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ABSTRACT

The paper presents experimental data generated for calibrating finite element models to predict the performance of reinforced concrete panels with a wide range of construction details under blast loading. The specimens were 1.2 m square panels constructed using Normal Weight Concrete (NWC) or Fiber Reinforced Concrete (FRC). FRC consisted of macro-synthetic fibers dispersed in NWC. Five types of panels were tested: NWC panels with steel bars; FRC panels without additional reinforcement; FRC panels with steel bars; NWC panels with glass fiber reinforced polymer (GFRP) bars; and NWC panels reinforced with steel bars and external GFRP laminates on both faces. Each panel type was constructed with three thicknesses: 152 mm, 254 mm, and 356 mm. FRC panels with steel bars had the best performance for new construction. NWC panels reinforced with steel bars and external GFRP laminates on both faces had the best performance for strengthening or rehabilitation of existing structures. The performance of NWC panels with GFRP bars was strongly influenced by the bar spacing. The behavior of the panels is classified in terms of damage using immediate occupancy, life safety, and near collapse performance levels. Preliminary dynamic simulations are compared to the experimental results.

INTRODUCTION

A significant amount of research has been carried out in recent years to improve the blast resistance of new and existing reinforced concrete (RC) structures. In general, blast resistant structures must prevent progressive collapse and catastrophic failure while reducing the penetration of projectiles. The present paper is concerned with the prevention of progressive collapse and catastrophic failure. Recently, steel and synthetic fibers have been used in concrete to reinforce structural elements. Fiber reinforced concrete (FRC) has been used in RC barriers to examine its performance under blast loads (Coughlin et al. 2010). The barriers were constructed with nylon, carbon, or synthetic/steel fibers using various fiber concentrations. Lan et al. (2005) conducted full scale blast experiments on 74 different RC panels. Some of the panels were made from FRC using two types of commercially available steel fibers with concentrations by volume of 0.5%, 1.0%, and 1.5% fiber. The panels with longer fibers performed better in resisting cracking and spalling of concrete due to blast load than shorter fibers.

Several studies have been carried out to improve blast resistance of existing RC elements. Muszynski and Purcell (2003) tested a bidirectional E-glass fabric as a retrofit of RC walls and columns of a structure subjected to a large blast from relatively small standoff distances. They concluded that the pressures caused by the blast should have catastrophically destroyed the structure. The columns failed but the wall remained relatively intact even though it had suffered large displacements. Lawver et al. (2003) tested the blast resistance of RC bridge decks. Four types of bridge decks were tested including a typical reinforced deck, a deck built to current blast resistant standards, a typical deck with carbon fiber reinforced polymer (CFRP) laminates, and a typical deck with glass fiber reinforced polymer (GFRP) laminates. The full scale tests validated simulation models and proved that fiber reinforced polymer (FRP) composite retrofits could perform as well as current blast resistant construction. Silva and Lu (2007) conducted blast tests on one-way RC slabs. The slabs were either covered with CFRP or steel fiber reinforced polymer (SFRP) laminates; one of each slab type was covered only on one side and another was covered on both sides. The slabs that were covered with laminates only on one side were severely damaged by the blast. In comparison, the slabs that were covered with laminates on both sides displayed significant increase in blast resistance. The slabs failed in shear. Razaqpur et al. (2007) conducted a study of RC panels subjected to blast with some panels reinforced with GFRP laminates. The laminates were applied externally in a crucifix form. The study concluded that in general, GFRP reinforced panels performed significantly better in residual strength than the control panel. No studies are known that examine the performance of GFRP bars used as reinforcement in concrete to mitigate the effects of blast.

EXPERIMENTAL RESEARCH

To develop further insight into the performance of various types of concrete panels with different reinforcement schemes, blast tests were carried out to evaluate materials pertaining to new construction as well as rehabilitation of existing RC panels with these variables; 1) panel thickness, 2) type of concrete (in particular normal weight concrete (NWC) and FRC), 3) internal reinforcement type (steel rebar or GFRP bars), 4) internal reinforcement spacing and ratio, and 5) external reinforcement (GFRP composite laminates).

