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On the Dynamic Behaviour of Strengthened Stone Masonry Walls

ELMENSHAWI Abdelsamie[1,a], SOROUR Mohamed[1,b], DUCHESNE Don[2], PAQUETTE Jocelyn[2], MUFTI Aftab[3], JAEGER Leslie[4] and SHRIVE Nigel[1,c]

[1]Department of Civil Engineering, University of Calgary, Canada

[2]Public Works and Government Services Canada, Canada

[3]Deprtment of Civil Engineering, University of Manitoba, Canada

[4]Department of Civil Engineering, Dalhousie University, Canada

[a]asmensh@ucalgary.ca, [b]msorour@ucalgary.ca, [c]ngshrive@ucalgary.ca

Abstract Unreinforced stone masonry is common in heritage structures worldwide. Unfortunately, these structures are susceptible to failure or severe damage when subject to dynamic or seismic loading. Conservation of historic structures is a challenge as the heritage and cultural values need to be preserved while the advent of new seismic codes may require major strengthening to be implemented. The new seismic codes demand high seismic strength and ductility for such structures, whereas neither the strength nor the ductility of an existing stone masonry building can be quantified easily. The Parliament buildings of Canada fall into this category. Therefore, an extensive experimental program was carried out to investigate the dynamic and seismic behaviours of stone walls representative of Canada's Parliament buildings. The walls were constructed of double stone wythes with the cavity between being filled with weak mortar, shards and small stones, constituting a rubble core of the walls. The experimental program included in-plane quasi-static, free vibration and high frequency loadings, together with out-of-plane shake table tests. The tests were aimed at investigating the integrity, strength, damping, stiffness degradation, and ductility of the walls. Different potential strengthening methods were assessed, methods that would minimize structural intervention and preserve the heritage values of the building. The methods involved different metallic anchors and traditional stone interlocking to tie the two outer wythes together. Fortunately, the stone walls exhibited satisfactory performance in all cases. In addition, the test results suggested that plain un-strengthened stone walls had strength and other characteristics similar to those of the rehabilitated walls, in the range of the imposed load scenarios.

Keywords: Unreinforced masonry, stone, seismic evaluation, rehabilitation, damping, ductility

Introduction

Structures built with unreinforced stone masonry can be vulnerable to seismic activity and show very low strength as evidenced by several failures in past earthquakes, although most of these structures did have thick and heavy walls (EERI 2005). The Parliament buildings of Canada are typical historic buildings that were built with unreinforced stone masonry and possibly warrant seismic strengthening provided that their heritage values is not affected. The buildings were built on "rule of thumb" in that no seismic code was developed at that time. Of the Parliament buildings, West Block (Fig. 1) is of particular interest currently. West Block was built of multi-wythe stone masonry. This type of construction creates possible bond deficiencies between the wall wythes that affect structural integrity and consequently the structural seismic performance. The absence of integrity during a seismic event can lead to wall delamination and wall crumbling (Meyer et al. 2007). It has been deemed that seismic rehabilitation of old stone structures is warranted in all seismic regions, and tying of the walls needs to be provided in every case (Tomaževič 2000). Some techniques have been suggested for strengthening of such heritage structures such as re-bonding, tie-back, reinforcing by means of grouting, and cemented networks for the masonry (Lizzi 1981). For seismic purposes, strengthening of unreinforced stone walls by cement grouting is preferred in regions of moderate and high seismic activity.

Increasing the grout strength has little effect on the in-plane seismic strength of the wall (Tomaževič 1996). CSA-A371-04 (2004) recommends bonding as a means of strengthening of rubble stone masonry. For multi-wythe stone walls, if the cavity between the wythes is filled by loose sand and gravel (e.g. dray-stacked masonry walls), there is a high possibility of the wall crumbling during a seismic event with high frequency content because of the increase in the outward thrust from the densification and fluidification of these loose filling materials (Meyer 2007). Therefore, the use of metal anchors to tie the wythes across a wall might avoid such wall crumbling. Numerical modeling showed that metal anchors would improve the seismic strength of stone structures if they were mounted in the plane of the walls; however, no trial was made to study the effect of lateral tying of wall wythes (Jordan and Brookes 2004). In other experimental work, metal anchors did not improve the compressive strength of three leaf walls when they mounted across a rubble core on their own, but when combined with grout injection, they did increase the strength slightly beyond that of grout injection alone (Valluzzi et al. 2004). The objective of the current research was to provide a closer look at the in-plane and out-of-plane behaviours of plain and strengthened stone walls subjected to static and dynamic loading scenarios. The strengthening schemes examined involved different metal anchors as well as traditional stone interlocking.

