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Blast loading response of reinforced concrete panels reinforced with externally bonded GFRP laminates

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Abstract

The behavior of reinforced concrete panels, or slabs, retrofitted with glass fiber reinforced polymer (GFRP) composite, and subjected to blast load is investigated. Eight $1000 \times 1000 \times 70$ mm panels were made of 40 MPa concrete and reinforced with top and bottom steel meshes. Five of the panels were used as control while the remaining four were retrofitted with adhesively bonded 500 mm wide GFRP laminate strips on both faces, one in each direction parallel to the panel edges. The panels were subjected to blast loads generated by the detonation of either 22.4 kg or 33.4 kg ANFO explosive charge located at a 3-m standoff. Blast wave characteristics, including incident and reflected pressures and impulses, as well as panel central deflection and strain in steel and on concrete/FRP surfaces were measured. The post-blast damage and mode of failure of each panel was observed, and those panels that were not completely damaged by the blast were subsequently statically tested to find their residual strength. It was determined that overall the GFRP retrofitted panels performed better than the companion control panels while one retrofitted panel experienced severe damage and could not be tested statically after the blast. The latter finding is consistent with previous reports which have shown that at relatively close range the blast pressure due to nominally similar charges and standoff distance can vary significantly, thus producing different levels of damage.

Keywords: A. Glass fibres; A. Laminates; B. Strength; B. Stress transfer; Blast

1. Introduction

Fiber reinforced polymers are currently used in civil applications to extend the structural integrity of deteriorated structures or to increase the load bearing capacity of deficient structures. In addition, seismic retrofitting of shear walls, columns, beams and structural joints is an important field of application for externally bonded FRP wraps. FRP composites have high strength, high-energy absorption and lightweight, which means that a small quantity of FRP can significantly increase the resistance of a structural member to resist tensile loads and bending moments, without significantly increasing its mass and stiffness.

In the design of structures to resist blast loads, there are two important considerations, prevention of catastrophic failure or progressive collapse and reduction of projectiles due to fragmentation. Control of deflection, crack width, vibration and other serviceability related criteria are not normally deemed essential. The flexible nature of FRP laminates, their thinness and lightweight and the ease with which they can be bonded to most surfaces render them attractive because they do not alter in any significant way the original mass, geometry and appearance of a structure. The addition of mass to a structure generally increases its blast resistance, but it also increases its dead load; the latter may be undesirable due to the increased sustained load on the columns and foundation before and after the blast

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event. In blast resistance design both high strength and ductility are important; FRP retrofit normally increases the strength substantially but at the expense of some reduction in ductility of flexural members. This trade-off between strength and ductility and its effect on blast resistance of retrofitted structures need investigation.

Another important factor that need be studied is the multiple reflections that the pressure wave will experience due to the various interfaces that exist among the FRP laminas and the concrete surfaces in a retrofitted flexural member. While intuitively it may be reasonable to assume that retrofitting of flexural members, particularly slabs and beams, may improve their blast resistance, practically one must test actual structural members under realistic blast loads to verify the extent of the improvement that can be realized.

Extensive testing has been performed by military research establishments to determine the response of concrete structures to various types and levels of explosion, but much of the key findings are not in the public domain. In the case of the response of FRP retrofitted concrete structures, there is little detailed information available in the open literature. There is ample information available in design manuals such as the TM 5-1300 [18] and other similar documents, but these are intended mainly for designing hardened military facilities rather than evaluating existing civilian structures.

Previous studies on concrete structures subjected to high strain rates have revealed some important differences between the behavior of structures subjected to relatively low strain rate dynamic loads, such seismic forces, and those experiencing very high strain rates generated by blast. Toutlemonde and Boulay [20,21] investigated the high strain rate dynamic behavior of plain and reinforced concrete circular slabs using a compressed air shock tube. They reported that significant reduction in the pulse duration resulted in a change in the failure mode from pure bending, manifested by radial cracks, to a mixed bending-shear failure, which involved radial and circumferential cracks near the supports. Furthermore, the cracks were reported to be mostly parallel to the steel reinforcing bars and the type of failure was strongly influenced by both the peak pressure and the duration of the pulse. From the point of FRP strengthening, this change in failure mode is significant because it is not evident whether FRP laminates bonded to the surface of a slab can necessarily enhance its direct shear strength significantly.

A somewhat relevant study on the effectiveness of FRP as blast enhancement reinforcement was performed by investigators at Wilfred Baker Engineering [22]. They tested $2.5 \times 2.5\ m^2$ concrete masonry unit (CMU) walls with and without openings and the walls were retrofitted with externally bonded Kevlar fabric on their two faces to enhance their out-of-plane bending resistance. In the non-retrofitted control specimens they noted bending failure at mid-height of the wall and/or shear failure near the supports. It was reported that the retrofit increased the resistance of the walls against both modes of failure. El-Domiaty [9] reported a similar behavior for un-reinforced CMU walls retrofitted with either near surface mounted FRP bars or surface bonded FRP strips.

