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# Effects of Blast Load on Historic Stone Masonry Buildings in Canada: A Review and Analytical Study

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***Abstract*—**The global ascendancy of terrorist attacks on building infrastructure with economic and heritage significance has increased awareness of the possibility of terrorism in Canada. Many structures in Canada that are at risk of terrorist attacks include government buildings, built many years ago of historic stone masonry construction. Although many researchers are investigating ways to retrofit masonry stone buildings to mitigate the effect of blast loadings, lack of knowledge on the dynamic behavior of historic stone masonry structures under blast loads makes it difficult to ascertain the effectiveness of the retrofitting techniques. This paper presents a review of open-source literature for the experimental and numerical stone masonry structures under blast loads. This review yielded very little information of the response of the historic stone masonry structures under blast loads. Thus, a comprehensive study is needed to understand the blast load effects on historic stone masonry buildings. The out-of-plane response of historic masonry structures to blast loads is investigated by using single-degree-of-freedom analysis. This approach presents equations that can be used effectively in the analysis of historic masonry walls to out-of-plane blast loading.

***Keywords*—**Blast loads, historical buildings, masonry structure, single-degree-of-freedom analysis.

## I. Introduction

TERRORIST 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 behavior.

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 paper reviews available literature on blast load effects on historic stone masonry, 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.

## II. 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 [1]. 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 [2]. 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 help Canadians develop a better understanding of the past, the present, and helps prepare for the future [2]

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 $19^{th}$ and in the early part of the $20^{th}$ century. These

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include the Parliament Buildings in Ottawa (Fig. l) and British Columbia Legislature Building (Fig. 2).

Fig. 1 Parliament Buildings- Center Block

Fig. 2 British Columbia Legislature Building

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 [3]. 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 [4]. 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 [5]. The exterior lower stories and foundation of the building were 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 [5].

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.

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.

## III. Historic Masonry Wall Construction

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 consists 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 [6]. These stress concentrations lead to lower compressive strength of dry layup masonry walls in comparison to wet layup masonry walls.

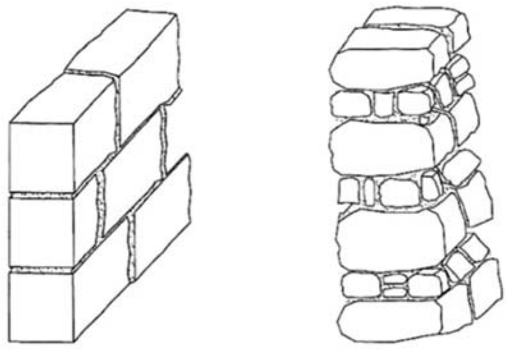
The stone or brick units in masonry wall construction are laid in a regular pattern in horizontal layers. The layers are staggered in a vertical direction to eliminate continuous vertical joints [7]. The order and regularity of stone or brick masonry influences the properties of the composite masonry 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 [8].

Characterization 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 [8]. Construction methodologies of historic masonry buildings vary 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). Fig. 3 presents typical single-wythe historic masonry wall typology.

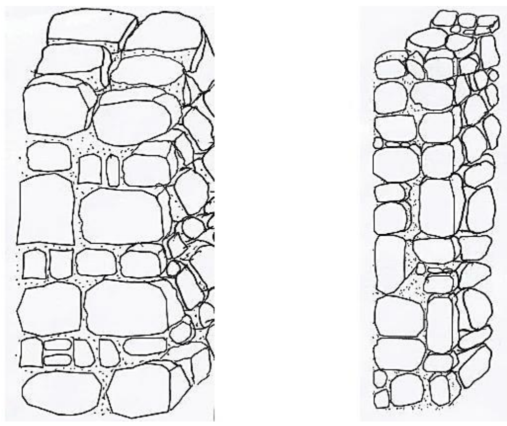
The single-wythe masonry walls can consist of full width stone units (Fig. 3 (a)) or multi-unit thick (Fig. 3 (b)). The double-wythe stone walls on the other hand can consist of two stone widths with ashlar stone facing with irregular stone or rubble masonry backing (Fig. 4).

