# RESPONSE OF ONE-WAY REINFORCED MASONRY WALLS TO BLAST LOADING

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## ABSTRACT

This paper focuses on estimating the damage levels and evaluating the out-of-plane behaviour of fully-grouted reinforced masonry shear walls when subjected to blast loading. Three third-scale reinforced concrete masonry walls with typical dimensions of 1m x 1m, which were designed based on the Canadian Standards CSA S304.1-04, were tested using different weights of live explosives. The vertical reinforcement ratio was the same for all three studied specimens. However, three different explosive charges were used to experimentally evaluate the three wall performance and damage levels. In general, the results show that reinforced masonry walls, even with low reinforcement ratios, can withstand substantial blast load levels with minor damage. The test results are expected to contribute to the growing masonry blast performance database that will facilitate possible changes to the current blast resistant construction standards CSA S850-12. These findings are also expected to significantly influence the development of masonry design approaches under blast loading, while maintaining the advantage of simplicity and cost effectiveness of such a construction system.

**KEYWORDS**: blast loading, blast scaling, experimental results, flexural resistance, out-of-plane capacity, reinforced concrete masonry, shear walls

## INTRODUCTION

In recent years, significant damage to structures due to accidental or deliberate explosions has created a requirement where several types of civilian structures must be designed to withstand this type of extreme dynamic loading. Due to the complex nature and the resulting devastating damage from blast loading, research efforts have been made during the past decade to develop blast resistance guidelines, such as the U.S.A. UFC 3-340-02 [2], for military purposes, and more recently, the CSA S850-12 [3] and the ASCE/SEI 59-11 [4], for civilian applications.

Figure 1. shows the typical pressure-time profile captured from an ideal explosion after a detonation of point source charge. The main characteristics of this pressure-time wave are the peak incident pressure "$P_{so}$", and the duration of blast event, which is a function of the charge weight, type, shape and standoff distance. Furthermore, the reflected pressure wave develops

from the reflection of the incident pressure wave, as illustrated in Fig. 1. In general, the reflected peak pressure is higher than the incident peak pressure and it depends on several factors related to the surrounding atmosphere, such as humidity, temperature and ground slope.

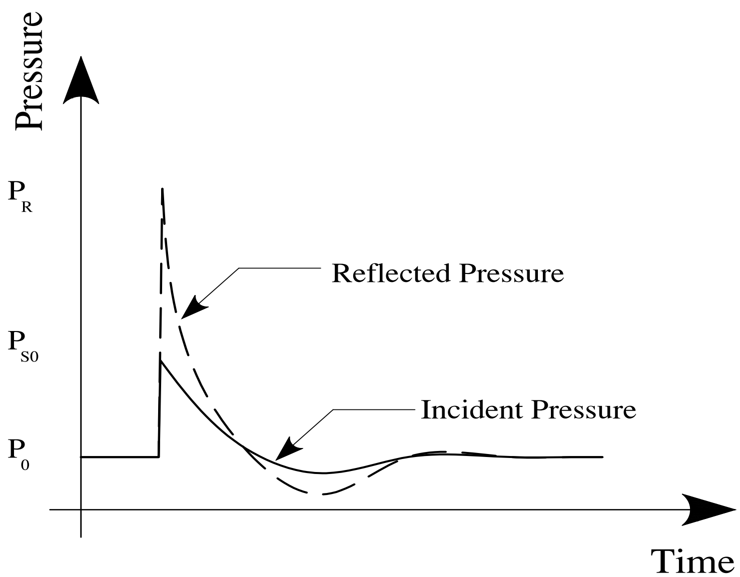



**Fig. 1: Pressure-Time variation for a free-air burst**

In order to facilitate a better understanding of blast phenomena and its impact on surrounding structures, it is essential to compare data obtained from carefully instrumented arena tests and the corresponding predicted from empirical methods for predicting blast effects on structures. Therefore, the experimental results obtained in this study are expected to contribute to better understanding of masonry walls subjected to blast loading.