Figure 1: The West Block building of the Parliament buildings of Canada

Experimental Program

Wall Specimens Eight walls were constructed at full scale, two plain and six containing different proposed strengthening schemes, and tested under different loading scenarios. Each wall was composed of three wythes: two stone wythes separated by a rubble stone core with weak strength mortar. The stone wythes were sandstone in a sneck pattern and limestone wythe in running bond as shown in Fig. 2. The wall dimensions were 2.0×2.75×0.54 m (width × height × thickness). To imitate the construction variability experienced throughout the life of the West Block building, the walls were built in two batches as shown in Table 1. In the first batch, the stones were dressed on the interior face so that the bond strength between the core and the surrounding wythes would be reduced. In addition, the walls of this batch had thicker joint mortars than the second batch. A ratio of 1 (lime) to 3 (sand) by volume was chosen to produce weak mortar to reflect the lower end of the spectrum expected in the existing building. The strengths of cubes of mortar from the first batch of walls were 1.67 and 2.58 MPa after six and nine months, respectively. Two cylinders (75×150 mm) were cored from the limestone and three from the sandstone. The compressive strengths of the limestone were 99.3 and 105.6 MPa, with moduli of elasticity of 21.6 and 62.1 GPa, respectively. The compressive strengths for the sandstone were 232, 247, and 202 MPa, with moduli of elasticity of 59, 67.2, and 58.7 GPa, respectively. Since the mortar strength is considerably less than the stone strength, the wall behaviour will be dominated by the behaviour of the mortar and the interface between the mortar and the stones.

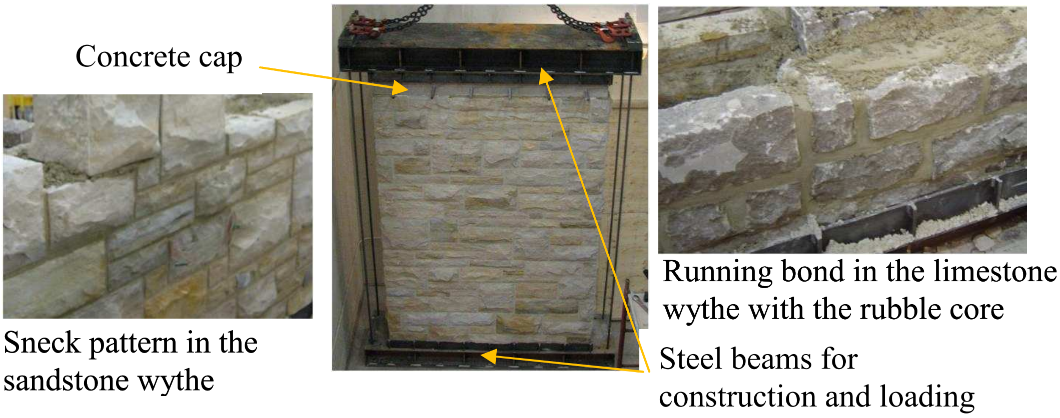
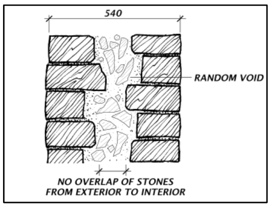
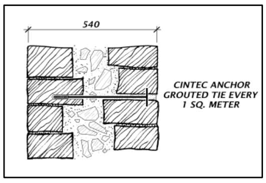
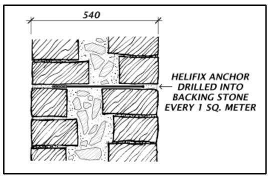
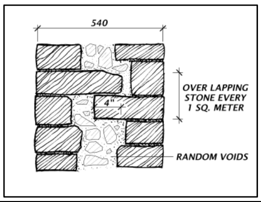


Figure 2: General layout of the wall showing the bonding patterns used in the stone masonry

