[

Blast Load Effects on Historic Masonry Buildings

Abass Braimah

Infrastructure Protection and International Security
Department of Civil and Environmental Engineering
Carleton University

Scientific Authority:
Darek Baingo
DRDC Centre for Security Science

Defence Research and Development Canada –
Technical Report
DRDC CSS CR 2013-045
December 2013

IMPORTANT INFORMATIVE STATEMENTS

[Include the sponsor of the work or a reference to a thrust or work unit, when deemed appropriate by author or CSA; relevant patent number(s), relating to protected intellectual property, should be noted. If there is no relevant information for this document, delete this text.]

Template in use: template-july2013-eng_V.03.01.dot

Blast Load Effects on Historic Masonry Buildings

Prepared by:
Abass Braimah, PhD., P.Eng.
Assistant Professor,
Infrastructure Protection and International Security
Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario

Executive Summary

This report presents a literature review on blast load effects on historic masonry buildings. The project was funded by the Royal Canadian Mounted Police (RCMP), Protective Operations, who have a mandate to protect the Federal Parliament Buildings of Canada and many other properties in Ottawa.

The current spate of international and "home-grown" terrorism has increased awareness of the possibility of a terrorist attack in Canada. The foiled efforts of the "Toronto 18" have forever changed the way governments in Canada perceive the terrorist threat and are seeking ways to minimize the effects of a terrorist attack on infrastructure systems, the economy, and the Canadian society. Many structures in Canada at risk of a terrorist attack include government buildings, buildings in the financial, security, and transportation sector, built many years ago of historic stone masonry construction – a time when no building codes existed. Thus their response to blast load is not very well-known. Currently many projects are being undertaken to retrofit historic stone masonry buildings to mitigate the effects of blast loading on the structures and their occupants. The lack of knowledge about the response of historic stone masonry under blast loading makes it difficult to ascertain the effectiveness of the retrofit schemes employed by blast consultants.

The objective of this report was to review open source literature for experimental and numerical research on historic stone masonry structures subjected to blast loading. Also, efforts were made to get classified experimental or numerical research from the USA and the UK, if there was any, on the response of historic stone masonry under blast loading. These efforts were led by Mr. Bert von Rosen of the Canadian Explosives Research Laboratory (CERL) and Dr. Darek Baingo of Centre for Security Science, Department of National Defence who contacted the Centre for the Protection of National Infrastructure (CPNI), UK, Technical Support Working Group (TSWG), and the US State Department. The author, Dr. Abass Braimah also contacted U.S. Army Engineer Research and Development Center and engineering consultants from the United Kingdom (Arup – Resilience Security and Risk) and United States (Hinman Consulting Engineers, Inc.) to solicit any test results on blast load effects on historic masonry structures.

The review of open source literature yielded very little information on the response of historic masonry under blast loading. Most of the information available is on the blast load effects on concrete masonry block walls. Also, information on the seismic response of historic masonry and the material characterization of stone and mortar is available in the literature. Requests to the CPNI and TSWG and other engineering firms in the US and UK yielded no experimental research work on the blast load effects on historic masonry structures.

To protect Canadian infrastructure constructed of historic masonry against terrorist attacks using explosives, a good understanding of their response under blast must be developed. This understanding can be developed through a comprehensive experimental test program

complemented with finite element modelling to investigate the behaviour of historic masonry structures, the effects of material properties, construction methodology (coursing), and support conditions on their response.

The literature review presented in this report reviews the construction methodologies of stone masonry structures in Canada, the material properties (stone and mortar) and coursing of stone masonry walls, and the out-of-plane response of stone masonry walls. The project also investigates the use of the single-degree-of-freedom method for dynamic analysis of out-of-plane response of stone masonry. The literature review also serves to guide future experimental and numerical research work on blast load effects on stone masonry buildings.

Table of Content

1. Introduction

Terrorists' attacks on infrastructure systems and "iconic" buildings and monuments with national, economic, and heritage significance are on the ascendancy globally. The attacks of September 11, 2001 in New York, the London Subway bombings of July 7, 2005 and the foiled actions of the "Toronto 18" in Canada in 2006 have drastically increased awareness of the possibility of terrorist action in Canada. Now more than ever, federal government departments and owners of "iconic" structures are seeking to understand the vulnerabilities of their structures to blast loading. Additionally, there is an increased desire to understand what retrofit/mitigation measures are available to increase the survivability of these structures and their occupants in the event of an attack against or proximate to the structures.

Among "iconic" buildings and monuments with national and heritage significance for Canada are the Federal Parliament Buildings, 24 Sussex Drive, Ontario Legislature Building, Quebec Legislature Building, British Columbia Legislature Building, and Victoria Museum of Nature. These buildings were constructed many years ago, with load-bearing masonry or non-load-bearing infill masonry. The material properties of masonry, especially of the era of construction of these "iconic" buildings are variable and not very well studied. Thus, the blast load resistance of these structures is also not very well defined. Additionally, efforts are afoot to increase the blast resistance of many of Canada's historic stone masonry buildings and to develop retrofit schemes for mitigating the attendant vulnerabilities of these buildings. Unfortunately most of the vulnerability assessments and retrofit designs are based on unproven methodologies, material properties, and structural behaviour.

Currently, the Parliamentary Precinct in Ottawa, Ontario, encompassing the Centre Block, East Block and West Block, is undergoing a comprehensive revitalization and retrofit/upgrade including enhancement of the seismic and blast resistance of the buildings. The methodologies used in assessing the blast resistance of the masonry walls have not been experimentally verified. Thus, there is a need to investigate the effectiveness of the methodologies used in the historic masonry wall analysis under blast loading.

This report reviews available literature on blast load effects on historic stone masonry, material properties, and construction methods of Canadian stone masonry buildings in an effort to inform and guide vulnerability assessments and retrofit designs of buildings on the Parliamentary Precinct.

This literature review effort is funded by the Royal Canadian Mounted Police (RCMP) Protective Operations in Ottawa who have responsibility to ensure protection of the grounds at Parliament Hill.

2. Historic Masonry Building Construction in Canada

Historic masonry buildings are defined as those that have significant historic, architectural, or social meaning and are constructed of masonry materials such as natural stone or brick (ASTM 2012). Thus, historic masonry is defined by a given society or nation to include buildings that are significant to its history or national character regardless of their age.

In Canada, the federal heritage buildings include buildings and monuments recognized as having heritage value. The designation is carried out in accordance with the Treasury Board of Canada policy on management of real property (Lefebvre 2009). The heritage buildings recount the lives and history of the citizens of Canada and raises awareness about how the society developed over the years. Heritage buildings helps Canadians develop a better understanding of the past, the present, and helps prepare for the future (Lefebvre 2009).

Figure 1: Parliament Buildings – Centre Block[1]

Historic buildings, in the Canadian context, include buildings that are designated, have significant cultural or political importance to Canada. Historic stone masonry buildings are those that are constructed with load-bearing or non-load-bearing stone or brick masonry walls. Most of the buildings considered historic masonry in Canada were built towards the end of the 19th and in the early part of the 20th century. These include the Parliament Buildings in Ottawa (Figure 1), Ontario Legislature Building (Figure 2), British Columbia Legislature Building (Figure 3), Quebec National Assembly Building (Figure 4).

[1] source: http://commons.wikimedia.org/wiki/File:Parliament-Ottawa_edit1.jpg

Figure 2: Ontario Legislature Building2

The Federal Parliament Buildings in Ottawa were constructed between 1859 and 1866 after Queen Victoria choose Ottawa as the permanent capital and seat of the government of the new Dominion of Canada (Subercaseaux 1999). The buildings were constructed of Nepean sandstone, quarried locally, and a mix of red sandstone from Potsdam, New York, and grey Ohio freestone. The walls were primarily double-wythe with a rubble infill core containing shards from the dressing stones, small stones, and mortar (Elmenshawi et al. 2010). The Ontario Legislature Building was completed in 1893 and the walls were constructed, primarily, of Credit Valley stone quarried from the Credit River near Toronto (Bayer and Vogel 1980, Freeman 2003). The British Columbia Parliament Buildings were constructed in two stages. The first phase was constructed between 1893 and 1898 while the second phase was built between 1912 and 1915 (Hora and Hancock 2008). The exterior lower stories and foundation of the building was constructed using Nelson Island granite quarried from the mouth of the Jervis Inlet about 100 km from Vancouver. The exterior upper stories were built with Haddington Island andesite. The compressive strength of the Nelson Island granite is reported to be about 240 MPa while that of the Haddington Island andesite is about 127 MPa (Hora and Hancock 2008). The Quebec National Assembly building on the other hand was constructed between 1877 and 1886 (Commission de la Capitale Nationale 2012).

[2] Source: http://en.wikipedia.org/wiki/File:Ontario_Legislature_LOC_npcc_18546u.jpg

Figure 3: British Columbia Legislature Building[3]

Figure 4: Quebec National Assembly Building[4]

The above sample of historic stone masonry buildings in Canada gives a time period for most of the construction. It is important to note that the stone used in the construction of each building was primarily sourced locally and could have significant variability in physical, mechanical, and chemical properties. Thus, the assessment of each building has to consider the properties of the stone and mortar used in the original construction and any modifications or rehabilitation work undertaken to upgrade the response of the building. Including repointing to maintain the aesthetics and moisture transport capability of the masonry structure.

The construction of the walls of Canadian historic stone masonry structures is not uniform, even when constructed in the same period of time. Historic masonry construction flourished at a time when there were no building design codes in Canada nor was the construction of masonry buildings and monuments regulated. The masonry construction trade and expertise were passed from master artisan to apprentice and were based on experience acquired from several years of practice and rules-of-thumb.

[3] Source: http://en.wikipedia.org/wiki/File:British_Columbia_Parliament_Buildings_-_Pano_-_HDR.jpg
[4] Source: http://en.wikipedia.org/wiki/File:Quebec_national_assembly.jpg

Generally, the building walls were either single wythe, double wythe, or double wythe with rubble core infill. The coursing of the stone varied with stone properties, coursing details, mortar properties, and wall composition.

3. Historic Masonry Wall Construction

The use of stone masonry in construction of buildings and monuments dates back thousands of years. From the pyramids of Egypt and the Mayan civilisation to the churches and legislative buildings of the 19th and 20th centuries, stone has been used as raw materials for construction (Drysdale and Hamid 2005, McKinley 2011). The use of stone as a building material blossomed with the invention of tools for shaping and carving rock into intricate blocks, columns, and cornices. In contrast, the use of brick in construction dates back to about 3000 B.C. Brick was manufactured as a mixture of clay, cattle dung, and straw. Later, the manufacturing process was improved to include firing clay brick in kilns to increase its strength and durability (Drysdale and Hamid 2005). Further development of the manufacturing process of brick led to cheaper and a more durable product. Brick thus became a more dominant material for masonry construction. However, stone continued to be used in the construction of monuments, heritage and nationally significant buildings.

Brick used in masonry before 1870's was pressed into moulds and fired. The firing could be uneven, greatly affecting the quality. Terra cotta, a kiln-dried clay product, was also used in buildings until the 1930s in the United States (Morton III et al. 1991), and mostly in Canada as an interior partition walls.

Natural stone is mined from the earth crust and can be categorized as igneous (e.g. granite), sedimentary (e.g. limestone, sandstone), or metamorphic rock (e.g. marble, slate). The properties of natural stone can vary widely even if mined from the same quarry. The properties of stone that are necessary for use in buildings include strength, hardness, workability, porosity, durability and appearance (Beall 1997). Table 1 presents typical physical and mechanical properties of stone used in stone masonry construction.

