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EXPERIMENTAL EVALUATION OF THE MECHANICAL PARAMETERS FOR SEISMIC ASSESSMENT OF TRADITIONAL BRICK AND STONE MASONRY BUILDINGS IN EASTERN CANADA

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Abstract: Eastern Canada has a large stock of old unreinforced masonry buildings, made with brick or stone and with architectural heritage value. To reduce the potential earthquake induced damage to these URM load-bearing walls structures, architects and engineers face the challenge of evaluating their lateral resistance and seismic performance. Evaluation of URM walls lateral resistance is also key information in damage prediction for seismic risk studies. However, there is limited information regarding the mechanical properties of URM walls, leading to difficulty in providing a reliable prediction of their seismic resistance. This paper presents an experimental programme initiated to evaluate the mechanical parameters for seismic assessment of typical traditional brick and stone URM buildings. The first phase aims to characterize the mechanical properties of the masonry and its constituent materials: manufactured moulded bricks typically used as replicas of traditional masonry, limestone blocks and cement-lime mortar used to match the mechanical properties of the original traditional cement-lime mortar. The second phase evaluates the diagonal shear strength of brick or stone masonry wallets. The third phase evaluates the lateral force-deformation behaviour of representative wall specimens under cyclic loading to capture the complex in-plane dynamic response and nonlinear behaviour of the masonry. Results are compared with predictive relations between the constituent material and the masonry mechanical properties. Although most equations from standards could predict the flexural failure mode of the masonry walls tested under cyclic loading, they all tend to overestimate the lateral resistance by up to 38%.

1 INTRODUCTION

During the 1988 Saguenay earthquake, with a magnitude Mw=5.9, damages were observed up to a distance of 350 km from the epicentre. Most of the damages to structures were to brick and stone masonry buildings (Mitchell et al. 1990). This is in agreement with worldwide post-earthquake damage surveys reports where unreinforced masonry (URM) buildings are typically associated with the highest proportion of damage (Ingham and Griffith 2011, Indirli et al. 2013). The older sectors of Montréal and Québec City are characterized by an important concentration of URM buildings many of which represent immeasurable architectural and cultural heritage (Abo El Ezz et al. 2013).

The in-plane resistance of an URM wall, and to a lesser extent its out-of-plane resistance, are highly correlated with the strength of the masonry assembly, which in turn is determined by the mechanical properties of the brick and mortar. The current challenge in evaluating the lateral resistance and performance of existing URM walls is in the identification of those material mechanical properties. Only few authors have focused on the characterization of older type of brick masonry constructed using weak cement-lime mortar (Lumantarna et al. 2012). Furthermore, in Eastern Canada, little is known about the characteristics of load-bearing walls in older structures, such as wall composition, geometry and materials and only limited information regarding their mechanical properties is reported, leading to difficulty in providing a reliable prediction of their seismic resistance.

The objective of this paper is to present the results of an experimental programme developed to characterize the mechanical parameters of typical masonry used in old URM buildings in Eastern Canada. Considered materials include manufactured moulded bricks, typically used as replicas of traditional masonry, limestone blocks and cement-lime mortar used to match the mechanical properties of the original traditional cement-lime mortar. The testing program is described and the results are analyzed and discussed. Results include: (1) compressive strength of the masonry assembly and its constituents, joint shear bond strength parameters, (2) diagonal shear strength of brick and stone masonry panels, and (3) lateral force-deformation behaviour of wallets under static-cyclic loading to capture the complex in-plane dynamic response and nonlinear behaviour of the masonry. Results are compared with predictive relations between the constituent material and the masonry mechanical properties and with predictive equations of the lateral resistance.

2 EXPERIMENTAL PROGRAMME

This project focuses on brick and stone masonry typically used in buildings in old sectors of Montreal or Quebec City. These buildings have load-bearing walls or infills URM made of clay bricks. Stone buildings are made of multi-leaf walls, often made of two or three layers of materials of different qualities and properties (stone, rubble stone, brick or tiles).

