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IN SITU AND LABORATORY TESTING OF THE CANADIAN PARLIAMENT BUILDINGS' HISTORIC MASONRY

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ABSTRACT

Centre Block, Canada's heritage designated federal parliament building, was constructed in 1916 after fire destroyed an earlier building of the same name that occupied the site. It was reconstructed using techniques considered state-of-the art at the time. This included hybrid walls consisting of an exterior sandstone wythe built integrally with clay brick backing and using a cement-based mortar.

Currently, Centre Block is undergoing a major rehabilitation, including a seismic upgrade. As a precursor to the upgrade design, a testing program was completed to investigate the structural properties of Centre Block's unique masonry walls. Both in situ testing following ASTM C1197 (masonry deformability) and ASTM C1531 (joint shear strength) protocols and their equivalent destructive laboratory tests of extracted prisms were conducted.

This paper reviews and provides commentary on the test procedures and discusses the advantages and limitations of each. The results of the in situ and destructive laboratory tests are presented and compared. It was found that the in situ ASTM C1197 test protocol provided a reliable means of estimating masonry deformability and was significantly easier to perform than the equivalent destructive laboratory test on extracted prisms. The opposite was observed for the ASTM C1531 tests where more consistent results were obtained with the equivalent laboratory test.

KEYWORDS: *Centre Block (Canadian Parliament Buildings), clay-brick masonry, in situ testing, masonry prism, Nepean sandstone, seismic material design properties*

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INTRODUCTION

The current rehabilitation and seismic upgrade of Centre Block, Canada's heritage designated parliament building, requires structural material properties for analytical modelling and upgrade design. These properties cannot be readily estimated because of Centre Block's unique construction. Its walls consist of a locally quarried stone (Nepean sandstone), built integrally with clay brick backing and a 1:3 Cement : Sand mortar. A testing program was undertaken to measure compressive strength, compressive elastic modulus and shear strength and behaviour (coefficient of friction and cohesion). In situ compression and shear tests and similar laboratory tests on prisms extracted from the building were performed. This paper reports the results and provides commentary on the advantages and challenges of each pair of experimental methods.

COMPRESSIVE STRENGTH AND STIFFNESS

In Situ Modulus of Elasticity Test Using Flatjacks per ASTM C1197

In situ tests to determine the modulus of elasticity, E_m, of Centre Block's masonry using 200 mm x 600 mm flatjacks were performed at five locations in Nepean sandstone and at six locations in clay brick following the requirements of ASTM C1197-14a [1]. Two flatjacks were inserted into slots made in masonry bed joints and pressurized, compressing the masonry between them. At discrete intervals, the flatjack pressure and corresponding displacements were recorded. Figure 1 depicts stone and clay brick masonry test locations. The difference between subsequent and the initial displacement measurement divided by the gauge length was used to calculate the imposed strain, ε. The flatjack pressure, p, was converted to masonry stress, σ, using a flatjack calibration factor, K_m, and the flatjack/slot area ratio, K_a, per Equation 1.

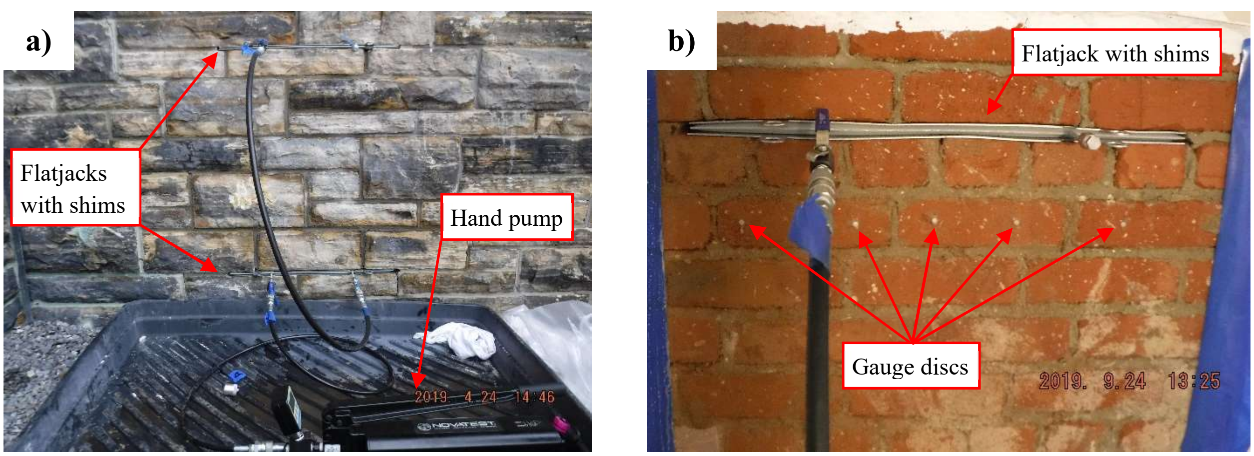

Figure 1: Typical Test Set-up for ASTM C1197 in a) Nepean sandstone masonry, and b) Clay brick masonry