Fig. 5 presents double-wythe masonry wall typology with rubble core infill. The rubble infill consists of cut stone, rubble, brick, loose materials, or any material available on site and grouted with lime-based grout or mortar [10], [11]. The double-wythe with rubble infill core typically has an ashlar (dimensioned stone) exterior wythe with rubble masonry interior wythe (Fig. 5 (b)) since the interior wythe is often covered in interior wall finishing.



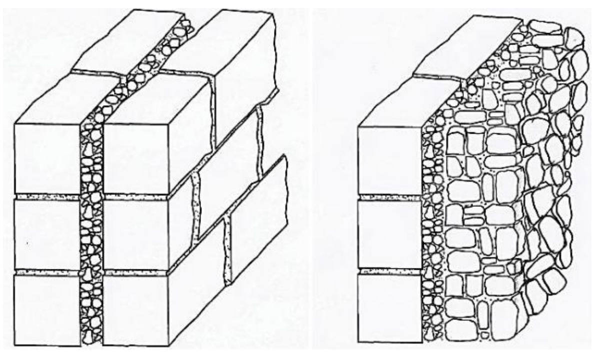
(a) (b)

Fig. 3 Single-wythe masonry wall typology



(a) (b)

Fig. 4 Two-wythe masonry wall typology



(a) (b)

Fig. 5 Double-wythe with rubber infill core wall typology

Single wythe walls are rarely used for exterior walls in historic masonry buildings [12]. According to [9], a survey in Italy, especially around Sicily, single-wythe masonry buildings made up between 0-8% of all stone masonry buildings.

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 behavior 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 [7]. These inter-wythe connections are conspicuously absent in historic masonry structures. Thus, monolithic behavior 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 [13]. 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 [11], [12].

## IV. Review of the Behavior of 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 [4], [13], [14]. Also, some work is available on the behavior of concrete block masonry subjected to blast loading [15]-[22], besides, some work on the response of blast loaded arched masonry [23], [24].

Pereira et al. [17] after experimental study on infill masonry walls against blast loading found decreased maximum deflection in masonry with increasing compressive and tensile strengths up to certain level of infill masonry for small-scaled distances.

Oesterle et al. [18] used a blast simulator to test carbon fiber reinforced plastic (CFRP) retrofitted concrete masonry walls. 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 model 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 modeling shows that LS-DYNA finite element code is capable of modeling masonry structures under blast loading.

Keys and Clubley [19] reported structural failure in masonry walls when exposed to positive overpressure with positive phase duration exceeding 100 ms. Failure mode, initial fragmentation and distribution of debris were found dependent on overpressure and impulse of blast, and wall geometry.

Badshah et al. [20] conducted eight successive experimental blast tests with an increasing TNT equivalent charge weights ranging from 0.56 kg to 17.78 kg on unreinforced, ferrocemented overlay masonry and confined masonry walls. The pressure-time history caused by the blast was recorded by pressure sensors installed on the test specimen. The results provide a basis for determining the response of each masonry system against blast loading. Consequently, efficiency of ferrocemented overlay masonry and confined masonry was found established in mitigation against blast loads.

Parisi et al. [21] reported enhanced dynamic performance with increasing thickness of brick masonry wall against blast loading.

Alsayed et al. [22] studied the performance of strengthening scheme for infill masonry walls using GFRP sheets against blast loads. Strengthened walls were tested against blast loads for the evaluation of the out-of-plane performance of strengthened walls as against the unreinforced masonry) walls. The indicated that the most significant parameter for assessing the severity of damage in structures under blast loads is the scaled distance. In addition, it has been demonstrated that the use of GFRP composites with proper end anchorage offers great potential for the retrofitting of URM infill walls to resist low or moderate blast loads and contain flying debris.