2.1 Materials

Only limited data was found in the existing literature on the mechanical properties of brick and stone material usually used in traditional URM walls. Materials were therefore selected based on a survey of past rehabilitation and conservation projects and consultation with a professional mason involved in such projects. Three types of URM were considered for this project: brick masonry, masonry made of sawn stone cubes and masonry made of stone blocks cut by guillotine replicating the shape and surface finish of old stones cut by chisel.

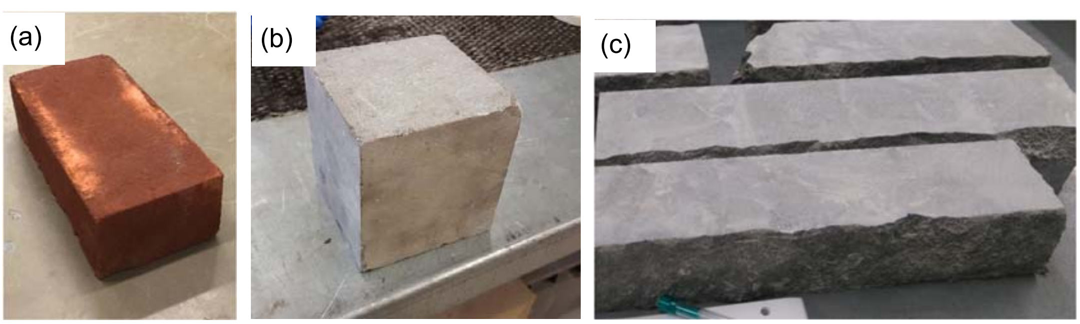

Figure 1: Masonry units used for testing: (a) Moulded full clay bricks, (b) Sawn limestone cubes, and (c) Limestone blocks cut by guillotine

URM brick specimens were made with manufactured 52-DD Glen-Gery moulded full clay bricks, typically used as replicas of traditional masonry (Figure 1a). Nominal dimensions of units were 92 mm thickness x 57 mm height x 203 mm length. Limestones, from Saint-Marc Des Carrières in the Province of Quebec, were used for the URM stone specimens. A first series of tests were carried out on specimens made with sawn cubes 100 mm x 100 mm x 100 mm (Figure 1b), and a second series on specimens made with stone blocks cut by guillotine (Figure 1c). Dimensions of the blocks as cut at the quarry varies between 90 and

135 mm in width, 70 and 80 mm in thickness and 150 and 190 mm in length. The blocks were then handed cut with a chisel to obtain a length varying between 95 and 105 mm.

The choice of mortar composition was based on reported data from mortar testing programmes in conservation projects to develop new repair mortar for restoration with properties matching the old original lime or cement-lime mortar. It was found that Portland cement/lime mortars could reproduce compressive strength values normally found in restoration projects. In order to cover a reasonable range of mortar strength, two types of cement/lime mortars were selected. A mortar Bétomix Plus type O was used for the brick masonry. For the stone masonry, mortar was mixed according to recommendations from a professional mason using Portland white cement from *Federal White Cement,* and high plasticity dolomitic lime with entrained air. These two mortar mixes provide compatible properties to old URM assemblies and have the following volume proportions: Mix 1 (1 cement / 2 lime / 6 sand) and Mix 2 (1 cement / 2 lime / 8 sand).

2.2 Testing programme

Figure 2 illustrates these three phases testing programme and indicates the number of specimens considered for each type of URM: brick masonry, masonry made of sawn stone cubes and masonry made of stone blocks cut by guillotine.