Scaling is a cost-effective approach especially in the area field tests conducted to evaluate the performance of structures under blast actions. Scaling can refer to either structural or charge scaling. Structural scaling is utilized to create a replica model, in which the dimensions of the prototype are scaled in addition to the possible use a similar material. On the other hand, Hopkinson-Cranz or "cube root" scaling is the most common scaling rule for charge scaling. Using this scaling rule, a self-similar blast wave can be reproduced by keeping the same scaled-distance "*Z*" (given in Eq. 1) for the same type of blast source and surrounding atmospheric conditions [4, and 5].

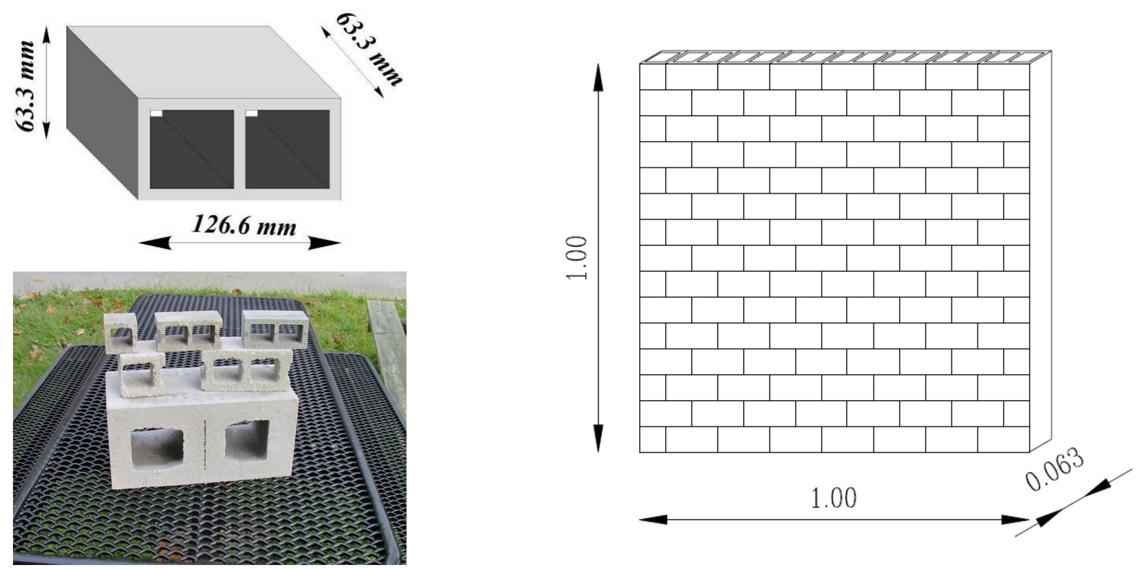
$$Z = R / \sqrt[3]{E} \qquad (1)$$

where *R* is the standoff distance from the detonation source, and *E* is total released energy of the explosion. Another form can be used by replacing the explosion energy "*E*" by the charge weight "*W*" as follow:

$$Z = R / \sqrt[3]{W} \qquad (2)$$

**EXPERIMENTAL PROGRAM**

The experimental program adopted in this study was designed to evaluate the flexural out-of-plane behaviour of fully-grouted masonry shear walls under blast loading. In this regard, three third-scale fully-grouted masonry walls with typical dimensions of 1m x1m and a vertical reinforcement ratio of 0.33 %, were constructed by professional masons. The walls were subsequently under free field explosive using three different charge weights of: 5, 10, and 25 kg. All walls were built using third-scale concrete blocks with specified dimensions (63.3 mm depth x 63.3 mm width x 126.6 mm length), as shown in Fig. 2(a). Each specimen consists of 15 courses high, seven and a half blocks in each course, as shown in Fig 2(b), all constructed with a running bond following common construction practice in North America.




**(a)** **(b)**

**Fig. 2: (a) Third-scale concrete block dimensions, and Fig. 2(b) typical wall dimensions in meters**

In order to ensure the quality of the grouting process, the masonry walls were built in two stages (i.e. to allow for low-lift grout). Following this approach, the first eight courses were laid and grouted first then the remaining seven courses were built and grouted. The first stage of grout did not exceed the mid-height of the $8^{th}$ course, to reduce the possibility of creating a weak surface at the shear key, thus creating a shear key between the $8^{th}$ and $9^{th}$ courses.

