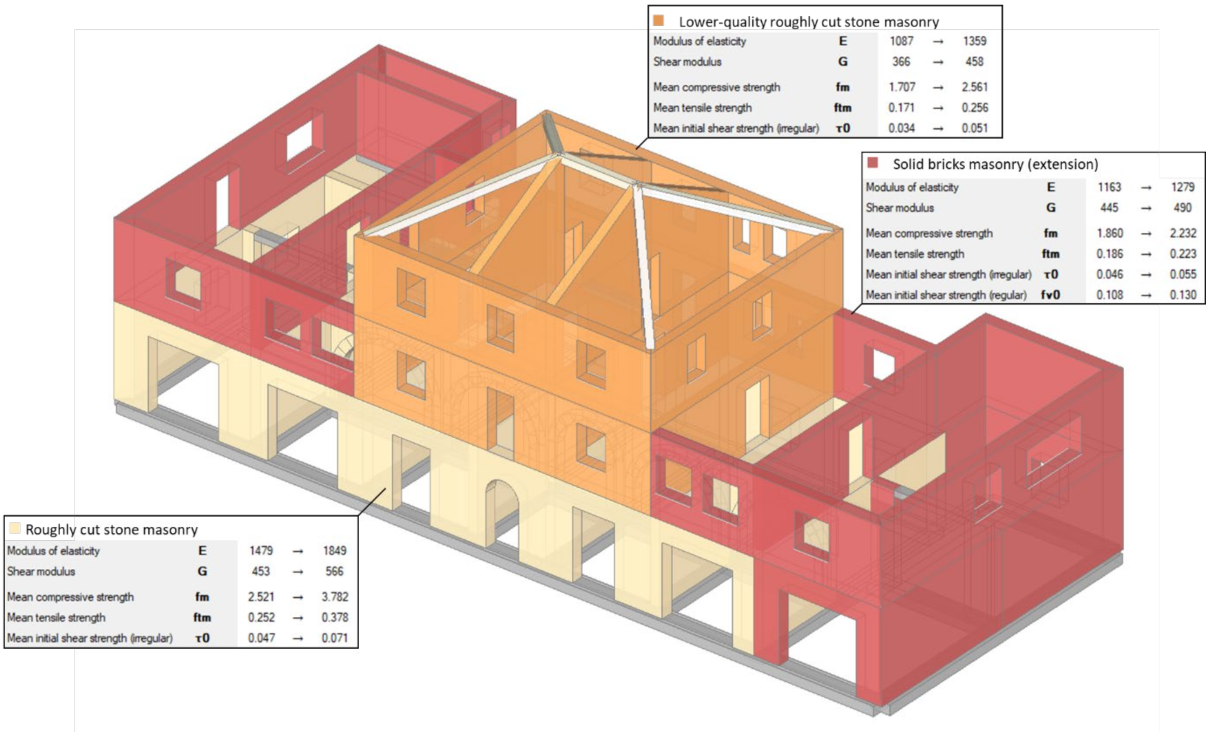


Figure 33. Model (A.5): Strengthening intervention with reinforced repointing applied to the entire building

Figure 34 shows the disaggregation verification implicitly satisfied through the strengthening intervention with reinforced repointing.