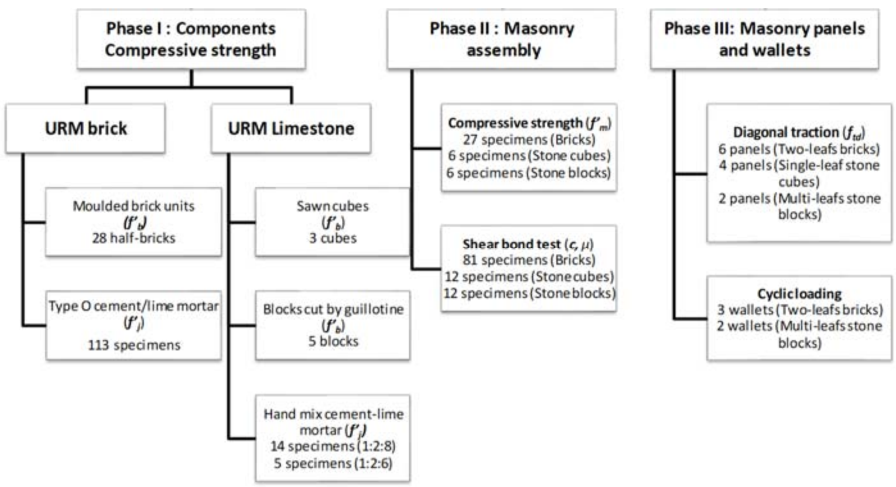

Figure 2: Experimental testing programme

The first phase of the testing programme aims at characterizing the compressive strength of the masonry constituent materials: brick (f'_b), stone units (f'_b) and mortar (f'_j). The second phase evaluates the mechanical parameters of small masonry assemblies: compressive strength (f'_m), cohesion c and coefficient of friction μ from shear bond test. The third phase focuses on the evaluation of the lateral behaviour panels and wallets. First two-leaf panels made of bricks, single-leaf panels made of stone cubes diagonal and multi-leaf panels, made of stone blocks cut by guillotine, were tested for diagonal tension (f'_{td}). Then, the lateral force-deformation behaviour of representative brick and stone wallets was assessed under cyclic loading tests. Specimens using stone blocks cut by guillotine were constructed to reproduce traditional stone masonry. The multi-leaf panels and wallets had two leafs of stone units separated by a layer of mortar and pieces of stones randomly placed.

2.3 Testing Procedures and Equipment

Tests were carried out according to ASTM standards for mortar, brick, stone and masonry as well as literature on experimental testing on masonry assembly and walls. Standards and references for tests are listed in Table 1. Compressive strength of 50 mm mortar cubes was determined using a 500 kN "Matest S.p.A Treviolo" actuator under force control. All other compression tests and shear bond tests (Table 1) were conducted using an MTS 815 press with axial compression force capacity up to 4500 kN under displacement control.

Table 1: Standards and references for tests

Test	Material	Standard or references
Compression (f'_b)	Bricks	ASTM C67 (ASTM 2016a)
	Stone	ASTM C170/C170M (ASTM 2017)
Compression (f'_j)	Mortar for URM brick	ASTM C109/C109M (ASTM 2016b)
	Mortar for URM stone	ASTM C109/C109M (ASTM 2016b)
Compression test (f'_m)	Masonry assembly	ASTM C1314-14 (ASTM 2014)
Shear bond test (c, μ)	Masonry assembly	RILEM MS-B.4 (RILEM 1996)
Diagonal tension (f'_{td})	Masonry panels	ASTM E519/E519M-15 (ASTM 2015)
Static-cyclic lateral loading tests	Two-leaf brick and multi-leaf stone wallets	ASTM E2126-11 (ASTM 2011) (Petry 2015, Mazzon 2010, Vanin et al. 2017)

For the diagonal tension tests, two sets of three brick panels with different dimensions were considered: Panels A (459 mm x 459 mm x 204 mm) and Panels B (861 mm x861 mm x 204 mm) (Figure 3a). Four stone cubes panels had average dimensions 618 mm x 618 mm x 100 mm (Figure 3b), while dimensions of two multi-leaf stone blocks panels were 625 mm x 625 mm x 270 mm (Figure 3c).

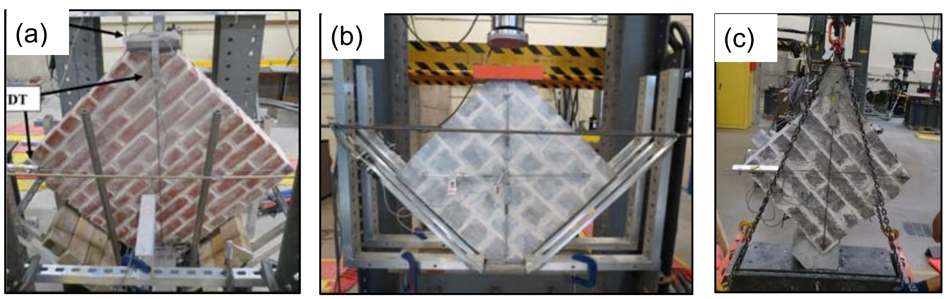

Figure 3: Masonry panels for diagonal tension tests: (a) Two-leaf bricks, (b) Single-leaf stone cubes, and (c) Multi-leaf stone blocks.