The strength of stone depends on the crystalline structure and the interconnectedness of the particles. The strength is generally characterised by the compressive strength. Depending on the thickness-to-height ratio of the wall, different compressive strength requirements maybe specified for a building project.

The porosity of the stone influences the capacity of the stone to absorb moisture and to resist the effects of freeze-thaw action, especially in cold climate areas such as Canada. Generally, sedimentary rocks are more porous than igneous and metamorphic rocks. Durability of stone on the other hand is its resistance to wearing and/or weathering. Durability of stone is very important in the performance of the stone as a building material and affects the life span of the building.

Table 1: Physical and mechanical properties of natural stone (after Beall 1997)

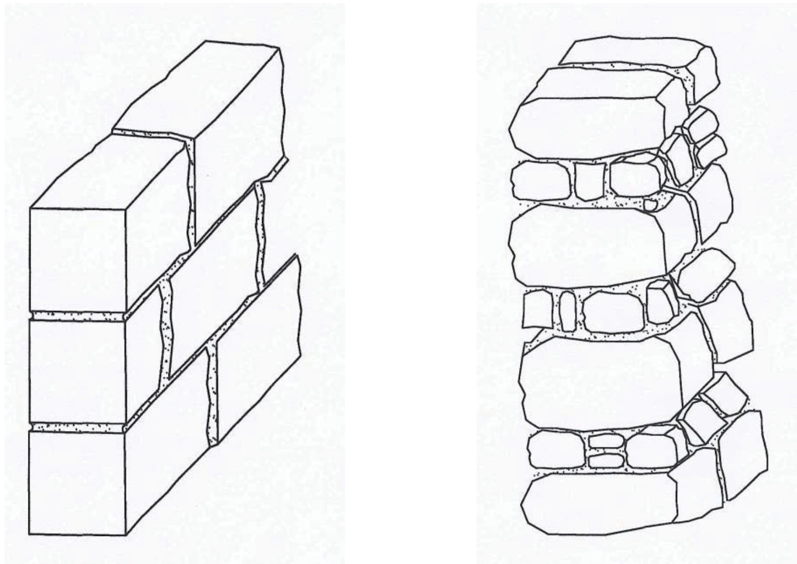
Stone Type	Density	Compressive Strength [MPa]	Tensile Strength [MPa]	Shear Strength [MPa]	Modulus of Rupture [MPa]	Modulus of Elasticity [GPa]
Metamorphic Rock						
Marble		51.7	0.3 - 15	10 - 33	6.9	14 - 150
Slate		-	21 - 30	14 - 25	50 - 62	67 - 125
Sedimentary Rock						
Limestone		27.6	2 - 5	6 - 12	3.5	23 - 37
Sandstone		13.8	1.9 – 3.5	2 - 20	2.1	13 - 53
Igneous Rock						
Granite		330	4.1 – 6.9	13.8 - 33	10.3	40 - 57

Historic masonry walls consist of a layup (coursing) of stone or brick blocks in a wet or dry layup. The dry layup of masonry construction consist of coursing stone or brick unit in layers without mortar. The wet layup on the other hand has coursing of stone or brick units on a bed of mortar. The mortar bed has several functions in the composite masonry wall. Firstly the mortar fills any voids between successive courses and units (stone and brick). The mortar also distributes the compressive pressure between successive courses. Unlike in wet layup, projections of the stone or brick in the dry layup bear against others in the adjacent courses and lead to stress concentrations (Felice 2011). These stress concentrations lead to lower compressive strength of dry layup masonry walls in comparison to wet layup masonry walls.

The stone or brick units in masonry wall construction are laid in a regular pattern in horizontal layers. The layers are staggered in a vertical direction to eliminate continuous vertical joints (D'Ayala 2007). The order and regularity of stone or brick masonry influences the properties of the composite masonry wall. A highly regular stacking order, for example, in dimensioned brick and stone masonry results in higher mechanical properties while random and rubble stone or brick stacking yields comparatively lower mechanical properties.

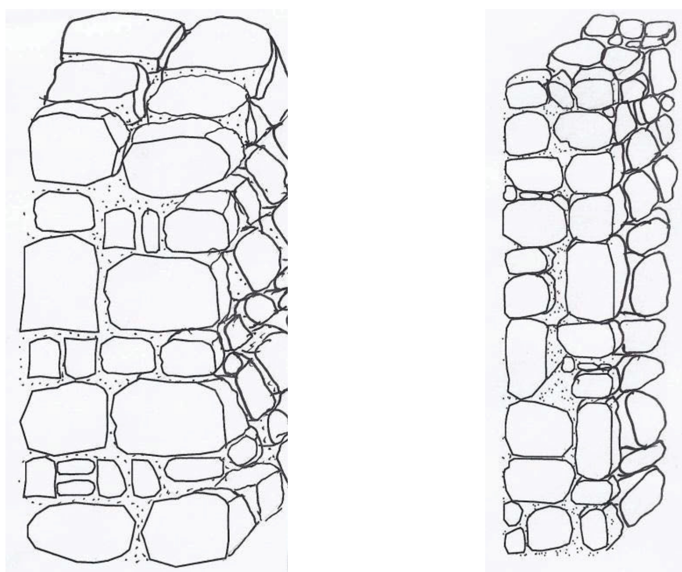
The mortar in historic masonry consists of fine aggregate, sand, water and a cementitious binder, usually lime. Lime-based mortar has very little tensile strength and is thus used to transmit compressive stress between stone or brick units. Depending on the state of the lime-based mortar bed, it can increase the friction stress between units in the masonry wall (Rhodes 1974).

Characterisation of the typology and morphology of historic stone masonry is not an easy task and can include type of stone, shape, and their assembly in the masonry wythes (da Porta et al. 2003). Construction methodologies of historic masonry buildings are varied and in most cases can be single-wythe when the wall thickness is small, double-wythe, or double-wythe with a rubble core infill (also referred to as three-wythe). Figure 5 presents typical single-wythe historic masonry wall typology.



(a) (b)

Figure 5: Single-wythe masonry typology

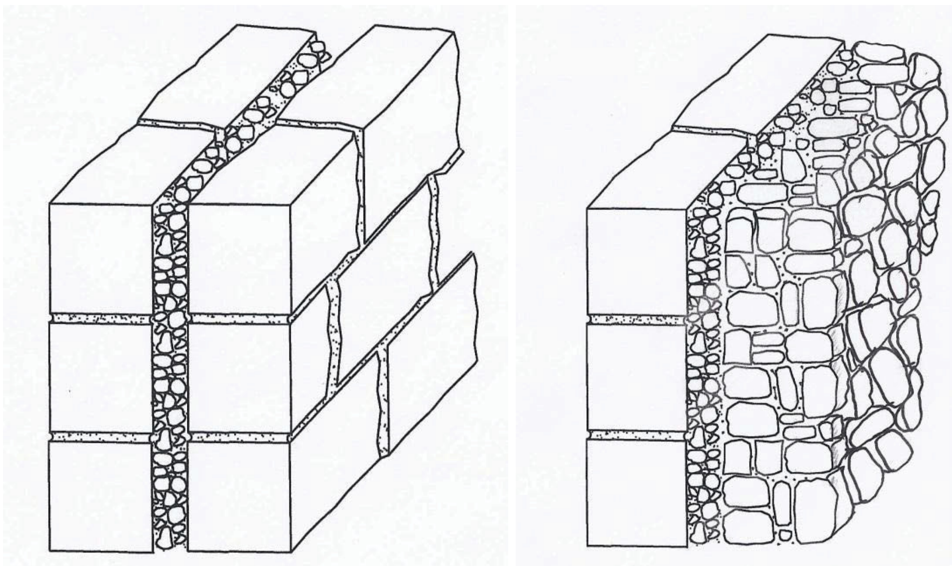


(a) (b)

Figure 6: Two-wythe masonry wall typology

The single-wythe masonry walls can consist of full width stone units (Figure 5a) or multi-unit thick (Figure 5b). The double-wythe stone walls on the other hand can consist of two stone widths with ashlar stone facing with irregular stone or rubble masonry backing (Figure 6). Figure 7 presents double-wythe masonry wall typology with rubble core infill. The rubble infill consist of cut stone, rubble, brick, loose materials, or any material available on site and grouted with lime-based grout or mortar (Jeffs 2001, Oliveira et al. 2012). The double-wythe with rubble infill core typically has an ashlar (dimensioned stone) exterior wythe with rubble masonry interior wythe (Figure 7b) since the interior wythe is often covered in interior wall finishing.

Single wythe walls are rarely used for exterior walls in historic masonry buildings (Binda et al. 1997). According to da Porta et al. (2003), a survey in Italy, especially around Sicily, single-wythe masonry buildings made up between 0-8 percent of all stone masonry buildings.



(a) (b)

Figure 7: Double-wythe with rubble infill core wall typology

The exterior walls of most Canadian historic stone masonry buildings were constructed as double-wythe to increase the thermal capacity of the wall. In many instances the cavity between the two wythes was filled with rubble masonry. In modern masonry building construction, multi-wythe wall systems are required to be connected with regularly spaced ties to ensure monolithic behaviour and redistribution of load between individual wythes. The wall ties are typically made of headers placed in the body of each wythe at regular intervals (D'Ayala 2007). These inter-wythe connections are conspicuously absent in historic masonry structures. Thus, monolithic behaviour of the walls is limited.

For ordinary buildings, load-bearing double-wythe walls are 500 – 1200 mm thick. The exterior wythe is usually composed of dimensioned stone (ashlar masonry) or brick units while the interior wythe is of rubble stone masonry (Mistler 2006). There is often no connection between multi-wythe masonry walls in historic masonry structures. This has an adverse effect on the load carrying capacity of the wall as the walls are liable to separate and act as slender walls under compression loading (Binda et al. 1997, Oliveira et al. 2012).

The strength characteristics and behaviour of historic stone masonry structures depends on the constituent components of the wall: units (stone or brick) and the mortar. The next section reviews the properties of natural stone and mortar used in historic masonry buildings.

4. Material Properties

In order to understand the behaviour of masonry as a structural material its mechanical properties such as strength and stiffness (modulus of elasticity, shear modulus, compressive strength, and tensile strength) must be defined (Elmenshawi et al. 2011). The relationship between stresses and strains is expressed by a material constitutive law that is generally defined over the range of usefulness (applicability) of the material. For masonry, the constitutive laws of the components (stone or brick and mortar) as well as those for the composite masonry are required.

The strength and deformation characteristic of natural stone used in masonry is varied and depends on the type of stone (sedimentary, igneous, or metamorphic), the chemical composition, and level of saturation. Corradi, Borri, and Vignoli (2003) tested calcareous stone to characterize the properties of stone masonry materials in Italy. The authors reported that the compressive strength of calcareous is very variable and dependant on the inhomogeneity of the stone due to the presence of large voids. The compressive strengths varied from 36 to 57.5 MPa.

Elmenshawi et al. (2011) tested sandstone and limestone samples representative of the stone masonry used for the West Block of the Canadian Parliamentary Precinct. The sandstone had a compressive strength of 227 MPa and a Young's modulus of 61.6 GPa while the limestone on the other hand had a compressive strength of 102.5 MPa and a Young's modulus of 41.9 GPa. Lawrence (2001) also reported that the original stone for the exterior walls of the Centre Block and Peace Tower of the Parliament Buildings of Canada consists of Nepean sandstone with a compressive strength of 152 MPa (22032 psi) and a density of 2460 kg/m^3 (153.5 lbs./ft^3). 100-mm cylindrical core samples of granite were tested by Oliveira et al. (2012) to characterise their properties for masonry construction. The authors reported a compressive strength and modulus of elasticity for granite of 52.2 MPa and 20.6 GPa, respectively.