Muszynski [13,14] reported on the results of air blast tests on two reinforced concrete cubical structures some elements of which were retrofitted with carbon fiber or aramid (Kevlar) composites. Also three $2.8 \times 2.6\ m^2$ masonry walls were retrofitted with the same type of FRP. The structures were subjected to air blast from the detonation of 830 kg TNT at a standoff of 14.5 m. It was reported that compared to the un-retrofitted or control specimens, the reinforced concrete elements retrofitted with carbon and aramid, respectively, had 25% and 40% smaller maximum displacement. In the case of the masonry walls, the aramid was reported to be more effective because it resulted in more energy absorption, a reduction of 30% in maximum displacement and a more ductile failure mode compared to the control specimens.

Crawford et al. [5] used the computer program DYNA-3D to study the effectiveness of FRP jacketing in increasing the blast resistance of reinforced concrete columns. They concluded that jacketing will increase the column resistance and reduce its displacement by 50 to 60%. The blast loads used in the analysis were assumed to have originated from detonation of high explosives ranging from 682 to 13654 kg of TNT at standoff distances ranging from approximately 3.0 to 12.2 m. Similarly, [10] reported the benefits of retrofitting reinforced concrete buildings using FRP.

A number of other studies have also been performed to examine the suitability of FRP as a blast resistance enhancing material. Ross et al. [16] tested simply supported beams at 4.57 m standoff using a 110.6 kg ANFO charge and based on qualitative assessment of the results reported the effectiveness of CFRP in increasing the blast resistance of reinforced concrete beams. Crawford et al. [6–8] performed a number of numerical simulations and field testing on reinforced concrete structural members and indicated the benefits of using FRP to increase the blast resistance of reinforced concrete and masonry structures. Mossallam and Mossallam [12] performed a series of simulations using dynamic analysis to study the response of reinforced concrete slabs retrofitted with CFRP to blast loading. They validated their modeling with laboratory tests using statically applied pressure on slabs similar to those analyzed dynamically. The static test results revealed an almost 200% increase in the load carrying capacity of the retrofitted slabs compared to the companion unretrofitted slabs. However, since the response of members to blast loads in the impulse regime is influenced mainly by their ductility rather than their strength, it is difficult to extrapolate from the static to the dynamic strength and response. Nevertheless, such preliminary analyses are useful to gain some insight into the nature of the response and its sensitivity to certain structural parameters.

These studies, and other similar ones that have not been cited here, have generally indicated the benefits of FRP in enhancing the blast resistance of concrete and masonry structures. However, the information and results provided by the studies are not sufficiently detailed to develop rational design methods for blast design of FRP retrofitted reinforced concrete elements. To develop such methods, more systematic, well-documented and detailed data are needed, including quantitative assessment of damage, measured pressure and impulse as well as concrete, steel and FRP deformations. The objective of this study is to experimentally investigate the extent to which GFRP laminates, adhesively bonded to the surface of reinforced concrete panels, can improve the panels' resistance to blast loads at relatively close standoff. The dynamic response and the nature of the damage are of particular interest, with special attention to the damage mechanisms, failure modes, and the bond of FRP with the concrete surfaces. The information that is provided can also be used to check the accuracy of numerical software for blast analysis of structures.

2. Blast load characterization

The principal effects of blast due to conventional explosives on structures are the imposition of a transient pressure pulse of high amplitude and relatively small duration compared to the fundamental period of the structure [1]. An explosion of a high-energy material such as trinitrotoluene (TNT), or mixtures of ammonium nitrate and fuel oil (ANFO) causes a very rapid release of energy that compresses and pushes the surrounding air out and away from the detonation source to form a blast wave. At a certain distance from the explosion center, regardless of the source, all blast waves have almost the same shape [11]. The blast pressure variation with time has generally two phases, the phase with pressure above ambient, called positive phase, and the one with pressure below ambient, termed negative phase. In structural evaluation, blast wave parameters associated with the positive phase, such as shock front velocity, peak overpressure and its duration, blast impulse, i.e. the area under the pressure-time curve, and dynamic pressure are of primary importance [2]. The dynamic pressure or blast wind is a function of the velocity of the air particles behind the shock front and is similar to conventional wind pressure.

The structure response to blast loading is governed by many factors, including charge mass and standoff distance, structure size and orientation, proximity of the target to other structures or to significant land features. Due to the pulse nature of the blast pressure and its short duration, the dynamic response of structures is mainly governed by the ratio of the positive phase duration, t_o, to the natural period of vibration of the target, T, referred to as duration ratio, t_o/T [3]. In case of gas explosions, where t_o/T is larger than 0.1, the structural response is dominated by the peak overpressure, but the structural behavior of above ground structures, where t_o/T is often less than 0.1, structural damage is deemed to be due to the reflected impulse [15]. Although the preceding effects of blast on structures are well-known, close range blasts may significantly deviate from the more uniform conditions that develop at some distance from ground zero. Therefore, from the point of smaller scale tests, the choice of the appropriate charge size and standoff would be an important consideration to obtain useful data.