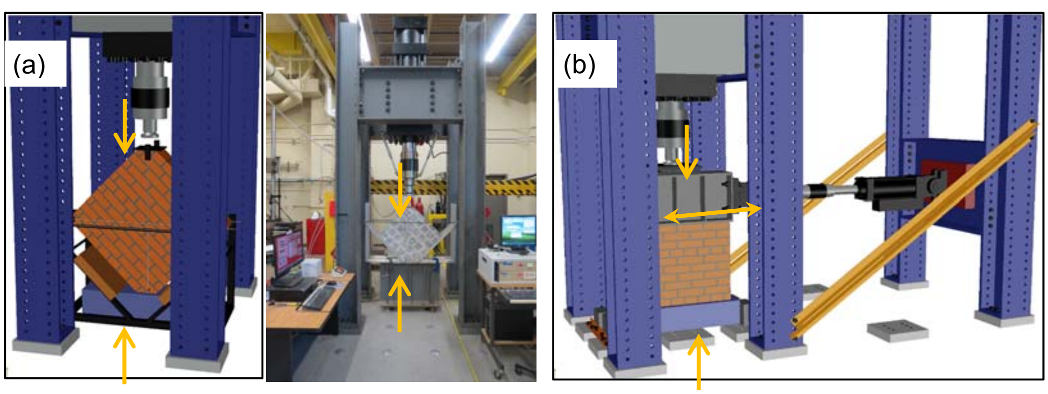

Figure 4: Test setup: (a) Diagonal tension, (b) Static-cyclic.

Panels were constructed in a horizontal position and then rotated 45 degrees for installation in the test setup as shown in Figure 4a. To induce a tension/shear failure mechanism, a compressive load is applied progressively on one diagonal of the panel, resulting in a tension stress on the other diagonal. The vertical load was applied in displacement control by a 1500 kN “Material Testing System” (MTS). Two LVDTs were positioned on both sides of the panels to measure the deformation according to each diagonal (strain shortening of the vertical diagonal and the extension of the horizontal diagonal), and thereby evaluate the masonry panel shear modulus G_m.

Static-cyclic lateral loading tests were carried out with two MTS actuators (1500 kN and 200 kN) on three two-leaf brick wallets and two multi-leaf stone wallets. Brick and stone wallets dimensions were, respectively (861 mm x 660 mm x 204 mm) and (890 mm x 940 mm x 270 mm). The objective of this test is to identify the in-plane lateral strength, the failure modes and the hysteresis behaviour of the URM wallets under different levels of vertical stress. For the three brick wallets, applied vertical stress was 0.4 MPa, 0.8 MPa and 1.7 MPa (i.e 7.5%, 15% and 42.3% of mortar compressive strength), and for the two stone wallets it was 0.2 MPa and 0.5 MPa (i.e. 3.8% and 9.4% of mortar compressive strength). The lowest level of vertical stress corresponds approximately to the weight of upper floors on a ground floor wall of a 2 to 3-storey URM residential building. The lateral cyclic load was applied on the top of the wallets by a horizontal actuator (200 kN). Loading protocol was conducted under displacement control and defined according to the method B of ASTM E2126 and Petry (2015). LVDTs measured the top drift of the wallet to obtain the force-displacement relation.

3 RESULTS FOR BRICK MASONRY

3.1 Mechanical characteristics of masonry and its components

Table 2 gives the results for the mechanical parameters obtained from tests on units, mortar and brick masonry assemblies as defined in Figure 2. Average compressive strength of mortar is representative for type O mortar. Average compressive strength of brick agrees with the value specified for 52-DD Glen-Gery brick as 27.5 MPa and the Young modulus was determined by the method of least squares between $0.15f'_b$ and $0.70f'_b$. In both cases the standard deviation is relatively important which is expected for such material.