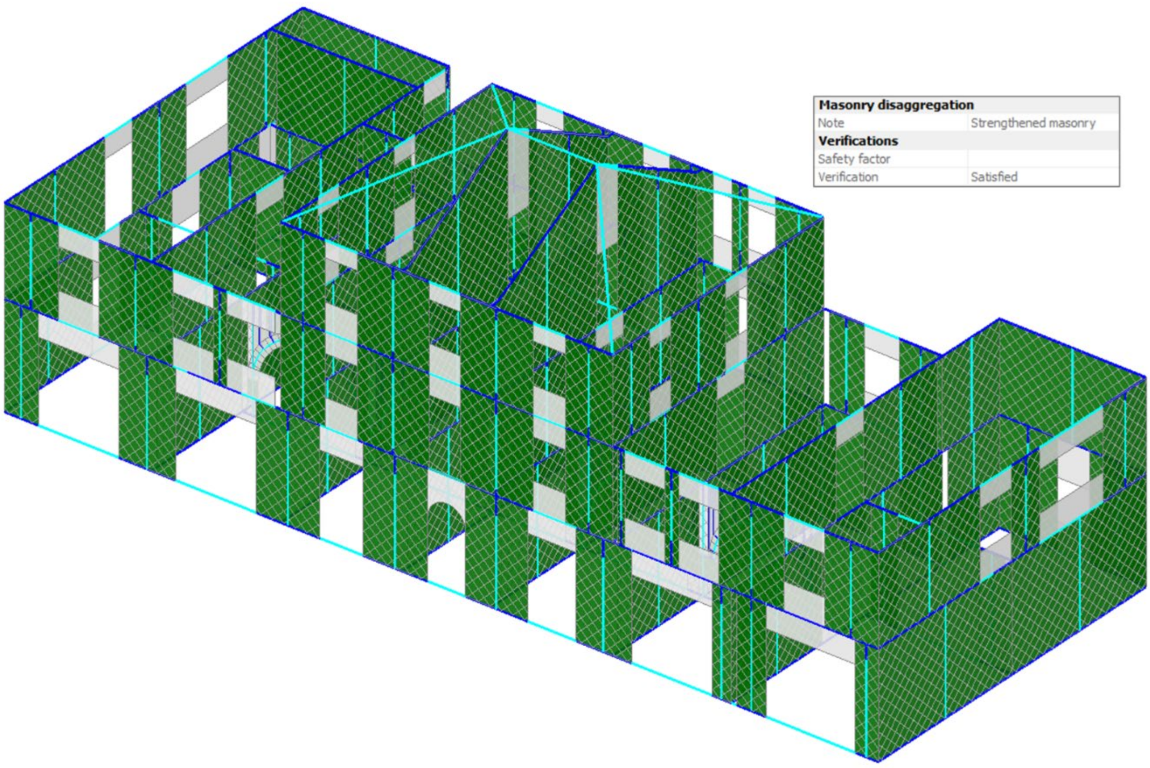


Figure 34. Model (A.5): Disaggregation verification implicitly satisfied

The complete analysis yields the results summarized in Table 8.

SLV: Local mechanisms	**0.478**
SLV: Pushover	**0.733**
SLV: Masonry disaggregation	**>>1**

Table 8. Model (A.5): Summary of analysis results for the project state (Amatrice)

Safety verification is no longer governed by disaggregation; both local (collapse mechanisms) and global structural behaviors now acquire physical meaning and produce reliable (not merely theoretical) results. The improvement in masonry quality not only prevents disaggregation but also leads to increased values of mechanical parameters (strength and elastic moduli), which in turn contribute to the rise in the pushover analysis result, from 0.626 (Table 5) to 0.733 (Table 8). It is worth noting that strengthening against local mechanisms was not developed in this example. However, the implementation of well-known retrofit measures (such as tie rods, wall-to-wall and wall-to-floor connections, and elimination of thrust forces at roof level) can increase the corresponding safety index beyond that of the pushover analysis. Once these interventions are defined, the pushover result becomes the definitive seismic risk index for the retrofitted design.

Therefore, the retrofitted design meets the required criteria:

- disaggregation has been fully addressed, achieving compliance with the adequacy threshold;
- The increase in the seismic risk index exceeds 0.10: $\Delta\zeta_E = 0.733 - 0.414 > 0.10$, indicating a shift from disaggregation mechanisms in the current state ($\zeta_E = 0.414$) to a global resistant mechanism in the retrofitted state ($\zeta_E = 0.733$).

If the required increase $\Delta\zeta_E > 0.10$ is to be adopted as a reference for each individual structural behavior, the proposed intervention still meets the requirement. In fact, even considering only the pushover analysis: ζ_E,retrofit,pushover = 0.733 > ζ_E,current,pushover = 0.626.

For local mechanisms, the strengthening measures are designed to ensure that the seismic risk index exceeds the pushover result. This guarantees, compared to the current-state value of 0.478, an increase of $\Delta\zeta_E > 0.10$: ζ_E,retrofit,local ≥ 0.733 > ζ_E,current,local = 0.478.

5 CONCLUSIONS

Structural damage to masonry buildings caused by increasing seismic actions can be identified through the hierarchy of collapse mechanisms, organized as follows:

- disaggregation, typically associated with low-quality masonry, such as that found in historic or spontaneus constructions;
- local collapse mechanisms, involving the overturning of rigid bodies;
- global collapse mechanisms, related to the overall strength of the walls.

This study has clarified and developed the intrinsic link between masonry quality, disaggregation, and the high-frequency content of seismic action (Jerk). The tendency toward disaggregation is not only an inherent property of the masonry typology but also influenced by vertical vibrations, which affect the mechanical characteristics of the mortar and cause displacement of the stone elements, destabilizing the walls and making them more susceptible to the expelling action triggered by high horizontal accelerations.

By identifying disaggregation as a function of masonry quality under static conditions and its degradation due to the effects of Jerk, it becomes possible to conduct more realistic assessments and to estimate more accurate seismic risk indexes in vulnerability analyses of existing structures.