Finally, it has been observed [17] that when a blast wave impinges on a small-size target, reflected pressure is created on its front face. The reflected pressure does not persist because the finite boundaries of the target allow a part of the blast wave to propagate around the edges. This phenomenon is referred to as blast wave clearing, which is typically associated with a pressure drop. Due to the time it takes the reflected pressure to drop (clearing time), and the lack of adequate knowledge about the precise value of the reduced pressure, there is often uncertainty about the blast wave parameters in such cases. Accordingly, in assessing blast loads on small-size targets, the clearing effect need be considered. The present test set-up was designed to minimize the effects of the latter phenomenon on the relatively small test panels and to capture important data about the behavior of control and retrofitted reinforced panels in order to asses the benefits of the retrofit in terms of the increased blast resistance of panels [19].

3. Test program

3.1. Test specimens

Eight $1000 \times 1000 \times 70$ mm reinforced concrete panels, Fig. 1, were doubly reinforced with welded steel mesh of

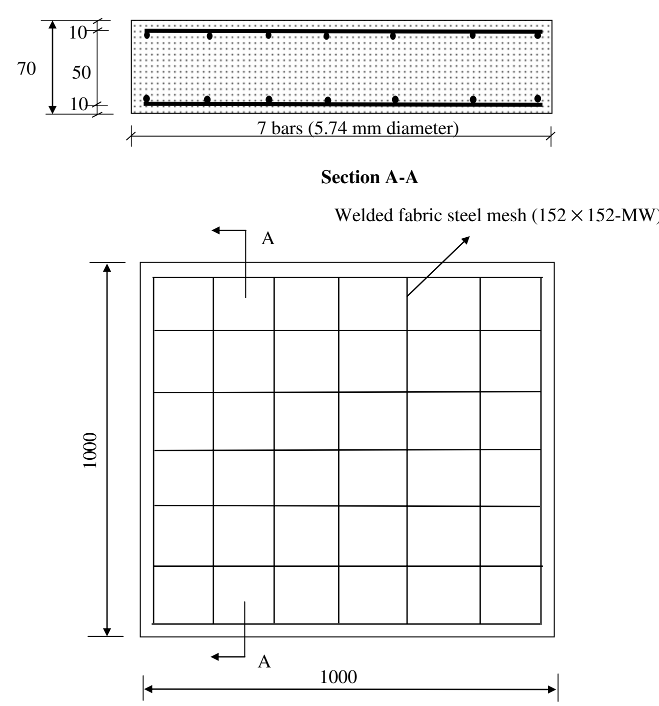


Fig. 1. Test specimen geometry and reinforcement details (all dimensions in mm).

designation MW 25.8, which has bar cross-sectional area of 25.8 mm^2, mass per unit area of 2.91 kg/m^2 and center-to-center spacing of 152 mm in each direction. The bar yield stress and ultimate strength are 480 MPa and 600 MPa, respectively. The concrete had an average 28 day compressive strength of 40 MPa, with its average strength at the age of testing the panels being 42 MPa.

The panels are divided into two groups, with specimens in each group being nominally identical. The panels in Group CS, i.e. CS1 to CS4, were as-built and were used as control specimens while those in Group GSS, GSS1 to GSS4, were retrofitted on each face with two laminates of GFRP arranged in a crucifix form, with each laminate being parallel to one of the edges of the panel, Fig. 2. Each laminate was 500 mm wide and it covered the middle half of the panel. The laminate is basically a unidirectional composite fabric with E-glass fibers in the main reinforcing direction and a small amount of aramid fibers perpendicular to the main direction. According to the manufacturer, the laminate has density of 2100 kg/m^3, thickness of 1.3 mm, tensile strength, elastic modulus and ultimate elongation of 580 MPa, 27.5 GPa and 2.1%, respectively, in the main direction. In the orthogonal direction, its tensile strength is 26.1 GPa. The epoxy used to bond the composite to the concrete has tensile strength, elastic modulus and maximum elongation of 54 MPa, 3.1 GPa and 5%, respectively.

As illustrated in Fig. 2, the GFRP covered only the middle half of each panel. The reinforcement scheme was based on the assumption that each panel would act as simply supported slab with the uniform blast pressure causing maximum moment in the central region; curling effects near the corners were ignored and was thought to be adequately resisted by the internal reinforcement. Furthermore, the support system was designed to prevent complete uplift of the panels. To simulate more closely practical situations, the laminates were not wrapped around the edges of the panel because it was assumed that in practice such wrapping in slabs and panels may not be possible.

3.2. Test set-up

If a charge is to be detonated in the air above a test panel, one way of avoiding the clearing effect is to place the panel in a horizontal position and fix it to a box buried in the ground. Such a set-up eliminates the clearing effect referred to earlier because the ground surface acts as a target of infinite dimensions with the test specimen being part of this unlimited surface. Accordingly, as shown in Fig. 3, a steel box was constructed and to its top was attached a steel frame made of four welded angels. The frame was provided with a clamping system to prevent the test specimens from uplifting. In essence the test specimen served as the lid of the box. The box also housed electrical cables and instrumentation equipment.

