BLAST RESPONSE OF REINFORCED CONCRETE BLOCK MASONRY SHEAR WALLS

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ABSTRACT

In the past two years, blast resistant design has been introduced in the USA and Canada through ASCE 59-11 “Blast Protection of Buildings” and CSA S850-12 “Design and Assessment of Buildings Subject to Blast Loads”. The introduction of these new design standards has led to a situation where the behaviour of reinforced masonry shear wall systems, designed and detailed essentially for in-plane loading, would need to be quantified under such extreme out-of-plane loading. The current study focuses on experimentally assessing the performance of masonry shear wall building components under large-scale explosions using third-scale specimens. The effect of the explosion level on the stability of the wall, and the possibility of partial or complete progressive collapse, as a result of the explosion, was quantified in terms of the residual wall mid-height deflection. In general, the results show that reinforced masonry shear wall systems can withstand relatively high blast load levels with minor damage. However, with more damage, designers should consider the overall system-level performance (such as the role of the floor diaphragm, load re-distribution and other perpendicular walls) on stabilizing the damaged wall and preventing progressive collapse.

KEYWORDS: blast loads, concrete masonry, reinforced masonry blast response, third-scale testing

INTRODUCTION

Recent events such as the Oklahoma City bombing in 1995 have seen an increased emphasis on the design of structures to accommodate the accidental or non-accidental effects of blast loading. Due to the threat that this type of loading imposes to structures, the development of standards to analyze and design for this type of loading condition has come to the forefront.

The current Canadian and American standards for the design of blast resistant structures (CSA S850-12 “Design and Assessment of Buildings Subject to Blast Loads” and ASCE 59-11 “Blast Protection of Buildings”) have only been introduced within the past two years. With the introduction of these codes, structural masonry walls that were primarily detailed and subject to in-plane loading must now be analyzed for their out-of-plane capacity in order to determine a level of protection for the structural system.

To quantify the different levels of protection, an experimental program was introduced to investigate the levels of damage imposed by blast loading and the potential of blast induced partial or complete progressive collapse. This study aims to further increase the knowledge base that has been laid out in the current design codes through the design and detailing of four third scale masonry walls having different levels of protection and out-of-plane resistance.

EXPERIMENTAL PROGRAM

Four third-scale reinforced concrete masonry walls were constructed and tested with free-field blast loads, which were produced using various weights of Pentex-D explosives. All walls were constructed at the Canadian Masonry Design Centre (CMDC) in Mississauga, Ontario by qualified masons having prior experience in the construction of third-scale masonry walls. The walls were constructed using a properly scaled version of the standard 190mm stretcher block. Each wall was approximately 7.5 blocks wide (1000mm) by fifteen courses high (1000mm), as shown in Figure 1. All the walls, which represent typical sections of a masonry shear wall, were constructed with a running bond and built to common North American construction standards. At the base and top of each wall, a C127 x 47.8 steel section was placed, which aided in the transportation of the walls, in addition to providing the necessary hinged-hinged wall boundary conditions.

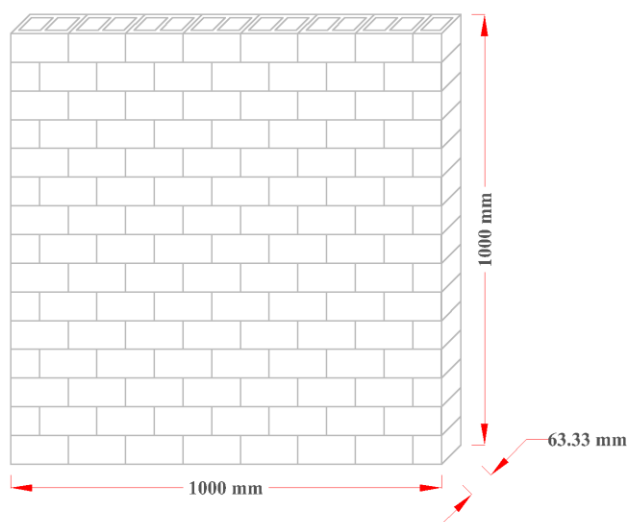


Figure 1: Wall Dimensions

MATERIALS

During construction, Type-S mortar was used and the compressive strength of each mortar batch was determined through the casting of three 50.8mm cubes. All mortar tests were performed in accordance with CSA A179-04. The average flow of the mortar was approximately 127.8%, with a coefficient of variance (COV) of 4.7%. A total of 15 mortar cubes were tested and the average compressive strength was found to be 30.3MPa, with a COV of 5.2%. For each mortar batch, 2 prisms 4 courses high were constructed with the same construction standards as those used during the construction of the walls. A total of 8 prisms were tested to determine the compressive strength as specified by CSA S304.1. The average compressive strength and elastic modulus were 20.8MPa and 11811.9MPa, respectively. The corresponding COV of the specimens was 9.6% and 9.8% respectively.