Table 2: URM brick mechanical parameters results from Phase I and II

Mechanical parameter	URM type	Number of specimens	Average Value ± SD
Compressive strength f'_j	Mortar Type O	113 cubes	5.35 ± 1.04 MPa
Compressive strength f'_b	Half-brick	28	26.3 ± 4.8 MPa
Young modulus E_b	Half-brick	28	3.63 ± 1.1 GPa
Compressive strength f'_m	Five stacked bricks	27	14.8 ± 2.1MPa
Young modulus E_m	Five stacked bricks	28	3.21 ± 0.83 MPa
Cohesion c	Three stacked bricks	81	0.29 MPa
Coefficient of friction μ	Three stacked bricks	81	0.94
Diagonal traction f'_{td}	Two-leaf brick panels	6	0.79 ± 0.03 MPa
Diagonal traction G_m	Two-leaf brick panels	6	1.98 ± 0.29 GPa

The masonry compressive strength, f'_m can be related to the brick and mortar compressive strengths, f'_b and f'_j, by Eq. (1) proposed by Lumantarna and al. (2014).

[1] $f'_m = Kf'^{\theta}_b f'^{\lambda}_j$

Figure 5 shows three curves drawn from Eq. 1 for the median, lower and upper values of the brick compressive strength f'_b according to the standard deviation given from the tests on brick samples. Values for parameters in Eq. 1 were taken from Eurocode 8 (CEN 2005), that is: $K = 0.75$; $\theta = 0.75$; $\lambda = 0.31$. Most results are included between the upper and the lower limits of the model, and 68.2% of the bricks have a compressive stress between 21.5 MPa and 31.1 MPa as in a normal distribution.

Joint shear bond tests were carried out for three levels of compressive stresses, 0.2 MPa, 0.6 MPa and 1.0 MPa, by applying a load perpendicularly to the joints of the specimen. Results were used to obtain the cohesion c and the coefficient of friction μ from the Mohr-Coulomb envelope. Cohesion and coefficient of friction are within the range of values reported in the literature for brick masonry with similar mortar strength (Lumantarna et al. 2014, Singhal and Rai 2013).

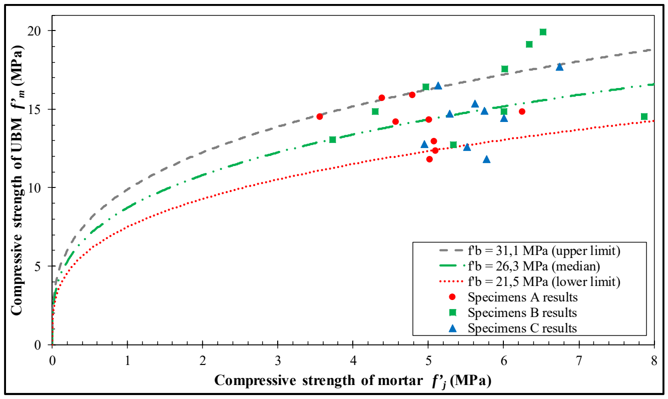

Figure 5: Relation between the masonry compressive strength f'_m, and the mortar compressive strength f'_j, for median, lower and upper values of the brick compressive strength f'_b.

Diagonal tensile strength f'_{td} is calculated according to the assumption of pure shear state of stress in the centre of the specimen (ASTM 2015). The shear modulus G_m of the panels is defined as the slope on the linear part of the shear stress-strain curve using the method of least squares between 0.05 τ_{max} and 0.70 τ_{max}. The respective coefficients of variation for f'_{td} and G_m are 4.1% and 15%, which is within the expected values considering the heterogeneity of material studied. Size of the panels does not seem to influence the results. The failure pattern of all six panels was characterized by stair-stepped cracking along the mortar joint, with some cracks going through the bricks (see Figure 7a).

