The 11th Canadian Conference on Earthquake Engineering

Canadian Association for Earthquake Engineering

11CCEE

SEISMIC VULNERABILITY OF HISTORIC STONE MASONRY: A CRITICAL REVIEW

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ABSTRACT: Historic structures are of great importance to current and future generations as they convey historical and cultural aspects of past civilizations. Many historic structures were built of stone masonry in the form of residential and religious buildings, bridges, and monuments. Some of those structures have survived earthquakes for centuries while others collapsed, revealing our lack of knowledge concerning the seismic behaviour of such structures. Historic structures are typically massive and stiff and can be vulnerable to seismic events - even ones of low to moderate severity. The seismic vulnerability of such structures arises possibly due to the attraction of high inertial forces, the lack of ductility to dissipate seismic energy and the deterioration and weakening of the material over time. Historic structures were typically built based on the builders' experience: seismic codes did not exist at the time. To conserve those structures, seismic resistance should be evaluated carefully, and an appropriate rehabilitation scheme used if necessary. The seismic vulnerability of a stone structure is a function of the interaction of ground motion parameters and the structure itself. In this paper, we review and discuss some factors in current practice affecting the seismic vulnerability of stone masonry, considering seismic demand and capacity parameters.

1. Introduction

Historic structures are tangible records of the history and culture of past civilizations, and how they could have influenced current generations. Thus, many societies see conservation of historic structures as important. Masonry units such as stone, bricks, and adobe are the fundamental components of many historic buildings since they were easy to extract or form from local resources. The mortar used with those masonry units was typically based from pozzolan, clay, or lime, or more recently Portland Cement, or lime/cement. Structures built solely on such masonry composition – i.e. units and mortar – are defined as unreinforced masonry (URM) structures. Stone masonry is a typical example of URM, in which the URM walls are the indispensable elements that resist an earthquake, in both the in- and out-of-plane directions. Seismic activities represent the most critical hazard that put heritage structures at risk of damage or collapse, since those structures were built using the builders' expertise, rule-of-thumb, with little/or no knowledge of how to resist seismic action at the time. In addition, historic structures may have very thick and massive walls, which can have catastrophic consequences if they fail, especially, in crowded urban areas. Some historic stone buildings have survived earthquakes for centuries, while others did not survive even the first ground shaking. However, just because a heritage stone structure is still standing does not confirm it has adequate seismic resistance because earthquakes, particularly ones of high severity, occur at a lower probability than other natural hazards. In general, failure of stone structures to earthquakes was manifested during past activities; e.g., as shown in Fig. 1 (e.g. ERRI, 1992; ERRI, 2005). The seismic design of URM (generally other than stone masonry) structures is now regulated, to some extent, and analytical models have been developed to predict their behaviour (e.g. FEMA 356, 2000).

(a) Heavy damage to stone bearing wall buildings in Egypt (EERI, 1992).

(b) Heavy damage to stone house in Chile (EERI, 2005).

Fig. 1 – Failure of Stone Masonry to Earthquakes (Permission to Reproduce is Granted by the Earthquake Engineering Research Institute)

Stone masonry used in heritage structures is site-dependent and may be coursed or uncoursed. The former can be solely stone units or stone units combined with other URM masonry (e.g. bricks), and the latter can be randomly placed (rubble), or a combination thereof, as shown in Fig. 2. Multi-wythe walls often have more than one stone wythe and a rubble material, with or without mortar, between them. Such walls are common in historic structures and can be more vulnerable than single wythe walls.

Mixed stone and brick masonry

Coursed and uncoursed stone with different masonry in a wall.

