Performance and Blast Design for Non-Load Bearing Precast Concrete Panels

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ABSTRACT

This paper summarizes recent testing, analysis, and applications to blast design for non-load bearing reinforced concrete wall panels. This includes solid, precast, and insulated sandwich panels reinforced with conventional and pre-stressed reinforcement. The measured peak response of these types of reinforced concrete test panels in blast tests is compared to the response calculated with the single-degree-of-freedom (SDOF) method commonly used for blast resistant design. The observed damage in test panels is also used to develop response criteria, which correlate the amount of peak panel response to the corresponding level of damage in the panel. The observed response of connections in precast test panels is also used to assess connection design for blast resistant precast panels. This paper recommends that cast-in-place and precast concrete non-load bearing panels, including solid concrete panels and insulated sandwich panels, can be designed to resist blast loads using the SDOF design methodology with new recommended response criteria.

Introduction

Traditionally, only cast-in-place concrete panels reinforced with conventional steel rebar have been used for blast resistant design. Recent blast tests conducted on different types of precast concrete panels have shown that solid and insulated sandwich precast concrete panels with pre-stressed and conventional steel reinforcement can resist high design blast loads, with impulses of up to 300 psi-msec. These tests have included a wide range of precast panel types with different types of steel reinforcement and typical connections to the test frame. This testing has been used to validate single-degree-of-freedom (SDOF) analyses for design for the different types of reinforced concrete panels and to recommend less conservative response limits. Additional full scale blast testing of precast concrete panels is on-going to validate the findings in this paper.

Shock Tube Tests on Solid Precast Panels and Connections

Protection Engineering Consultants (PEC) designed a test program conducted by Baker Engineering and Risk Consultants (BakerRisk) where 17 shock tube tests were performed on 13 precast panels (Lowak et al, 2012). The testing was funded by the U.S. State Department Bureau of Diplomatic Security Physical Security Division's Research and Development Branch. These panels were connected to the shock tube frame using conventional precast connections so that the connections were tested with the panels. The panels were tested to sustain heavy damage or failure. The connection capacities were generally equal to connection design loads based on the ultimate resistance of the test panels, as they would be in a typical blast design. In some cases, specifications for blast resistant precast construction require (or recommend) that the connection capacity is controlled by a ductile mechanism (e.g. yielding of the embed studs rather than stud pullout from the concrete). This approach was not included in the connection design for the test panels. Also, the connection angles (i.e. clip angles) were only designed to resist the panel reaction load in shear, and not to resist moment caused by the application of the panel reaction load at an eccentricity from the angle connection to the shock tube frame. The clip angles yielded and plastically rotated during the tests, but no other damage was noted in any tests indicating the clip angles were near failure. Gusseted clip angles were used to connect some test panels and there was no noticeable difference between the measured dynamic loads from gusseted clip angles and conventional clip angles.

Table 1 shows a summary of the test results, including the peak measured dynamic reaction forces and deflections. The peak measured deflections are also shown in terms of support rotation, which is a commonly used response term for blast design. The blast loads, acceleration of the panel at midspan, and dynamic reactions of the test panels were measured in the tests. Also, a high speed video camera recorded the midspan movement of the panels against a background grid. This was used to verify the panel displacement history that was determined from double integration of the measured acceleration histories. The panels responded with varying amounts of damage and corresponding amounts of support rotation. The amount of support rotation prior to failure depended heavily on the type of reinforcement. The panel damage level descriptions are shown in Table 2. Figure 1 shows an example of a test panel with heavy damage.

The tests showed that conventional bolted and welded angle connections designed to resist the equivalent static reaction force from the ultimate resistance of the panels were sufficient to resist the dynamic panel reaction loads up to panel failure. The connections were the weak point in only one out of seven tests with panel failure or borderline failure. The one case of connection failure occurred in the bolt in the horizontal leg of the bottom angle connection for Test Panel 6. This was a 5/8 inch diameter A490 bolt, which was oversized compared to the design reaction force because the test engineer substituted it for a similar A325 bolt designed for the connection that did not fit the test frame. Figure 2 shows the measured reaction history for Test 6 up to panel failure and the measured reaction histories from Tests 1A and 10, which had higher measured reaction loads and a similar connection as Test 6, including the same substituted A490 bolt at the bottom connection, and the original, weaker A325 bolt with same bolt diameter at the top connection that did not fail during the tests. Therefore, it does not seem like the failed bolt was overloaded and other connections with similar capacity in the test panels were subjected to higher stresses without failing. The connection failure in Test 6 may have occurred because the bolt had a flaw.