Three different types of steel reinforcement were used during construction. Deformed D7 (45mm^2 average area) and D4 (26mm^2 average area) bars were used as vertical reinforcement in the specimens. Smooth W1.7 bars (average area of 11mm^2) were used as horizontal reinforcement and hooked at the end to accommodate the outermost vertical reinforcement. The D7 deformed steel bars were found to have an average yield strength of 483.9 MPa with a COV of 4.15%. The D4 bars tested had an average yield strength of 477.6 MPa and a COV of 0.99%.

TEST MATRIX

Shown in Table 1, the walls were split into two separate groups. Group M1 consisted of walls that had each cell fully grouted and fully reinforced with D4 bars. The second group, group M2, consisted of walls that had each cell fully grouted and fully reinforced with D7 bars. Both groups had a single W1.7 bar as horizontal reinforcement at every course. The reinforcing layout is shown in Figure 2. Each wall was only subjected to one blast load. The purpose of these groups was to quantify the amount of damage, assess the performance of walls subjected to blast load, and monitor stability, based on the reinforcement in each wall.

Table 1: Test Matrix

Group	Wall	Grouting	Vertical Reinforcement		Horizontal Reinforcement		Stand-off Distance	Charge Weight (TNT eqv.)
			Number and Size	ρ_V	Number and Spacing	ρ_H		
M1	M11	Fully Grouted	15 D7 Bars (15 X 45mm²)	1.07%	15 W1.7 Bars every 63.3mm	0.26%	5m	11kg
	M12						5m	27.5kg
M2	M21	Fully Grouted	15 D4 Bars (15 X 26mm²)	0.62%	15 W1.7 Bars every 63.3mm	0.26%	5m	11kg
	M22						5m	27.5kg

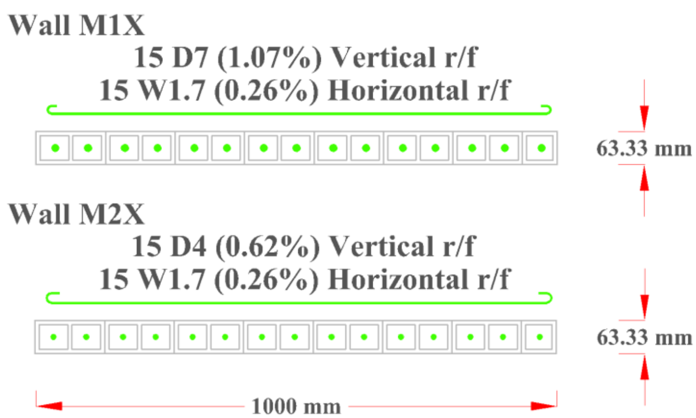


Figure 2: Steel Layout

TEST-SETUP

In order to counteract the blast wave clearing effect that occurs during free field explosions, which would significantly alter the results of the tests, the walls were enclosed in a reaction frame that prevented the pressure wave from propagating around the edges of the wall.

The reaction frame consisted of two vertical components. The first vertical component had 6 HSS sections, 101.6 x 152.4 x 11.1mm, welded together to provide support for the wall while being tested. The reaction supports for the steel channels where provided by two pieces of 50.8mm diameter cylindrical solid steel sections, which were welded to the middle of both the top and bottom HSS sections resulting in hinged boundary conditions. The second vertical component had an additional 6 HSS 101.6 x 152.4 x 11.1mm sections welded together at the rear of the structure to provide support for the instrumentation during the testing of the specimen. The entire frame was enclosed with 6.4mm thick steel plates, which counteracted the blast wave clearing effects as well as simulated the interior conditions on the inside of the wall.