3.2 Cyclic Behaviour of Masonry Wallets

Despite the three different compression stress conditions, all three URM brick wallets failed in a rocking failure mode characterized by the occurrence of a horizontal crack along the lower mortar joint due to a tension failure between mortar and brick. Hysteresis curves for each wallet are shown in Figure 6a. They exhibit typical characteristics of a rocking failure: narrow loops typical of fragile behaviour with low energy dissipation by hysteresis, and large displacements without a loss of lateral strength (Magenes and Calvi 1997). The resulting in-plane lateral force-displacement relations are obtained from the backbone curve of the hysteresis idealized by a bilinear curve (Figure 6b) defined by the idealized lateral strength V_e, the idealized drift δ_e and the equivalent stiffness K_{eq}. Results for each wallet are given in Table 3. As expected, idealized strength and equivalent stiffness increase with the vertical load ***P*** applied on the wallet. Idealized yield drifts (at the limit of elastic behaviour) are similar for all wallets and vary between 0.103% and 0.127%. The closest values for the predicted lateral resistance are given by Eurocode 8 (CEN 2005) with 29.4 kN, 58.7 kN and 125.8 kN but for a shear failure through the bed joint. Equations from ASCE-41 (ASCE/SEI 2013), Magenes and Calvi (1997) and NZSEE (2009) all predict flexural failures (rocking or toe-crushing failures) but overestimate the lateral resistance by 15% to 38%.

Table 3: Idealized strength, drift and stiffness for URM brick wallets

Wallets	σ_v ***(MPa)***	V_e **(kN)**	δ_e **(%)**	K_{eq} **(kN/mm)**
C-W1	0.4	33.9	0.103	46.9
C-W2	0.8	60.6	0.109	79.3
C-W3	1.7	110.4	0.127	123.9

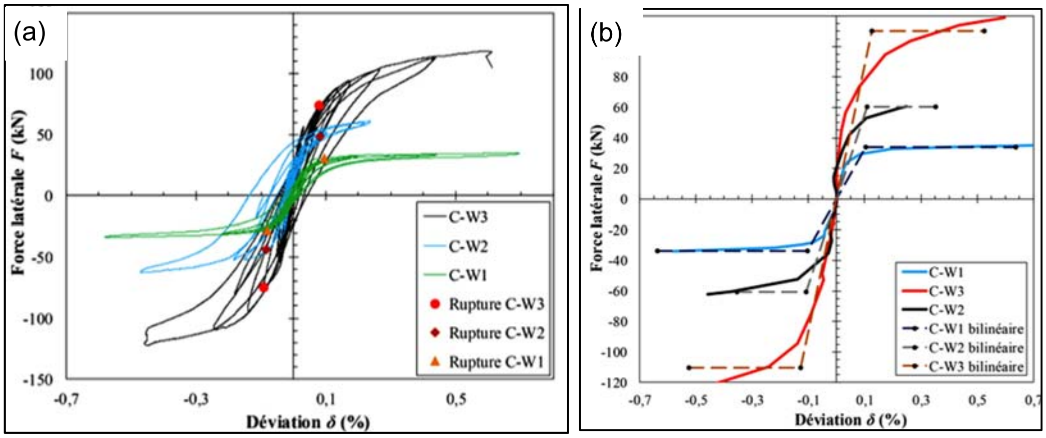

Figure 6: Cyclic behaviour of URM brick wallets: (a) hysteretic curves, and (b) backbone and bilinear models.

4 RESULTS FOR STONE MASONRY

4.1 Mechanical Characteristics of Masonry and its Components

Table 4 gives the results for the mechanical parameters obtained from tests on units, mortar and stone masonry assemblies as defined in Figure 2. Average compressive strength of mortar at 28 days is lower than the average value of 7.6 ±2.9MPa for mortar used in restoration projects, but still representative (Moretti 2016). Average compressive strength of stones is lower for guillotine blocks than for sawn cubes. Both stones are from the same career but were extracted at different time, which illustrates the large variation seen for limestone compressive strength reported in literature (12.3 to 193 MPa) (Beall 2004).