Rubble stone

Fig. 2 – Configuration of Stone Masonry Walls

The seismic deficiency of stone structures can be ascribed to deteriorated mortar and/or stone units, e.g. due to environmental effects, defective bond between the core filling and the surrounding wythes, diaphragm flexibility, high slenderness of the walls, lack of or weak anchorage between walls or walls and floors, low tensile and shear strength, and changes in a structure’s function. In addition, irregular layouts in plan and/or elevation can reduce the seismic resistance of these structures. In addition, stone masonry structures generally lack regular sources of ductility, e.g. reinforcing steel, which provide the structure an ability to resist inelastic seismic excursions. A typical existing historic stone structure may have some, or all, of these shortcomings. Thus, a holistic evaluation scheme needs to be followed for every structure.

Although the assessment of an existing historic structure is a crucial step in determining its vulnerability and strengthening needs, ground motion parameters are no less important. Vulnerability is a mutual interaction between the structure itself and the imposed seismic demands. In fact, there is an ever-present dilemma when evaluating an existing structure. Part of this dilemma is insufficient information on the material properties, construction, stone masonry strength, and renovation history. Moreover, stone masonry does not have unique shapes, topologies, or dimensions, and so the variations in constituent materials lead to significant uncertainties in expected mechanical properties and structural behaviour. The other part of the dilemma is the possible seismic forces compelling the evaluation process. Are current codes ideal in both predicted magnitudes and method of assessment, or should a different approach be used instead? Misjudging either part could yield an incorrect assessment of vulnerability and thus subsequent rehabilitation strategies. Therefore, the current study aims at discussing earthquake demand parameters, the seismic characteristics of stone structures, with particular emphasis on shear strength and deformation, and the philosophy of possible interventions.

2. Earthquake Demand Parameters

Recent advances in earthquake engineering and seismology enable us to understand better the significance of ground motion parameters like acceleration amplitudes, frequency content and earthquake duration: all depend on the epicentral distance, site condition and geology, earthquake magnitude (energy released), fault mechanism and proximity. Evaluating earthquake severity solely using peak horizontal ground accelerations provides only partial information. Egypt's 1992 earthquake is an example of the effect of site condition on historic buildings struck by a moderate earthquake: although the magnitude was M_b = 5.9 (M_s = 5.4), significant damage was observed for large inventory of stiff historic buildings (EERI, 1992). In Canada, URM is classified as the most vulnerable construction material, and so structures built pre-1970 are, in general, thought to be vulnerable to severe damage or complete failure when subjected to moderate to strong earthquakes (NBCC, 2005). Inasmuch as the natural period of vibration of historic structures is ever low (high frequency), any ground motions with filtered high frequency components can cause significant damage even though the earthquake is moderate. In addition, for massive structures, like historic ones, the amplitude of vertical acceleration is as equally important as the amplitude of horizontal acceleration. High-frequency ground motions can even be associated with earthquake magnitudes as low as 5.0 and their negative effects increase with the decrease of inherent ductility and integrity of stone structures – brittle structures (Stewart et al., 2001).

In past earthquakes, the observed ratio of vertical to horizontal peak ground acceleration has ranged from 0.5 to 1.4 and those events were associated with significant damage to URM and, in particular, stone structures (EERI, 2005). Such amplitudes of vertical acceleration exceed the two-thirds ratio typically used in seismic evaluation and design by codes of practice. Total collapse of a stone masonry building was observed due to the high amplitude of the vertical acceleration, which was close to the horizontal acceleration (Costa, 2002). Costa explained that the vertical acceleration has particular importance in multi-wythe stone structures because the response of loose materials inside the walls depends on the frictional forces between particles – forces that reduce with increasing vertical acceleration. Vertical ground motion is mainly associated with the propagation of P-waves (compressive waves in the vertical direction) that have higher frequency components than the S-waves which propagate in the horizontal direction, because of their possession of shorter wavelengths (Elnashai and Sarno, 2008).