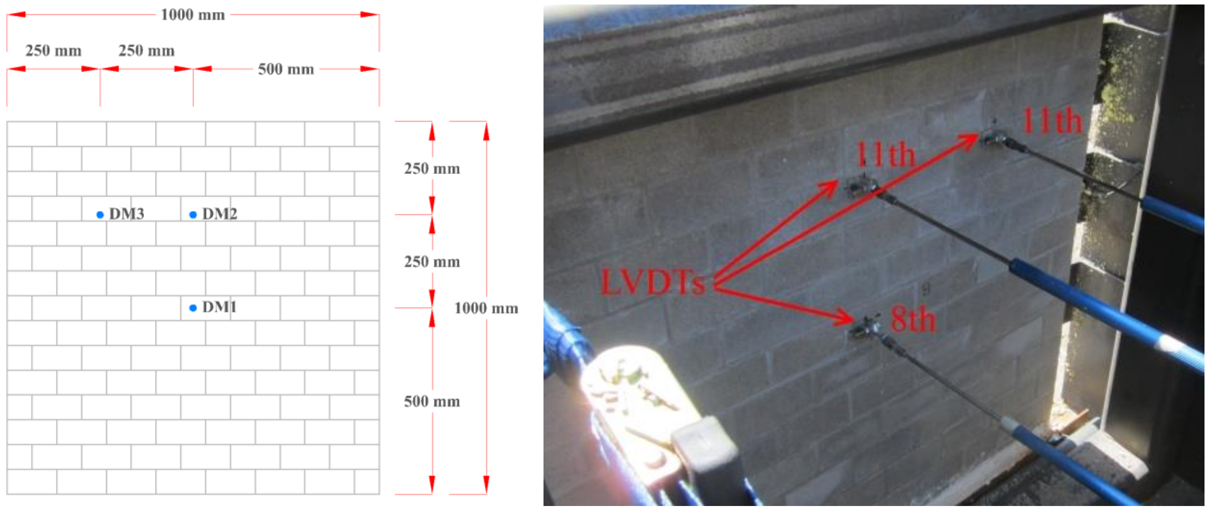
In order to increase the total surface area and to create a more uniform blast wave across the height and width of the wall, 6.4mm thick steel wing walls were attached to the frame on both sides of the wall and the hinged lid. The hinged lid was bolted down during testing, but was opened prior to testing in order to access and attach the instrumentation to the back of the wall. The test setup can be seen below in Figure 3.

(a) **(b)**

Figure 3: Test Setup: (a) Reaction frame pre setup; (b) Reaction frame post setup

INSTRUMENTATION

Three LVDTs were installed on each wall in order to record the horizontal displacements. Typical values of the displacements from these LVDTs are shown further in the study. These LVDTs were placed at critical locations, two along the centre-line of the wall, at mid-height (DM1, 8th course) and ¾ height (DM2, 11th course), and an additional LVDT placed at the ¾ height, ¼ width (DM3, 11th course). The locations of the LVDTs are shown below in Figure **4**.