$$\sigma = K_m \cdot K_a \cdot p \tag{1}$$

ASTM C1197 stipulates that tangent and secant moduli be calculated at each interval but does not indicate how these values are to be combined to report a single E_m value for the test location. The approach taken here was to perform a least-squares linear regression on the straight portion of the σ–ε data, excluding any data that did not conform to the linear trend of the middle part of the data. The slope of the regression was taken as E_m. Initial soft behaviour was interpreted as the closing-up of micro-cracks in the masonry during pressurization and seating of the flatjacks. Flattening-off of σ–ε plots at the end of the compressive stress range was possibly due to the onset of localized mortar crushing. Figure 2 depicts the σ–ε behaviour of all tests in Nepean sandstone and clay brick and the average linear regression straight line for each material type. The average moduli of elasticity determined by in situ flatjack testing are reported in Table 1. The measured variation appears to be reasonable, given the inherent variability in masonry.

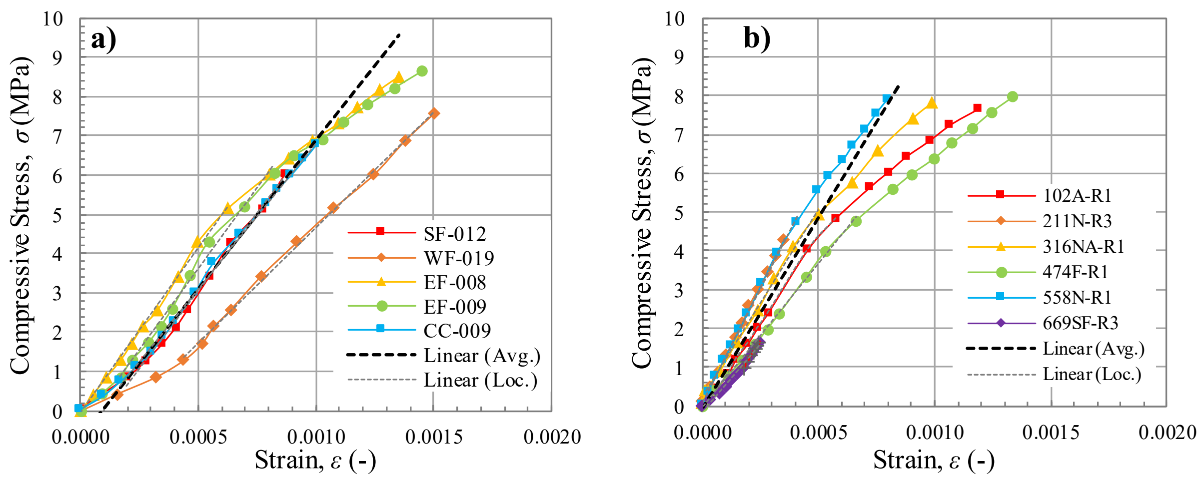

Figure 2: σ–ε Plots for Flatjack Tests in a) Nepean sandstone and b) Clay brick masonry

Table 1: Modulus of Elasticity, E_m, Results

	Nepean Sandstone	**Clay Brick**
Average E_m [GPa]	7.59	9.88
Std. Dev. [GPa]	1.03	1.76
COV[a] (%)	13.5%	17.8%
Average R^2 [-]	0.9962	0.9984
# of tests with result/ # of attempted tests	5/5 (100%)	6/6 (100%)

[a] Coefficient of Variation (Standard deviation / Average)

ASTM C1197 indicates that “the in situ deformability test (modulus of elasticity) typically over-estimates the average compressive modulus of the masonry by up to 15%” [1]. This is in part due to this test treating the masonry between the flatjacks as a stand-alone prism when in fact the masonry test zone is connected at its sides and at the back to the surrounding masonry. The exceptionally high strength and stiffness of Centre Block’s cement-based mortar may exacerbate this effect, resulting in an even greater overestimation of E_m. However, this may have been partially mitigated by the larger than typical width of the flatjacks. The equipment used for this test only allowed for discrete measurement of stress and strain compared to set-ups with continuous data recording. This did not prove to be problematic since the data was smooth, allowing interpolation of the behaviour between recorded data points. Overall, this test method proved to be a relatively problem-free and simple method to determine masonry stiffness, requiring less effort than extraction and laboratory testing of prisms.

Compressive Strength and Stiffness of Masonry Prisms per ASTM C1314

Tests to determine the compressive strength, σ_{ult}, and elastic modulus, E_m, were performed following ASTM C1314-16 [2], adapted for convenience and typical practice of the testing agency. Tests were performed on 13 three-unit Nepean sandstone prisms and 25 four-course tall clay brick prisms which were extracted from well-distributed sampling locations throughout the building. The prisms were instrumented either with pairs of linear variable displacement transducers (LVDTs) held by magnetic stands resting on the lower load platen and facing in opposite directions, or with compact LVDTs affixed to the face of the prism (Figure 3). Displacement was measured at one or two locations on each of the front and back faces.