Seismic waves do not have unique frequency amplitudes: rather, they contain a wide range of frequency and energy contents. High-frequency/low energy components of seismic waves are too high to resonate with the fundamental frequency of historic structures, but these waves can trigger two failure mechanisms in dry-rubble, multi-wythe, stone walls (Meyer et al., 2007). The first is wall delamination, triggered by high-frequency amplitudes that cause small vertical inter-stone vibrations and result in irreversible relative displacements between the outer layers and the core, leading to wall failure. The second is wall crumbling that is associated with an increase in out-ward thrust from the fluidification and densification, or loss of shear strength of the materials filling the core of the wall; i.e. the filling materials can contribute to wall failure. Furthermore, Meyer et al. (2007) showed that wall delamination occurred faster at higher frequencies and worsened when vertical and horizontal accelerations occurred simultaneously, as in a real earthquake. Again, the vertical acceleration decreased the frictional forces between the stone wythes and thus weakened, or diminished, the wall's resistance to horizontal accelerations. In summary, earthquake demand parameters are very significant for historic structures even if they are seemingly minor or moderate. Seismic activities of moderate magnitudes may become as important as those with high magnitudes, depending on the other influencing demand parameters.

3. Strength and Deformation of Stone Masonry Walls

Current trends of seismic evaluation and design rely not only on strength evaluation but also on evaluation of deformation capacity. The strength or deformation capacity of masonry depends on the mechanical properties of the constituent units like stone and mortar as well as quality, integrity, topology…etc. Compressive strength, elastic moduli, and shear strength are typical examples of these mechanical properties, and they are site-dependent, as are the masonry strength and deformation capacity. Our need to rehabilitate heritage stone masonry structures requires assessment of their strength and deformation capacity and is warranted in all seismic regions (Tomaževič, 2000; Vasconcelos and Lourenco, 2009). In general, masonry structures resist earthquake-induced forces through in-plane wall strength and stiffness insofar as out-of-plane responses are prevented or deferred until the required in-plane responses are achieved. Retaining the integrity of walls – particularly in multi-wythe masonry– and structures is typically a means to overcome undesired out-of-plane responses. Structural integrity can be assured by tying the wythes of a wall together, e.g. by transverse masonry units; connecting walls to walls and floors; and avoiding slender walls, by having wall thicknesses exceed the requirements of a design earthquake. The thickness of a wall is as important as the section strength in the out-of-plane direction (Doherty, 2002; Elmenshawi et al., 2010b; Priestley, 1985). Achieving structural integrity also enhances the arching action of walls in the out-of-plane direction. Thus the resistance of a masonry structure to earthquake-induced forces is dependent on the strength and deformation capacity in the in-plane direction as long as the out-of-plane responses are negated.

3.1. In-Plane Strength and Deformation

Experimental data concerning the relationship between the in-plane shear strength and deformation of stone masonry walls show nonlinearity. For example, Elmenshawi et al. (2010a) tested multi-wythe stone walls (one external wythe of sandstone and the other of limestone, with a rubble core between) and obtained the relationship shown in Fig. 3. They observed that the nonlinearity between the lateral shear load, V, and corresponding displacement, Δ, commenced at an early stage. The nonlinear relationship was modeled with an equivalent bi-linear format, as shown in Fig. 4, such that the areas under the experimental and bi-linear curves were equal. The bi-linear relationship was used to define an associated equivalent ductility, the term of *equivalent ductility* being used instead of *ductility* to emphasize that the inelastic behaviour being defined was not due to the yielding of mild steel, as in reinforced concrete, reinforced masonry or steel structures.