Table 4: URM stone mechanical parameters results from Phase I and II

Mechanical parameter	Number of specimens	Sawn cubes	Guillotine blocks
Compressive strength f'_j	5	3.30 ± 0.20 MPa	5.34 ± 0.13 MPa
Compressive strength f'_b	3	100.6 ± 13.8 MPa	77.6 ± 7.6 MPa
Compressive strength f'_m	6	33.2 ± 3.2 MPa	52.7 ± 6.6 MPa
Young modulus E_m	6	2 823 ± 186 MPa	n/d
Cohesion c	12	0.56 MPa	0.32 MPa
Coefficient of friction μ	12	0.85	0.78
Diagonal traction f'_{td}	4 and 2	0.41 ± 0.08 MPa	0.31 ± 0.09 MPa
Diagonal traction G_m	4 and 2	487 ± 10 MPa	n/d

The stone masonry compressive strength, f'_m. can be related to the stone units and mortar compressive strengths, f'_b and f'_j, by Eq. (1) using the following values for the coefficients as recommended by Eurocode 8 (CEN 2005): $K = 0.45$; $\theta = 0.70$; $\lambda = 0.30$. The computed compressive strength of sawn cubes assemblies is 16.24 MPa compared to the experimental value of 33.23 MPa. For the guillotine blocks assemblies, these values are 9.46 MPa and 52.7 MPa, respectively.

In general shear bond tests results are within the range of values for cohesion (0.33 MPa-0.58 MPa) and coefficient of friction (0.58 – 0.85) reported in the literature for stone masonry with cement-lime or lime mortar (Binda et al. 1994, Vasconcelos and Lourenço 2009). The difference in cohesion for sawn cubes guillotine block assembly is possibly due to the better quality of the surface finish for the sawn stones.

Diagonal tension tests were completed successfully for sawn cubes panels with a typical tension rupture on the diagonal as shown in Figure 7a. However, panels made with stone blocks cut by guillotine rupture along a horizontal joint as shown in Figure 7b. This is mainly due to the size of the unit blocks relative to the size of the panels (limited by the testing installation). Computed diagonal tension strength for the guillotine stone blocks is therefore a lower bound value that did not even reach the theoretical 10% of the mortar strength (0.53 MPa). This illustrates the complexity of measuring diagonal tension strength for multi-leaf masonry made of irregular large units. Diagonal tensile strength f'_{td} for the sawn cube panels is calculated according to the assumption of pure shear in the centre of the specimen (ASTM 2015). It is slightly higher than 10% of the mortar strength (0.33 MPa) and in the upper range of experimental values (0.06 and 0.37 MPa) as reported by Mazzon (2010). The shear modulus G_m is defined as the slope on the linear part of the shear stress-strain curve at 33% of f'_{td}. Shear modulus of 487 MPs is about 17 % of Young modulus E_m (2 823 MPa) and within the range of experimental values (79 and 837 MPa) (Mazzon 2010).

Figure 7: Diagonal tension failure of panels: (a) Brick panel A, (b) Sawn stone cubes, and (c) Guillotine stone blocks

4.2 Cyclic Behaviour of Masonry Wallets

Two multi-leaf URM stone wallets made of blocks cut by guillotine were tested under static-cyclic loading under two different compression stress conditions. Both wallets failed in a rocking failure mode characterized by the occurrence of a horizontal crack along the lower mortar joint due to a tension failure between mortar and brick (see Figure 8). Hysteresis curves for each wallet are shown in Figure 9a. They exhibit typical characteristics of a rocking failure: narrow pinched loops typical of fragile behaviour and low energy dissipation by deformation, and large displacements without a loss of lateral strength. For Wallet 1 tested under a vertical stress of 0.2 MPa, the idealized lateral strength V_e and drift δ_e are 19.3 kN and 0.075%, respectively, and the equivalent stiffness K_{eq} is 29 kN/mm. For Wallet 2 tested under vertical stress of 0.5 MPa the corresponding values are 43.3 kN, 0.084% and 61 kN/mm, respectively. Idealized yield (elastic limit) drifts are similar for both wallets for an average of 0.08% in the lower range (0.06% to 0.13%) of reported values for first damage state to URM stone walls (Abo El Ezz 2013). Equation from ASCE-41 predicted a rocking failure with a lateral resistance of 21.7 kN and 51.4 kN, overestimating the experimental resistance by 12% and 19%.