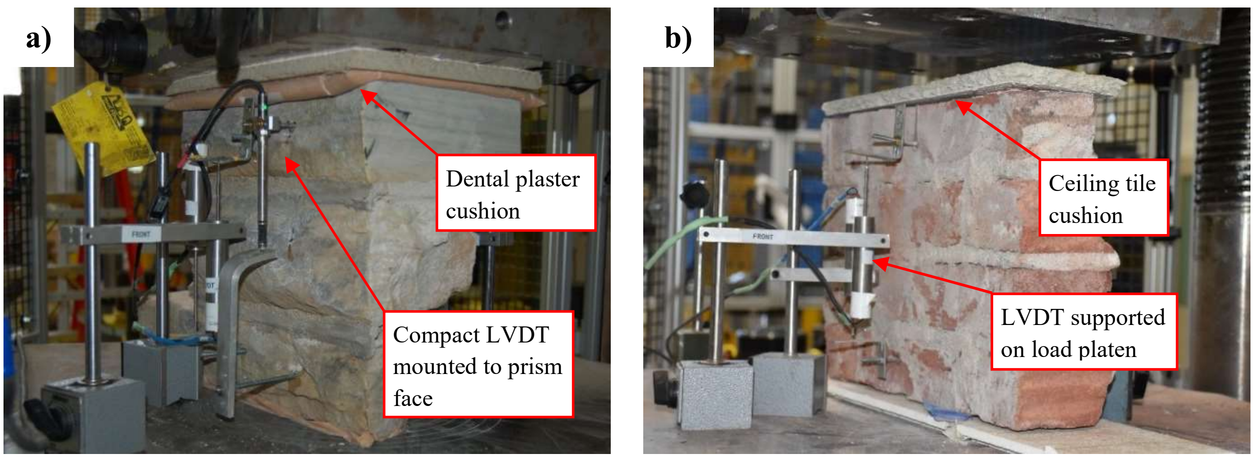

Figure 3: Prism Compression Test Apparatus a) Nepean sandstone prism WC-002-NP-001 b) Clay brick prism 458N-001

σ_{ult} was determined by dividing the maximum load by the least cross-sectional area of the prism, estimated by making several measurements prior to the test. E_m was determined using one of three methods: the slope between the points on the stress-strain (σ–ε) curve corresponding to a) 5% and 33% of σ_{ult} (per ASTM C1314), b) 5% and 70% of σ_{ult} (per [3]) and c) the slope from a least-squares linear regression performed on linear data between selected low and high percentages of σ_{ult}. This last approach was found to be preferable since fixed low and high percentages of σ_{ult}, although consistent and simple, do not always exclude erratic behaviour often present at the start and end of loading. The quality of data was examined and results for many sensors rejected on account of the plots showing tensile strains due to flexural rotation of the prisms or localized movements at the sensor locations. Figure 4 depicts σ–ε relationships of two prisms, one with good quality data (approximately linear behaviour except at the beginning and end of loading) and one with rejected data. The average value of E_m from all strain measurements for a prism was reported as E_m for the prism. The average value of E_m for all prisms of each material type, along with other pertinent results, are provided in Tables 2 and 3 for stone and brick, respectively.

This laboratory test was expected to provide reliable results of E_m that are more accurate than the equivalent in situ test because the problems of test zone interaction with surrounding masonry in ASTM C1197 are eliminated. However, the results are highly variable with large coefficients of variation (31 to 51%, depending on material and analysis method). Furthermore, a significant percentage of the data was rejected. Data from 41% and 64% of the sensors on stone and brick prisms, respectively, were excluded, resulting in high overall rates of specimens without usable data and casting doubt on the overall quality of results. The success rate of extracting intact, three-stone prisms was also very poor, at 46%. While the masonry compressive strength results, particularly for brick, may appear less problematic/less variable, it is noteworthy that the average σ_{ult} for brick, 10.3 MPa, is on the same order of magnitude as the maximum stresses imparted by the flatjacks in four of the six in situ tests, during which no signs of distress were observed.