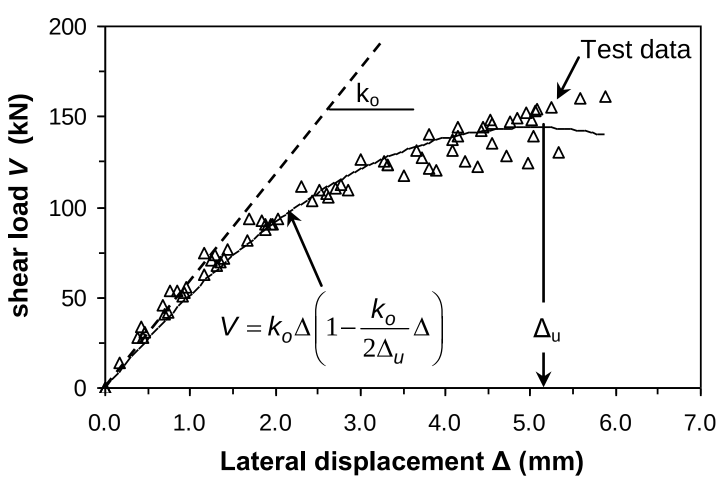

Fig. 3 – Experimental Lateral Shear Load and Displacement of Stone Walls (Elmenshawi et al., 2010a)

The equivalent ductility is the ratio of the displacement at ultimate strength to the displacement at a yield point as shown in Fig. 4. The bi-linear relationship could be even further simplified to an elasto-plastic model (α = 0 in Fig. 4) because the variation in α did not significantly affect the equivalent ductility values. In contrast, the ratio of effective stiffness k to the initial stiffness k_o did have a profound effect. The equivalent ductility, μ_{eq}, was therefore consequently computed as in Eq. 1 (Elmenshawi et al., 2010a).

$$\mu_{eq} = \frac{\Delta_u}{\Delta_y} = \frac{k}{k_o}\left[1.5 + 1.23\sqrt{1.5 - \frac{k_o}{k}}\right] \qquad : \frac{k}{k_o} \geq \frac{2}{3} \tag{1}$$

The term of k/k_o in Eq. 1 indicates the stiffness degradation of the stone walls in the elastic stage. Theoretically, if there is no stiffness degradation ($k = k_o$) before virtual yielding, i.e. no tensile cracks develop, the equivalent ductility can reach 2.4. However, it is prudent to consider a practical limit to the ratio k/k_o, e.g. 0.8. In this case, the equivalent ductility has a maximum of 1.7. Elmenshawi et al. (2010a) found further that the ratio k/k_o was correlated to the lateral drift, Δ/h, of the wall as shown in Fig. 5. In fact, Fig 4 and Fig. 5 reveal that continuing maintenance of masonry is necessary to minimize degradation of k/k_o over time, in order to keep the equivalent ductility as high as possible and the wall's lateral drift as low as possible. The post-peak behaviour of the walls was not monitored in Elmenshawi et al. (2010a), so the equivalent ductility factor obtained is deemed to be a lower-bound of the actual ability of the walls to deform inelastically.

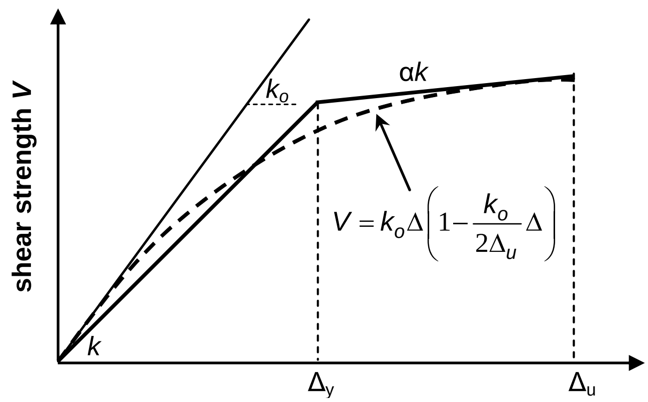

Fig. 4 – Idealization of Nonlinear Behaviour of Stone Walls (Elmenshawi et al., 2010a)