**Material Properties**

A third true scale replica of the standards 190 mm hollow concrete block (190 mm depth x 190 mm width x 390 mm length) was used for the construction of the walls. According to the standards of concrete masonry units (CSA A165), eighteen blocks were selected randomly and tested to obtain their compressive strength. The average compressive strength and the corresponding coefficient of variation (*COV*) are 20.26 MPa, and 17.17%, respectively. These values are calculated based on net block area of 4,789 mm$^2$.

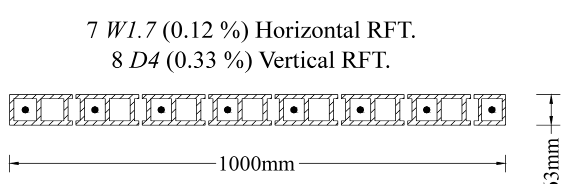
Type-S mortar with an average flow of 125% was used in all the batches and proportioned by weight for more quality control. In accordance to the CSA A179-04 [10], three

51 mm cubes were cast for each batch, and the average compressive strength and *COV* were 17.26 MPa, and 17.05 %, respectively. An average grout compressive strength of 16.75 MPa, with a corresponding *COV* 18.5%, was determined from compression tests carried out on grout cylinders (150 mm diameter, and 300 mm height). In order to estimate the compressive strength as well as the ultimate compressive strain of the masonry walls in accordance with CSA S304.1-04 [8], compressive tests should be carried out on fully grouted prisms. During each construction stage, three four-block high by one block long (270 mm high x 126 mm long x 63 mm thick) grouted masonry prisms were constructed and grouted. The average compressive strength of the grouted prisms was 18.42 MPa (*COV*=17.02%), which is equivalent to 21.18 MPa for standard two-blocks high prisms in accordance to ASTM Standards [11].

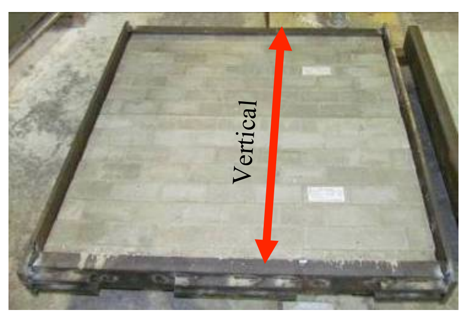
Two different types of reinforcement bars have been used in the walls, *D4* and *W1.7* (with area of 26 and 11 $mm^2$, respectively). By performing tensile test on reinforcement samples, the average yield stress was found to be 488 MPa with a Young's Modulus of 200 GPa.

**Test Matrix**

The main objective of this study is to quantify the performance of shear walls that are not designed for out-of-plane loads and to evaluate the possibility of progressive collapse due to partial or complete shear wall failure. It is often not practical to conduct full-scale explosive tests to evaluate the structural response to blast loads, due to the large amount of explosive charge required, specimen transportation and handling issues, cost, etc. In this regard, testing third scale masonry walls provides an attractive alternative due to the significantly reduced charge weights, transportation costs and wall handling complexities in the field. Three masonry walls with the same horizontal and vertical reinforcement ratios and the same properties, as illustrated in Fig. 3 (a) are studied under three different charge weights. *D4* bars, which are the one-third scale equivalent of full-scale *15M*, were used in the three specimens as vertical reinforcement, while *W1.7* bars, which are equivalent to full-scale *10M* bars, were used as horizontal reinforcement.




(a)




(b)

**Fig. 3: Specimens characteristics: (a) cross section of the test walls, and (b) fixed-fixed boundary conditions**

This group of walls, as shown in Fig. 3(b), consists of three one-third scale concrete masonry walls that represent external shear walls subjected to a blast event in the out-of-plane direction. In order to simulate the one-way out-of-plane behaviour for these types of walls, fixed-fixed boundary conditions were selected. To simulate such boundary conditions each wall was built over steel C-Channel 101.6 mm (4 in) height by 40.2 mm (1.6 in) flange width. The vertical reinforcement *D4* bars have been welded to both the upper and lower previously mentioned C-Channel sections. In addition, 12.5 mm (1/2 in) steel plates have been welded at each side of the wall to C-Channel steel section.