The foregoing highlights the variability in the mechanical properties of stone and the importance of determining the in-situ strength and stiffness values in assessment of the behaviour of historic masonry building to in-plane and out-of-plane loading.

Mortars used in historic masonry construction generally consist of a binder, aggregates and water. The mix proportion of these constituent materials is important for the performance of the mortar and the masonry structure (Forster and Carter 2011, Uranjek 2012, Garcia et al. 2012). Lime-sand volume proportion of 1:3 is assumed to produce mortars with properties representative of mortars recovered from historic buildings (Forster and Carter 2011, Uranjek 2012, Garcia et al. 2012). Lime binders are either naturally occurring from stone deposits or are manufactured by mixing calcium carbonate and clay-based minerals (Forster and Carter 2011). The aggregate (fine sand) acts as bulk filler in the mortar, reduces shrinkage, and contributes to the compressive strength of the mortar.

Lime-based mortars were used in masonry construction prior to the 1900 and before the development of ordinary Portland cement. The use of lime mortars, however, persisted in natural stone masonry due to observed construction defects attributed to use of ordinary Portland cement mortars. According to Forster and Carter (2011), ordinary Portland cement leads to moisture entrapment in stone masonry leading to freeze-thaw related deterioration. Additionally, the high modulus of elasticity of ordinary Portland cement based mortars leads to fracturing of masonry units under the effects of structural loading and thermal or moisture deformations in the masonry structure. The lower modulus of elasticity of lime mortars in comparison to modulus of elasticity of ordinary Portland cement mortars enables the structure to accommodate such deformations (lateral deformations) without distress (suter et al. 1998, Hendry 2001, Forster and Carter 2011).

During repointing work on the Canadian Federal Parliament buildings in 1994 and1995, two test programs were carried out to find most appropriate mortar for deep repointing of Nepean sandstone masonry (Suter et al. 1998). Out of several mix proportions evaluated two mortar mixes with mix proportions of cement, type S lime, and sand mix of 1.5:0.5:6.25 and Portland cement, type S lime, and sand mix of 1:3:9 were adjudged the performers for Nepean sandstone masonry. The compressive strength and modulus of elasticity of the two mortars were 4.2 MPa and 4.4 GPa respectively for the first mortar mix and 3.4 MPa and 4.2 GPa respectively for the second mortar mix. Mortar compressive strengths ranging between 2 and 5 MPa were considered good performance while for modulus of elasticity a value of 1000 times the compressive strength (1 to 8 GPa) was considered good performance.

The characterization of mortar used in historic masonry buildings is complicated. Unlike the stone or brick from which core samples with sufficient size can be sampled and tested to establish the mechanical properties, it is not possible to get mortar samples from existing historic masonry walls for testing (Garcia et al. 2012). According to Garcia et al. (2012), mortars used in Romanesque buildings in Spain have very low compressive strengths varying between 0.2 and 0.50 MPa. Suter et al. (1998) categorised mortars for Nepean sandstone as poor, adequate, and good depending on various performance parameters including compressive strength, ratio of split tensile to compressive strength, Young's modulus, bond strength and freeze-thaw expansion.

Historic stone masonry is a inhomogeneous material composed of two constituent components: stone and mortar. The stone and mortar properties and the method of construction of the masonry wall have a significant influence on its behaviour and mechanical properties. The construction of historic masonry walls can be with stone or brick units laid in regular or irregular patterns with various sizes and shapes in mortar joints. The mortar joints vary in thickness from 5.0 to 20 mm. Reddy et al. (2009) studied the effect of mortar joint thickness on the compressive strength of masonry using soil-cement block units. The 28-day compressive strength of the mortar varied between 3.45 – 8.34 MPa. According to the authors a 16% reduction in masonry compressive strength is achieved when the mortar joint thickness was increased from 6 mm to 30 mm.

A number of researchers (Vasconcelos and Lourenco (2009), Elmenshawi et al. (2001), Garcia et al. (2012), Almeida et al. (2012)) have investigated the effect of stone (or brick) unit and mortar properties on the behaviour and mechanical properties of masonry walls. Garcia et al. (2012) constructed and tested two types of masonry prisms using two stone types and a lime-cement mortar to investigate the strength and behaviour of composite masonry prisms. The stones used were two varieties of sandstone with compressive strengths of 40 MPa and 64 MPa and corresponding tensile strengths of 6.28 MPa and 5.14 MPa respectively. The moduli of elasticity were very similar. The lime-cement mortar was representative of mortar used in historic masonry construction in Spain with a mix proportion of 0.5:1.5:19 (lime: white cement: sand). The mortar compressive strength determined after 365 days was 0.28 MPa.

The stone masonry prisms were constructed by either dry or wet (mortared) layup. The authors reported that the dry jointed masonry prisms exhibited lower compressive strengths in comparison to the mortar jointed masonry prisms. In the dry jointed masonry prisms, stress concentration at a few discrete contact points led to lower failure strengths. The first cracks appeared at an average stress of 1.27 MPa while the average ultimate strength was 7.21 MPa. The masonry prisms with mortared joints on the other hand had average stress at first cracking of 2.58 MPa and average compressive strength of 8.07 MPa. In either stone masonry layup type the ultimate strength was much less than the ultimate compressive strength of the stone. Also, the effect of mortar strength on the modulus of elasticity of the masonry was negligible.

Vasconcelos and Lourenco (2009) tested three types of masonry walls to determine their capacity. The walls were one-third scale models with a thickness of 200 mm and a height of 1200 mm. The masonry wall construction used two stone types: two-mica and medium-coursed-granite in dry stack, irregular stone blocks, and rubble stone masonry layup. The irregular stone and rubble stone masonry walls were coursed with mortar with a compressive strength or 3.0 MPa, representative mortar in historic stone masonry buildings. Even though their tests were primarily in-plane the authors reported that the coursing of the stone in the wall did not significantly affect the wall performance under low precompression. At high levels of precompression, however, the masonry wall strength was found to decrease as the randomness of the stone masonry (or bond) increased.

Sorour et al. (2011) evaluated stone masonry walls representative of the West Block of the Canadian Parliamentary Precinct. The walls were composed of three wythes - a limestone wythe in running bond, sandstone wythe in sneck pattern formed the exterior wythes while the interior wythe was rubble-stone infill. The mortar used in the wall was a 1:3 lime to sand mix proportion by volume with a minimum 28-day compressive strength of 0.34 MPa. The sandstone had a compressive strength of 227 MPa and a Young's modulus of 61.6 GPa while the limestone had a compressive strength of 102.5 MPa and a Young's modulus of 41.9 GPa (Elmenshawi et al. 2011).

The tangent modulus of the walls at compressive stress of 0.55 MPa ranged between 2.1 and 3.0 GPa while the axial ultimate compressive capacity of the wall varied between 1.0 MPa and 1.5 MPa (Sorour et al. 2011). The capacities of the walls were, also, significantly lower than the capacity of the sandstone and limestone used in the construction of the walls, similar to observations reported by other researchers (Garcia et al. 2012, Garcia et al. 2012).

Almeida et al. (2012) tested single-wythe stone masonry wall panels under axial compression. The stone masonry walls were cut out of an existing building constructed in 1916. The stone was granite and was representative of construction materials in historic buildings around the city of Porto, Portugal. The compressive strength, tensile strength and elastic modulus of the granite were determined in accordance with the European Standards (EN 1926 and EN 14580) and ASTM C496-71 and were reported to be about 60 MPa, 3 MPa and 26 GPa, respectively. The wall panels were 1.20 m wide, 2.50 m high and 0.40 m thick and consisted of various rectangular size granite stones with diagonal lengths between 500 – 900 mm on mortar joints with variable thickness between 5 and 20 mm thick. The walls also had smaller granite stones and brick pieces wedged between large blocks.

Almeida et al. (2012) tested the wall panels up to compressive stresses of 3.94 MPa without significant distress observed. The goal of the test program was to test mortar injected walls thus the walls were not taken to failure. The elastic modulus of the walls was reported to be much lower than predictions from code equations.

Oliveira et al. (2012) tested stone (granite) masonry walls under compression to characterise the masonry properties. The compressive strength test on cylindrical granite samples gave compressive strength of 52.2 MPa, elastic modulus of 20.6 GPa and a Poisson's ratio of 0.24.The mortar was low tensile strength and representative of mortar in historic masonry. The compressive strength of the mortar at 28 days was 2.9 MPa.

The walls were three wythes with a thickness of of 100 mm each. The exterior wythes were irregular granite masonry while the core was filled with alternate layers of granite splinters and mortar. The compressive strength of the three-wythe wall was about 4% of that of the granite stone strength. This underscores the importance of testing wall that are representative of the wall construction instead of the strength of the stones (Oliveira et al. 2012).

The above shows that the strength of composite masonry walls is much less than the capacity of the stone components making up the walls. Under uniaxial compression, masonry structures behave as a composite material. The stiffer units (stone or brick) expand laterally due to Poisson effect but to a lesser extent than the mortar layers which are much less stiff. The units experience a compression-bilateral tension stress state due to the higher deformation in the mortar while the mortar layer experiences a tri-axial state of stress (Mosalam 2009, Reddy et al. 2009). The compressive strength of masonry thus depends, primarily, on the tensile strength of the unit. Moreover, the elastic moduli and Poisson's ratio of the units and mortar also affect the compressive strength.

To design masonry walls under various loading actions it is essential to determine the strength of the composite wall given the mechanical properties of the components: stone and mortar. A number of researchers have proposed formulae for determining the compressive strength of masonry, f_m', as a function of the properties of the constituent components. Eurocode 6 (CEN 2003) proposed

$$f_m' = K f_s^{0.75} \times f_m^{0.3} \qquad [1]$$

while Garcia et al. (2012) report on the Huerta and Rozza equations expressed in equations 2 and 3 respectively. Equation 3a is used for ashlar (dimensioned) masonry while equation 3b is for rubble masonry.

$$f_m' = \frac{1}{3} f_s + \frac{2}{3} f_m \qquad [2]$$

$$f_m' = \frac{0.8 \nu_s f_s + 1.2 \nu_m f_m}{10} \qquad [3a]$$

$$f_m' = \frac{\nu_s f_s + 0.8 \nu_m f_m}{12.5} \qquad [3b]$$

f_s is the compressive strength of the stone and f_m is the compressive strength of mortar. ν_s and ν_m are the relative volume of the stone units and the relative volume of the mortar respectively. K is a constant that depends on the quality of the masonry unit.

Similarly the modulus of elasticity and the flexural bond strength of the masonry material can be expressed by equation [4] and equation [5] respectively.

$$E_m = 300 f_m' \qquad [4]$$

$$f_b' = 0.025 f_m \qquad [5]$$

At the time of construction of most historic stone masonry structures there were no established design codes and thus master masons resorted to use of rules-of-thumb to design and build masonry structures.

In Canada historic stone masonry construction generally consisted of double-wythe walls with rubble infill core. The wythes consisted of dimensioned stone (ashlar masonry), brick, or rubble stone masonry. The mortar was often lime-based with very low to no tensile strength. Unlike multi-wythe walls in modern masonry construction which have regularly spaced ties or header bond between the wythes to ensure monolithic behaviour and to redistribute the stresses between the wythes, in historic masonry these ties could be missing. Monolithic behaviour is assured through the use of infill rubble core which most often cannot transfer shear stresses between the exterior wythes.