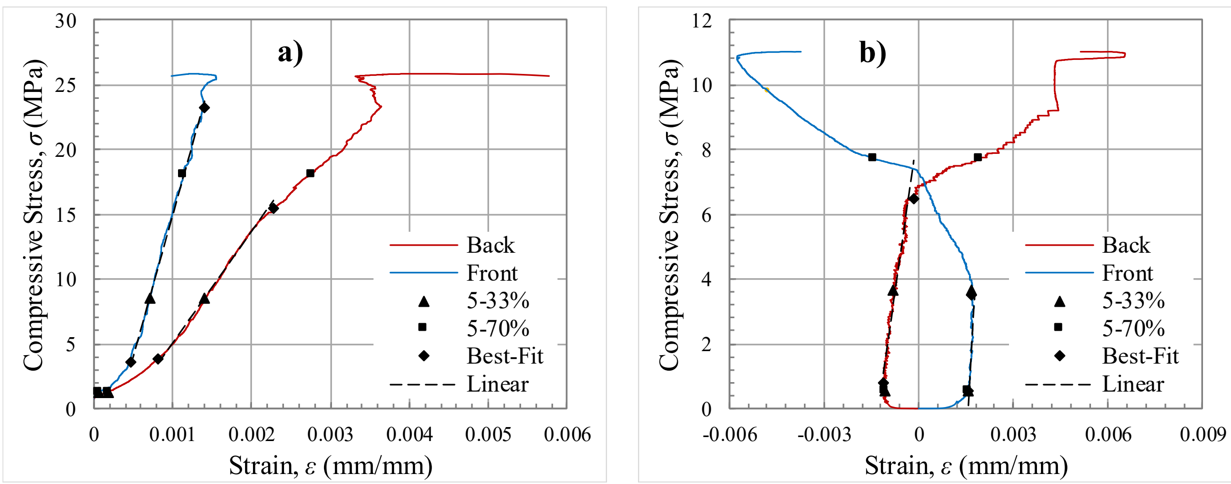

Figure 4: Stress-Strain Plots (Compressive Stress and Strain Plotted on Positive Axes) for a) Nepean sandstone prism WF-001-NP-001 and b) Clay brick prism 648D

Table 2: Compressive Strength and Elastic Modulus of Nepean Sandstone Prisms

	Average	**Std. Dev.**	**COV (%)**	**# of tests with result/ # of attempted tests**
σ_{ult} [MPa]	25.6	9.38	36.6%	13/13 (100%)
E_m (5–33% of σ_{ult}) [GPa]	9.13	4.70	51.4%	11/13 (85%)
E_m (5–70% of σ_{ult}) [GPa]	10.6	3.75	35.3%	11/13 (85%)
E_m (Best-Fit) [GPa][a]	13.3	4.07	30.6%	11/13 (85%)
ε at σ_{ult} [-]	0.00338	0.00231	68.2%	8/13 (62%)

[a] Avg. low % of σ_{ult}: 12.2%; Avg. high % of σ_{ult}: 65.3%; Avg. R^2: 0.9830 [-]

Table 3: Compressive Strength and Elastic Modulus of Clay Brick Prisms

	Average	Std. Dev.	COV (%)	# of tests with result/ # of attempted tests
σ_{ult} [MPa]	10.3	2.72	26.3%	25/25 (100%)
E_m (5–33% of σ_{ult}) [GPa]	5.51	2.23	40.5%	15/25 (60%)
E_m (5–70% of σ_{ult}) [GPa]	4.95	2.54	51.2%	15/25 (60%)
E_m (Best-Fit) [GPa][a]	5.81	2.55	43.8%	15/25 (60%)
ε at σ_{ult} [-]	0.00323	0.00199	61.6%	12/25 (48%)

[a] Avg. low % of σ_{ult}: 7.4%; Avg. high % of σ_{ult}: 64.6%; Avg. R^2: 0.9904 [-]

SHEAR STRENGTH AND SLIDING SHEAR BEHAVIOUR

In Situ Shear Strength per ASTM C1531

In situ shear tests following ASTM C1531-16 [4] were conducted at the same locations as the ASTM C1197 tests described in the previous section (five stone, six brick tests). The objective was to determine the relationship between shear stress, τ, and normal stress, σ, of Centre Block's masonry walls. This relationship is given by Equation 2 where μ is the coefficient of friction and c is the cohesion:

$$\tau = \mu \cdot \sigma + c \tag{2}$$

A single stone or brick (the test unit), centred between the flatjacks, was isolated by removing masonry on either side. While subject to a normal compressive stress, σ, imposed by the flatjacks, a horizontal force was applied by a calibrated hydraulic ram to one end of the test unit. Horizontal displacement, d_h, of the test unit was measured with a dial gauge and pairs of force-displacement data recorded at discrete intervals. The horizontal force was gradually increased until sliding occurred. After releasing the horizontal force, σ was increased to the next pre-determined level and the test repeated. Five to seven runs were thus performed with increasing normal stress at each location, the instrumentation reversed and as many repetitions completed in the opposite direction. Typical test configurations are depicted in Figure 5 for stone and clay brick test locations.

The normal stress imparted to the masonry by the flatjacks given in Equation 1 must be modified to account for the removed masonry. The appendix to ASTM C1531 recommends the use of a factor, j, which, for the particular case where the angle between the end of the flatjack and the corner of the test unit is 45°, is given as 1.7. With the varying geometry at the Centre Block test locations, the approach adopted here was to interpolate between j of 1.7 at 45° and j of 1.0 for 90°, the case described in Equation 1. The revised expression is given in Equation 3.