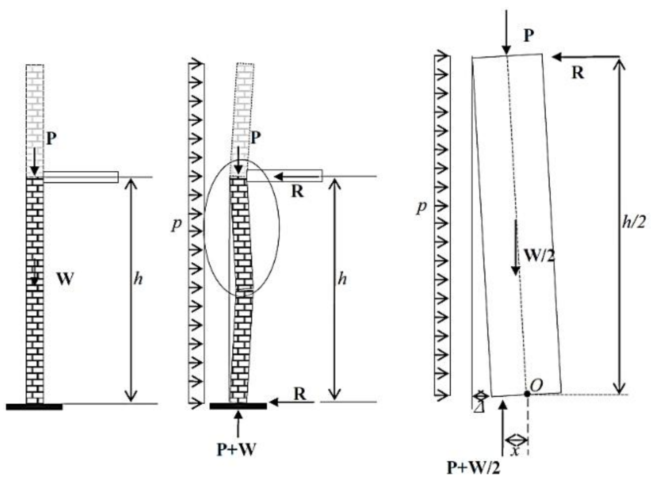
The research papers reviewed, however, are lacking in detail and cannot be directly used in the analysis of historic stone masonry walls to assess their behavior under blast loading. It is therefore important to undertake a focused research work aimed at modeling stone masonry construction in Canada and to understand the response of these structures to blast loading.

## V. Single Degree of Freedom Analysis

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 (1):

$$M_e\ddot{y}(t) + C_e\dot{y}(t) + K_e y(t) = F_e(t) \tag{1}$$

where $M_e$ is equivalent mass, $C_e$ is equivalent dumping coefficient, $K_e$ is equivalent stiffness of 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 masonry wall and is necessary for the completion of SDOF analysis.




(a) (b) (c)

Fig. 6 Out-of-plane response of masonry wall under blast loading

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. Fig. 6 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 [29].

Prior to the lateral load (blast load) acting on the wall the stress state midspan of the wall, $\sigma_D$ , precompression, is given by,

$$\sigma_D = \frac{P+\frac{W}{2}}{t} \tag{2}$$

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 (l[st] floor elevation and foundation) the reactions R developed in the floor diaphragm system will be given as:

$$R = \frac{ph}{2} \tag{3}$$

where $p$ is the lateral loading on the wall.

The lateral load (resistance) at cracking (elastic limit of the wall), $p_e$, is given by,

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

while the wall deflection under the action of lateral load resistance is given by,

$$\Delta_e = \frac{5p_e h^4}{384EI} \tag{5}$$

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 as shown in Fig. 6 (b).

$$R = \frac{ph}{2} + \frac{W\Delta}{2h} \tag{6}$$

where Δ is the mid-height deflection of the wall. Taking moments about the point O in Fig. 6 (c) gives the resistance of the wall at a given displacement Δ in accordance with (7):

$$p_e = \frac{8}{h^2}\left(P + \frac{W}{2}\right)(x - \Delta) \tag{7}$$

where $x$ is the distance between the resultant compressive force in the masonry and the center of the wall section (point O). The distance $x$ depends on the compressive stress block (Fig. 7). 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 Fig. 7 (a) as t/6. The maximum value of occurs when the resultant force approaches the edge of the wall on the loaded face (Fig. 7 (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.

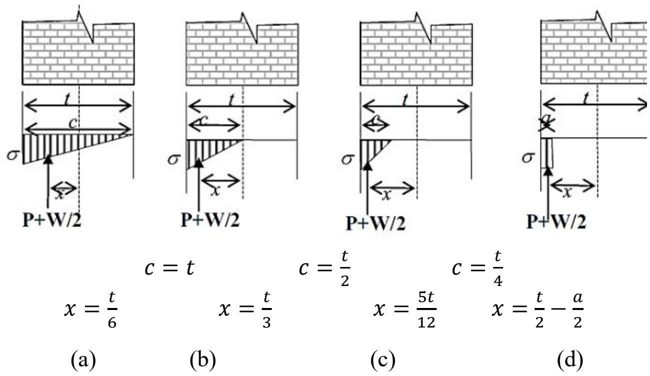



Fig. 7 Compressive stress distribution at mid-height of masonry wall [25]

According to [25], at the elastic limit (onset of cracking) the stress distribution in the masonry walls is as shown in Fig. 7 (a) and the blast pressure and elastic deflection at this stage are calculated by (4) and (5), respectively. At the elastic limit the curvature, $\varphi_e$, of the wall is determined with (8) which expresses the relationship between compressive and tensile strains in the masonry wall.