When a double-wythe with rubble core wall is subjected to uniaxial compression, the load distribution is dependent on the mechanical properties of the individual wythes, the thickness of the each wythe and the connectivity between them (Binda et al. 2006). The load distribution is in proportion to the axial stiffness of the wythes. Binda et al. (2006) tested double-wythe walls with rubble core infill. The exterior wythes consisted of ashlar masonry of sandstone and limestone on a 10-mm thick mortar joints. The masonry wythes were tested individually under compression and also as a double-wythe with rubble masonry infill (da Porta et al. 2004). Binda et al. (2006) reported that the individual ashlar masonry wythes failed at about 40-45% of the strength of the stone. When the double-wythe with rubble masonry core was tested the failure load was similar to that of the individual wythe even though the area was substantially increased i.e. the failure stress was substantially decreased.

Binda et al. (2006) proposed three equations for estimating the compressive strength of the multi-wythe masonry walls. However, only two of the equations compared favourably with experimental results and are shown below.

$$f_c = \frac{2t_e}{2t_e+t_i} f_e + \frac{t_i}{2t_e+t_i} f_i \qquad [6]$$

$$f_c = \frac{2t_e}{2t_e+t_i} \theta_e f_e + \frac{t_i}{2t_e+t_i} \theta_i f_i \qquad [7]$$

f_c is the compressive strength of the composite masonry section, f_e and f_i are the compressive strength of the exterior and interior masonry wythes respectively, and t_e and t_i are exterior and interior wythe thickness. The coefficients θ_e and θ_i are correction parameters for the exterior and interior wythes respectively. The authors proposed values of θ_e = 0.7 and θ_i = 1.3 for the correction parameters.

It has been established that the strength of construction materials are sensitive to high strain-rates. Most research to date on material response under high strain-rate has been conducted for concrete and steel reinforcement (Malvar and Ross 1998, Ross et al. 1993, Ross et al. 1995, Cadoni et al. 2001, Bischoff and Perry 1991, Soroushian and Choi 1987, Malvar 1998). Only scant information is available on the strain-rate effects on material properties of stone masonry.

Asperone et al. (2009) reported that natural stone exhibits higher strengths under high strain rates and attributed the strength increases to the microscopic inhomogeneity. The authors investigated the strain-rate effects on a classic Mediterranean natural stone (Neapolitan Yellow Tuff) at strain-rate levels between 10^{-1} and 50^{-1} s^{-1}. The tensile strength of the Mediterranean natural stone was reported to be about three times the static tensile strength. Asperone et al. (2009) used the Comite Euro-International du Beton (CEB) formulation to model the strain-rate effects of natural stone and observed that the tensile strength was grossly overestimated by the code equations.

Islam and Bindiganavile (2011) investigated the stress rate sensitivity of Paskapoo sandstone, quarried from the Rocky Mountain formations from approximately 60 million years ago. The compressive strength of the stone was determined to be 27 MPa with an elastic modulus of 3.8 GPa and Poisson's ratio of 0.22. The authors proposed that dynamic increase factor (DIF) for the flexural strength of sandstone at higher stress rates. They proposed that the DIF may be described by equations modified from the CEB-FIB (1990) formulation. The DIF is expressed by a bilinear expression given in terms of stress rate as:

$$DIF = \left(\frac{\dot{\sigma}}{\dot{\sigma}_s}\right)^{\delta} \; for\; \dot{\sigma} \leq E\; s^{-1} \qquad [8]$$

$$DIF = \gamma\left(\frac{\dot{\sigma}}{\dot{\sigma}_s}\right)^{1/3} \; for\; \dot{\sigma} > E\; s^{-1} \qquad [9]$$

Where $\dot{\varepsilon}_s = 10^{-6}\, s^{-1}$, $\log \gamma = 6\delta - 2,\; and\;\; \delta = \frac{1}{1+\left({8f_c'}/{f_{co}'} \right)}$, $f_{co}' = 10\; MPa$

5. Analytical and Numerical Modelling of Historic Stone Masonry

Modelling of historic stone or brick masonry is complex. Most modelling techniques consider historic masonry as a homogeneous material while in reality the properties of the mortar, brick or stone, and of the construction methodology of the assemblage has a significant effect on the properties and behaviour of the masonry structure. Furthermore, the interaction of the constituent materials and their interaction in multi-wythe walls make the modelling even more difficult (Cohen 2011). According to Bosiljkov (2004) and Lourenco (2002), the factors leading to the complexity of modelling historic stone masonry include difficulty in characterising mechanical properties of the original component materials, large variability in workmanship, lack of consistency in core infill material and connection between exterior wythes in multi-wythe walls, and lack of understanding of damage levels in existing walls.

There are several methods available for modelling historic masonry structures with varying complexity and difficulty. These methods include rules-of-thumb, hand calculations, rigid-block rotation, finite element analysis, and discrete element method. Regardless of the complexity of the method chosen, the properties of the constituent components: stone (or brick) and mortar are required together with constitutive models of the materials that relate stresses and strains in the constituent component under load.

The rigid-block rotation method is based on a collapse mechanisms idealised as an assemblage of rigid blocks separated by fracture lines (Bosiljkov 2004), usually at the supports and mid-height in one-way spanning walls. In this case, the masonry is assumed to behave in a rigid-perfectly plastic manner with very low or zero tensile strength and infinite compressive strength. The rigid-block rotation method is suited for hand calculations.

Finite element and discrete element methods are the most sophisticated of all analysis methods and study the relationship between stresses and displacement. For accurate modelling of historic masonry structures a good understanding of the material properties and behaviour is required. A mathematical representation of the behaviour of the material (constitutive model) is required to express the relationship between stress and strain in the material. The historic masonry structure can be modeled as a two-dimensional or three-dimensional finite element model or discrete element model depending on the problem and the level of accuracy desired. When the response of a single structural element is sought, the distribution of the stone units and mortar joints can be accounted for by using either the finite element or discrete element method (Giordano et al. 2002).

According to researchers (Giordano et al. 2002, Lourenco 2002, Bosiljkov 2004, Lemos 2007, Fodi and Bodi 2011, Linse and Gebbeken) the strategy employed in modelling historic masonry structures can be broadly grouped into two classes: micro-modelling (detailed and simplified) and macro-modelling.

In micro-modelling (detailed) the stone or brick units and mortar are represented by continuum elements whereas the unit-mortar interface is represented by interface elements. Micro-modelling is a two-material modelling technique. The element sizes are assumed the same as the real unit sizes and mortar thickness and the stone and mortar are modelled with two distinct (different) material models representative of the stone and mortar. Micro-modelling accounts for all failure modes of the masonry structure including unit and mortar failure and failure in the unit-mortar interface. The elastic modulus, lateral deformation, and inelastic material properties of the units and mortar can be modelled. This enables the study of the interaction between the units and the mortar. The disadvantage of using the detailed micro-modelling technique is the large number of elements needed to model structures of substantial size and the complexity and computer resource requirements (Giordano et al. 2002). The simplified micro-modelling technique or macro-modelling techniques can also be used in such situations.

The simplified micro-modelling technique is used where the stone or brick units are modelled as continuum elements and the unit-mortar interfaces are lumped and modelled with interface elements (Figure 8a). Unlike the micro-modelling technique, in the simplified micro-modelling technique the mortar joints are modelled with interface elements and thus the element size is modelled to be greater than the unit sizes to accommodate mortar joint thickness (Figure 8b). The interface elements are modelled as nonlinear springs.

In the macro-modelling technique the stone or brick units, unit-mortar interface and mortar are considered as one homogeneous material and represented by one constitutive model (macro-modelling is also referred to as equivalent material technique) (Figure 8c). In the macro-modelling technique, the properties of the units (stone or brick) and the mortar are "smeared" (Linse and Gebbeken). The macro-modelling technique is more suited for practical use as the computer resources required is less in comparison with micro-modelling techniques. With the macro-modelling technique it is not required to model each masonry unit, mortar and unit-mortar interface. The macro-modelling technique is not able to give information about failure mechanisms, which most often is not required in the analysis.

The discrete element method considers the stone blocks as rigid or deformable elements acting on unilateral elasto-plastic contact elements. The contact elements follow the Coulomb slip criterion for simulating the contact forces between stone and mortar (Giordano et al. 2002).

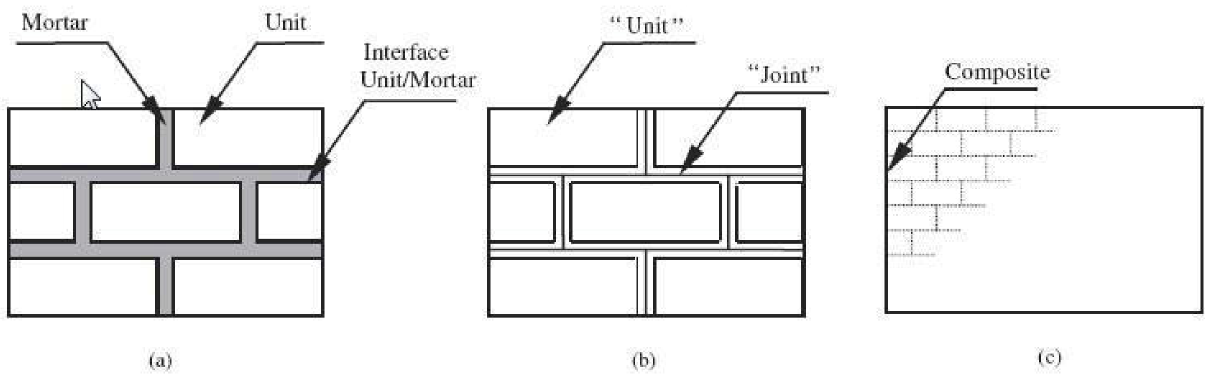


Figure 8: Modelling techniques for masonry structures (a) detailed micro-modelling, (b) simplified micro-modelling, (c) macro-modelling (Lourenco 2002)

In order to develop a design and analysis methodology for a masonry wall, its material properties, particularly strength and stiffness, must be established (D'Ayala 2007). For numerical analysis a constitutive model relating stresses and strains and failure criterion are required. The development begins with understanding the behaviour of the individual components and then deriving a constitutive model for the composite. The stone or brick is often considered the stronger and stiffer material while the mortar is considered the weaker and less stiff (flexible) material. Thus the strength of the masonry wall depends on the strength of the mortar while the stiffness is related to the stone or brick.

The contact elements in the finite element and discrete element methods can be modelled using either hard contact (or rigid contact) or soft contact (or deformable contact) models (Lemos 2007). The rigid contact model enforces the condition of no overlap between adjacent masonry units while for the deformable contact model contact stiffness is defined in the normal and tangential (shear) axes to relate contact stresses and adjacent masonry block displacement in the normal and tangential axes.

Lofti and Shing (1994) identified mortar joints as a source of major weakness and material nonlinearities in finite element analysis of masonry structures. The authors also reported that the behaviour of mortar joints dominated the behaviour of unreinforced masonry structures subjected to severe dynamic loading. They suggested that the proper modelling of mortar joints was crucial to successfully modelling unreinforced masonry structures. Lofti and Shing (1994) developed a constitutive model for the dilatant interfaces. The constitutive model of the dilatant interfaces can simulate the initiation and propagation of masonry fracture under normal and shear stress conditions. The authors verified the proposed model against experimental test results and reported good agreement.