The test matrix adopted in this paper was designed to simulate an external explosion that might occur from the detonation of a car bomb. Assuming that the standoff distance between the detonation source and the target masonry wall is 15 m, and the weight of charge varies from 100 to 1,000 kg. For such scenario, the scaling laws that relate the prototype to the model considering a geometrical scale factor of $S_L$=1/3, and same material properties, manufacturing details and supports are given in Table 1 [12].

**Table 1: Scale factors for blast loading**

| Group | Quantity | Dimension | Model |
|---|---|---|---|
| Geometry | Linear dimension, *l* | $L$ | 1/3 |
| | Area, *A* | $L^2$ | 1/9 |
| | Displacement, $\delta$ | $L$ | 1/3 |
| Loading | Pressure, *P* | $ML^{-1} S^{-2}$ | 1 |
| | Force, *F* | $M L S^{-2}$ | 1/9 |
| | Time, *t* | $T$ | 1/3 |
| | Impulse, *I* | $ML S^{-1}$ | 1/3 |
| | Velocity, *v* | $LT^{-1}$ | 1 |

Based on this, the scaled charge should be placed at 5 m (corresponding to 15 m stand-off distance in the full-scale scenario), and charge weights must be selected to maintain the proper scale distance. Table 2 represents the test matrix of the reinforced masonry walls, while the explosive charge used in this test is *Pentex-D*, which has an equivalent *TNT* factor of 1.2. The three charge weights were selected to induce three damage levels: minor, moderate and sever.

**Table 2: Experimental Matrix of free field blast tests**

| Shot No. | Specimen | Charge Size *Pentex-D* (kg) | Equivalent *TNT* (kg) | Scale Distance (m/kg$^{1/3}$) |
|---|---|---|---|---|
| 1 | *W1* | 5.0 | 6.0 | 2.75 |
| 2 | *W2* | 10.0 | 12.0 | 2.18 |
| 3 | *W3* | 25.0 | 30.0 | 1.61 |

**Test Set-up and Instrumentation**

In order to reduce the effect of the *clearing* phenomena, which causes a non-uniform pressure distribution (drop of the reflected wave pressure) at the edges of the specimen as the finite boundaries of the target allow part of the wave to propagate around the edges, a built-up steel frame was built and used in testing the masonry walls. The main skeleton of the steel frame consists of two components, the reaction structure and the LVDT mounting system, as illustrated in Fig. 4. Both components are welded to the 6.4 mm (1/4 in) base steel plate. The right, left, and back faces are closed using gusset plates that were welded to the built-up steel frame. Additionally, the upper side of the frame was covered using a rotatable steel door, which permitted access to the LVDTs. To overcome the clearing phenomena, three steel plates (wing walls) were bolted to the test frame, as illustrated in Fig 4(b).

(a)


(b)

**Fig. 4: Testing frame (a) main skeleton, and (b) final shape after installing wing walls**

Three displacement transducers were used to monitor the out-of-plane displacement behaviour, as shown in Fig. 5. A pre-tensioned string-potentiometer was placed at the centre of each wall specimen, which had a maximum stroke length 360 mm (14.2 in). In addition, two 300 mm stroke Linear Variable Displacement Transducers (LVDT), LVDT #1 and LVDT #2, were placed at the horizontal quarter points of each wall specimen. Finally, three piezoelectric pressure transducers were used; one was placed inside the test frame to measure the inner pressure behind the wall, while the other two transducers were attached to the front of the frame.