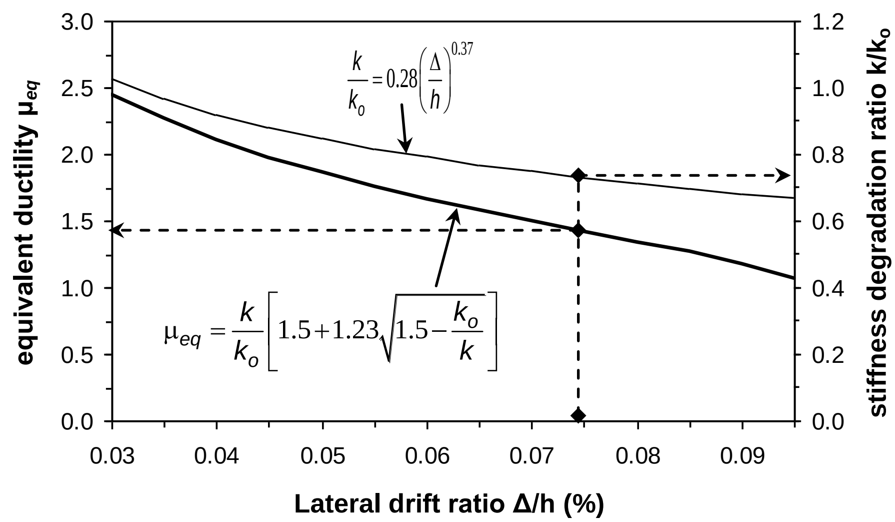

Fig. 5 – Trinity of Drift, Ductility, and Stiffness of Stone Walls (Elmenshawi et al., 2010a)

Other factors that can significantly affect the shear strength and deformation capacities of stone masonry walls are the bond (texture) patterns and the level of axial compression. Vasconcelos and Lourenco (2009) investigated the effect of axial compression and bond patterns on the in-plane shear strength and deformation of single-wythe stone walls subjected to cyclic displacement excursions. They examined three distinctive textural topologies; dry-stack stone (no mortar in the joints), irregular stone units (with mortar and coursed stones), and rubble masonry (with mortar and uncoursed masonry). The ductility factors decreased as the irregularity of the textural arrangement and vertical axial compression increased, giving the lowest ductility factor to the rubble stone walls. On the other hand, the bond pattern did not affect the lateral shear strength of the walls for axial compression values between 0.5 to 0.875 MPa, whilst for higher axial compression the shear strength decreased as the irregularity of bond pattern increased (Vasconcelos and Lourenco, 2009). Correlating the lateral shear strength of stone walls (macro level) to the basic mechanical properties (micro level) is yet another challenge that merits more discussion and thus is reviewed next.

3.2. Shear Strength of Stone Masonry

The lateral shear strength (macro level) of stone masonry walls is determined during testing based on failure modes attained locally: shear, flexural, or combined modes. Flexural-based failure modes are rarely observed in real structures, with shear-based failure modes indeed dominating the behaviour of walls. Hence, the shear resistance determines the seismic resistance of the walls primarily and thence that of the structure (Tomaževič, 2009). Generally, there are two main shear failure modes in masonry - diagonal tension cracks and bed-joint sliding. Diagonal tension cracks can develop in either a stair-step pattern through head and bed joints if the mortar/unit interface is weaker than the masonry units, or as a roughly straight diagonal path through masonry units and mortar joints if the mortar is stronger than the units. The stair-stepped cracking pattern is preferable because the vertical compressive stress normal to the bed joints results in the development of frictional forces that will remain active at nearly any amount of lateral deflection and thus continue to absorb energy, typically without collapse of the wall (FEMA 356, 2000). Eq. 2 is recommended for evaluating the shear strength of stone masonry walls despite its apparent simplicity, to allow for the potential chaotic configurations in historic buildings, (Elmenshawi and Shrive, 2015). The relationship in Eq. 2 was first suggested by Turnšek and Čačovič (1970), then verified for old and new URM by Tomaževič (2009) and Calderini et al. (2009). The shear strength, V_u, of a stone masonry wall can be estimated as:

$$V_u = \frac{A}{b} f_t \sqrt{1 + \frac{\sigma}{f_t}} \qquad (2)$$

where: σ is the axial compressive stress, A is the cross-sectional area of the wall, f_t is the referential tensile strength of the masonry, and b is a factor accounting for the shear stress distribution and depends on the aspect ratio of the wall (h/ℓ: ℓ is the section's depth) and the ratio of shear to vertical stress. For a fixed-fixed wall, $1.0 \leq b = h/\ell \leq 1.5$ (Magenes and Calvi, 1997). Eq. 2 is based on the calculation of the principal tensile stress at the middle of the walls, assuming the masonry is elastic, isotropic, and homogeneous. Elmenshawi and Shrive (2015) investigated the validity of Eq. 2 in multi-wythe stone walls thoroughly and found that if the wall integrity is achieved by a means of tie beams or transverse stone or anchor reinforcement, the wythes displace laterally as one unit without any distortion in the in-plane direction. Accordingly, the cross-sectional area, A, represented the whole section regardless of the dimensions of individual wythes and inner cores; likewise, the shear, tensile, and normal stresses could be considered as smeared values for the whole section. Nonetheless, the assumptions would yield better results if the width of the middle core becomes small and the mechanical properties of the outer wythes are close to one another. Furthermore, Eq. 2 successfully predicted the shear strength of non-regular stone masonry walls better than regularly coursed walls. That is, when a stone masonry wall has a chaotic bond pattern, the propensity of the wall to behave according to the assumptions of Eq. 2 – elastic, isotropic, and homogeneous– increases (Calderini et al., 2009).

The most ambiguous parameter in Eq. 2 is the referential tensile strength f_t because it is not a real strength for a specific surface; rather, it is the tensile strength that verifies the experimental shear strength and is evaluated by inverting Eq. 2. That is why it is called a referential tensile strength. Masonry types play a central role in determining the design values of f_t: for brick/block URM walls, an f_t = 0.2 MPa was obtained (Tomaževič, 2009; Turnšek and Čačovič, 1970); for mixed stone-brick masonry, an f_t = 0.08 MPa was determined (Sheppard, 1985); and for three-wythe stone walls, an f_t = 0.09 MPa was obtained (Elmenshawi and Shrive, 2015). Thus, it is clear that the referential tensile strength of stone masonry is lower than that of other URM. It is worth stating that FEMA 356 (2000) suggests that as an approximation, the referential tensile strength f_t can be replaced by the bed-joint shear strength for URM other than stone. The effect of variation of the terms in Eq. 2 is shown in Fig. 6 for the value of f_t = 0.09 MPa, through the aspect ratio (*b*-variable), and the ratio of σ/f_t. These both influence the shear strength V_u. For heritage structures, however, the ratio σ/f_t is expected to be between 2.0 to 5.0; therefore, one can anticipate that the shear strength of a wall would vary between 0.1 and 0.22 MPa regardless of the *b*-value, for the fixed-fixed boundary condition.