Materials and specimen details

Two types of concrete were used in this project: (i) NWC and (ii) FRC with a macro-synthetic fiber. The FRC had 8.9 kg of polypropylene fibers per cubic meter of concrete, which resulted in 1% of fibers by volume. The fibers were 51 mm long, 0.9 mm in diameter and were added to the concrete during mixing using a minimum mixing time of 5 minutes. The fibers had a unique sinusoidal wavelike shape that increased anchorage to concrete; the fibers had a specific weight of 0.91, a tensile capacity of 338 MPa, and a modulus of elasticity of 3.0 GPa. The average static 28 day compressive strength of NWC was 51 MPa, while that of FRC was 46 MPa. The average static tensile strength of NWC using a split cylinder test was 4.0 MPa and that of FRC 4.3 MPa.

Steel rebars and GFRP reinforcing bars were used in this research. The steel rebars used had a nominal tensile strength of 420 MPa and a modulus of elasticity of 200 GPa. The 16 mm diameter (ϕ16) GFRP bars had a tensile strength of 717 MPa and a modulus of elasticity of 43 GPa for the specific lot of bars used; the tensile strength of ϕ10 GFRP bars was 758 MPa and the elastic modulus was 41 GPa.

Unidirectional glass fabric was adhered to both sides of panels for Type E (Table 1) for the full panel area. The fabric had a tensile strength of 2276 MPa and a modulus of elasticity of 72 GPa. The fabric had a weight of 913 gm/m^2 with a nominal thickness of 0.35 mm. A high-modulus high-strength impregnating two part epoxy was used to attach the GFRP composite fabric to the concrete. Two layers of fabric were applied to each side; the layers were applied perpendicular to each other, one at zero and one at 90 degrees with respect to the horizontal axis. Eighteen panels were tested under blast load; a summary of each panel type is shown in Table 1.

Table 1. Description of panels.

1.2 m x 1.2 m panels		Thickness, Reinforcement and Designation		
Type	Description	152 mm	254 mm	356 mm
A4	Normal Weight Concrete with Steel Rebar	Φ10 @ 305mm A4-6	Φ13 @ 305mm A4-10	N/A
B4	Fiber Reinforced Concrete Only	No Rebar B4-6	No Rebar B4-10	N/A
C4	Fiber reinforced Concrete and Steel Rebar	Φ10 @ 152mm C4-6	Φ13 @ 152mm C4-10	Φ16 @ 152mm C4-14
D4	Normal Weight Concrete and GFRP Rebar	Φ10 @ 152mm D4-6	Φ16 @ 229mm D4-10	Φ16 @ 152mm D4-14
E4	Normal Weight Concrete with Steel Rebar and two layers of GFRP laminate on each face	Φ10 @ 305mm E4-6	Φ13 @ 305mm E4-10	Φ16 @ 305mm E4-14
COND	Control Panels with GFRP Rebar	Φ16 @ 305mm CON-1, CON-2, CON-3, CON-4	N/A	N/A
CONB	Control Panels with Only Fibers	CON - 5	N/A	N/A

Test Setup

Explosives (C4 or ANFO) were used for the blast tests conducted at the National Security Testing Range (NSTR) at the Idaho National Laboratory (INL). The panels were arranged in a square pattern with a centrally located charge, as shown in Fig. 1.

The test specimens were 1.2 m square panels constructed using NWC or FRC. One specimen was placed on each of three sides of the layout, while the fourth side was left open for working space. The layout provided an equal standoff distance of 1 m from the center of the explosive to the face of each panel. The panels were placed on the ground and large concrete blocks were placed on each side of the specimens to provide support, as shown in Fig. 2. The charge was placed on a table at the center of the test layout with the explosive at the mid-height of the specimens. A mylar break screen was used as the trigger for the data acquisition system. A series of nine blast tests were carried out. Table 2 summarizes the following information for each panel: panel designation, standoff distance, weight of charge expressed as a mass of equivalent TNT, and internal reinforcement ratio ρ.