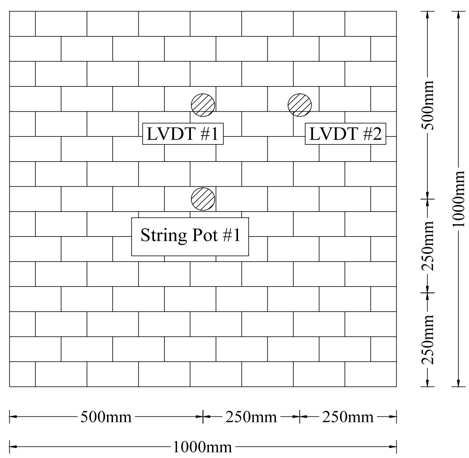



**Fig. 5: Instrumentation layout for wall specimens**

## TEST RESULTS

These results include visual observations, such as crack patterns and failure modes, in addition to presenting the recorded data like pressure profiles and peak deformations. It was noticed that the behaviour of all walls was dominated by a flexural response as evident by the observed level of horizontal cracking along the bed joints and in the concrete blocks along wall height.

### Post-blast Observations

The tested masonry walls were carefully checked visually after each shot. It was noticed that cracks propagated mainly within bed joints after the detonation of 5.0 kg *Pentex-D* charge. However, for the 10.0 kg charge test cracks were observed in both mortar and concrete masonry blocks. No compression spalling was observed to have occurred on the front face of Wall *W1*, while tension cracks were observed along the rear face of the masonry wall after the first shot (Table 2), as illustrated in Fig. 6, while a tension crack was only observed above the 7th course.

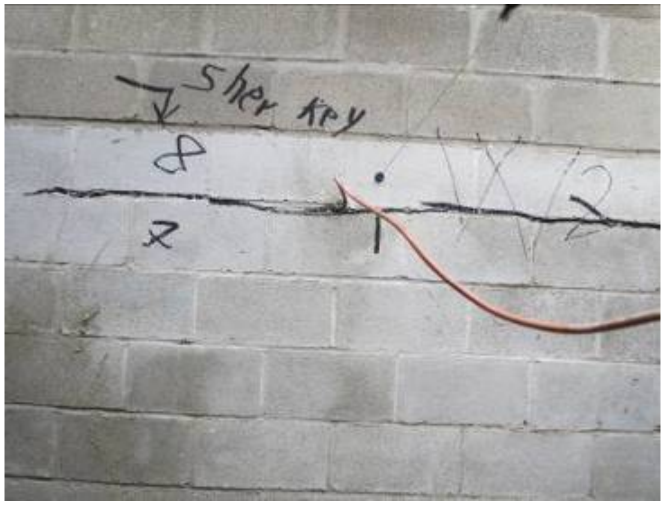



**Fig. 6: Tension crack pattern of first masonry specimen after the detonation of 5.0 kg *Pentex-D* charge**

For wall *W2*, which was subjected to twice the weight of charge as wall *W1*, a majority of tension cracks occurred on the rear face of specimen along the mortar joints, with some minor cracks propagating through the concrete blocks, as shown in Fig. 7. Wall *W2* cracked between the 6th and 8th courses, compared to the smaller size cracks observed above the 7th course in wall *W1*.