$$\sigma = K_m \cdot K_a \cdot j \cdot p \tag{3}$$

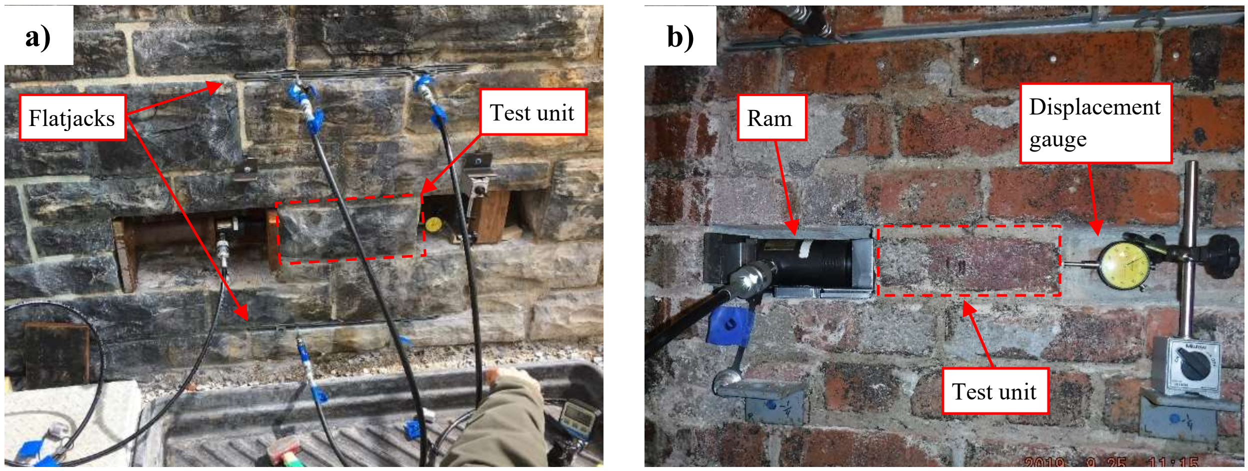

Figure 5: Typical Test Set-up for ASTM C1531 in a) Exterior stone masonry, and b) Interior brick masonry

Shear stress, τ, was obtained by dividing the horizontal force by the area of both top and bottom bed joints of the test unit. The τ–d_h plots generally exhibited initial linear, positive-slope behaviour, followed by a crisp transition to sliding behaviour (horizontal displacement with nearly constant load), except for the initial run at each test location where the transition to the sliding mode was gradual. To determine the relationship between τ and σ, it is necessary to select a value of τ to represent the onset of sliding, $\tau_{sliding}$, from the τ–d_h data set from each run. For plots which depicted obvious bi-linear behaviour, $\tau_{sliding}$ was taken as the intersection of straight lines fit to the bi-linear response. Where bi-linear behaviour was not as well defined, a data point was selected at the onset of near-zero stiffness. Figure 6a) depicts the τ–d_h plots for the first test direction at a clay brick test location, and Figure 6b) illustrates the selection of $\tau_{sliding}$.

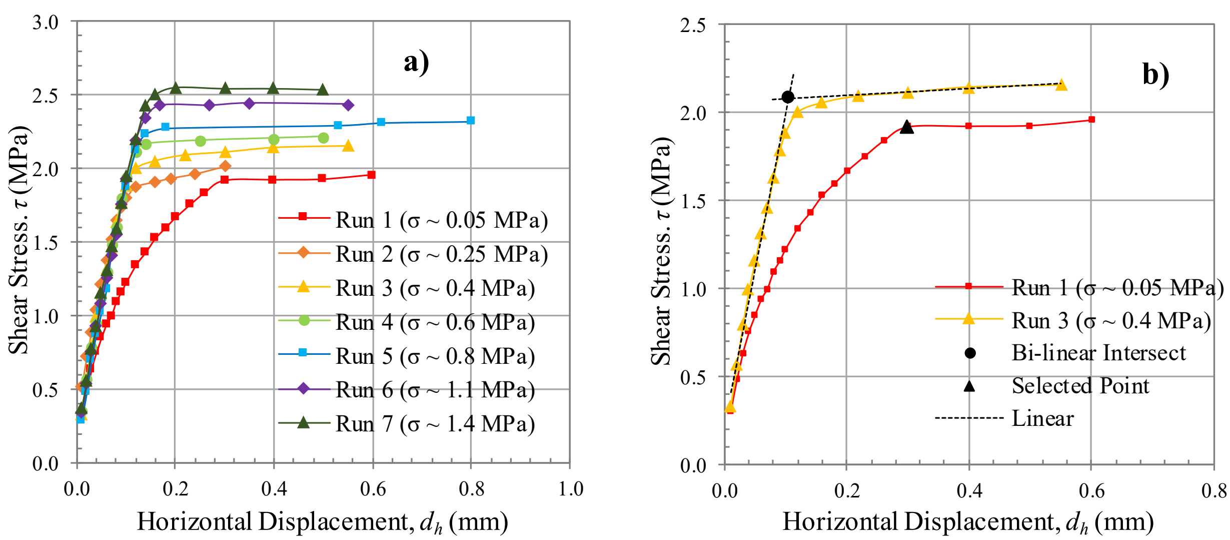

Figure 6: a) τ–d_h Plots for Test Location 474F-R1 b) Selection of τ to Represent Onset of Sliding Behaviour