AFRL Blast Tests

In another recent large testing program on precast panels, high explosive blast tests were conducted at the Air Force Research Laboratory (AFRL) on insulated concrete sandwich panels (Naito et al, 2011b, Naito et al, 2013). In the main phase of the testing, blast loads were applied in two separate tests to eight test panels as described in Table 3. These panels were constructed to meet the design standards used for the Precast/Prestressed Concrete Industry (PCI) to resist typical conventional design loads. There were four “M” series panels, which had only one span with simple supports, and four “F” series panels, which were two-span continuous over an interior support with a 3 ft. cantilever above the top support. In no case were the panel walls or connections to the test frame designed for blast. The design of the reinforcement for these panels was controlled by loads during lifting and handling, so the panels were considerably stronger than required only to resist lateral wind loads.

Table 1. Summary of Precast Concrete Panel Shock Tube Tests

Test No. [1]	Panel Thick[2] (inch)	Vert. Panel Reinforce-ment	Peak Pressure (psi)	Impulse (psi-ms)	Connection (2 per support)	Maximum Deflection[3] (inch)	Support Rotation[3] (degrees)	Max. Reaction[4] (kips)	Damage	Comments
1	6	#4@12" EF	10	140	Bolted Angles				LOW	No data – recorder malfunction
1A	6	#4@12" EF	16.8	232	Bolted Angles	3.0	4.8	35	MODERATE	
2	6	#4@12" EF	20.7	393	Direct bearing	10	11.8	46	HEAVY	The wall had a permanent displacement of 8.5 inches.
3	5	#5@8"	10.3	141	Bolted Angles	1.75	2.1	22	LOW	Tension steel was 2.16 inches from compression face.
3A	5	#5@8"	20.6	277	Bolted Angles	**6**	**7.1**	31	FAILURE	
4	5	#5@8"	20.6	176	Direct bearing	**4**	**4.8**	23	FAILURE	Reinforcing located 1.5" from compression face.
5	6	#4@12" EF	17.7	119	Bolted, gussetted Angles	1.1	1.5	22	LOW	The wall had a very slight permanent displacement estimated at 1/8".
5A	6	#4@12" EF	20.1	277	Bolted, gussetted Angles	4.6	5.5	38	MODERATE	The wall had an estimated 2" permanent displacement.
6	6	(2) 4x4/ W2.1xW4 EF	20.9	255	Bolted Angle (Bottom), Halfen w/ rod (Top)	n/a	n/a	31	FAILURE	**Bolt in connections failed with little damage to concrete panel.**
7	6	(2) 4x4 W2.1xW4 EF	17.8	121	Welded angles	1.3	1.8	25	LOW	The wall was measured to have a 0.25" permanent displacement.
7A	6	(2) 4x4/ W2.1xW4 EF	20.3	303	Welded angles	**3.8**	**4.5**	31	FAILURE	
8	6	#4@12" EF	20.4	311	Welded, gusseted angles	3.5	4.8	41	MODERATE	Rebound damage to connections
9	6	#4@12" EF	7.6	105	Welded angles	0.5	0.9	19	LOW	Panel installed 'backwards" so connections tested in rebound direction. Damage to connection.
10	6	#4@12" EF	20.8	350	Bolted angles	7.5	8.9	37	HEAVY	
11	6	#6@5" EW	19.7	269	Bolted angles	**4**	**4.8**	36	BORDERLINE FAILURE	Tension steel was 2.6 inches from compression face.
12	6	#6@5" EW	20.3	370	Welded angles	**5**	**6**	57	FAILURE	Tension steel was 2.6 inches from compression face.
14	6	(2)4x4/W2.9 xW2.9 EF	20.5	253	Welded angles	**2.8**	**3.3**	33	BORDERLINE FAILURE	Tension steel had 2-inch cover. Instead of 1.25 inches.