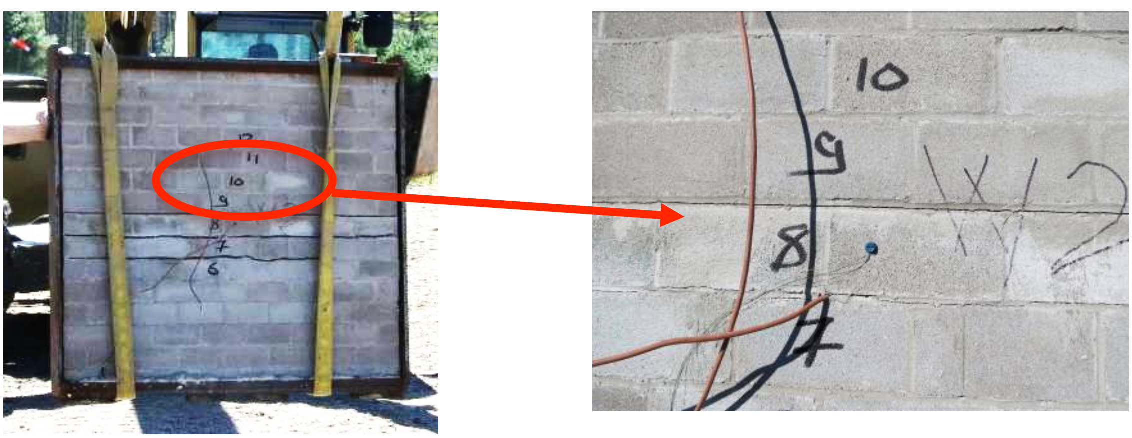



**Fig. 7: Tension crack pattern of second masonry specimen after the second shot**

The deformations experienced by walls *W1* and *W2* were significant, however, it is likely that they are repairable, and the possibility of progressive collapse is considered to be unlikely. Conversely, the remaining wall, *W3*, was split into two separate halves under the action of the third shot, as shown in Fig. 8. Two failure edges above the 8th course and between the top of the masonry wall and the steel C-Section in case of Wall *W3* were observed. As shown in Fig. 8, necking of the steel bar was observed at the failure edges, which indicates that the failure occurred as a result of high tensile stresses.

**Fig. 8: Sample of compression spallings, tensions cracks, and bar fracture after the third shot**

**Pressure-Time History**

In order to predict the air blast parameters a set of empirical relations, which are plotted in the form of charts, can be used [2]. Software, such as ConWep [13] is also available to determine blast parameters. Some investigators, such as Baker [6], have suggested various relations to represent the pressure time history, such as the modified Friedlander equation:

$$P(t) = P_{\max}\left(1 - \frac{t - t_a}{t_0}\right)e^{\left(-\gamma\frac{t-t_a}{t_0}\right)} \tag{3}$$

In this study, the pressure time history is determined experimentally based on free-field tests. Fig. 9 shows two typical pressure-time histories captured by piezo-electric pressure transducers after detonation of the 5 kg *Pentex-D* charge at a standoff distance 5 m.

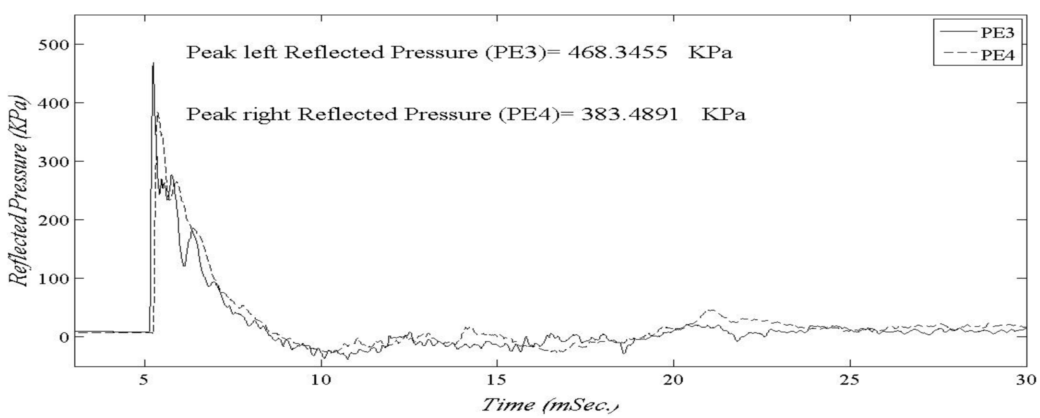



**Fig. 9: Reflected pressure-time history developed from 5 kg explosive charge**

Two piezo-electric pressure transducers were attached to the steel frame at the mid-height level of specimens, PE3 and PE4. A nonlinear curve fitting technique based on the modified Friedlander equation (Equation 3) was used to remove noise from the recorded measurements. The average computed reflected pressures using ConWep [13] for the 5, 10 and 25 kg are

416.17, 807.31, and 2032.37 kPa, respectively, and the corresponding impulses are 449.33, 745.25 and 1,474.17 kPa.msec, respectively. The average recorded reflected pressures and impulses for the three shots are presented in Table 3. The difference between the theoretical and actual records was found to remain below 10%.