The test set-up was commenced by burying the steel box in the ground, with its top being level with the ground surface. Two cables connected the explosion source to the instrumentation bunker located 150 m away. Rubber pads of the same width and length as the steel angle legs were placed between the angles and the test specimen bottom to ensure uniform support conditions. Similar pads were used between the test specimens and the clamps used to prevent the uplift. Subsequently, the tripod holding the charge was centered above the center of the panel and the charge was hung with a wire. The explosive used was ANFO, comprising 5.7% fuel oil and 94.3% ammonium nitrate, shaped into an approximately spherical form. The explosive energy of ANFO is 3717 kJ/kg, which is 82% of the energy of one kilogram of TNT. Fig. 4 shows the test specimen in place and the tripod holding the charge.

3.3. Instrumentation

The free field incident pressure was measured by at least two transducers located, at 3.2 and 5.9 m from the center of the test panel. Reflected pressure was measured 3.1 m from the charge center by four transducers located at the mid-length of the four angles holding the panel. An LVDT was placed inside the steel box to measure the panel central displacement.

To measure steel reinforcement and GFRP strains, two 6 mm long electrical resistance strain gauges were attached to the top and two to the bottom steel mesh. One gauge was placed at midspan while the other was located at the 1/3 point of the panel diagonal. In addition, three 30 mm long strain gauges were installed on the top surface and

a b

Fig. 2. Typical test panels: (a) Control specimen; (b) retrofitted specimen.

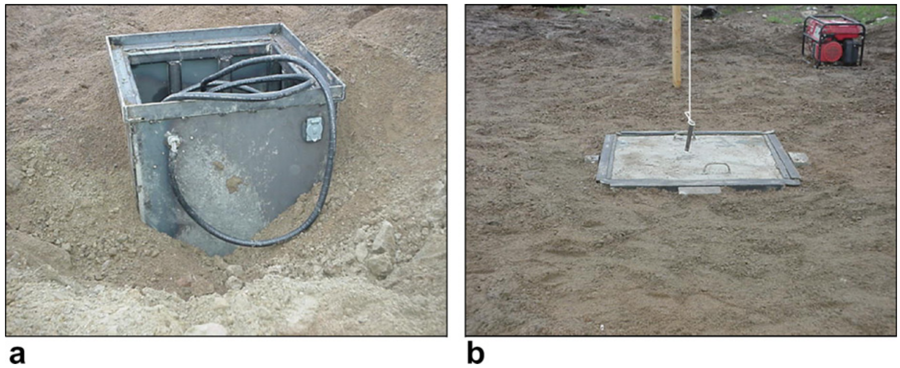


Fig. 3. Preparation steps of the final blast test set-up: (a) Partially buried box supporting the blast panels; (b) typical text panel placed on top of the buried box.

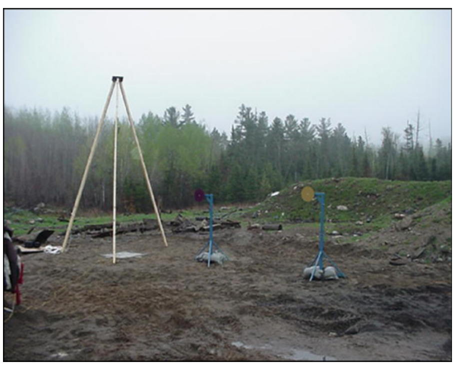


Fig. 4. The test specimen and the tripod holding the explosive charge.

two strain gauges on the bottom surface of each panel. The surface strain gauges were glued to the concrete in the control panels and to the GFRP in the retrofitted panels. Fig. 5 schematically shows the location of these strain gauges.

3.4. Test procedure

The blast tests were conducted on a Canadian Armed Forces Base. The following procedure was typically followed for each test. (1) The test panel was placed in top of the box and the instrumentation was connected. (2) The ground around the specimen was leveled and compacted. (3) The wooden tripod supporting the explosive charge was erected, ensuring a standoff distance of 3 m. (4) Locations of the incident pressure gauges from the charge center were measured and recorded. (5) All personnel were evacuated to a safe distance and the explosive charge was initiated at 1.5 km away from ground zero. (6) After the operator called the area clear, the state of the specimen was observed and recorded. Panels CS4 and GSS1 were subjected to the blast load due to the detonation of 22.4 kg of ANFO while the remaining five panels, i.e. CS2, CS3, GSS2, GSS3 and GSS4 were subjected to the load generated by the detonation of 33.4 kg of ANFO. In each case the distance from the center of the charge to the center of the test panel was 3.0 m.