The properties of stone and brick in historic masonry buildings can be defined through testing of samples from the buildings. A mean value can be obtained for compressive strength, tensile strength, and elastic modulus of the units. It is however, not easy to obtain samples of the mortar, with sufficient size, for testing. Once the properties of the components of a masonry structure are determined they can be used to define a homogeneous orthotropic material with a smeared crack model and Mohr-Coulomb failure criterion for use in modelling.

In their attempt to model load-bearing unreinforced masonry vaults in historic cathedrals, Boothby et al. (2006) used the finite element package - ANSYS to develop three-dimensional models of two buildings in New York and Washington DC. Of interest were the natural frequencies and mode shapes of the structures. The authors used the 8-noded iso-parametric element (SOLID45) and the 20-noded iso-parametric element (SOLID95) in ANSYS with linear elastic constitutive material models. An experimental study was also carried out using an instrumented impulse hammer to excite the buildings and monitor their response. The authors reported very good performance of the numerical models in estimating the mode shapes. The paper, however, did not include more details about the material properties, masonry composition, or modelling techniques used.

Hao (2009) proposed a homogenised brick masonry material model that incorporated high strain rate effects on masonry expressed as dynamic increase factors (DIF). The homogenised material constitutive models were programmed and linked to AUTODYN to investigate the brick masonry damage and post-failure fragmentation under blast loading. Even though the authors did not provide details for the AUTODYN modelling, they concluded that for scaled distances less than 4 $m/kg^{1/3}$ the masonry wall was completely blown-out and that the strain rate effects were inconsequential at this scaled distance. At scaled distances greater than 7 $m/kg^{1/3}$ no cracking of the brick masonry wall was observed. The strain-rate effects were prominent at scaled distances between 4 and 7 $m/kg^{1/3}$.

Jordan et al. (2009) investigated the effects of proximate mining operations on historic masonry buildings constructed in about 1830 in Australia. A three-dimensional linear elastic finite element model of the building was developed and subjected to ground vibrations due to mining operations. The mesh size was developed to be equivalent to the masonry unit size (as in the macro-modelling technique) but no effort was made to refine the mesh for better results nor was a mesh sensitivity study undertaken. It is not clear if the mortar was modelled. The authors reported that the peak particle velocity profiles from blasting were applied to the finite element models. The maximum peak frequency was noted to be outside the range of natural frequencies of the building and thus resulted in minimal structural response. Increasing the input loading (by upto 5 times) had negligible effect on the response of the building.

Luccioni et al. (2004) modelled the AMIA building in Argentina, using AUTODYN, under blast loading from a 400 kg TNT detonation. The building was a reinforced concrete frame building with concrete masonry infill walls. The masonry was modelled as an elasto-plastic model, similar to models used for plain concrete. The properties of the concrete were replaced with the mechanical properties of masonry and consisted of a linear equation of state, reference density of 2.40 g/cm^3, bulk modulus of 7.8×10^6 kPa, shear modulus of 2.6×10^6 kPa, and failure tension of 1×10^3 kPa. The shear model was Mohr Coulomb while the failure criterion was set to principal stresses.

The authors reported good agreement between the actual damage recorded during the attack on the AMIA building and the results of the numerical analysis. Similarly to the results obtained from the Luccioni et al. (2004) analysis, Linse and Gebbeken modelled a brick wall in ANSYS AUTODYN and subjected to blast loading and reported a good ability of the software package to reproduce experimental results and brick masonry structure behaviour under blast loading. Brencich and Gambarotta (2005) also used ANSYS to model a brick-mortar stack to study the behaviour of the assemblage in both concentric and eccentric axial compression loading. The authors reported that under concentric loading the assemblage exhibited linear response to about 90% of ultimate while in the eccentric loading case a nonlinear response was observed. The major difference between experimental and ANSYS modelling results was that the experimental post-peak behaviour showed softening behaviour while the ANSYS model predicted a catastrophic collapse.

The numerical modelling studies presented are primarily based on either clay brick or concrete masonry structure response. Very few researchers have studied the response of natural stone masonry using numerical modelling and fewer still under the response of blast loading. Most of the researchers looking at natural stone masonry have developed custom constitutive models for masonry using either the micro-modelling or macro-modelling techniques. The research papers reviewed, however, are lacking in detail and cannot be directly used in the analysis of historic stone masonry walls to assess their behaviour under blast loading. It is therefore important to undertake a focussed research work aimed at modelling stone masonry construction in Canada and to understand the response of these structures to blast loading.

6. Analysis of Historic Masonry Structures under Blast Loading

Review of the literature shows very little information on the response of historic stone masonry to blast loading. Most of the work available with regards to historic masonry is for analysis and response of these structures to seismic loading (Mistler et al. 2006, Sorour et al. 2008, Elmenshawi et al. 2010, Sorour et al. 2011). Also, some work is available on the out-of-plane response of concrete block masonry subjected to blast loading (Moradi et al. 2009, Irshidat et al. 2009, Oesterle et al. 2009).

The methods of analysis proposed for out-of-plane (lateral) loading due to blast or wind are normally based on the homogenised or rigid-block rotation modelling techniques. The response of the wall is assumed to be linear elastic until tensile stresses due to flexure exceed the sum of the tensile strength of the masonry and the precompression from self-weight and applied loads (Kelly 1996, Godinho 2007) when cracking is formed. Post-elastic (or post-crack) response of the wall is dependent on the boundary condition of the wall. For non-load-bearing walls built into rigid peripheral frames, the wall develops arching action due to confining effects of boundary frames while for load-bearing walls the portion of wall between the mid-height and support crack acts as a rocking rigid block. The failure mode for walls under arching action is geometric instability due to snap-through and for non-load-bearing walls geometric instability due to exceedance of the stability limit (deflection). Other failure modes of note are shear failure, friction failure, and bending failure (Mistler et al. 2006).

The shear failure mode is characterised by cracking in the head and bed joints or in cases of low strength stone or brick units, cracking can be through the units. Friction failure mode involves horizontal sliding of masonry units relative to one another. Friction failure modes are common in cases of low vertical loads on the masonry walls or friction resistance. Bending failure mode occurs with high compression loading and cracking in the wall due to flexural stresses. The bending failure mode is a very brittle failure mode while shear failure mode would usually be more ductile with cracking to warn of incipient failure.

Oesterle et al. (2009) used a blast simulator to test carbon fibre reinforced plastic (CFRP) retrofitted concrete masonry wall. The masonry wall specimen consisted of 2650-mm high 190-mm concrete blocks with different layers of CFRP sheets. The tests were modelled using LS-DYNA finite element code. The material constitutive models used for concrete masonry was the K&C concrete model (LS-DYNA MAT_072_R3). Even though the authors did not include a concrete masonry wall without CFRP retrofit the numerical finite element modelling shows that LS-DYNA finite element code is capable of modelling masonry structures under blast loading.

The response of masonry walls, whether concrete block masonry, clay brick masonry, or natural stone masonry, to dynamic loading is determined with single-degree-of-freedom or finite element analysis. The response of historic masonry structures to blast loading is studied by using single-degree-of-freedom analysis expressed in equation [10].

$$M_e\ddot{y}(t) + C_e\dot{y}(t) + K_e y(t) = F_e(t) \qquad [10]$$

Where M_e is equivalent mass, C_e is equivalent damping coefficient, K_e is equivalent stiffness of the SDOF system. $F_e(t)$ is the equivalent load-time history representative of blast loading. The value of $K_e y(t)$, is the resistance function of the masonry wall, and is necessary for the completion of the SDOF analysis.

The resistance function of historic masonry is based on the methods accepted for out-of-plane response concrete masonry wall depending on the support conditions. For historic stone masonry designed primarily for gravity load resistance, there are no stiff boundary elements at the floor elevations. In fact it is typical to have the floor system supported on the stone masonry wall or on only the interior wythe in multi-wythe masonry construction. Thus the resistance function is based on the tensile strength of the masonry components and the precompression from self-weight and applied floor and roof loading. Figure 9 presents lateral loading and out-of-plane response for masonry walls. The resistance curve is bilinear. The peak resistance occurs at the equilibrium displacement beyond which the masonry structure becomes unstable with failure dependent on the inertial restoring force (Kelly 1996).

Prior to the lateral load (blast load) acting on the wall the stress state midspan of the wall, σ_D, precompression, is given by equation [11].

$$\sigma_D = \frac{P + \frac{W}{2}}{t} \qquad [11]$$

Where P is the applied loading and the floor level above the wall plus the half the weight of the section of wall, W, under consideration, and t is the thickness of the wall.

Assuming simple support at the top and bottom of the wall (1st floor elevation and foundation) the reactions R developed in the floor diaphragm system will be given as;

$$R = \frac{ph}{2} \qquad [12]$$

where p is the lateral loading on the wall.

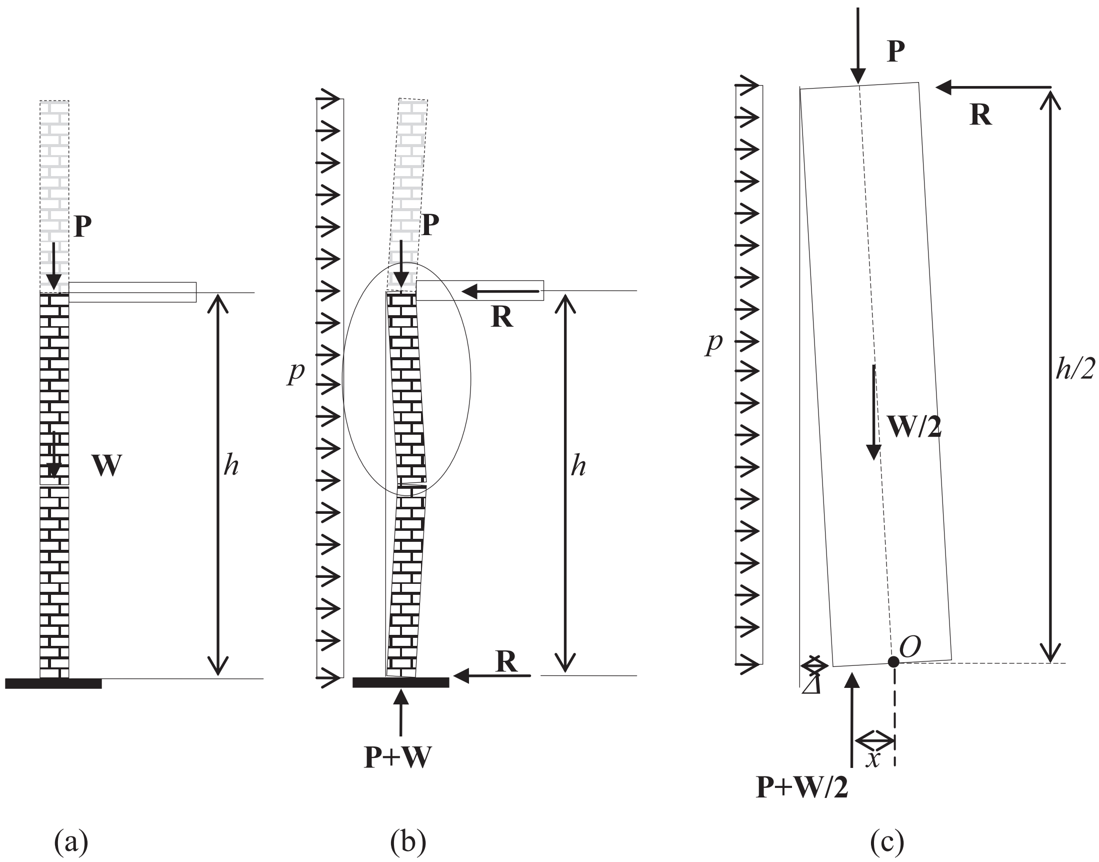


Figure 9: Out-of-plane response of masonry wall under blast loading

The lateral load (resistance) at cracking (elastic limit of the wall), p_e, is given by equation [13]

$$p_e = \frac{8S}{h^2}\sigma_D = \frac{4}{3}\left(P + \frac{W}{2}\right)\frac{t}{h^2} \qquad [13]$$

While the wall deflection under the action of lateral load resistance is given by equation [14]

$$\Delta_e = \frac{5p_e h^4}{384EI} \qquad [14]$$

The reaction at the foundation and floor elevation, R, is also dependent on the level of displacement of the wall at mid-height and is calculated by taking moments about the base of the wall in the deflected shape (9b).