Table 1: Description of the wall batches and rehabilitation schemes

Wall No.	*Batch No.*	*Anchorage description*	*Remark*
W1	1		Plain
W2	1		
W3	2		Cintec Anchor
W4	2		
W5	1		Fully-socked Cintec
W6	1		Helifix
W7	2		Stainless steel cramps every 1.0 m^2
W8	2		Overlapping stones every 1.0 m^2

Loading Scenarios Three types of in-plane loading were applied to the walls consecutively. First, axial compression was applied to obtain axial load-deformation data. This was followed by an in-plane push-pull lateral cyclic static test under displacement control with increasing displacement per cycle to a maximum of 5.0 mm. The walls had their tops and bottoms restrained against rotation during this racking which was applied with an imposed axial compressive stress of 0.3 MPa for all walls except the W2, where the axial compressive stress was doubled. Last, dynamic in-plane loading was applied at 5.0 Hz with sinusoidal amplitude. The load amplitude was increased monotonically from 1.0 to 10 kN at increments of 1.0 kN. After completing the in-plane static and dynamic loadings, the walls were subjected to series of out-of-plane shake table tests using real-time readings of actual and synthetic earthquakes. The actual readings were for the earthquake recorded on Parliament Hill in 2002, amplified 183 times to match the requirements of the Uniform Hazard Spectrum (UHS) of the NBCC (2005). The synthetic readings were developed to comply with the short period end of the UHS for the City of Ottawa with a probability of 2% being exceeded in 50 years. The shake tests were commenced by applying the 60% of the displacements of the amplified actual earthquake, followed the synthetic earthquake at 60% amplitude. Both records were repeated at 100% amplitude, and again at 110%. Therefore, each wall was shaken six times. More detailed descriptions of the tests are provided by Elmenshawi et al. (2010a), and Sorour et al. (2010). Damping characteristics of the stone walls were determined from free vibration tests as described in Elmenshawi et al. (2010b). The free vibration tests were conducted by applying an in-plane lateral displacement at the top of the wall through a tension cable. The cable was then cut to allow the wall to vibrate in-plane. The wall acceleration was monitored at different heights with accelerometers. The free vibration test was performed three times in the testing sequence, for different specific damage states. The first vibration test was conducted prior to the lateral in-plane tests to determine the damping values in the undamaged state. The second test was executed after the in-plane dynamic tests to investigate the effect of in-plane damage on the damping values. The final vibration test was performed after the shake table tests to examine the damping ratios after any out-of-plane damage had occurred.

Test Observations and Discussions

In-Plane Tests No cracks appeared until the maximum displacement of 5.0 mm was reached for the walls in the first batch constructed. By repeating the excursion at the maximum displacement, a stepped-diagonal "X-cracking" formed in the walls with different paths in the sandstone and limestone wythes. Reaching the maximum displacement for the walls in the second batch did not initiate cracking. However, repetition of the maximum displacement resulted in a similar cracking pattern developing. All walls showed the same cracking pattern regardless of the strengthening scheme used. The use of different strengthening techniques affected neither the hysteretic behaviour of the walls, Fig. 3, nor other characteristics such as lateral strength (V_u), dissipated energy (E_t = total integration of the load-displacement loops), and stiffness degradation (k/k_o = ratio of secant stiffness at ultimate to initial tangent stiffness), Table 2. The normal sort of variation in masonry specimens was observed. The results of W2 were excluded as W2 was subjected to a different axial stress to the other walls. Since no differences were observed between the walls, only samples of hysteretic behaviours are shown in Fig. 3. Further discussion can be found in Elmenshawi et al. (2010a).

Table 2: Test results of the in-plane cyclic static tests

Wall No.	*W1*	*W3*	*W4*	*W5*	*W6*	*W7*	*W8*
$V_u/V_{u\text{-}W1}$	1.0	1.11	1.07	0.94	0.90	1.12	1.16
$E_t/E_{t\text{-}W1}$	1.0	0.85	0.91	1.04	0.81	1.12	1.01
k/k_o	0.44	0.54	0.51	0.42	0.41	0.45	0.51