Note 1: Panels with "A" were retests of a panel with minor damage.
Note 2: All panels are nominally 8 ft x 8 ft with an 8 ft vertical simply supported span. Measured concrete compression strength (f'_c) during testing was 7500 psi.
Note 3: All red maximum deflections were deflections at panel failure time as determined from measured panel reaction forces.
Note 4: Total reaction force at bottom of panels summing the two load cells (one load cell near each connection). Load cell measurements were reduced by a factor of 1.7 or 1.77 (Tests 2 and 4) to account for overturning/prying forces on the dynamic load cells as recommended by BakerRisk.

Table 2. Damage Descriptions for Precast Panel Shock Tube Tests

Damage Level	Damage Description
LOW	Moderate cracking on both faces of the panel. No permanent deflection or very small permanent deflection.
MODERATE	Significant cracking on the non-loaded face of the panel. Light spalling on the loaded face. No significant damage at the connections. Small but noticeable permanent deflection.
HEAVY	Significant cracking and spalling on both faces of the panel. Complete loss of cover over tension reinforcement at midspan on the non-loaded face. Not significant damage at connections. Significant permanent deflection.
FAILURE	Complete rupture tension reinforcement. Potential failure at connections and complete detachment of panel from supports.

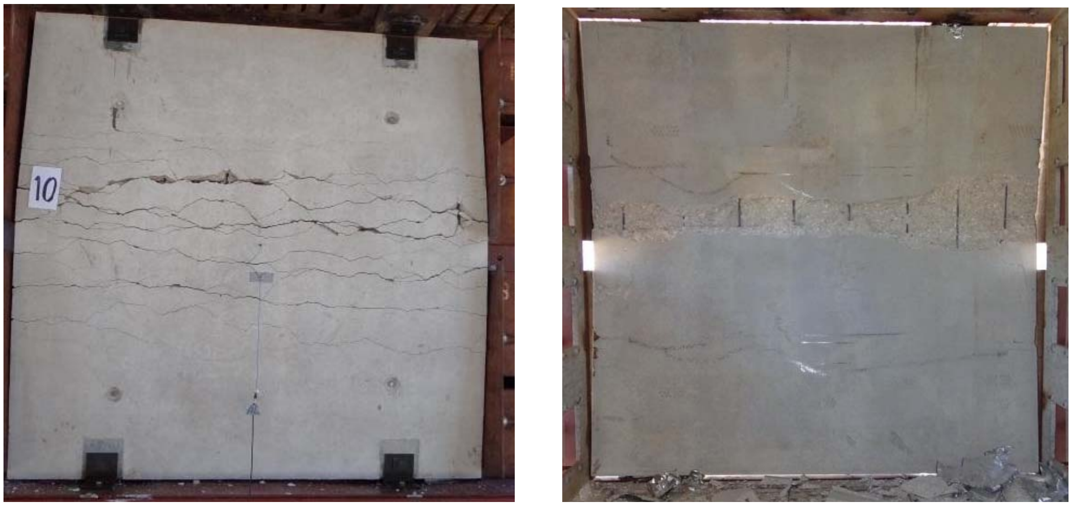

Figure 1. Exterior and Interior (Blast Loaded) Face of Test Panel with Heavy Damage

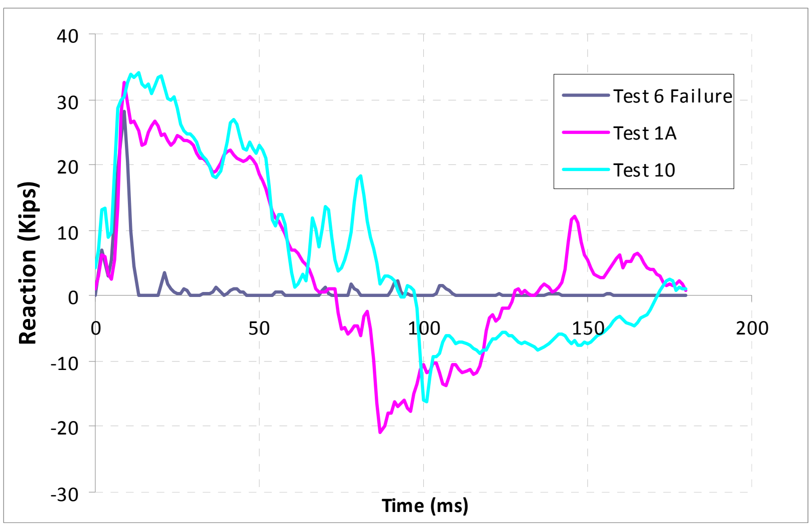

Figure 2. Measured Reaction Forces for Test 6 (Bolt Failure) and Tests 1A and 10 (No Connection Failure) with Same Bottom Connection