**Table 3: Recorded reflected peak-pressures and impulses**

| Shot No. | Charge Size *Pentex-D (kg)* | Equivalent *TNT* (*kg*) | Scale Distance ($m/kg^{1/3}$) | Average Reflected Pressure (*kPa*) | Average Reflected Impulse (*kPa. msec*) |
|---|---|---|---|---|---|
| 1 | 5.0 | 6.0 | 2.75 | 372.35 | 430.62 |
| 2 | 10.0 | 12.0 | 2.18 | 869.35 | 685.46 |
| 3 | 25.0 | 30.0 | 1.61 | 2095.93 | 1423.94 |

**Displacement Response History**

The behaviour of masonry specimens under different charge weights has been captured experimentally using a pre-tensioned string-potentiometer at wall centre and two LVDTs at ¾ the height of masonry walls, LVDT #1 and LVDT #2 as shown previously in Fig. 5. In general, a sinusoidal wave is observed from the displacement-data recorded, and the assumed one-way out-of-plane behaviour is verified by the similarity of the two profiles obtained from LVDT data, as illustrated in Fig. 10. The peak deformations captured at the mid-height of the masonry shear walls by the pre-tensioned string pot were 14 mm for Wall *W1* and and 29 mm for Wall *W2*.

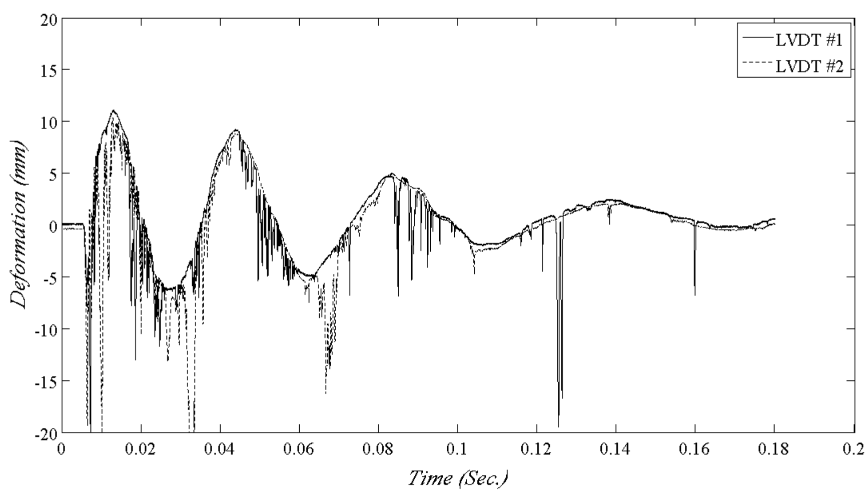



**Fig. 10: Displacement-time profiles at ¾ wall height from 5 kg explosive charge**

**CONCLUSIONS**

Most existing buildings are not designed to withstand blast load. Therefore, masonry veneers and external shear walls of high explosive risk must be evaluated considering the effect of such extreme loading. The current study focuses on evaluating the performance of fully-grouted reinforced masonry shear walls, with low reinforcement ratio, subjected to out-of-plane blast loading. The three third-scale concrete block walls reported in this study had a vertical reinforcement ratio of 0.33% and were tested under different levels of live explosions.

In view of the presented study, although not designed to resist out-of-plane loading, these reinforced masonry walls were able to resist substantial blast loads. Low vertical reinforcement ratio can provide significant resistance to medium explosion levels, with peak reflected pressure and impulse up to 800 kPa and 750 kPa.msec, respectively, with minor damage and fragmentation. In addition, boundary conditions affected the size and propagation of cracks. Therefore, it is recommended that a special reinforcing detail at the lower and upper edges of the walls be developed in order to avoid zero moment resistance at wall ends. By comparing the recorded air blast parameters, such as pressure and impulse, to the predicted values using the ConWep software, it is acceptable to predict such parameters using ConWep or the code empirical relations, which are plotted in the form of charts.

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