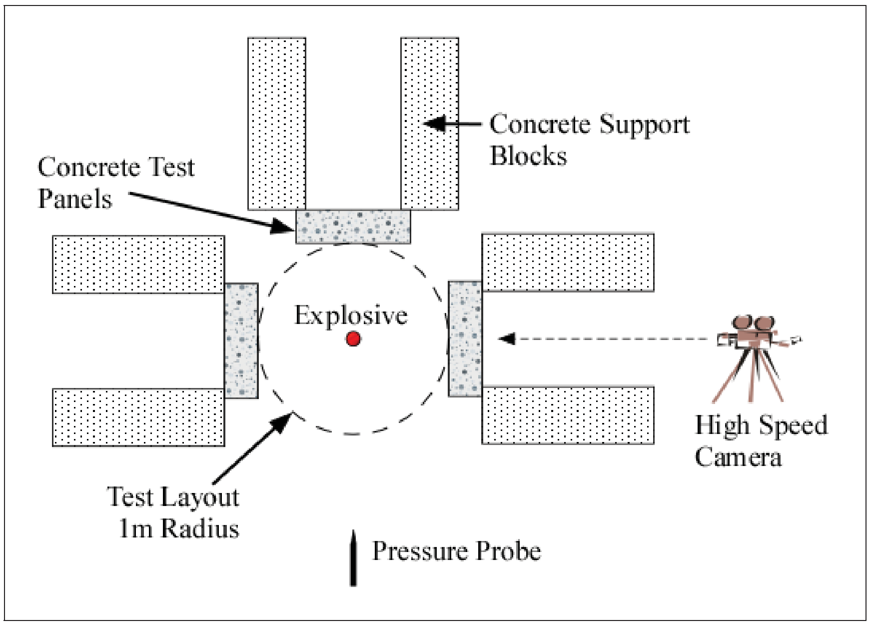


Figure 1. Test setup.

Figure 2. Support details.

Table 2. Blast characteristics for each panel and reinforcement ratio.

Panel	A4-6	A4-10	B4-6	B4-10	C4-6	C4-6	C4-10	C4-14	D4-6	D4-10	D4-14
ρ (%)	0.20	0.19	N/A	N/A	0.40	0.40	0.39	0.41	0.40	0.38	0.41
Standoff Distance (m)	1.02	0.97	1.02	1.02	1.02	1.04	0.97	0.97	1.04	0.97	0.97
Equivalent TNT (Kg)	6.2	13.2	6.2	13.1	6.2	6.2	13.2	13.2	6.2	13.2	13.2

Panel	E4-6	E4-10	E4-14	CON-1	CON-2	CON-2	CON-2	CON-3	CON-4	CON-5
ρ (%)	0.20	0.19	0.21	0.58	0.58	0.58	0.58	0.58	0.58	N/A
Standoff Distance (m)	1.02	1.02	0.97	1.02	3.05	3.05	1.02	1.02	1.02	1.04
Equivalent TNT (Kg)	6.2	13.1	13.2	6.2	7.4	15.6	0.8	6.2	13.1	6.2

A data acquisition system was used to record strain, acceleration, and pressure during the blast. Data from accelerometers and free-field blast pressure transducers were digitized at 10^5 data points per second. Data from strain gauges were digitized at 10^4 data points per second. The accelerometers had a capacity to record from 5g to 5000g. The free field blast pressure transducers had a capability to record from 34 kPa to 3.4 MPa. Strain gauges were installed at the center of the rebar mats located inside of the panels. On each reinforcing mat, one gauge was installed in the horizontal x-direction and one in the vertical y-direction, each concrete panel used two mats so there were a total of four gauges per panel. Two external concrete strain gauges were installed on the exterior face of each test panel.

EXPERIMENTAL RESULTS

Figure 3 shows internal and external strain, acceleration, and pressure data for panel A4-6. From this figure and the fact that the two pressure transducers were placed 3m and 6m from the explosive, respectively, it can be deduced that the blast wave had a velocity of approximately 600 m/s in the free field. As anticipated, very high pressures, accelerations and strains were observed in the blasts. The results of the tests are described based on three performance categories: (a) immediate occupancy, (b) life safety, and (c) near collapse. These categories are accepted in performance based seismic design (Fardis 2009) and are proposed here for blast evaluation. Post-blast testing was performed on nine panels that confirmed the definition of the performance categories.