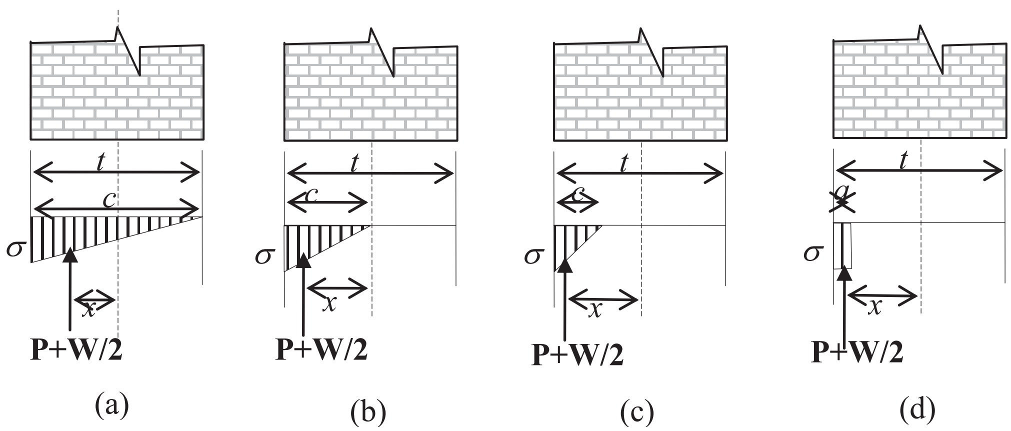
$$R = \frac{ph}{2} + \frac{W\Delta}{2h} \qquad [15]$$

where Δ is the mid-height deflection of the wall. Taking moments about the point O in Figure 9(c) gives the resistance of the wall at a given displacement Δ in accordance with equation [16].

$$p = \frac{8}{h^2}\left(P + \frac{W}{2}\right)(x - \Delta) \qquad [16]$$

where x is the distance between the resultant compressive force in the masonry and the centre of the wall section (point O).

The distance x depends on the compressive stress block (Figure 10). At cracking of the masonry wall, the stress at the unloaded face of masonry wall is zero and the value of x is given in Figure 10(a) as t/6. The maximum value of x occurs when the resultant force approaches the edge of the wall on the loaded face (Figure 10(d)) where the compressive stress could be very high. In this analysis the wall is assumed to have infinite compressive strength and thus crushing failure of the masonry wall is neglected.



(a)	(b)	(c)	(d)
$c = t$	$c = \frac{t}{2}$	$c = \frac{t}{4}$	
$x = \frac{t}{6}$	$x = \frac{t}{3}$	$x = \frac{5t}{12}$	$x = \frac{t}{2} - \frac{a}{2}$

Figure 10: Compressive stress distribution at mid-height of masonry wall (Paulay and Priestley 1992)

According to Paulay and Priestlay (1992), at the elastic limit (onset of cracking) the stress distribution in the masonry walls is as shown in Figure 10(a) and the blast pressure and elastic deflection at this stage are calculated by equations [13] and [14], respectively. At the elastic limit the curvature, ϕ_e, of the wall is determined with equation [17] which expresses the relationship between compressive and tensile strains in the masonry wall.

$$\phi_e = \frac{\sigma}{Et} = \frac{2\left(P+\frac{W}{2}\right)}{Et^2} \qquad [17]$$

When the crack has grown to the centreline of the wall (Figure 8(b)), the curvature can be approximated by equation [18] and expressed to have a curvature of four times the elastic curvature.

$$\phi = \frac{\sigma}{Et} = \frac{8\left(P+\frac{W}{2}\right)}{Et^2} = 4\phi_e \qquad [18]$$

Paulay and Priestly (1992) reports that mid-height displacement of the wall can conservatively assumed to increase in proportion to the central curvature and hence the displacement at associated with the stress state in Figure 8(b) can be expressed as;

$$\Delta = 4\Delta_e \qquad [19]$$

Thus the response of the masonry wall can be represented as a ratio, β, of the elastic displacement (equation [14]).

$$\beta = \frac{\Delta}{\Delta_e} \qquad [20]$$

For different values of β the pressure p and the mid-height deflection of the masonry wall can be determined (Moradi et al. 2009) and used to develop the resistance function of the wall. Figure 11 presents a typical resistance function of a masonry wall. The stability (deflection) limit of the wall is equal to the thickness of the wall i.e. when the eccentricity, *e*, of the wall at mid-height is outside of the wall thickness (Paulay and Priestley 1992, Brencich and Gambarotta 2005, Davidson and Moradi 2007, Griffith et al. 2004).

After cracking, the resistance of the wall to lateral loading is afforded by the internal couple moment from the compressive stresses in the rigid block above the mid-height crack.

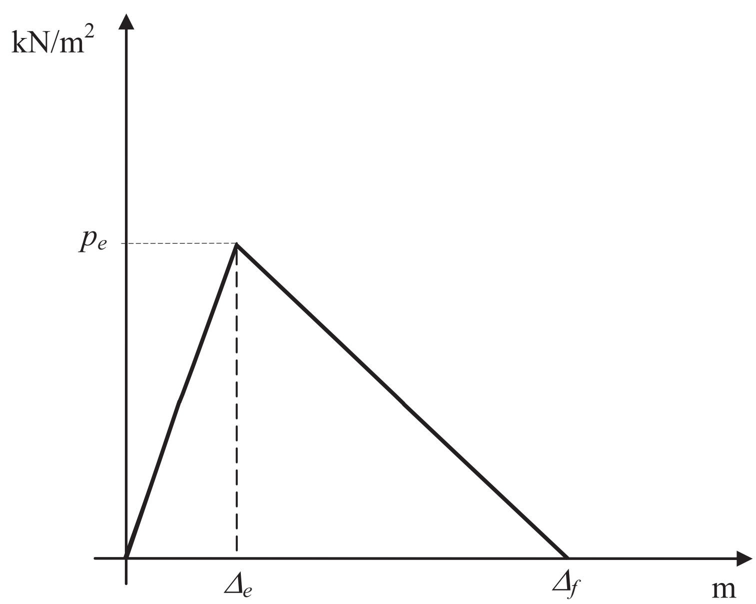


Figure 11: Resistance function of masonry wall under lateral loading

In modelling the out-of-plane response of a masonry wall to blast loading of importance are the representation of the structural mass and the pressure-displacement relationship or resistance function (Hoemann et al. 2010, Lam et al. 2003). The uniformly distributed mass of the wall is represented by the equivalent mass by using a mass factor – determined from an assumed deformed shape while the pressure-displacement (resistance function) relationship is as presented above.

The equation of motion of the SDOF system, equation [10], is often solved by direct numerical integration. The Newmark β-method is a very versatile method used for solving the SDOF equation of motion (Moradi et al. 2009). The average or linear acceleration Newmark method is often regarded the best way to formulate the numerical integration procedure. The timestep has to be chosen to ensure accuracy and convergence of the solution.

7. Recommendation for Historic Masonry

The resistance function development procedure presented in the last section has been developed for concrete masonry units under axial load arching response for out-of-plane loading. Consulting engineers tasked with evaluating the response of historic masonry buildings subjected to blast loading resort to this methodology (sometimes with some modifications) to investigate their response.

The difficulty in using this resistance function development methodology for historic masonry is that most historic masonry construction is non-uniform and consists of multi-wythe stone masonry with either ashlar (dimensioned) stone masonry or rubble stone masonry. The axial loading from upper stories could be applied to only the interior wythe and thus the stress state in the various wythes could be different, especially since there is usually a lack of connection between the wythes. Even when the load from upper stories and floor loading are applied across the thickness of the multi-wythe historic masonry wall, the load is distributed to the wythes in relation to their stiffness (Binda et al. 2006), especially in walls with different wythe thickness, material properties, or coursing.

The material properties of the historic masonry wall are highly variable and could be lacking for the particular buildings being assessed. It is thus difficult to find representative material properties of the building, which is essential for an accurate modelling, without conducting in situ testing of material sampled from the wall. Also, with the variability in material properties for historic masonry construction it is essential to conduct a sensitivity analysis to establish the effect of various properties (for example stone strength, mortar strength, and coursing (ashlar or rubble)) on the performance or response of the structure.

In developing the resistance function of concrete masonry walls, the supports are often assumed to be simply supported. This assumption is often used in seismic response of masonry walls (Paulay and Priestlay 1992) and essentially means that the point of application of the loading from upper stories is at the centroid of the section at the floor and foundation elevations (Figure

12a). This assumption could be true for walls where the response of two adjacent stories is 180^o out of phase (Paulay and Priestlay 1992). This situation is unlikely under blast loading. Thus it is appropriate to consider the moments developed at the supports in the analysis of historic masonry structures. According to Moradi et al. (2009) and Davidson and Moradi (2007) the support moments can be accounted for by assuming fixity at the supports. The authors propose the same level of fixity at the top and bottom supports of the wall.

The support moments can be expressed as the product of the applied axial loading from upper stories and the eccentricity of the load (Figure12). Since historic masonry walls are usually massive with wall thickness varying between 600 mm and 1500 mm and could have large inter-story heights, the support moments at the bottom of the wall could be substantial higher than at the top of the wall. The difference in the magnitude of these moments is the product of the weight of the wall and the eccentricity (Figure 12(c) and Figure 12(d)).