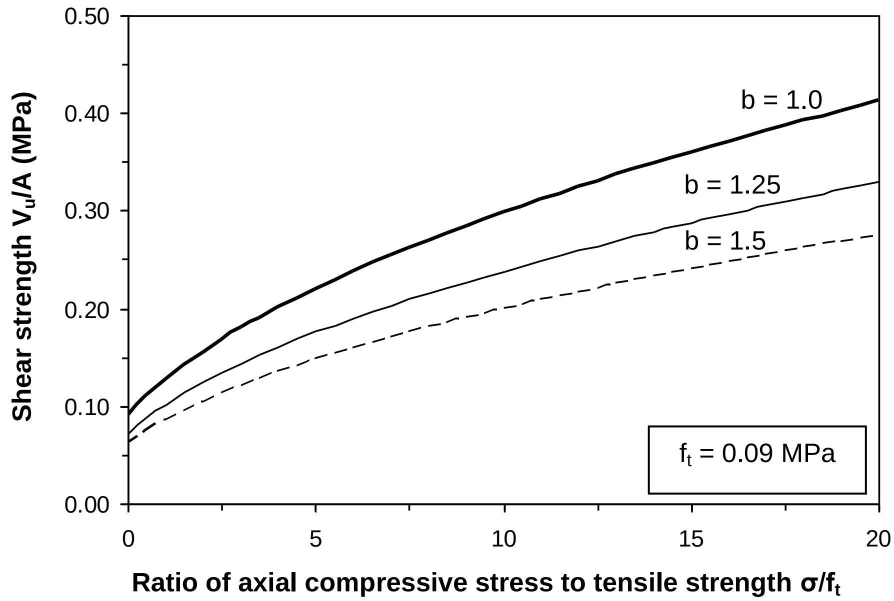

Fig. 6 – Factors Affecting Shear Strength of Stone Walls (Fixed-End Conditions)

In Eq. 2, there is no difference between failure modes associated with diagonal tension through masonry units, shear-sliding along a horizontal plane, or stair-stepped cracks. Thus one may query why a friction analogy cannot best explain the latter two failure modes. Recent experimental work showed that the friction analogy theory could not predict the shear strength of three-wythe stone walls despite those walls having stepped cracks. This was attributed to the propensity of irregular stone walls not to follow the Coulomb friction criterion (Elmenshawi and Shrive, 2015). Furthermore, the cohesion and friction coefficient values available in the literature are based on well-defined stone patterns, which was not the case in the stone walls tested: they had a form of running bond in the limestone wythe, and a sneck pattern in the sandstone wythe. The ability of the friction analogy theory to predict the shear strength of masonry walls was also questioned by Tomaževič (2009) even though Eurocode 6 (2004) adopts this analogy. To mimic severely deteriorated mortar joints in historic stone walls, Lourenço et al. (2005) and Vasconcelos et al. (2006) conservatively presumed that such walls would behave as dry stacked stone units. Therefore, Lourenço et al. (2005) proposed a different formula to determine the shear strength of a single-wythe dry stone wall as:

$$V = \tan\phi\left[1 - \frac{h}{2\ell}\tan\phi\left(\frac{1}{1-\sigma/f_m}\right)\right]N \quad (3)$$

where: V is the maximum horizontal shear force, N is the applied vertical force, f_m is the average compressive strength of the wall, and ϕ is the angle of friction. Eq. 3 was found to underestimate the shear strength of mortared-stone walls (Elmenshawi and Shrive, 2015). Mortar type, quality, and strength are known to affect the masonry shear strength and failure modes (Bosiljkov et al., 2003).

Failure modes obtained in laboratory testing of masonry walls can be different to those in real structures. For example, in the laboratory, one can observe the formation of horizontal tensile cracks in the tension zone of the wall, for example, in the bed joints at the support. Masonry units have also been observed to crush at the compressed corners of walls, sometimes before the formation of diagonal shear cracks in the central part of the wall. However, in-situ, neither horizontal cracks nor the crushing of masonry have been observed (Tomaževič, 2009). In addition, shear failure can be one of several types seen in masonry walls. Lastly, the foregoing shear strength formulae are not expected to change due to the cyclic excursions of a seismic event (Vasconcelos et al., 2006). Further studies are certainly warranted on this subject.

4. Intervention Philosophy

High shear demand is to be expected in massive, stiff historic structures should they be analyzed in accordance to current seismic codes. Those structures may not be able to withstand such high shear without adding new structural elements, but such additions could well alter the heritage values of buildings negatively, in contradiction to conservation principles. Thus before adding new elements or modifying existing topology, one needs to make sure that the shear forces are accurately estimated, given that the codes being used are meant to regulate new construction with new materials. Tomaževič (2000) argued that the seismic demand for old stone structures could be reduced from that of code recommendations for new buildings, so as to optimize broader social, economic, and historic goals without jeopardizing the structural safety of those structures. He based his rationale on observed experience in high and moderate seismic zones. Likewise, Eurocode 8 (2004) suggested a reduction in the design ground acceleration if i) the anticipated total cost of strengthening the entire building increases significantly, and ii) the code's required acceleration for redesign led to unacceptable architectural alteration for monuments. The reduction ratio depends on the seismic zone selected for the analysis and can be as high as 0.67 for one zone and as low as 0.0 for another. Performing the correct seismic analysis for a heritage structure should lead to an appropriate intervention for strengthening.