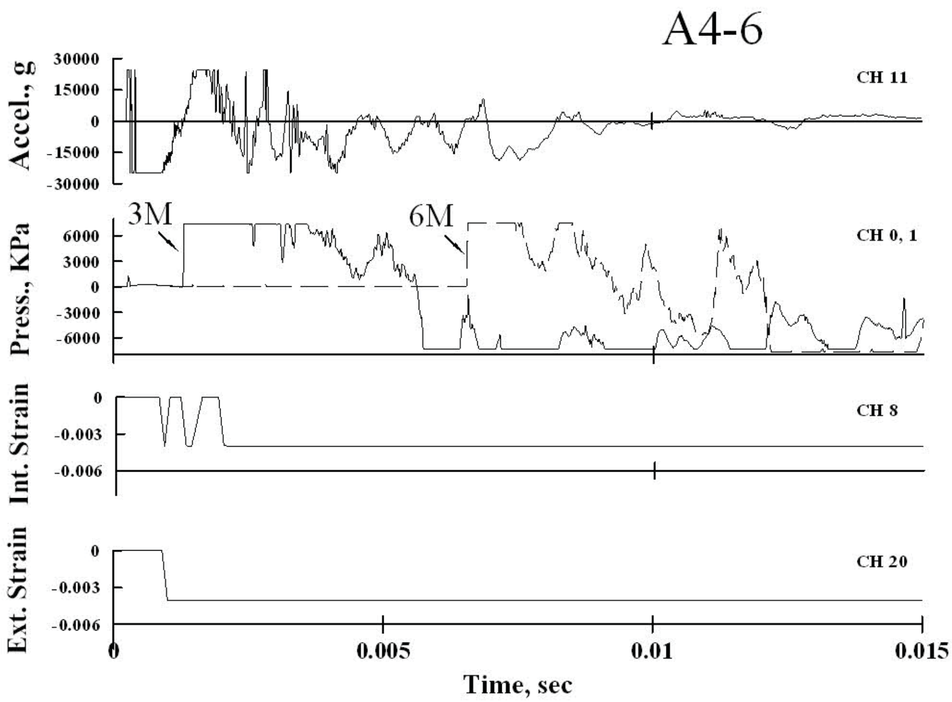


Figure 3. Blast data for panel A4-6.

Immediate Occupancy (IO)

The damage in this category is such that it could be possible to resume normal activities in a structure with the characteristics of the panels described and the charge used for the respective blast. The structure should retain its pre-blast strength and stiffness. The immediate occupancy (IO) performance category was met by Type C panels (C4-6, C4-10, C4-14) and panel E4-14. Panel Type C was designed to determine the effects of using the macro-synthetic fibers in addition to traditional steel rebar as reinforcement. Panel C4-6 experienced minimal damage with a largest measured crack width in the panel of 1 mm. Since the panel was not visibly damaged, it was tested a second time. The panel again experienced minimal damage as shown in Fig. 4(a). The largest measured crack width after the second blast was only 2mm with few new cracks. Panels C4-10 and C4-14 behaved similar to C4-6 with even smaller crack widths as shown in Table 3. Thus, thicker panels are shown to absorb a blast more efficiently.

Panel E4-14 had two GFRP laminate layers on each side of the panel. Due to the considerable thickness of 356 mm, Panel E4-14 had very few signs of damage. The front face GFRP laminate sustained no visible damage and did not debond from the concrete, as shown in Fig. 4(b). The back face GFRP overlay debonded from the concrete surface except in a few locations. The only sign of cracking was on the side of the panel where a crack was visible starting from the back face and progressively closed as the crack propagated to the front face of the panel. The maximum measured crack width was 3 mm. The deflections analytically calculated for panels in the IO performance category ranged from 0.3 mm to 5 mm.

(a)

(b)