The τ–σ relationship was determined for each test direction by performing a least-squares linear regression on the data excluding the first run from the first test direction because the bond was still intact. The slope of this line is μ. The cohesion, however, cannot be directly calculated since it is one of several factors that resist sliding, including frictional resistance of the bed joints and collar joint and mechanical interlock at the back of the test unit (particularly for irregularly shaped stones). Cohesion also acts on both bed joints as well as the collar joint. The bed joint cohesion cannot be separated from these other parameters. Of great importance, however, is that only the frictional resistance of the bed joints is dependent on σ: the other parameters described are assumed to be independent of σ and did in fact appear to be constant in each run, evidenced by the linear behaviour of the τ–σ relationship. Plots of this relationship are provided in Figure 7. Variability between different directions at the same test location is evident and could be explained by the degradation of the mortar interface as testing proceeds or by the shape of the failure surface. The average values from both test directions are reported in Table 4. Results are not reported for some test locations/test directions because the attempt did not result in sliding, the test unit could not be displaced, or the results were erroneous. The measured values agree reasonably well with the CSA S304-prescribed design value of 1.0 for masonry-to-masonry sliding planes [5].

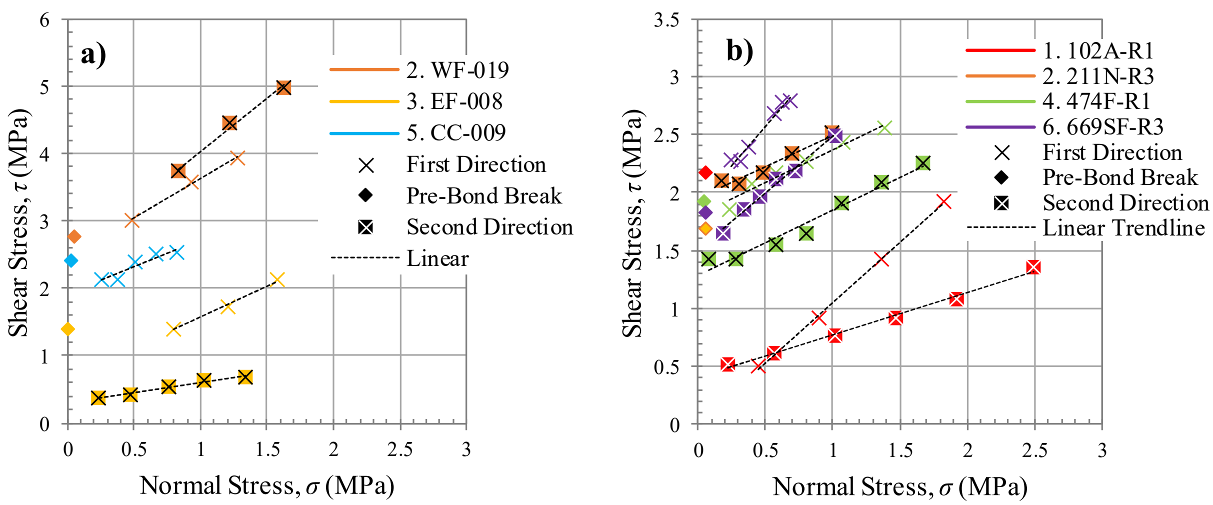

Figure 7: Relationship Between τ and σ for a) Nepean sandstone and b) Clay brick

Table 4: Coefficient of Friction, μ, Results

	Nepean Sandstone	Clay Brick
μ [-]	0.93	0.74
Std. Dev. [-]	0.39	0.28
COV (%)	41.8%	38.1%
Average R^2 [-]	0.9580	0.9734
# of tests with result/ # of attempted tests	5/8[a] (63%)	7/11[b] (64%)

[a] At two of the five stone test locations, tests could only be attempted from one direction.

[b] At one of the six brick test locations, joint did not fail from first test direction and was not attempted from other direction.

This in situ test is a simple, minimally intrusive means of estimating μ. However, a cohesion value cannot be calculated due to the unknown magnitude of the interaction of the test unit with the backing masonry and must be determined by other means for analysis and design purposes. The proportion of normal force applied to the test unit versus dissipated into the surrounding masonry is also not precisely known, resulting in some uncertainty in the results.

Direct Shear Testing of Two-Unit Prisms per ASTM D5607

The coefficient of friction and cohesion strength of the mortar joint of six two-unit Nepean sandstone prisms were determined following an adaptation of ASTM D5607-16 [6]. The test method is similar to ASTM C1531 but the normal force was applied directly to the top unit of the prism while the bottom unit was clamped in place. Five or six runs were performed on each specimen in a single direction. Figure 8 depicts the test apparatus. Both stone units were instrumented with two horizontal LVDTs, one at each side of the prism. The average displacement of the top LVDTs less the average of the bottom LVDTs provides the net horizontal displacement, d_h. Shear stress, τ, was determined by dividing the shear force by the area of the bed joint.