Table 3. AFRL Sandwich Panel Test Panels

Panel	Wythes[1] (inches)	Reinforcement	Boundary Conditions[2]	Width	Span	f'c	Non-prestress		Prestress	
							A_s^+	A_s^-	A_s^+	A_s^-
				(ft)	(ft)	(psi)	(in^2)	(in^2)	(in^2)	(in^2)
M1	3-2-3	Pre-stressed	Simply supported (single span)	4.92	9.75	8723	0.077	0.077	0.34	0.34
M2	3-2-3	Pre-stressed		4.92	9.75	7641	0	0	0.34	0.34
M3	3-2-3	Non-pre-stressed		4.92	9.75	4828	0.8	0.8	0	0
M4[3]	3-2-6	Non-pre-stressed		4.92	9.75	5093	0.8	0.285	0	0
F1	3-2-3	Pre-stressed	Fixed-simple (continuous two span)	7.33	14/12.5	8723	0.116	0.116	0.51	0.51
F2	3-2-3	Pre-stressed		7.33	14/12.5	7907	0	0	0.51	0.51
F3	3-2-3	Non-pre-stressed		7.33	14/12.5	4828	1.2	1.2	0	0
F4[3]	3-2-6	Non-pre-stressed		7.33	14/12.5	4828	1.2	0.425	0	0

Note 1: The exterior wythes on each face are reinforced concrete and the interior wythe is foam insulation.
Note 2: The "M" panels are single span and the "F" panels are continuous two span panels with a 3 ft cantilever above the top span.
Note 3: All panels have shear connectors extending from each concrete face through the insulation layer. The shear connectors are designed to cause the panel to act 100% compositely for all tests except M4 and F4, where the shear connectors are only designed to cause the panel to act 50% compositely.

The test panels resisted the blast test loads, which were in the middle to upper range of loads used for most blast design, without failing. Detailed information on the blast loads is provided by Naito et al (2011). Detailed test results are shown in the next section compared to corresponding calculated response. The test panels responded to the highest blast load with moderate damage and some connection failures in the two-span continuous panels that did not cause total panel failure (i.e. the panel was still supported upright by the test frame after the test). Many of the two-span continuous test panels had heavy localized damage around the connections including failures at the interior span connection, where the highest reaction loads occurred. Since the connections were only sized for typical wind loads, they were under-designed compared to typical blast design requirements. The test results showed that test panels with nearly equal moment capacity using conventional reinforcing steel and pre-stressed had nearly equal damage levels and support rotations. This is somewhat unexpected since pre-stressed panels are often considered to be significantly less ductile than conventionally reinforced panels, so that they fail at much lower support rotations. Panels with both types of reinforcement had relatively large support rotations (i.e. in the range of 5 to 6 degrees) with only moderate damage to the panel from the higher test blast load.

Use of SDOF Method to Determine Maximum Panel Response

The precast concrete panels from the previously described tests were analyzed using the SBEDS program (SDOF Blast Effects Design Spreadsheets) from the U.S. Army Corps of Engineers, Protective Design Center (PDC) (PDC-TR 06-01, 2008). The SDOF approach in SBEDS has compared well (i.e. generally within 20% on the average to over 120 tests on 8 different component types) as summarized in Bazan and Oswald (2014). Table 4 and Table 5 show comparison between the measured peak deflections and dynamic reactions of the solid precast panel tested in the shock tube tests (see Table 1) and sandwich panels from the AFRL blast tests (Table 3) and corresponding values calculated with SBEDS. All the test panels were analyzed using the measured pressure histories (including the negative phase load from the high explosive tests) and the test panel configurations in Table 1 and Table 3. The default values in SBEDS were used to calculate the dynamic moment capacities for the test panels, including the static and dynamic increase factors.