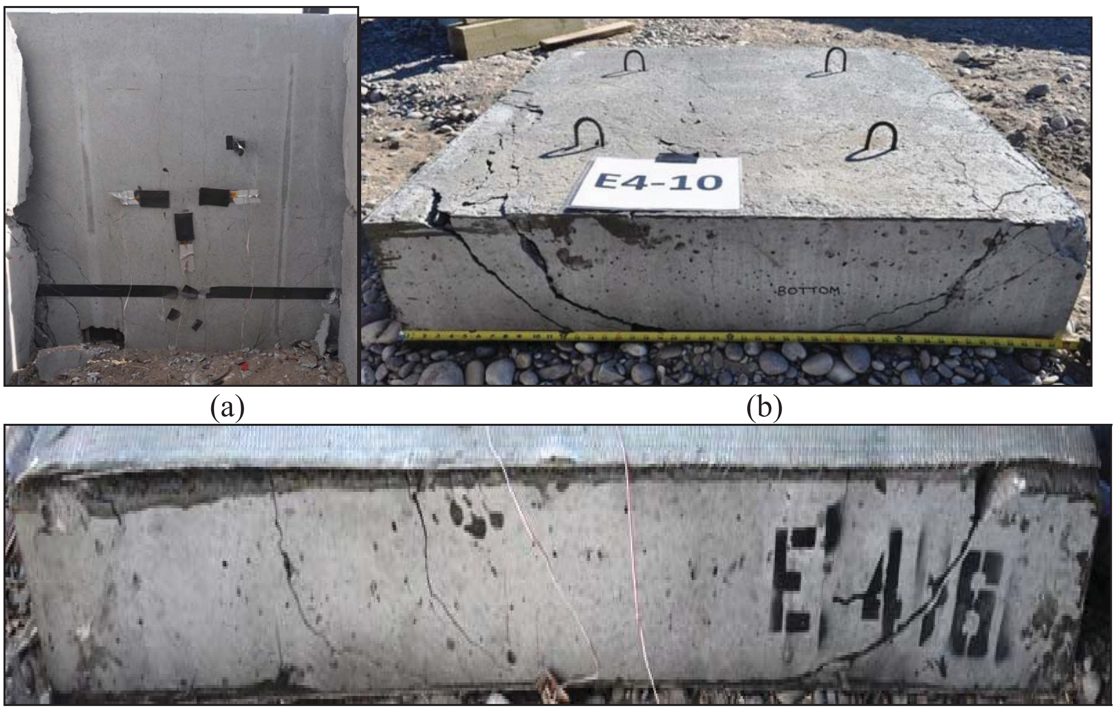
Figure 4. Immediate Occupancy (IO) performance level: (a) front of panel C4-6 after second blast; (b) front of panel E4-14 after blast.

Table 3. Performance Levels.

Specimen	Maximum Crack Width (mm)	Deflection (mm)	Performance Level
A4-6	16	82	Near Collapse
A4-10	13	31	Near Collapse
B4-6	11	N/A	Near Collapse
B4-10	2	N/A	Near Collapse
C4-6	2	5	Immediate Occupancy
C4-10	1.3	1.5	Immediate Occupancy
C4-14	0.1	0.3	Immediate Occupancy
D4-6	2	9	Life Safety
D4-10	13	12	Near Collapse
D4-14	6	2	Life Safety
E4-6	3	8	Life Safety
E4-10	13	3	Life Safety
E4-14	3	1.3	Immediate Occupancy
CON-1	10	11	Near Collapse
CON-3	16	43	Near Collapse

Life Safety (LS)

The structure in this category does not collapse, retaining integrity and residual load capacity after the blast. The structure may have some damage and permanent drift but retains its strength and stiffness. The panels in this performance category are D4-6 and D4-14 reinforced with GFRP bars at 152 mm and E4-6 and E4-10 which are RC panels with external GFRP laminates. Panel D4-6 experienced radial cracks that started at the center of the panel and propagated outwards with maximum measured crack width of 2 mm. Panel D4-14 experienced two small vertical cracks in the middle of the panel as shown in Fig. 5(a) with maximum measured crack width of 6 mm. It was observed that the boundary condition at the bottom of the panel caused some additional resistance as indicated by the cracks in Fig. 5(a); the panel had a portion of the concrete spall off in the lower left corner of the back face.



(a) (b)

(c)

Figure 5. Life Safety (LS) performance level: (a) back of panel D4-14; (b) panel E4-10 after the removal of the GFRP laminate; (c) side of panel E4-6 after blast.

Type E panels were designed to determine the effect of retrofitting typical RC panels reinforced with internal steel rebar using externally applied GFRP overlays. Panel E4-6 had some cracking at the edges of the panel with a maximum measured crack width of 3 mm, as shown in Fig. 5(c), which also shows blast-induced radial shear cracks on the panel side. The crack pattern shows that the dynamic response is dominated by shear effects. The GFRP overlay on the back face of the panel debonded during the blast and the laminate was easily removed after the blast. The front face laminate sustained minimal damage and was still securely bonded after the blast. Panel E4-10 experienced some damage, because the GFRP overlay on the back face debonded during the blast. This debonding likely occurred when the strain at the concrete surface exceeded the strain capacity of the resin used to bond the laminate to the concrete. The largest measured crack was 13 mm wide, and was located on the bottom side of the panel at 45 degrees from the front to the back face of the panel, as shown in Fig. 5(b), similar to those observed in Fig. 5(c) for panel E4-6. The deformed shape and radial shear cracks under blast-induced shear are different compared to static shear failure. Anchors should be investigated to determine whether global debonding of the GFRP laminate can be avoided. The deflections analytically calculated for panels in the LS performance category ranged from 2 mm to 8 mm.