4. Test results

Typical data recorded for each test specimen are shown in Table 1. The data for specimen CS1 are not shown because it was basically used as a trial sample for establishing the level of the charge and for checking the operation of the instrumentation. It was subjected to two successive

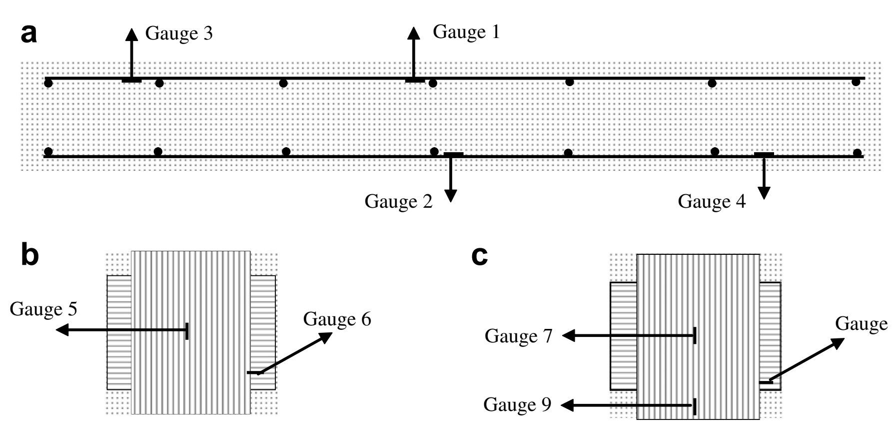


Fig. 5. Locations of the reinforcement and concrete/GFRP surface strain gauges: (a) Reinforcement strain gauges (6 mm length); (b) strain gauges on the bottom surface (30 mm); (c) strain gauges on the top surface (30 mm).

Table 1
Typical test results obtained during the blast and post-blast test of the panel

Panel	Reflected pressure (kPa)		Average reflected impulse (kPa ms)	Maximum strain				Maximum central deflection (mm)	Post-blast residual static strength (kN)	Post-blast observed damage
				Reinforcing steel		Concrete surface/ FRP layer				
	Max.	Ave.		Top	Bottom	Top	Bottom			
CS4	4823	3842	571	NC	4375	NC	NC	8.33	80.02	Moderate
GSS1	4203	3741	1344	1480	6500	NA	10000	10.83	140.05	Light
CS2	5528	5059	1954	1875	2730	5000	1650	13.12	68.00	Heavy
CS3	5712	5507	2412	6350	2900	1700	3800	9.53	93.00	Heavy
GSS2	3838	3050	1819	2000	NC	NC	10000	14.58	–	Severe
GSS3	5215	4995	1492	20000	17000	14000	750	P	112.51	Moderate
GSS4	5030	4933	2239	11000	16000	8750	16000	9.10	85.04	Heavy

Panel	Experimental measurements			CONWEP Predictions		Maximum strain				Maximum central deflection (mm)	Post-blast residual static strength (kN)	Post-blast observed damage
	Reflected pressure (kPa)		Reflected impulse (kPa ms)	Reflected pressure (kPa)	Reflected impulse (kPa ms)	Reinforcing steel		Concrete surface/FRP layer				
	Max.	Ave.	Ave.	Ave.	Ave.	Top	Bottom	Top	Bottom			
CS4	4823	3842	571	3323.7	1247	NC	4375	NC	NC	8.33	80.02	Moderate
GSS1	4203	3741	1344	3323.7	1247	1480	6500	NA	10000	10.83	140.05	Light
CS2	5528	5059	1954	4937.6	1686	1875	2730	5000	1650	13.12	68.00	Heavy
CS3	5712	5507	2412	4937.6	1686	6350	2900	1700	3800	9.53	93.00	Heavy
GSS2	3838	3050	1819	4937.6	1686	2000	NC	NC	10000	14.58	–	Severe
GSS3	5215	4995	1492	4937.6	1686	20000	17000	14000	750	P	112.51	Moderate
GSS4	5030	4933	2239	4937.6	1686	11000	16000	8750	16000	9.10	85.04	Heavy

charges and therefore its results could not be compared with those of the remaining specimens because they were each subjected to a single shot.

Table 1 gives the largest reflected pressure measured for each panel as well as the average of the maximum pressures measured by the four transducers located at the middle of its four sides. In some cases, some of the instrumentation did not capture any data, designated as NC in Table 1, therefore, the average values shown are based on the data captured by either two, three or four functioning transducers.

The maximum concrete and steel strains are also given in the table as well as the maximum central deflection of each panel. Fig. 6 shows typical captured incident and reflected pressure profiles for the control panel CS3 and its companion retrofitted panel GSS3. The symbols FF1 and FF2 in the legend refer to the incident pressure transducers while PR1, PR2, etc. refer to the reflected pressure transducers. The average positive phase duration of the latter panels were determined to be 3.17 ms and 1.68 ms. If we use the blast prediction software CONWEP [4] to calculate the positive phase duration for 33.4 kg of ANFO detonated at a 3.1 m standoff (note that the reflected pressure transducers were located at a 3.1 m standoff), we would obtain 5.5 ms, which is significantly greater than the 1.68 ms measured for panel GSS3. If the two panels were assumed to have the same fundamental period, the difference between the two positive phase durations is expected to have a significant impact on their response. It should be stated that the conventional weapons software, known as CONWEP, developed by the US Army, uses empirical expressions fitted to actual blast experiments to calculate air blast parameters, such as pressure, impulse and phase duration.