Table 4. Comparison of Measured and Calculated Response for Shock Tube Tests

Test	Measured Values					Calculated Values with SDOF		SDOF/Measured		Observed Damage
	Peak Pressure (psi)	Impulse (psi-ms)	Max. Defl. (in)	Max. Support Rotation (deg)	Max. Reaction (kip)[1]	Max. Defl. (in)	Max. Reaction (kip)	Max. Defl. Ratio	Max. Reaction Ratio	
1A	16.8	232	3	4.5	35	3.6	34	1.21	0.97	Moderate
2	20.7	393	10	11.77	46	10.1	38	1.01	0.82	Heavy
3	10.3	141	1.75	2.09	22	1.1	26	0.65	1.19	Low
3A	20.6	277			31		36		1.15	Failure
4	20.6	176			23		31		1.37	Failure
5	17.7	119	1.1	1.49	22	1.0	29	0.86	1.31	Low
5A	20.1	277	4.6	5.47	38	5.9	37	1.28	0.97	Moderate
6	20.9	255			31		38		1.25	Failure
7	17.8	121	1.3	1.79	25	1.6	25	1.25	0.99	Low
7A	20.3	303			31		31		1.01	Failure
8	20.4	311	3.5	4.76	41	8.0	38	2.28	0.93	Moderate
9	7.6	105	0.5	0.60	19	0.4	24	0.86	1.28	Low
10	20.8	350	7.5	8.88	37	10.1	38	1.35	1.03	Heavy
11	19.7	269			36		62		1.72	Failure
12	20.3	370			42		63		1.49	Failure
14	20.5	253			33		34		1.02	Failure
Average								**1.20**	**1.16**	

The insulated sandwich panels were modeled in SBEDS assuming they were 100% composite, including the two panels in each test that were only designed to be 50% composite (M4 and F4, see Table 3). The insulated panels were modeled as monolithic panels with the full panel thickness and an input negative supported mass to account for the lightweight insulating panel-. The calculated peak deflections for the two panels designed to be 50% composite are shown as shaded rows in Table 5. Many SDOF-based blast design approaches (e.g. UFC 3-340-02, UFC 3-340-01) require that concrete components are analyzed with a Type II cross section (i.e. based only on a moment capacity from the compression and tension face reinforcing steel) when the support rotation exceeds 2 degrees. This approach assumes that the concrete does not carry any compression force at these support rotations. This approach was not used for the SDOF analyses in Table 4 and Table 5.

Additional blast tests by BakerRisk on 5.5 inch thick reinforced concrete slabs with a single layer of #5 bars at 11.25 inch spacing at mid-thickness, spanning 8 ft. with simple supports, are shown in Table 6 (Holland and Wesevich, 2010). This table shows that analyses with SBEDS also predicted the measured deflection for these solid conventionally reinforced concrete panels well.
In summary, the comparisons of maximum deflections calculated with SBEDS (i.e. SDOF analysis) and measured deflections in blast and shock tube tests strongly indicate that SDOF analysis in SBEDS can be used to accurately predict the maximum response of reinforced concrete walls for blast design purposes. This includes solid and insulated sandwich wall panels reinforced with conventional reinforcing steel and pre-stressing steel and with fixed and simple boundary conditions. The approach in SBEDS is generally based on documented SDOF blast

design procedures in UFC 3-340-01 and UFC 3-340-02, as described in the SBEDS Methodology Manual (PDC-TR 06-01). The peak dynamic reactions are also predicted well with SBEDS compared to measured values (i.e. within 20% on the average), although these reactions are typically not used directly in the blast design process.