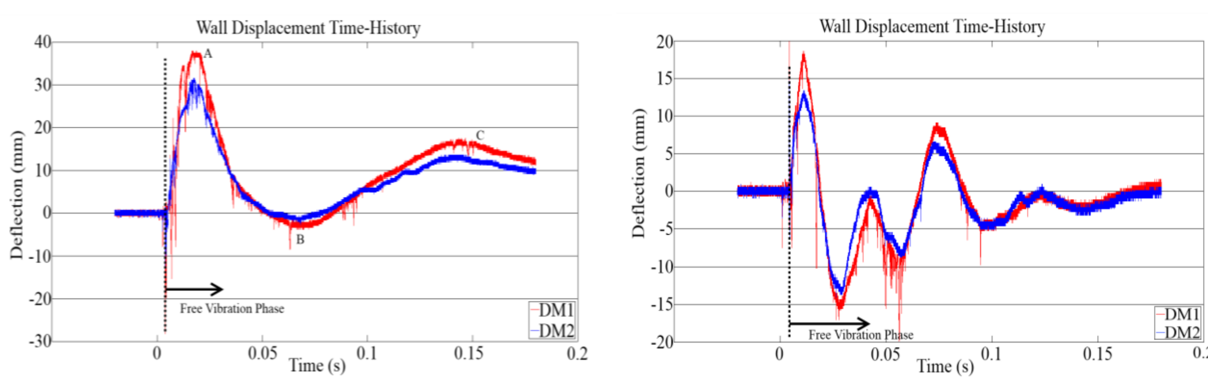


(a) **(b)**

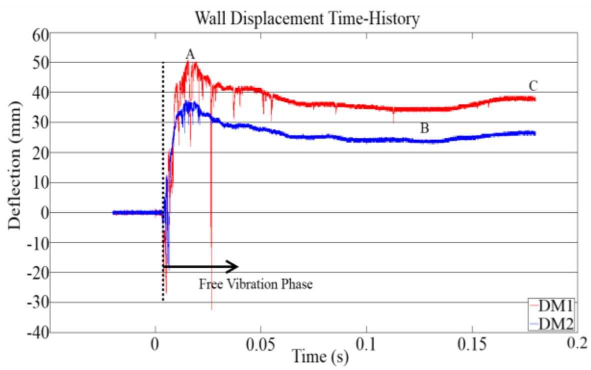
Figure 4: LVDT Locations: (a) Elevation; (b) Interior Instrumentations

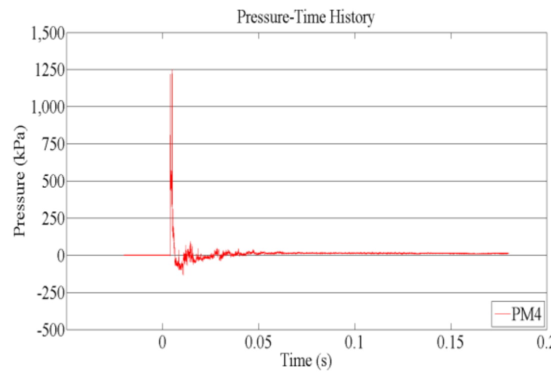
DISPLACEMENT RESPONSE AND FAILURE MODES

Sample displacement-time histories occurring at the 8th and 11th courses of Walls M21 and M11 are shown below in Figure 5(a) and (b), while the displacement time-history for wall M12 is shown in Figure 5(c). No data was recorded for wall M22 due to the blowout failure that the wall experienced. The positive values indicated on the graph are displacements inwards towards the rear of the frame as the blast occurs. Point A on both response curves represents the peak displacement that the specimens incurred during the blast event. Point B on both response curves represents the peak outward displacement due to the rebound of the wall after the initial pulse of the blast load. Finally, point C on both curves represents the permanent deflection of both specimens. Both DM2 and DM3 provide fairly consistent results between the two LVDTs, meaning that the specimen is exhibiting one-way bending because the displacement is not changing across the length of the wall, only the height. Examining the results between DM2 and DM3 to DM1 for the entire displacement-time history, it is seen that there is a non-linear variation between the results for the 8th and 11th courses. In addition, it is seen that the displacement of the courses are in phase with each other, showing that negligible debilitating damage has occurred in the wall because of the reinforcement in the walls. An example of a pressure time-history of the blasts that the specimens were subjected to is shown in Figure 5(d)



(a) (b)

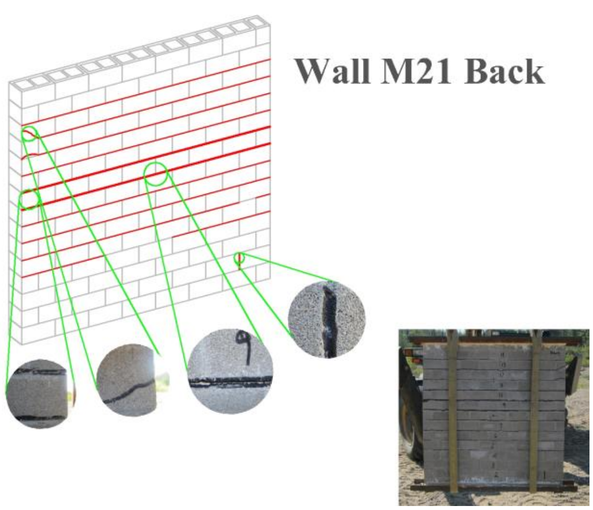


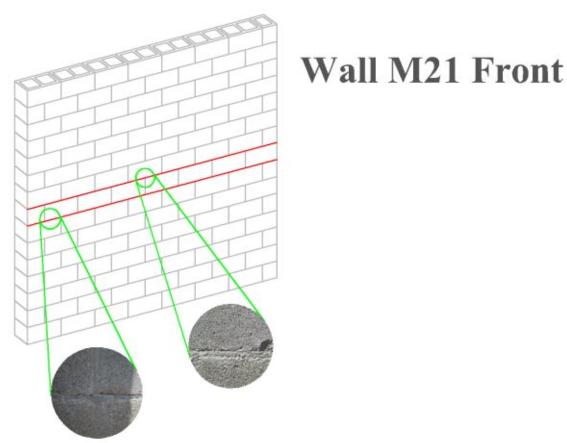


(c) (d)