We observe that the average reflected pressure and impulse of the control panel CS4 and its companion retrofitted panel GSS1 are reasonably close and if we use CONWEP, we would obtain maximum reflected pressure and impulse of 3323 kPa and 1247 kPa ms, respectively, for the 22.4 kg of ANFO detonated at 3.1 m standoff. These values are close to those recorded for panel GSS1 but the measured impulse for panel CS4 seems rather small compared to the calculated value.

The captured reflected pressure and impulse data for the panels subjected to the detonation of a 33.4 kg charge are generally in good agreement with each other albeit the pressure for panel GSS2 and the impulse for panel GSS4 are relatively low compared to the other panels. We observe that the maximum measured pressure in all the control panels is higher than the corresponding pressure in the companion retrofitted panels. This may be due to the presence of the GFRP composite and the lower density of the adhesive layer. On the other hand, there does not appear to be any clear trend with respect to the effect of the GFRP on the reflected impulse.

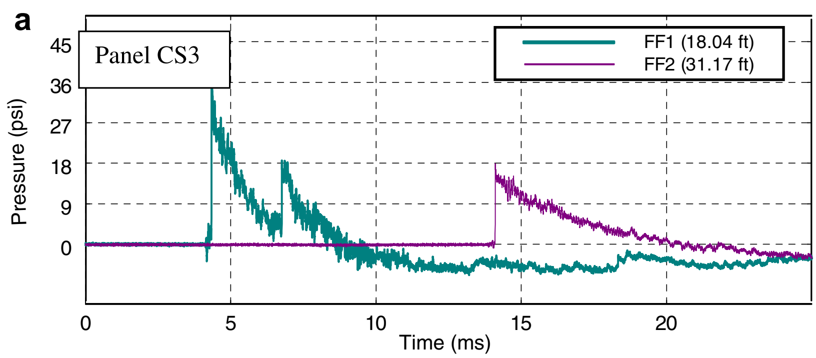
Measured strain values show yielding of the bottom reinforcement in all the panels while the top steel seems to have yielded in some cases. Both the top and bottom FRP layers experienced large strain, generally exceeding 10000 $\mu\varepsilon$, which indicate significant damage and plastic deformations. Concrete surface strains were generally smaller and they varied from 1650 $\mu\varepsilon$ to 5000 $\mu\varepsilon$. Given the fast strain rate, the 5000 $\mu\varepsilon$ measured on the top surface of panel CS2 appears to be quite surprising because high strain rates generally lead to a reduction in ductility. The foregoing strain values generally correlate well with the observed damage in the panels.

The maximum deflection measured in the center of each panel is given in the table. The deflection values vary from 8.33 mm to 14.58 mm. The deflection of the control panel CS4 is less than that of the companion retrofitted panel GSS1 despite the fact that the average maximum reflected pressure acting on CS4 is slightly higher than that acting on GSS1. This is because the maximum deflection is both a function of the maximum pressure and the duration of the positive phase. Assuming the same maximum pressure to be acting on both panels and assuming the two panels to have the same natural period, an increase in the positive phase duration of either panel will increase its maximum deflection. The preceding assumptions are reasonable for the current test panels because their measured maximum pressure values are reasonably close and it is not anticipated that the FRP would significantly alter the natural period of the panels, at least prior to the yielding of the steel. Nevertheless, the synergetic effect of these parameters makes the interpretation of data from blast tests rather challenging.

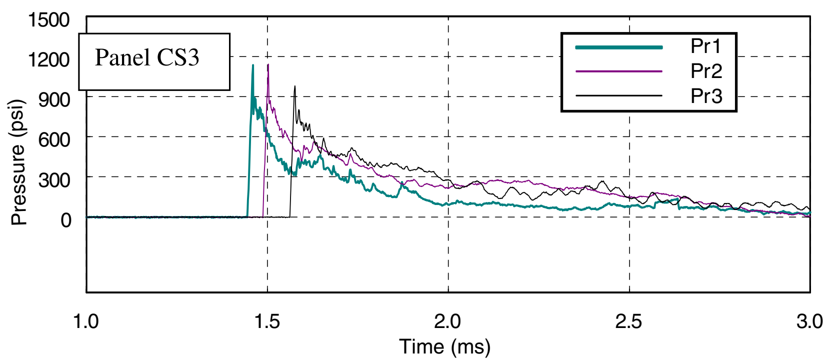
To assess the effectiveness of the GFRP in increasing the resistance of the panels to blast load, the panels were inspected for cracking, spalling and scabbing. In addition, those panels that survived the blast were subsequently statically tested by means of a central patch load to determine their post-blast residual strength. These results are summarized in Table 1.

The post-blast damage is classified as light, moderate, heavy or severe. Light damage implies appearance of hairline cracks on the exposed concrete surfaces with the bond between the FRP layers and the concrete remaining intact. Moderate damage refers to situations when bottom surface cracks of width up to 1.5 mm and minor concrete spalling occurred. Heavy damage means large cracks up to 4 mm wide together with large permanent deflections and heavy concrete spalling. In the case of moderate damage, the FRP experienced minor delamination while in the case of heavy damage fiber rupture and/or local concrete crushing occurred. Finally, severe damage refers to total crushing and failure of the concrete and the complete delamination of the FRP laminates. Fig. 7 shows some of the panels after their exposure to the blast event. Notice the crack pattern in panel CS3 and the inverted shear cracks on the panel sides in Fig. 7a.