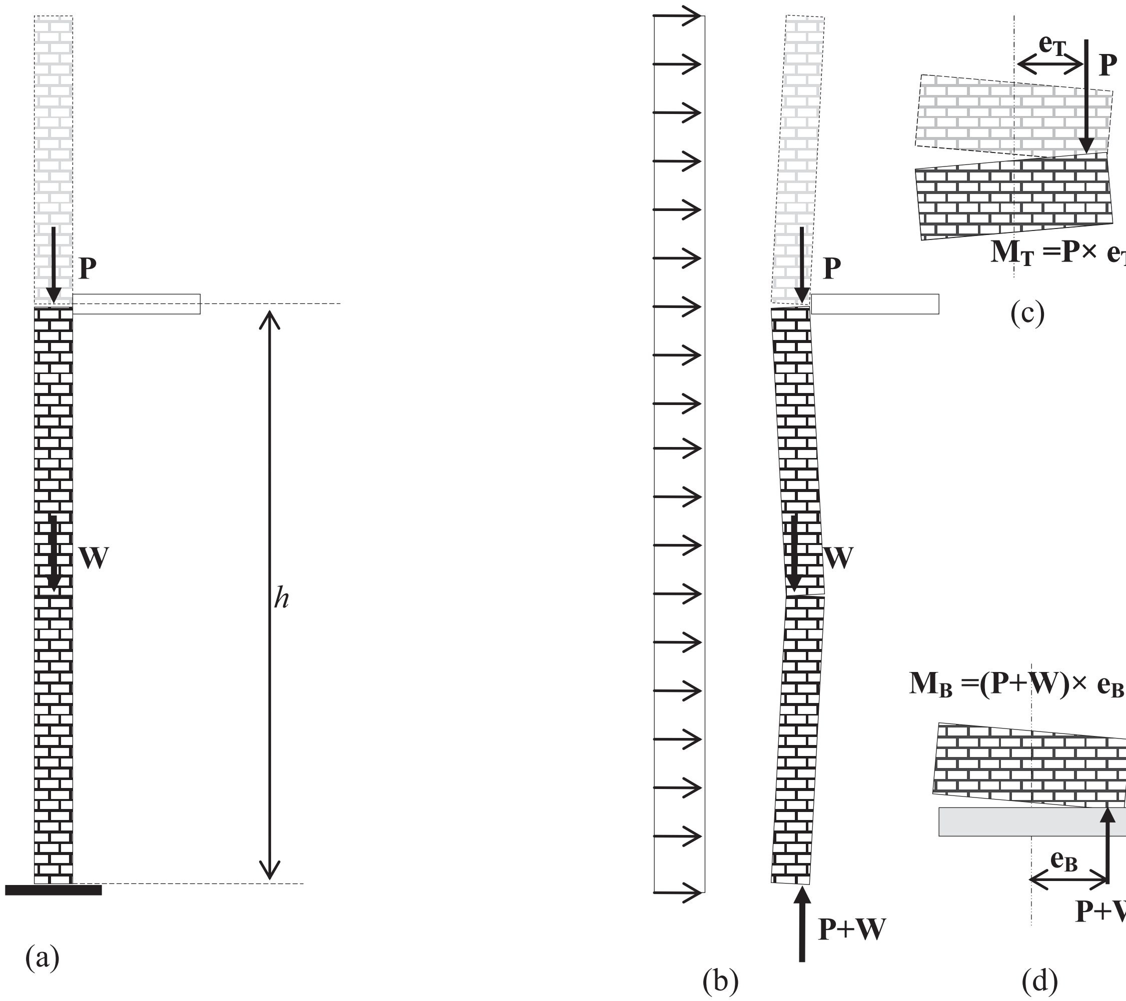


Figure 10: Support moments of the of historic masonry wall

The historic masonry walls investigated in this report are assumed to be supported at the foundation, floor, and roof elevations. The reaction force at these supports is dependent on the applied lateral blast loading and weight of the wall as shown in equation [15]. The floor or roof diaphragm must be capable of resisting this force. Where the roof or floor diaphragm is incapable of resisting the reaction force or where the stiffness of the floor diaphragm results in lateral displacement, the wall analysis must consider the partial cantilever response of the wall that results.

When a historic masonry wall subjected to blast (lateral) loading does not fail it could respond in a vibratory rocking mode, assuming rigid-body rotation. This response mode will lead to successive opening and closing of mortar joints. It is difficult to assess the response of multi-wythe walls, especially double-wythe with rubble core infill, in the vibratory rocking mode. Felice (2011) tested multi-wythe masonry wall in lateral vibratory rocking and noted that the lack of ties between wythes led to detachment of the external wythes and subsequent failure of the walls. It is important to investigate the effect of multi-wythes on the wall performance. Especially the slenderness of each wythe must be evaluated to establish its effect on the overall performance of the wall.

The literature review presented in this report has shown a dearth of information or test results of historic masonry structures under blast loading. Most of the work in the literature has concentrated on the response of historic masonry walls under seismic loading. Thus there is a need for experimental testing of historic masonry walls under blast loading to investigate the accuracy of the SDOF method of analysis in assessing the response of historic masonry walls. Also, the level of end fixity needs to be investigated. Unlike concrete masonry walls which could have limited mass resulting in insignificant end fixity the mass of historic masonry is expected to yield high end fixity that can enhance the response of these walls to blast loading.

The literature review also showed a significant number of numerical finite element analyses efforts at modelling the behaviour of historic masonry structures. The modelling efforts have primarily been in material response/behaviour quantification and in-plane response. However, the constitutive relations of historic masonry defined in these research works can be used effectively in the analysis of historic masonry walls to out-of-plane blast loading.

8. References

Almeida, C. et al., “Physical Characterization and Compression Tests of One Leaf Stone Masonry Walls”, Construction and Building Materials, Volume 3,May 2012, pp. 188-197.

American Society for Testing and Materials (ASTM), “Standard Guide for Repointing (Tuckpointing) Historic Masonry”, E2260 – 03, ASTM International, West Conshohocken, PA. 2012.

Asperone, D. et al., “Dynamic Behavior of a Mediterranean Natural Stone Under Tensile Loading”, International Journal of Rock Mechanics and Mining Sciences, Volume 46, Number 3, April 2009, pp. 514-520.

Bayer, F. and Vogel, P., “Conservation Problems within the Ontario Legislative Building”, Canadian Regional Review II & III, Regional Council of the Commonwalth Parliamentary Association, June 1980, pp. 16-21.

Beall, C., “Masonry Design and Detailing for Architects, Engineers, and Contractors”, 4th Edition, McGraw Hill, New York, 1997.

Binda, L. and Saisi, A., “State of the Art of Research on Historic Structures in Italy”

Binda, L. et al., “Repair and Investigation Techniques for Stone Masonry Walls”, Construction and Building Materials, Vol. 11, No. 3, 1997, pp. 133-142.

Binda, L. et al., “A Contribution for the Understanding of Load-transfer Mechanisms in Multi-leaf Masonry Walls: Testing and Modelling”, Engineering Structures, 28 (2006) 1132-1148.

Birhane, T. H., “Blast Analysis of Railway Masonry Bridges”, Master’s Thesis, University of Minho, Portugal, September 2009.

Bischoff, P. H. and Perry, S. H., “Compressive behaviour of concrete at high strain rates”, Materials and structures, Volume 24, Number 6, 1991, pp. 425-450.

Boothby, T. E., Atamturkur, H. S., and Hanagan, L. M., “Modal Analysis Methods for Validation of Vaulted Stone Masonry Models”, In Buiding Integration Solutions, ASCE, 2006, pp. 1-8.

Borri, A., Corradi, M., Speranzini, E., and Giannantoni, A., “Reinforcement of Historic Masonry: The Reticolatus Technique”, Italian Association of Forensic Engineering Conference, Napoli, December 2 – 4, 2009.

Bosiljkov, V., “Report on the State of the Art on Structural Modelling of Historical Masonry Structures”, European Community funded Project under Environment and Sustainable Development Programme. Project No. EVK4-2001-00091, 2004.

Brencich, A. and Gambarotta, L., “Mechanical Response of Solid Clay Brickwork Under Eccentric Loading. Part I: Unreinforced Masonry”, Materials and Structures, Volume 38, Number 2, 2005, 257-266.

Cadoni, E. et al., “Strain-rate Effects on the Tensile Behaviour of Concrete at Different Relative Humidity Levels”, Materials and Structures, Volume 34, Number 1, 2001, pp. 21-26.

Cohen, J. S., “A Simple Method in the Preliminary Assessment of Historic Masonry Buildings”, Proceeding of ASCE Structures Congress, Las Vegas, Nevada, April 14-16, 2011.

Commission de la Capitale Nationale, “Parliament Hill Walking Tour – A Capital Visit”, available at http://www.capitale.gouv.qc.ca [accessed: December 2012].

Corradi, M., Borri, A., and Vignoli, A., “Experimental Study on the Determination of Strength of Masonry Walls”, Construction and Building Materials, 12, 2003, pp. 325-337.

Davidson, J. S. and Moradi, L. G., “Resistance of Membrane Retrofit Concrete Masonry Walls to Lateral Pressure”, Report No. AFRL-RX-TY-TR-2008-4540, Airbase Technologies Division, Tyndall Air Force Base, Florida. 2007.

da Porto, F. et al., “Investigation for the Knowledge of Multi-leaf Stone Masonry Walls”, Proceedings of the 1st International Congress on Construction History, 20-24th January 2003, Madrid, Spain.

da Porta, F. et al., “Experimental Tests on Irregular Masonry”, European Community funded Project under Environment and Sustainable Development Programme. Project No. EVK4-2001-00091, 2004.

D’Ayala, D. F., “Numerical Modelling of Masonry Structures”, Chapter 9 of Structures and Construction in Historic Building Conservation, Editor: Forsyth, M, Blackwell Publishing, 2007.

Drysdale, R. G. and Hamid, A. A., “Masonry Structures – Behaviour and Design”, Canadian Edition, Canada Masonry Design Centre, Mississauga, ON. 2005.

Elmenshawi, A et al., “In-plane Seismic Behaviour of Historic Stone Masonry”, Canadian Journal of Civil Engineering, Vol. 7, 2010, pp. 465-476.

Elmenshawi, A. et al., “Elastic Moduli of Stone Masonry Based on Static and Dynamic Tests”, 11th North American Masonry Conference, Mineapolis, MN, June 5-8, 2011.

Felice, G., “Out-of-Plane Seismic Capacity of Masonry Depending on Wall Section Morphology”, International Journal of Architectural Heritage, Volume 5, Issue 4-5, pp. 466-482, 2011.

Fodi, A. and Bodi, I., “Basics of Reinforced Masonry”, Concrete Structures, Annual Technical Journal, Volume 12, 2011, pp. 69-77.

Forster, A. M. and Carter, K., “A Framework for Specifying Natural Hydraulic Lime Mortars for Masonry Construction”, Structural Survey, Volume 29, Issue 5, 2011, pp.373-396.

Freeman, E. B., “Geology of Parliament Buildings 3. Building Stones of Ontario’s Provincial Parliament Buildings”, Geoscience Canada Volume 30, Number 2, June 2003, pp. 43-57.

Friedman, D., “Historical Building Construction – Design, Materials, and Technology”, W.W. Norton & Company Inc., New York, NY. 1995.

Garcia, D. et al., “Experimental Study of Traditional Stone Masonry Under Compressive Load and Comparison of Results with Design Codes”, Materials and Structures, Volume 45, Issue 7, July 2012, pp. 995-1006.

Garcia, D. et al., “Comparison between Experimental Values and Standards on Natural Stone Masonry Mechanical Properties”, Construction and Building Materials, Volume 28, Issue 1, March 2012, pp. 444-449.

Giordano, A. et al., “Modelling of Historical Masonry Structures: Comparison of Different Approaches through a Case Study”, Engineering Structures, Number 24, Number 8, 2002, pp. 11057-1069.

Godinho, J. et al., “Resistance of historic unreinforced masonry walls to air-blast loads”, Structures Magazine, May 2007.

Griffith, M. C. et al., “Experimental Investigation of Unreinforced Brick Masonry Walls in Flexure”, Journal of Structural Engineering, Volume 130, Number 3, March 1 2004, pp. 423-432.

Hamed, E. and Rabinovitch, O., “ Nonlinear Dynamic Behavior of Unreinforced Masonry Walls Subjected to Out-of-Place Loads”, Journal of Structural Engineering, Volume 134, Number 11, November 2008, pp. 1743-1753.