In cases where disaggregation is expected, either due to poor mechanical properties of the materials or because of the percussive effects of Jerk under significant seismic accelerations, the strengthening intervention must be capable of counteracting and eliminating the disaggregation mechanism, thereby shifting the structural capacity of the building toward a global, unified response.

Mortar regeneration, the insertion of through-stones, reinforced repointing, and reinforced mortar coating are all effective approaches in cases where masonry quality is insufficient—either due to inherently poor construction in a non-seismic state or to originally modest quality degraded by Jerk during a seismic event. These types of interventions neutralize the effects of Jerk, improve masonry quality, and prevent the destructive consequences of disaggregation. When combined with targeted evaluations of structural connections, this preventive seismic strategy can be effectively implemented, even for low-quality masonry buildings subjected to high seismic demands.

ACKNOWLEDGEMENTS

The Authors would like to express their gratitude to Alessio Francioso (M.Eng.) for his essential contribution to the software implementation of the methodologies developed in this work. Special thanks also go to Alessandro De Maria (M.Eng.) and Prof. Giulio Castori for the insightful technical and scientific discussions on the relationship between Masonry Quality Index and disaggregation mechanisms.

REFERENCES

[1] M. Mariani, Terremoto e caos: un nuovo percorso di analisi del comportamento dei sismi, *Ingenio*, 2017. https://www.ingenio-web.it/articoli/terremoto-e-caos-un-nuovo-percorso-di-analisi-del-comportamento-dei-sismi/

[2] Ministero delle Infrastrutture e dei Trasporti, *D.M. 17 gennaio 2018: Aggiornamento delle Norme Tecniche per le Costruzioni*.

[3] Consiglio Superiore dei Lavori Pubblici, *Circolare 21 gennaio 2019, n. 7: Istruzioni per l'applicazione dell'Aggiornamento delle Norme Tecniche per le Costruzioni di cui al D.M. 17.1.2018*, Ministero delle Infrastrutture e dei Trasporti.

[4] F. Braga, C. Miozzi, G. Rossi, M. Sessa, *Edifici in muratura esistenti: interventi antisismici da adottare*, Agenzia del Demanio, Roma, 2024.

[5] A. Borri, M. Corradi, A. De Maria, The Failure of Masonry Walls by Disaggregation and the Masonry Quality Index, *Heritage*, 3(4), 1162–1198, 2020. https://doi.org/10.3390/heritage3040065

[6] A. Borri, A. De Maria, L'Indice di Qualità Muraria (IQM) e la disgregazione delle murature per effetto del sisma, *Structural*, n. 229, maggio–giugno 2020.

[7] S. Szabó, M.F. Funari, P.B. Lourenço, Masonry patterns' influence on the damage assessment of URM walls: Current and future trends, *Developments in the Built Environment*, 13, 2023, 100119. https://doi.org/10.1016/j.dibe.2023.100119

[8] Aedes Software, *Aedes.PCM: Progettazione di Costruzioni in Muratura*, Version 2025. https://www.aedes.it

[9] P. Meyer, J. Ochsendorf, J. Germaine, E. Kausel, The Impact of High-Frequency/Low-Energy Seismic Waves on Unreinforced Masonry, *Earthquake Spectra*, 23(1), 2007, 77–94.

[10] M. Tong, G.-Q. Wang, G.C. Lee, Time derivative of earthquake acceleration, *Earthquake Engineering and Engineering Vibration*, 4(1), 2005, 1–16.

[11] M. Mariani, F. Pugi, Effects of impulsive actions due to seismic Jerk and local failures in masonry structures, *Eurodyn* 2020, Athens, Greece, 23–26 November 2020

[12] E.M. Rathje, N.A. Abrahamson, J.D. Bray, Simplified frequency content estimates of earthquake ground motions, *Journal of Geotechnical and Geoenvironmental Engineering*, 124(2), 1998, 150–159.

[13] H. He, R. Li, K. Chen, Characteristics of Jerk Response Spectra for Elastic and Inelastic Systems, *Shock and Vibration*, Vol. 2015, Article ID 782748.

[14] Y. Xueshan, Q. Xiaozhai, G.C. Lee, M. Tong, C. Jinming, Jerk and Jerk Sensor, *14th World Conference on Earthquake Engineering*, October 12–17, 2008, Beijing, China.

[15] M. Mariani, F. Pugi, Impulsive actions due to seismic Jerk and design of reinforcements to counteract their effects in masonry buildings, *CompDyn*, 2023.

[16] M. Mariani, F. Pugi, A. Francioso, Vertical component of the seismic action: amplified vulnerability of existing masonry buildings, *CompDyn*, 2019.