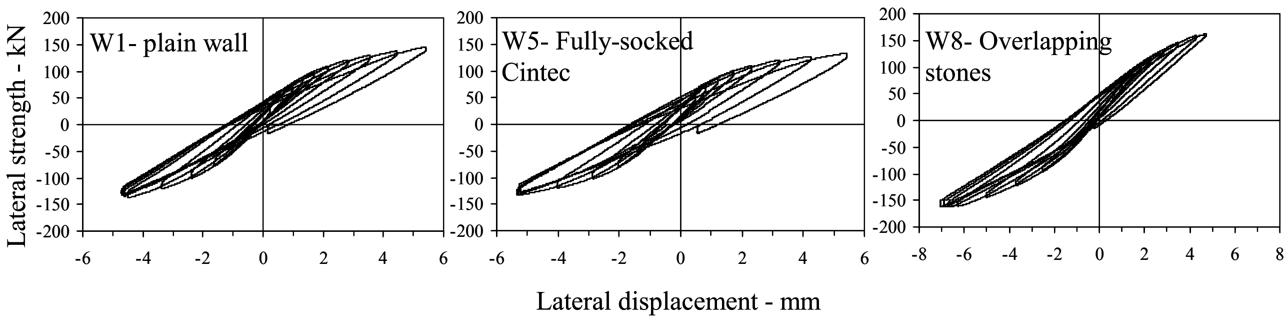


Figure 3: Hysteretic behaviour of stone walls strengthened with different schemes

The progressive nonlinear behaviour observed for the envelope of the load-displacement curves shown in Fig. 3 can be transferred to an equivalent elasto-plastic behaviour with limited plastic deformation (Elmenshawi et al. 2010a). Accordingly, formula (1) was deduced to anticipate the equivalent displacement ductility of the walls, in which for a cracking secant stiffness (k/k_o) of 0.8, the maximum ductility is 1.7. This very limited value represents the potential ductility in healthier stone structures. No strengthening scheme showed significant difference for the anticipated ductility.

$$\mu = \left({}^{k}\!/\!_{k_o} \right)\left(1.5 + 1.23\sqrt{1.5 - \left({}^{k}\!/\!_{k_o} \right)} \right): \ {}^{k}\!/\!_{k_o} \geq 0.67 \qquad (1)$$

The in-plane dynamic tests at 5.0 Hz which followed the quasi-static ones did not initiate new cracks, but rather widened the existing cracks. The high frequency loading did not trigger new failure modes, as the value used is probably smaller than that which could affect the walls as observed by Meyer et al. (2007). Delamination of the wythes or crumbling need higher frequencies than 5.0 Hz and the effect of frequency content was coupled with the imposed acceleration (Meyer et al. 2007).

Free Vibration Tests Damping values were extracted for the walls from the free vibration tests. Masonry elements provide more progressive cracking than concrete and steel elements, so their damping ratios should be different from elements of the latter materials. Also, the reduction in elastic modulus due to cracking would increase the damping level as compared to the uncracked behaviour. Although the wall responses were captured through accelerometers, the known dynamic formula to calculate the viscous damping ratio ζ is still valid by determining the logarithmic decrement δ as:

$$\zeta = {}^{\delta}\!\Big/\!_{\sqrt{4\pi^2 + \delta^2}} \qquad (2)$$

The experimental results showed variations in ζ, T (natural period of the wall), and δ, which means that the viscous damping is amplitude-dependant, and consequently is not linear as usually assumed in dynamic analysis. Regression analysis was used for each phase to extract the corresponding viscous damping ratio. For each phase, the relationship between the acceleration amplitudes and time (normalized to the average value of T for each wall) was used for the regression analysis (e.g. Fig. 4). The damping ratios for three damage levels are 3, 4, and 5%, respectively - the viscous damping ratio is damage-dependent. In addition, the rate of decay in the acceleration response revealed that Coulomb friction damping is a strong candidate for the damping mechanism, in addition to the viscous nature of the stone walls. However, it is very difficult to represent both damping mechanisms in dynamic analysis (Elmenshawi et al. 2010b).