Near Collapse (NC)

The structure in this category is heavily damaged, at the verge of collapse or parts of it have already collapsed, has little residual strength, is unsafe for use, and repair is not feasible. Significant cracking and amount of concrete spalling is also expected, as observed in the present tests. The panels in this performance category belong to Type A constructed with internal steel rebars, Type B and CONB constructed with only synthetic fibers, and Type D and COND with internal GFRP bars. Panel A4-6 experienced complete loss of structural integrity, as shown in Fig. 6(a). The blast created large spalling of concrete in the center of the panel and flexural cracks that propagated radially outwards. Severe damage was caused in the panel and concrete fragmented as a result of the radial cracks. The blast also caused large flexural cracking on the side of the panel and a plastic hinge along the vertical centerline. Panel A4-10 experienced two large vertical cracks with an average width of 10 mm, which separated the panel into thirds. The maximum crack width was 13 mm on the side of the panel. The entire left side of the panel was also damaged heavily near the support. The cracks on the back of the panel were located very close to the position of internal rebars. The back of the panel was so heavily damaged that the rebar was exposed.

Panels Type B and CONB had macro-synthetic fibers as the only reinforcement. Both panels broke into two pieces likely due to the lack of internal reinforcement.

Type D and COND panels were designed to determine the effects of using GFRP bars as internal reinforcement. Panel D4-10 experienced two large vertical cracks splitting the panel into thirds as shown in Fig. 7(b); the side of the panel was heavily damaged as a result of the cracks propagating completely through the thickness of the panel, as shown in Fig. 7(a). However, the blast-induced shear cracks were similar to those shown in Fig. 5(b) and Fig. 5(c) and different from static shear cracks.

Panel D4-10 had the same reinforcement ratio as the other two Type D panels, but with a wider GFRP bar spacing. The increased damage to this panel indicates that the GFRP bar spacing is important for good performance, not just the reinforcement ratio. This is also confirmed when comparing the performance of panel CON-3 to panel D4-6. Panels CON-1 and CON-3, which had a wider GFRP bar spacing but higher reinforcement ratio compared to Type D panels, were heavily damaged with large pieces of the concrete panel missing, as shown in Fig. 6(b). It was observed that the panels in this category experienced a progressive failure. The panels first cracked at the top because of lack of support, and as the cracks spread downward, the concrete spalled and fragmented. As the fragmentation progressed, the cracks propagated further through the panel and opened up until they reached the bottom edge of the panel. The deflections analytically calculated for panels in the NC performance category ranged from 8 mm to 82 mm. Some of the analytically calculated deflections were confirmed with measurements.

(a) (b)

Figure 6. Near Collapse (NC) performance level: (a) back of panel A4-6; (b) back of panel CON-3 after blast.

(a) (b)

Figure 7. Near Collapse (NC) performance level: (a) front; (b) back of panel D4-10.

ANALYTICAL RESULTS

Preliminary dynamic simulations of the panel tests were conducted using LS-DYNA (Hallquist 2006) with material model 159—a smooth surface cap model for strain rates similar to vehicle impacts. Figure 8 shows results from a model of a 152 mm thick plain concrete panel with no reinforcement. Rigid elements shown in light grey represent the support structure used during testing. A conversion factor of 1.34 kg of TNT to 1 kg of C4 was used to simulate an air blast consisting of 4.54 kg of C4 explosive at a 1.0 m standoff from the center of the panel front face. Concrete strength data (compression and tension) from cylinder tests conducted within one week of the blast tests were used in this simulation. The contours shown are a