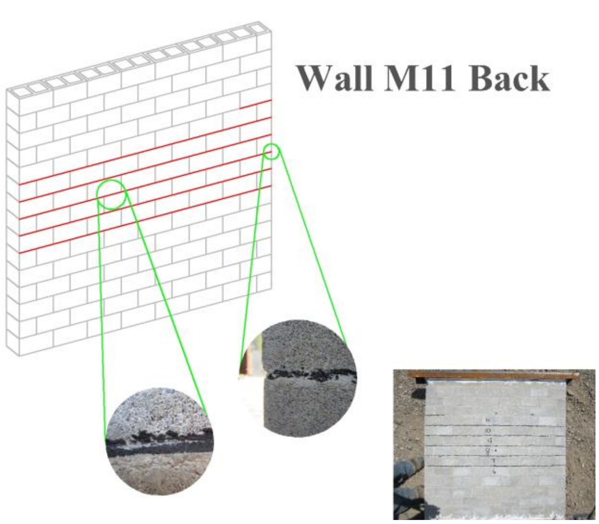
Figure 5: Displacements at the 8th and 11th courses (a) Wall M21; (b) Wall M11; (c) Wall M12; and (d) Pressure time-history for shot #07

Samples of the post-blast damage that occurred are shown in Figure 6(a), (b) and (c). Seen in Figure 6(d) is the typical flexural buckling that is occurred in the specimens following the detonation of the explosives. Because specimen M11 was significantly over-reinforced, no damage was noted at the front of the wall. However, at the back of the wall, bed joint cracks were found to have occurred at every course from the 7th to the 11th. This relates to the reduced amount of displacement specimen M12 underwent in comparison to specimen M22. M22 began to show signs of hairline bed joint cracks at the front of the wall, in addition to the bed joint cracks extending from the 5th course to the 14th course. With the higher reinforcing ratio, specimen M12 exhibited better performance based on both the displacement-time history as well as less visual damage than specimen M22.





(a) (b)



(c)

(d)

Figure 6: Post-Blast Observations (a) Back of Wall M21 after Shot 8; (b) Front of Wall M21 after Shot 8; (c) Back of wall M11 after Shot 7; (d) Post Blast Rotation of Wall M11

CONCLUSIONS

The recent implementation of the standards for blast protection of buildings in Canada (CSA S850-12 “Design and Assessment of Buildings Subjected to Blast Load”) and USA (ASCE 59-11 “Blast Protection of Buildings”) has focused the design of structural walls to certain levels of protection and different criteria for the performance of the walls. In both standards, reinforced masonry is shown to be able to withstand a large amount of flexural damage and still be within reasonable response limits. However, this does not quantify the stability of the wall given the damage or the possibility of partial or complete collapse of the wall. In this study, third scale reinforced masonry walls were experimentally exposed to free field explosions to determine the response and stability of different reinforcement configurations. Given the results both experimentally and analytically, the governing strength for out-of-plane resistance for heavily reinforced masonry walls is based on the flexural resistance, as opposed to the wall shear resistance. This is shown as the vertical reinforcement dominates the behaviour of the specimens. In addition, the deflected shape of the wall at the peak positive displacement prior to the plastic hinge forming at the mid-height of the wall is similar to a uniformly distributed load across the height of the wall.

ACKNOWLEDGEMENTS

This research was facilitated with funding provided by McMaster University Centre for Effective Design of Structures (CEDS), funded through the Ontario Research and Development Challenge Fund (ORDCF), the Natural Sciences and Engineering Research Council of Canada (NSERC), the Canadian Concrete Masonry Producers Association (CCMPA), and the Canada Masonry Design Centre (CMDC). The provision of mason time by Ontario Masonry Contractors Association (OMCA) is gratefully acknowledged. The authors are very grateful to the members of the Canadian Explosives Research Laboratory (CERL) who conducted the field blast tests and to the Canadian Forces for providing the range where the tests were conducted.

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