Hao, H., “Numerical Modelling of Masonry Wall Response to Blast Loads”, Australian Journal of Structural Engineering, Volume 10, Number 1, 2009, pp. 37-52.

Hora, Z. D. and Hancock, K. D., “Geology of the British Columbia Parliament Buildings, Victoria”, Journal of the Geological Association of Canada, volume 35, Number 2, 2008.

Hendry, A. W., “Masonry Walls: Materials and Construction”, Construction and Building Materials, Volume 15, number 8, 2001, pp. 323-330.

Hoemann, J. et al., “Boundary Condition Behavior and Connection Design for Retrofitted Unreinforced Masonry Walls Subjected to Blast Loads”, In Proceedings of 2010 Structures Congress, May 12-15, 2010, Orlando, Florida.

Irshidat, M. et al., “Blast Resistance of Unreinforced Masonry (URM) Walls Retrofitted with Nano Reinforced Elastomeric materials”, In 2009 Structure Congress: Don’t Mess with Structural Engineers – Expanding our Roles, ASCE, 2009, pp. 1-10.

Islam, M. T. and Bindiganavile, V., “Stress Rate Sensitivity of Paskapoo Sandstone Under Flexure”, Canadian Journal of Civil Engineering, Volume 39, Number 11, November 2011, pp.1184-1192.

Jeffs, P. A., “Core Consolidation of Heritage Structure Masonry Walls and Foundations using Grouting Techniques – Canadian Case Studies”, Proceedings of 9th Canadian Masonry Symposium, June 2001, Fredericton, NB.

Jordan, J. W. et al., “Blast Vibration Effects on Historical Buildings”, Australian Journal of Structural Engineering, Volume 10, number 1, 2009, pp. 75-84.

Kelly, T. E., “Earthquake Resistance of Unreinforced Masonry Buildings”, 11th World Conference on Earthquake Engineering. Paper No. 689, 1996.

Lam, N. T. K. et al. “Time-history Analysis of URM Walls in Out-of-plane Flexure”, Engineering Structures, Volume 25, Issue 6, May 2003, pp. 743-754.

Lawrence, D. E., “Building Stones of Canada’s Federal Parliament Buildings”, Geoscience Canada, Volume 28, Number 1, March 2001, pp. 13-30.

Lefebvre, C., “A Guide to Working with The Federal Heritage Buildings Review Office”, Parks Canada, 2009.

Lemos J. V., “Discrete Element Modeling of Masonry Structures”, International Journal of Architectural Heritage, Volume 1, Issue 2, 2007, pp. 190-213.

Linse, T. and Gebbeken, N., “Modeling Masonry under Dynamic Loadings, Material Models, Numerical Simulations”, …

Lofti, H. R. and Shing, B., “Interface Model Applied to Fracture of Masonry Structures”, Journal of Structural Engineering, Volume 120, Number 1, January 1994, pp. 63-80.

Lourenco, P. B., “Experimental and Numerical Issues in Modelling of the Mechanical Behaviour of Masonry”, Structural Analysis of Historical Construction II, 1998.

Lourenco, P. B., “Computations on Historic Masonry Structures”, Progress in Structural Engineering and Materials, Volume 4 Issue 3, July/September 2002, pp. 241-339.

Lourenco, P. B., "Structural Masonry Analysis: Recent Developments and Prospects, Proceedings of 14th International Brick and Block Masonry Conference (14IBMAC), 17-20 February, 2008, Sydney, Australia.

Luccioni, B. M. et al., "Analysis of Building Collapse Under Blast Loading", Engineering Structures, Volume 26, Number 1, 2004, 63-71.

Malvar, L. J., "Review of Static and Dynamic Properties of Steel Reinforcing Bars", ACI Materials Journal Volume 95, Number 5, 1998.

Malvar, L. J. and Ross, A., "Review of Strain Rate Effects for Concrete in Tension", ACI Materials Journal, Volume 95, Number 6, 1998.

McKinley, J. M., "Natural Building Stones", Geology Today, Volume 27, Number 3, May-June 2011, pp. 114-118.

Mistler, M. et al., "Modelling Methods of Historic Masonry Buildings under Seismic Excitation", Journal of Seismology, Volume 10, Number 4, 2006, 497-510.

Moradi, L. G. et al., "Response of Bonded Membrane Retrofit Concrete Masonry Walls to Dynamic Pressure", Journal of Performance of Constructed Facilities, Volume 23, No. 2, April 1, 2009.

Morton III, W. B. et al., "The Secretary of the Interiors Standard for Rehabilitation & Illustrated Guidelines for Rehabilitating Historic Buildings", U.S. Department of the Interior, National Park Service, Heritage Preservation Services, Washington D. C., 1991.

Mosalam, K. et al., "Mechanical Properties of Unreinforced Brick Masonry-Section 1", Lawrence Livermore National Laboratory, Report No. LLNL-TR-417646, October 2009.

Oesterle, M. G. et al., "Response of Concrete Masonry Wall to Simulated Blast Loads", In 2009 Structure Congress: Don't Mess with Structural Engineers – Expanding our Roles, ASCE, 2009, pp. 1-10.

Oliveira, D. V. et al., "Strengthening of Three-leaf Stone Masonry Walls: An Experimental Research", Materials and Structures, Volume 45, Issue 8, August 2012, pp. 1259-1276.

Paulay, T. and Priestley, M. J. N., "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley & Sons, Inc., 1994.

Reddy, B. V. V. et al., "Influence of Joint Thickness and Mortar-Block Elastic Properties on the Strength and Stresses Developed in Soil-Cement Block Masonry", Journal of Materials in Engineering, Vol. 21, No. 10, October 2009.

Rhodes, P. S., “The Structural Assessment of Buildings Subjected to Bomb Damage”, The Structural Engineer, Volume 52, No. 9, September 1974, pp. 329-339.

Ross, C. A. et al., “Moisture and Strain Rate Effects on Concrete Strength”, ACI Materials Journal, Volume 93, Number 3, 1996.

Ross, C. A. et al., “Effects of Strain Rate on Concrete Strength”, ACI Materials Journal, Volume 92, Number 1, 1995.

Schuller, M. and Boornazian, G., “Engineering for Heritage Masonry”, Structure Magazine, May 2010, pp. 26-29.

Sorour, M. et al., “An Experimental Programme for Determining the Characteristics of Stone Masonry Walls”, Canadian Journal of Civil Engineering, Volume 38, Number 11, 2011, pp. 1204-1215.

Soroushian, P. and Choi, K., “Steel Mechanical Properties at Different Strain Rates." Journal of Structural Engineering, Volume 113, Number 4, 1987, pp. 663-672.

Subercaseaux, M. I. et al., “Designing a Test Blasting Program for an Underground Building on Parliament Hill”, ATP Bulletin, 30 (2-3), 1999, pp. 67-73.

Suter, G. T. et al., “Mortar Study of Mechanical Properties for the Repointing of the Canadian Parliament Buildings”, APT Bulletin, Volume 29, Number 2, 1998, pp. 51-58.

UFC 3-340-02, “Structures to Resist the Effects of Accidental Explosions”, Department of Defense, Washington DC, 2008.

Uranjek, M. et al., “Lime Based Grouts for Strengthening of Historical Masonry Buildings in Slovenia”, Historic Mortars (2012): 393-409.

Vasconcelos, G. and Lourenco, P. B., “In-Plane Experimental Behavior of Stone Masonry Walls under Cyclic Loading”, Journal of Structural Engineering, Vol. 135, No. 10, October 2009, pp.1269-1277.

DOCUMENT CONTROL DATA

(Security markings for the title, abstract and indexing annotation must be entered when the document is Classified or Designated)

1. ORIGINATOR (The name and address of the organization preparing the document. Organizations for whom the document was prepared, e.g. Centre sponsoring a contractor's report, or tasking agency, are entered in section 8.)

Abass Braimah
Infrastructure Protection and International Security
Department of Civil and Environmental Engineering
Carleton University

2a. SECURITY MARKING (Overall security marking of the document including special supplemental markings if applicable.)

UNCLASSIFIED

2b. CONTROLLED GOODS

(NON-CONTROLLED GOODS)
DMC A
REVIEW: GCEC APRIL 2011

3. TITLE (The complete document title as indicated on the title page. Its classification should be indicated by the appropriate abbreviation (S, C or U) in parentheses after the title.)

Blast Load Effects on Historic Masonry Buildings

4. AUTHORS (last name, followed by initials – ranks, titles, etc. not to be used)

Braimah, Abass

5. DATE OF PUBLICATION (Month and year of publication of document.)	6a. NO. OF PAGES (Total containing information, including Annexes, Appendices, etc.)	6b. NO. OF REFS (Total cited in document.)
	37	77

7. DESCRIPTIVE NOTES (The category of the document, e.g. technical report, technical note or memorandum. If appropriate, enter the type of report, e.g. interim, progress, summary, annual or final. Give the inclusive dates when a specific reporting period is covered.)

Contractor Report

8. SPONSORING ACTIVITY (The name of the department project office or laboratory sponsoring the research and development – include address.)

9a. PROJECT OR GRANT NO. (If appropriate, the applicable research and development project or grant number under which the document was written. Please specify whether project or grant.)

CSSP-2013-TI-1041

9b. CONTRACT NO. (If appropriate, the applicable number under which the document was written.)

10a. ORIGINATOR'S DOCUMENT NUMBER (The official document number by which the document is identified by the originating activity. This number must be unique to this document.)

DRDC CSS CR 2013-045

10b. OTHER DOCUMENT NO(s). (Any other numbers which may be assigned this document either by the originator or by the sponsor.)

11. DOCUMENT AVAILABILITY (Any limitations on further dissemination of the document, other than those imposed by security classification.)

12. DOCUMENT ANNOUNCEMENT (Any limitation to the bibliographic announcement of this document. This will normally correspond to the Document Availability (11). However, where further distribution (beyond the audience specified in (11) is possible, a wider announcement audience may be selected.))

13. ABSTRACT (A brief and factual summary of the document. It may also appear elsewhere in the body of the document itself. It is highly desirable
that the abstract of classified documents be unclassified. Each paragraph of the abstract shall begin with an indication of the security classification
of the information in the paragraph (unless the document itself is unclassified) represented as (S), (C), (R), or (U). It is not necessary to

include
here abstracts in both official languages unless the text is bilingual.)

This report presents a literature review on blast load effects on historic masonry buildings. The project was funded by the Royal Canadian Mounted Police (RCMP), Protective Operations, who have a mandate to protect the Federal Parliament Buildings of Canada and many other properties in Ottawa.

Ce rapport présente une revue de la littérature sur les effets de la charge de souffle sur les bâtiments historiques de maçonnerie. Le projet a été financé par la Gendarmerie royale du Canada (GRC), des opérations de protection, qui ont pour mandat de protéger les édifices du Parlement fédéral du Canada et de nombreuses autres propriétés à Ottawa.

14. KEYWORDS, DESCRIPTORS or IDENTIFIERS (Technically meaningful terms or short phrases that characterize a document and could be
helpful in cataloguing the document. They should be selected so that no security classification is required. Identifiers, such as equipment model designation, trade name, military project code name, geographic location may also be included. If possible keywords should be selected from a
published thesaurus, e.g. Thesaurus of Engineering and Scientific Terms (TEST) and that thesaurus identified. If it is not possible to select indexing terms which are Unclassified, the classification of each should be indicated as with the title.)

Historic Masonry; Blast Loading; Physical Security