Visible interventions in heritage structures are not commonly preferred as they reduce the heritage values of the structures. Rehabilitation schemes need to comply with special requirements including preservation of the heritage value, protection of life safety, and being cost effective. Moreover, while the ductility, lateral strength and stiffness should be improved, the increase in lateral stiffness should not alter the seismic demand significantly. These conditions limit the available alternatives and increase the research and engineering efforts to seek feasible techniques. Of the available solutions passing these criteria, tying stone walls in the transverse direction can be one step for multi-wythe walls; tying the exterior wythes can postpone failure of a wall under triggering static or dynamic loadings. Meyer et al. (2007) noticed that the failure of walls with no transverse stones (overlapping between the wythes) tested under dynamic loadings, tested under dynamic loadings, occurred at a lower acceleration, in a catastrophic manner with very limited dissipated energy compared to walls with stones overlapping between the wythes. The latter walls deformed substantially before failing. For walls tested under static loadings, inadequate integrity between the wythes can also cause early separation of the outer wythes from the internal core, increasing the second-order effects under axial compressive stress (Valluzzi et al., 2004; Vintzileou and Tassios, 1995). Grouting of such walls had a better effect than using reinforcing bars to tie the outer wythes (Valluzzi et al., 2004). For such walls, the mutual interaction between gravity and seismic loads is conceived to result in a subtle situation. The use of transverse stone, or other techniques, to connecting the wythes can reduce the second-order effect and enhance wall integrity. The seismic performance level (damage level) of the structure after deploying the strengthening technique should be identified correctly according to the expected earthquake severity, cost, and structural function. If the result is not as desired, an alternative scheme should be considered.

5. Conclusions

The current paper discusses some aspects governing the seismic vulnerability of historic stone masonry. These include the earthquake demand parameters, in-plane strength and deformation, shear strength, and intervention philosophy. In view of the above discussions, the following conclusions are drawn:

- Assessment of seismic vulnerability of historic structures is essential. However, this requires evaluating the elements' strength, stiffness, and inherent ductility as well as the ground motion parameters. A great challenge exists in that there are limited or no plausible engineering data, leaving vagueness and uncertainty for existing historic structures. Nonetheless, overlooking parameters from either the structure or the design earthquake can lead to an inappropriate intervention with an unwanted cost burden and no benefits – certainly not to the structure.

- Earthquake parameters encompassing the frequency and energy content, magnitudes, acceleration amplitudes in both the vertical and horizontal directions can affect profoundly the vulnerability of a historic structure. Notably, the vertical component of the ground motion is as important as the horizontal component, so the recommended code value (the two-thirds ratio) needs a thorough study: higher values were typically observed in the past. In general, it is prudent not to ignore ground motions of small magnitudes in both horizontal and vertical directions to analyze heritage stone structures constructed of multi-wythe walls.

- Stone walls can resist earthquake demands in the in-plane direction as long as the unwanted out-of-plane response is postponed or eliminated. Of note, the inherent ductility of undamaged stone walls can be a maximum of 1.7 - a very limited value compared to other contemporary structural systems. To keep the ductility at this level, continuing maintenance is required to repair any damage induced by weathering or small seismic events.

- Although existing stone configurations may be complex, the shear strength can be evaluated with a simple shear formula that considers only diagonal cracks in the walls assuming elastic, isotropic, and homogeneous wall properties.

- The intervention philosophy for conservation of historic structures requires reversible and invisible means not to impair the heritage values of the structure, considering life safety of the users and economy. Techniques considered for improving the ductility, lateral strength and stiffness of walls should not alter the seismic demand significantly.

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