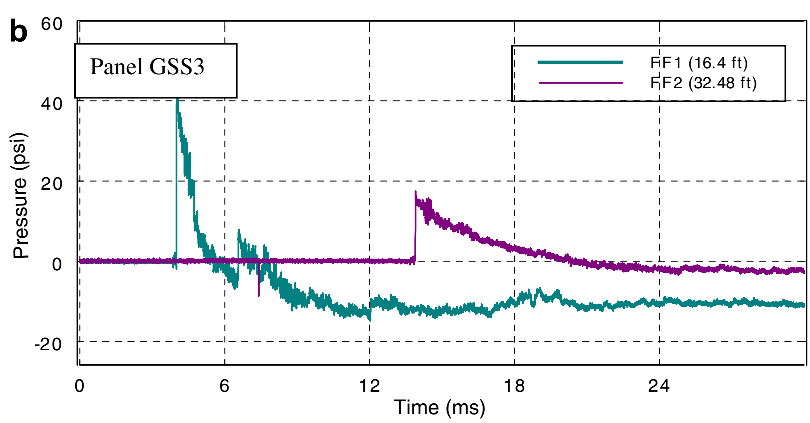
We notice in Table 1 that the retrofitted panels generally suffered less damage than the companion control panels. However, the retrofitted panel GSS2 was completely damaged by the blast to the extent that it could not be tested



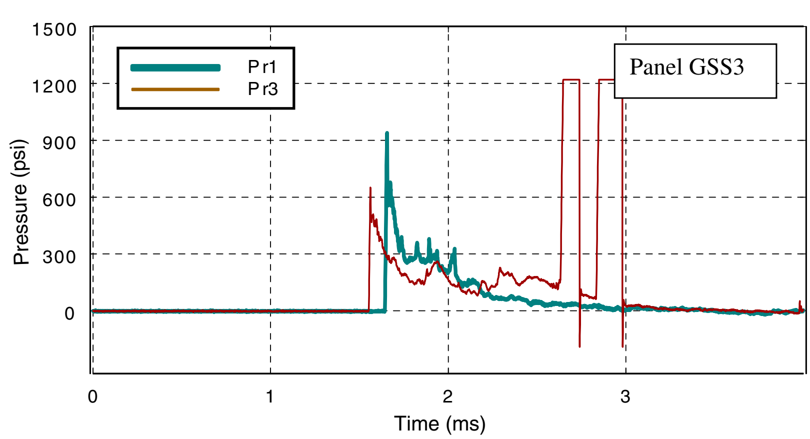
(a): Incident(free field) pressure-timeprofiles)



(b): Reflectedpressure-timeprofiles



(a): Incident (free field) pressure-time profiles



(b): Reflected pressure-time profiles

Fig. 6. Captured incident and reflected pressure profiles generated by the detonation of 33.4 kg ANFO at 3.1 m standoff, (a) Control panel CS3, (b) retrofitted GSS.

Fig. 7. Typical post-blast observed damage in the test panels (a) panel CS3, (b) panel GSS2.

statically, Fig. 7b. On the contrary, the replicate panel GSS3 suffered only moderate damage. Typically, all damaged panels had full depth inverted 45° shear cracks near their supports and on all four sides. These cracks were rather wide and in some cases greater than 4 mm, Fig. 7a and c. The application of FRP did not prevent or mitigate this mode of damage perhaps due to the fact that the FRP did not cover the entire surface of the panels. Based on the observed inclination of the shear cracks, they must have been caused by pressure acting upward on the bottom of the panels. In addition, on the bottom surface of most panels an array of 500 mm long cracks formed a square shape centered on the panel center and then propagated diagonally towards the corners of the panel, Fig. 7a, similar to the yield line pattern for a statically applied central patch load. On the bottom surface, additional minor cracks, which typically followed the reinforcement layout, were also observed.

The post-blast static strength of the panels is given in the penultimate column of Table 1. As mentioned earlier, panel GSS2 was severely damaged and therefore could not be tested statically. From the static test data, it is clear that the retrofitted panel GSS1 performed better than the companion control panel CS4 because the residual strength of the former is almost 75% higher than that of the latter. This may be attributed to the fact that the GFRP did not delaminate during the blast event and therefore was able to contribute to the post-blast strength. Also visual observations indicated lighter damage in the retrofitted panel compared to the control panel.

Under the effect of the larger 33.4 kg charge, it was more difficult to draw definitive conclusions about the effectiveness of the GFRP. While the retrofitted panel GSS3 had a residual strength of 112.5 kN and it suffered moderate damage, at the same time panel GSS2 completely disintegrated while panel GSS4 had 9% less residual strength than the companion control panel CS3. On the other hand, the residual strength of panel CS2 was only 60% of that of GSS3. In an average sense the retrofitted panels had higher residual strength than the companion control panels, but there was no consistent trend. Hence, from this study, at least from the results of the panels subjected to the higher charge, it is difficult to draw definite conclusions about the blast mitigation effectiveness of the GFRP bonded laminates.