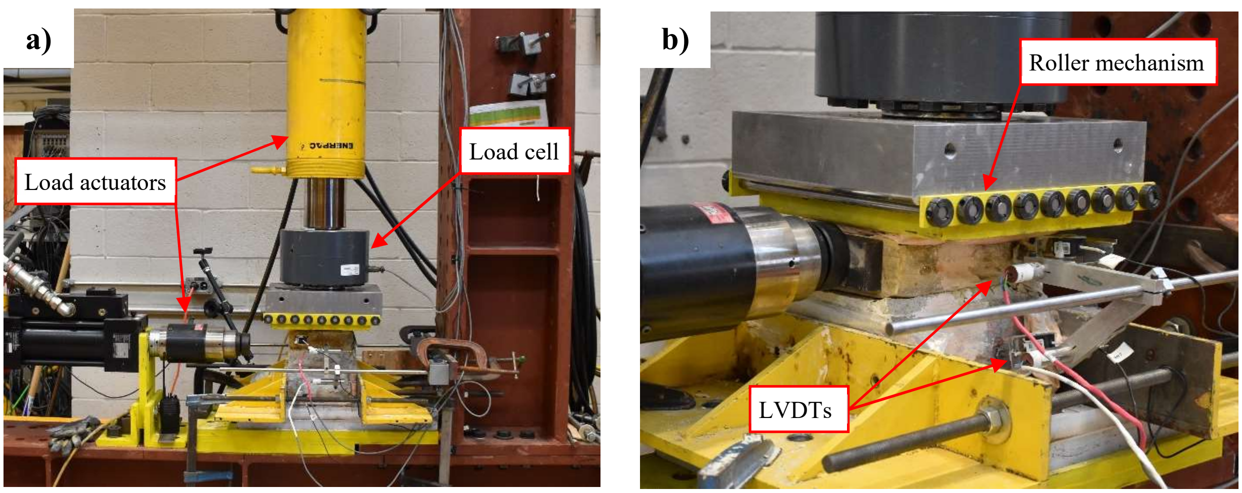

Figure 8: Direct Shear Test Apparatus a) Overall view b) Close-up view

Figure 9 depicts $\tau - d_h$ behaviour for one of the prisms. Run 1 typically had a high peak shear stress, τ_{peak}, followed by sliding (approximately horizontal behaviour). Subsequent runs did not exhibit a strong peak. The peak shear stress is interpreted as the stress required to overcome static friction as well as, in the case of the run 1, cohesion. Sliding resistance was taken as the average value of τ between two selected points during the horizontal/sliding behaviour.

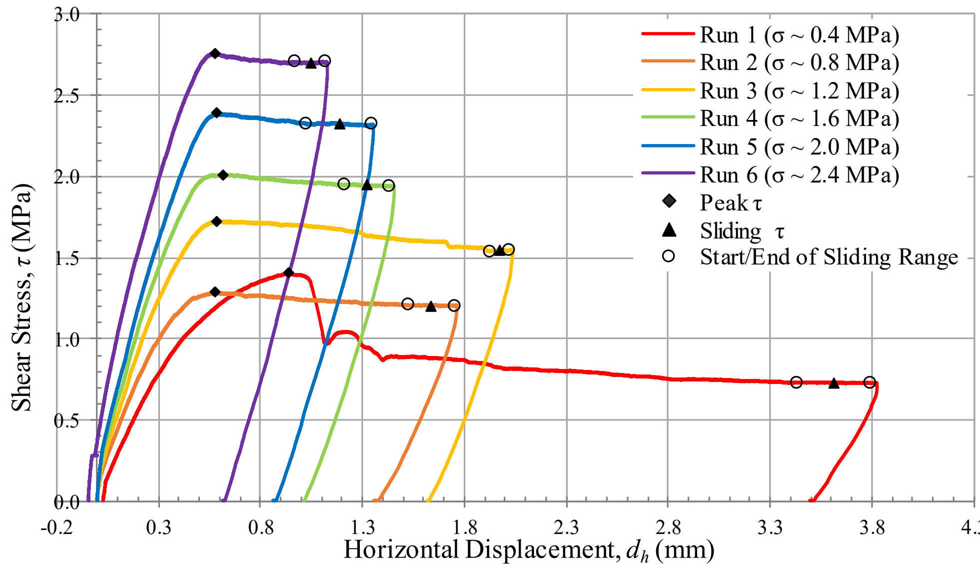

Figure 9: Shear Stress – Displacement Behaviour for Prism NF-006-NP-001

Plots of τ and σ (normal stress) for both peak shear (τ_{peak}) and sliding shear ($\tau_{sliding}$) are expected to be linear as described by Equation 2. Since distinct values of τ_{peak} and $\tau_{sliding}$ are obtained from the data, separate and corresponding values of μ_{static} and $\mu_{sliding}$ can also be determined. Figure 10 depicts these relationships for the six prisms tested, along with least-squares linear regression trendlines on the τ–σ data; there was little difference between μ_{static} and $\mu_{sliding}$.