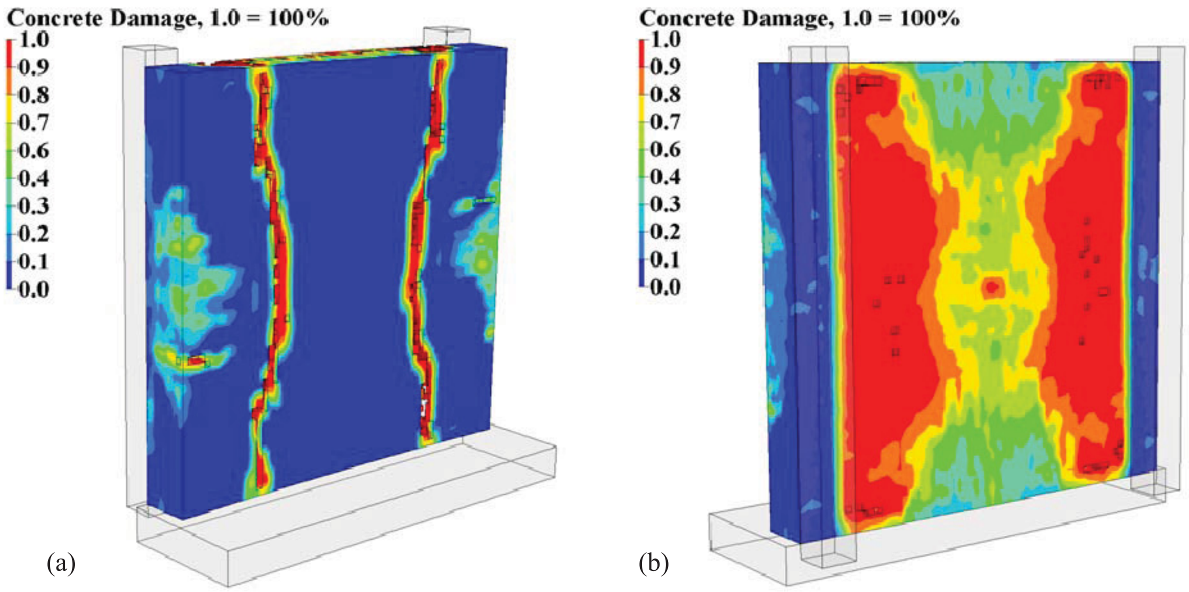


Figure 8. Preliminary front face LS-DYNA simulation: (a) front face; (b) back face.

combination of brittle and ductile damage with elements removed when damage reaches a value of 1.0. While the crack and damage patterns are similar to those for the lightly reinforced test panels, the extent of damage predicted is generally larger in the simulations. Corrections in the material model are underway to account for the very high strain rates experienced by the concrete during blast loading.

CONCLUSIONS

Performance-based categories of Immediate Occupancy (IO), Life Safety (LS), and Near Collapse (NC) were defined in terms of damage resulting from a blast load of a given charge and standoff distance. Damage was defined in terms of the amount of concrete spalling, crack width, and panel deflection. The IO category was met by Type C panels with internal steel rebars and FRC concrete for panel thicknesses of 152 mm, 254 mm, and 356 mm, and a 356-mm thick Type E panel with internal steel rebars and external GFRP laminates. The LS category was met by the 152 mm and 254 mm thick Type E panels, and the 152-mm and 356-mm thick Type D panels with internal GFRP bars spaced at 152 mm. Panels in the NC category included Type A panels with NWC concrete and internal steel rebars, Type B FRC panels with no internal or external global reinforcement, and 254 mm thick Type D panels with internal GFRP bars spaced at 229 mm.

The panels in the NC category experienced a progressive failure and damage was cumulative. The panels in this research first cracked at the top because of lack of support; as the cracks progressed, they spread to the bottom of the panel through the concrete, which in turn spalled and fragmented. The deformed shape and radial shear cracks under blast-induced shear were different compared to static shear failure, which typically produces linear cracks at a certain fixed angle to the member axis.

The reinforcement ratio was not a strong predictor of panel performance while spacing of the reinforcement was found to be very important. For panels with adequate thickness and when the bars were spaced 152 mm on center, GFRP and steel bars performed well, even though steel bars were more ductile. Increasing the thickness of the panel was an effective method to reduce damage, which is explained by the increased area of the concrete struts. Global reinforcement of Type B FRC panels is recommended for improved performance. Mechanical or FRP composite anchors are recommended to prevent global debonding of the GFRP laminate for Type E panels. The analytical models were able to replicate the level of damage that a panel experienced as a result of the blast.

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