5. Discussion of the test method and results

The present testing program and its result indicate that assessing the blast response and resistance of reinforced concrete elements by using actual explosives is a complex task. It is well-known that sometimes minor changes in material properties, test set-up and the surrounding environment could produce significantly different responses at close range. Although in this study replicate specimens were used to assess the effect of such variabilities, based on the quantitative and qualitative results, it is not possible to arrive at general conclusions regarding the effectiveness of GFRP for blast mitigation. Theoretically, the blast resistance of structures that are loaded in the impulse realm can be effectively increased by increasing their ductility rather than their strength [3]. Since the addition of FRP increases the strength of flexural members, but not their ductility, the case for the use of FRP in such cases is not obvious. On the other hand, structures that are loaded in the pressure realm would benefit most from an increase in strength rather than ductility. The loading realm is determined by the ratio of the positive phase duration to the natural period of the structure. In the current testing program for nominally similar blast scenarios, noticeably different positive phase durations were measured.

One of the reasons for the scatter in the results for the larger charge size may be that the selected charge size of 33.4 kg at 3.1 m standoff is too high even for the retrofitted panels to resist. Hence, if the pressure-impulse combination produced by this charge exceeds the resistance of both the control and the retrofitted panels, then it would not be possible to use the results of the test to assess the effectiveness of the GFRP in mitigating blast damage. The results of tests under the smaller charge size indicate that the retrofitted panel performed very well because it suffered only light damage and had a residual strength that was 75% higher than that of the companion control panel. Note that the smaller charge caused noticeable damage in the control panel, including a 2 mm permanent deformation, but the same did not happen in the retrofitted panel.

It may also be noticed that replicate tests using equal charge size and standoff produced significantly different values of impulse and positive phase duration. Since the response of a structural element subjected to blast loads is rather sensitive to the ratio of its natural period to the positive phase duration of the blast load, it is not easy to relate the measured deformations and strains to their theoretically expected values. It would appear that the current standoff distance is too close and therefore the results are significantly affected by the surrounding terrain and other test conditions. In addition, it is well known that the charge density and shape will affect its generated pressure amplitude and phase duration. Consequently, in performing tests using real ammunition, one must carefully control all the test parameters, including those related to the explosive charge and its characteristics. Variations in charge shape and density, ambient temperature and surrounding test environment, despite the use of a constant charge mass and standoff distance, can produce significantly different response in nominally identical test specimens. In the light of these observations, for close-in blasts, application of the results of refined numerical calculations, including detailed computational fluid dynamics and finite element models, to actual blast scenarios should be treated with caution. One way of reducing the variability of the test results may be to use an array of replicate specimens, involving control and retrofitted specimens, arranged in a circular pattern centered on the charge, with the surrounding ground uniformly landscaped and compacted. At least the blast load parameters would be generated by the same explosion, but such a set-up requires a large amount of instrumentations to capture the data for all the test specimens concurrently.

6. Conclusions

The results of the blast tests performed in this study on reinforced concrete panels retrofitted with externally bonded GFRP laminates support the following conclusions:

(1) The reflected blast pressure and impulse measured at the same location during different shots using the same charge size and standoff distance were generally reasonably close, but in some cases significant deviation occurred.
(2) Generally the reflected blast pressure and impulse values calculated using the software CONWEP were in reasonable agreement with experimental data.
(3) To determine the correct charge size and standoff distance, i.e. the scaled distance, for investigating the blast resistance of concrete elements may require several different tests with different scaled distances and a number of replicate specimens in each case.
(4) In the present tests, when control and GFRP retrofitted panels were subjected to a blast load produced by the detonation of 22.4 kg of ANFO at a standoff distance of 3.1 m, or a scaled distance of 1.137 $m/(kg)^{1/3}$, the GFRP retrofitted panel performed significantly better than the control or non-retrofitted panel in resisting the blast load. The post-blast static strength of the retrofitted panel was 75% higher than that of the companion unretrofitted panel.
(5) The performance of replicate retrofitted panels compared to the control panels when subjected to the blast load caused by the detonation of 33.4 kg of ANFO at 3.1 m standoff, or a scaled distance of 0.995 $m/(kg)^{1/3}$, was mixed. In some cases the retrofitted panel performed better than the companion unretrofitted panel while in other cases the opposite occurred. One of the retrofitted panels completely disintegrated while none of the unretrofitted panels suffered such catastrophic damage.
(6) In an average sense the retrofitted panels had higher residual strength than the companion control panels, but there was no consistent trend. Hence, from this study, at least from the results of the panels subjected to the higher charge, it is difficult to draw definite conclusions about the blast mitigation effectiveness of the GFRP bonded laminates.
(7) The results of this study indicate that the GFRP retrofit may not be suitable in every situation and that quantifying its strengthening effects will need more actual blast testing rather than merely theoretical modeling or pseudo-dynamic testing.

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