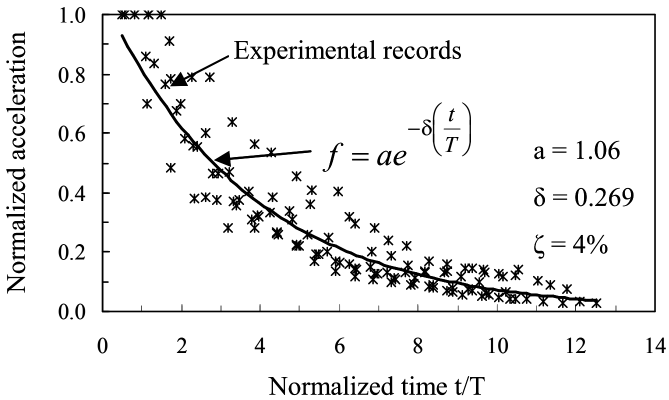


Figure 4: Viscous damping ratio of the walls after the in-plane loadings

Out-of-Plane Tests When the walls subjected to the out-of-plane shaking, they did flex, and most walls lost superficial mortar, especially in areas damaged by the in-plane racking which had loosened the surface mortar in some places. It is important to note that no wall collapsed and no wall split under these simulated earthquake displacements. All walls survived intact. One piece of sandstone became loose in one wall and one piece of limestone in another, at the intersection of the cracks in the stepped diagonal form. However these stones could not be removed from the walls. The accelerometer readings were dominated by noise, which has hindered full analysis of the results. The general trend of the out-of-plane movements was that the fixation of the frame holding the wall at the base did not enforce the top and bottom of the wall to move synchronously. However, the middle of the wall did displace differently to the top and bottom, indicating that some flexural displacement did occur.

Conclusions

Multi-wythe unreinforced stone masonry walls are frequently found in heritage structures, as is the case in the Parliament buildings of Canada. These structures may need to be strengthened after the advent of recent seismic codes. For such structures, keeping the integrity of the walls during a seismic event is essential to avoid wall delamination and crumbling. The use of metal anchors as a strengthening scheme to tie the wall's wythes together across the wall did not improve the seismic characteristics of the walls. The seismic characteristics that were not affected include lateral strength, dissipated energy, stiffness degradation, and ductility. The high frequency in-plane lateral loading applied (5.0 Hz) did not trigger any additional failure modes, but simply widened existing cracks. Viscous damping in the stone walls is not elastic as assumed in dynamic analysis, but is amplitude- and damage-dependent. Although the assumed viscous nature of the damping was verified by the experimental work, Coulomb friction damping is also a candidate for such structures.

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References

[1] Earthquake Engineering Research Institute (EERI) (2005). "Intensities and damage distribution in the June 2005 Tarapacá, Chile, earthquake," *EERI News Letter*, November.

[2] Elmenshawi, A, Sorour, M, Mufti, A, Jaeger, L, and Shrive, N (2010a). "In-plane seismic behaviour of historic stone masonry." *Canadian Journal of Civil Engineering*, 37, 465-476.

[3] Elmenshawi, A, Sorour, M, Mufti, A, Jaeger, L, and Shrive, N (2010b). "Damping mechanisms and damping ratios in vibrating unreinforced stone masonry." *Engineering Structures*, Accepted.

[4] Jordan, J and Brookes, C (2004). "The conservation and strengthening of masonry structures," in *Proc. of 7th Australasian Masonry conference*, Newcastle, Australia, 83-92.

[5] Lizzi, F (1981). "*The static restoration of monuments, basic criteria-case histories, strengthening of buildings damaged by earthquakes*." Sagep Publisher.

[6] *Masonry construction for buildings (CSA-A371-04)*, Canadian Standards Association, 2004.

[7] Meyer, P J O, Germaine, J, and Eduardo, K (2007). "The impact of high-frequency/low-energy seismic waves on unreinforced masonry." *Earthquake Spectra*, 23, 77-94.

[8] *National building code of Canada (NBCC)*, National Research Council, Canada, 2005.

[9] Sorour, M, Elmenshawi, A, Parsekian, G, Mufti, A, Jaeger, L, Duchesne, D, Paquette, J, and Shrive, N (2010). "An experimental program for determining the characteristics of stone masonry walls." *Canadian Journal of Civil Engineering*, In review.

[10] Tomaževič, M (1996). "*Post-earthquake rehabilitation and reconstruction.*" F. Y. Cheng and Y. Y. Wang, Ed(s). Elsevier Science, 317-331.

[11] Tomaževič, M (2000). "Seismic redesign of existing stone-masonry buildings." *European Earthquake Engineering*, 14, 59 -66.

[12] Valluzzi, M, Da Porto, F, and Modena, C (2004). "Behaviour and modeling of strengthened three-leaf stone masonry walls." *Materials and Structures*, 37, 184-192.