$$\varphi_e = \frac{\sigma}{Et} = \frac{2(P+\frac{W}{2})}{Et^2} \tag{8}$$

When the crack has grown to the centerline of the wall (Fig. 7 (b)), curvature can be approximated by (9) and expressed to have a curvature of four times the elastic curvature.

$$\Phi = \frac{\sigma}{Et} = \frac{8(P+\frac{W}{2})}{Et^2} = 4\varphi_e \tag{9}$$

Paulay and Priestly [25] report 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 Fig. 7 (b) can be expressed as:

$$\Delta = \Delta_e \tag{10}$$

Thus, the response of the masonry wall can be represented as a ratio, $\beta$, of the elastic displacement (5)

$$\beta = \frac{\Delta}{\Delta_e} \tag{11}$$

For different values of $\beta$ the pressure p and the mid-height deflection of the masonry wall can be determined [15] and used to develop the resistance function of the wall. Fig. 8 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 [15], [25].

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.

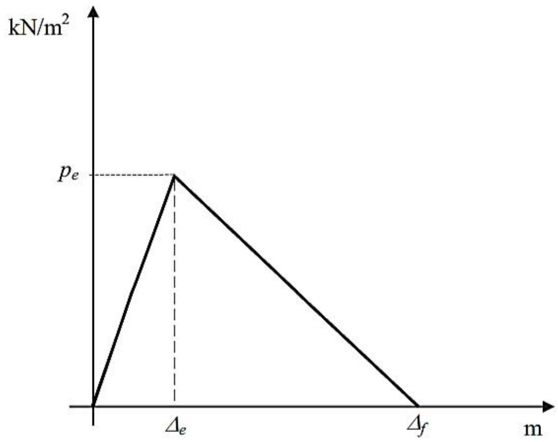



Fig. 8 Resistance function of masonry wall under lateral loading

In modeling the out-of-plane response of a masonry wall to blast loading, the important part is the representation of the structural mass and the pressure-displacement relationship or resistance function [26], [27]. 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 (1) is often solved by direct numerical integration. The Newmark method is a very versatile method used for solving the SDOF equation of motion [15]. 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.

## VI. Recommendations 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 are 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 constructions are non-uniform and consist 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 [19], especially in walls with different wythe thickness, material properties, or coursing.

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 [25] 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 (Fig. 9 (a)). This assumption could be true for walls where the response of two adjacent stories is 180° out of phase [25]. 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 [15], [28], 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 (Fig. 9). 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 (Figs. 9 (c) and (d)).

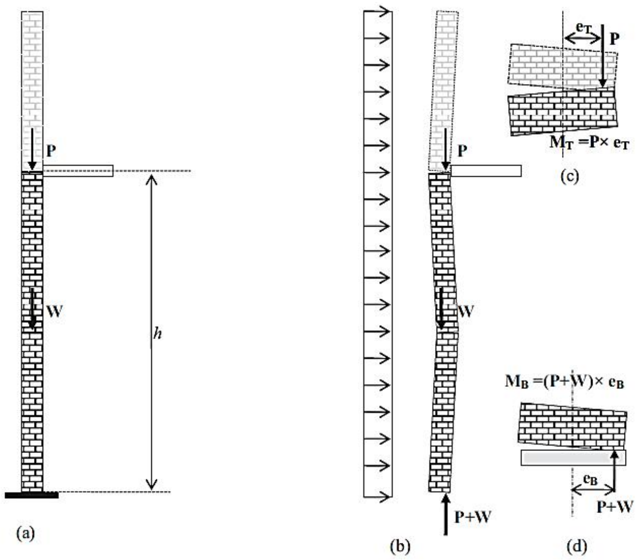



Fig. 9 Support moment of the historic masonry wall

The historic masonry walls investigated 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 (6). 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 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 of multi-wythe walls, especially double-wythe with rubble core infill, in the vibratory rocking mode. Felice [6] 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 paper 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.

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