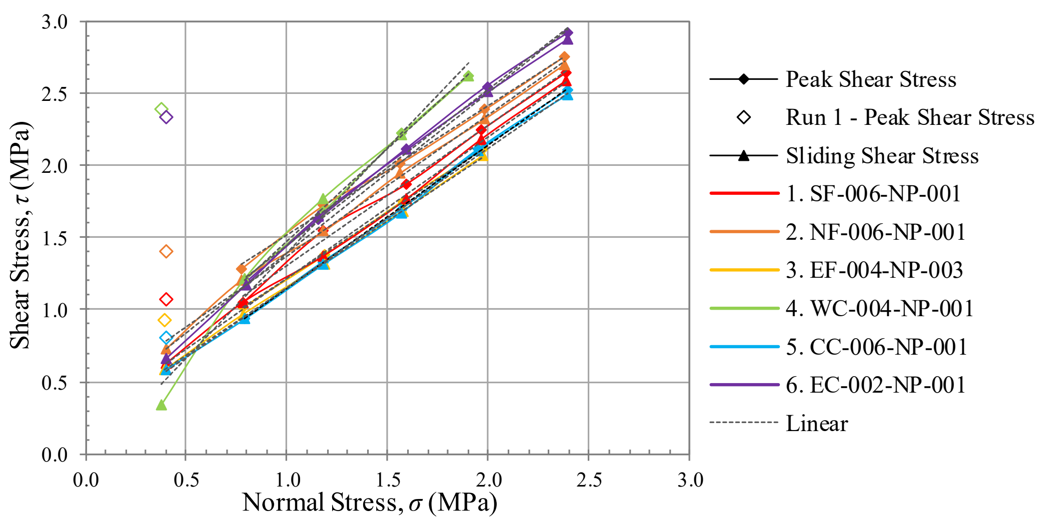

Figure 10: Relationship Between τ and σ and Influence of c

Since the bond (cohesion) is intact up to the peak during run 1, the τ_{peak} value for run 1 was not considered to be part of the τ_{peak}–σ data set used to calculate μ_{static} and to which subsequent runs belong. Equation 2 cannot be used to calculate c directly since this relationship is based on intact bond. Two methods were investigated to calculate c from the data. Method 1 takes c as the difference between τ_{peak} from run 1 and the linear equation fit to the remainder of the τ_{peak}–σ data; since the only difference between run 1 and subsequent runs is the state of the bond (intact/broken), the difference can be attributed solely to c. Method 2 assumes that c is equal to the intercept of a line of slope equal to μ_{static}, shifted up to pass through the τ_{peak}–σ data pair for run 1, as is suggested by the form of Equation 2. The two methods provide lower and upper bounds on c. Table 5 summarizes calculated values of μ and c; the values of μ agree very well with the CSA S304-stipulated value of 1.0 for masonry-to-masonry sliding planes [5].

Table 5: Coefficient of Friction and Cohesion Values Obtained from Direct Shear Test

	μ_{static} [-]	$\mu_{sliding}$ [-]	**Cohesion [Mpa] (Method 1)**	**Cohesion [Mpa] (Method 2)**
Average	1.03	1.07	0.761	1.08
Std. Dev. [-]	0.14	0.20	0.68	0.67
COV (%)	13.9 %	18.5 %	89.0%	61.5 %
Average R^2 [-]	0.9983	0.9949	—	—
# of tests with result/ # of attempted tests	6/6 (100%)			

This laboratory test is completed in a controlled manner, eliminating unquantifiable interaction of the back side of the test unit with the surrounding masonry as well as the unknown magnitude of normal force actually transmitted to the test unit, both encountered in the in situ test (ASTM C1531). Cohesion values can be readily calculated and extracting the small (two-unit) prisms did not prove overly difficult. One challenge was that the eccentricity of the shear force applied close to, but not directly at the bond surface, resulted in an overturning moment on the top unit. The normal force applied to the prism had to be continually adjusted to remain constant – it is not known what effect this had and is possibly related to the high intercepts obtained from linear regressions. This issue could be eliminated by using three-unit prisms with the shear force applied through the centroid of the middle unit; however, obtaining intact, three-stone prisms was difficult.

CONCLUSIONS

In situ tests are typically the first resort when faced with the problem of unknown material properties whereas laboratory testing is believed to provide better precision (but often at greater financial cost/effort). For the stiffness of Centre Block's masonry walls, in situ testing proved to be simpler and more reliable and is preferred over the equivalent laboratory prism compression test, which involves the delicate task of extracting intact prisms from the walls. The opposite was found to be true for sliding shear behaviour, where unknown effects of the interaction of the back of the test unit with surrounding masonry, as well as an unknown distribution of normal stress to the test unit led to lower reliability and greater variance in the results than for the equivalent laboratory test, performed under controlled conditions. The extraction of the smaller prisms required for the laboratory shear test was not overly difficult, and this test is preferred to the in situ equivalent.

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