Figure 8: Rocking failure rupture of stone wallets

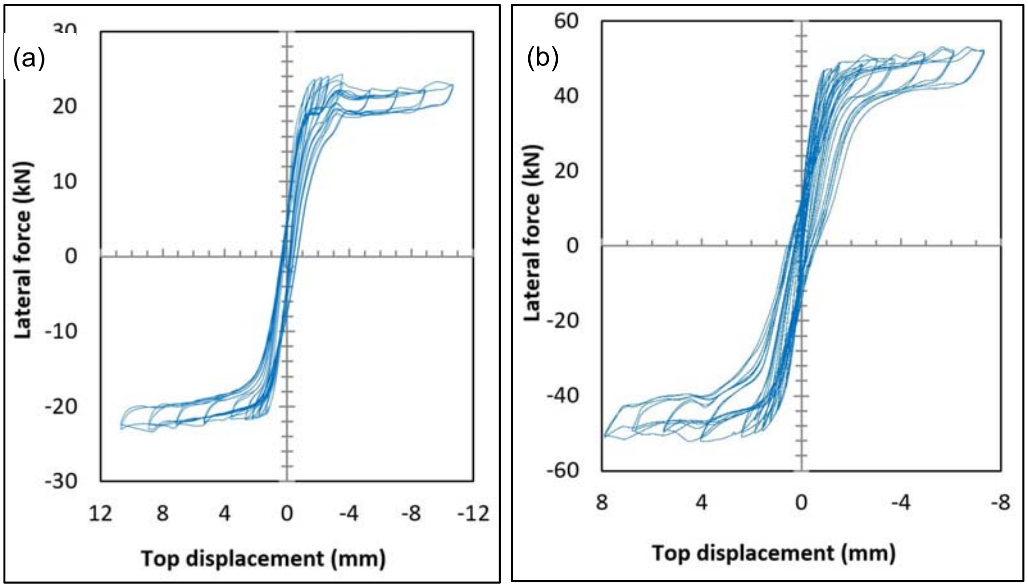

Figure 9: Cyclic hysteretic behaviour of URM stone block wallets: (a) Wallet 1 (σ_v=0.2 MPa), and (b) Wallet 2 (σ_v=0.5 MPa).

5 CONCLUSION

An experimental programme was developed to characterize the mechanical properties and parameters of unreinforced brick and stone masonry traditionally used in old existing buildings in Eastern Canada. The obtained results are used to validate equation recommended by Eurocode to predict the compressive strength of the masonry assemblies from the compressive strength of the units and mortar. For the brick and stone masonry, the difference between the computed strength and the experimental values is significant indicating that more experimental data with local material and a larger range of mortar strength is required to adapt the equation. Shear bond tests parameters are coherent with the values reported in the literature, with a cohesion c = 0.29 MPa and a coefficient of friction μ = 0.94 for brick masonry and, for stone masonry, average values of c = 0.44 MPa and μ = 0.82. Diagonal tension and shear modulus are within experimental values reported in the literature. Lastly, the failure modes and hysteresis behaviour of all brick and stone wallets tested under in-plane lateral cyclic loading were characteristic of rocking failure. Prediction equations proposed in standards could in general predict the flexural failure mode, but tend to overestimate the lateral resistance of brick wallets by up to 38% and by 20% for stone wallets. Yield drifts values for brick and stone wallets are in the same range of values. This reinforces the use of a deformation drift threshold (instead of a force threshold) for damage evaluation and risk studies as they can be used as threshold displacement for the evaluation initiation of first cracking and the development of simplified bilinear lateral force-deformation curves for in-plane response. The results obtained from this study are particularly useful for better evaluation of the seismic vulnerability of existing UBM buildings in Eastern Canada and for efficient selection of appropriate rehabilitations and strengthening strategies.

Acknowledgements

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