Table 5. Comparison of Measured and Calculated Maximum Deflections and Reaction Forces for AFRL Blast Tests

Wall[1]	Measured Values				Calculated Values with SDOF			Maximum Deflection Ratio	Maximum Reaction Ratio
	Max. Defl. (in)	Time (ms)	Max. Reaction (lbs)[2]	Support Rotation	Max. Defl. (in)	Time (ms)	Max. Reaction (lbs)	Calc./Meas.	Calc./Meas.
M1 – Test 1	3.44	38.0	45854	3.4	4.19	33.6	50755	1.22	1.11
M1 – Test 2	5.86	53.5	100813	5.7	5.37	33.1	78463	0.92	0.78
M2 – Test 1	2.97	30.1	72392	2.9	4.42	34.9	50329	1.49	0.70
M2 – Test 2	5.72	64.0	81072	5.6	5.59	34.1	77893	0.98	0.96
M3 – Test 1	4.87	38.9	65803	4.8	5.80	41.4	50198	1.19	0.76
M3 – Test 2	6.99	56.2	95686	6.8	7.02	45.7	76932	1.00	0.81
M4 – Test 1	3.47	40.2	76200	3.4	3.39	38.0	52924	0.98	0.69
M4 – Test 2	4.52	51.6	64367	4.4	4.20	38.4	86351	0.93	1.34
F1 bot – Test 1	3.23	38.9		2.2	2.73	34.5		0.85	
F1 top – Test 1	2.07	37.7		1.6	2.15	30.5		1.04	
F1 bot – Test 2	7.46	50.3		5.1	6.06	35.7		0.81	
F1 top – Test 2	n/a				5.40	33.4		n/a	
F2 bot – Test 1	3.27	43.0		2.2	2.90	35.6		0.89	
F2 top – Test 1	2.03	32.7		1.6	2.32	31.7		1.14	
F2 bot – Test 2	6.59	49.9		4.5	6.25	34.6		0.95	
F2 top – Test 2	4.07	35.7		3.1	5.60	34.1		1.38	
F3 bot – Test 1	4.64	47.7		3.2	3.87	41.9		0.83	
F3 top – Test 1	3.18	39.2		2.4	3.30	38.3		1.04	
F3 bot – Test 2	8.78	61.6		6	7.56	51.5		0.86	
F3 top – Test 2	5.65	41.6		4.3	6.80	41.7		1.20	
F4 bot – Test 1	2.86	51.2		2	2.59	42.0		0.91	
F4 top – Test 1	1.99	39.8		1.5	2.21	38.4		1.11	
F4 bot – Test 2	4.68	61.3		3.2	5.10	52.1		1.09	
F4 top – Test 2	2.88	38.4		2.2	4.57	42.1		1.59	
Average								**1.06**	**0.89**
Std. Deviation								**0.21**	**0.23**

Note 1: Shaded rows correspond to panels statically designed as 50% composite but are analyzed in SBEDS as 100% composite during dynamic blast response.
Note 2: Maximum reactions were only measured for M series test panels.

Response Criteria for Reinforced Concrete Panels

Response criteria correlate the peak component response (peak deflection) to the damage level of the component. These criteria are specific to a given component type because different types of components have differing levels of ductility under blast load, so they have larger or smaller maximum deflections associated with each damage level. Component response for reinforced concrete components is typically defined in terms of the maximum support rotation (UFC 3-340-02, 2008; ASCE/SEI 59-11, 2011).

Table 6. Measured Slab Response in BakerRisk Shock Tube Tests

Test	Peak Pressure (psi)	Impulse (psi-ms)	Measured		Calculated	Ratio of Calc./Meas. Deflection
			Deflection (inch)	Support Rotation	Deflection (inch)	
1	7.7	217	2.3	2.7	2.1	0.91
2	7.6	213	2.3	2.7	2.1	0.91
3	7.9	221	2.0	2.4	2.3	1.15
4	10.7	306	5.5	6.5	6.2	1.13
5	10.8	311	5.8	6.9	6.3	1.09
6	10.6	297	6.0	7.1	6.1	1.02
Note 1: 5.5 inch slab with #5 bars at 11.25 inch spacing in one-way simply supported 8 ft span. The ultimate resistance for panel is 4 psi.						

Figure 3 shows the observed damage levels for solid precast panels from the shock tube tests in Table 1 with different configurations of conventional reinforcement plotted against the observed maximum support rotations. The support rotations for panels with failure were determined based on the measured deflection at the failure time, where failure time was determined by noting the time when the measured reaction load dropped very suddenly. The one panel with connection failure (i.e. Test 6) and the two over-reinforced test panels (i.e. Test 11 and 12) from this test series are not included in Figure 3. Therefore, all the panels in Figure 3 have sufficient connection strength and are not over-reinforced.

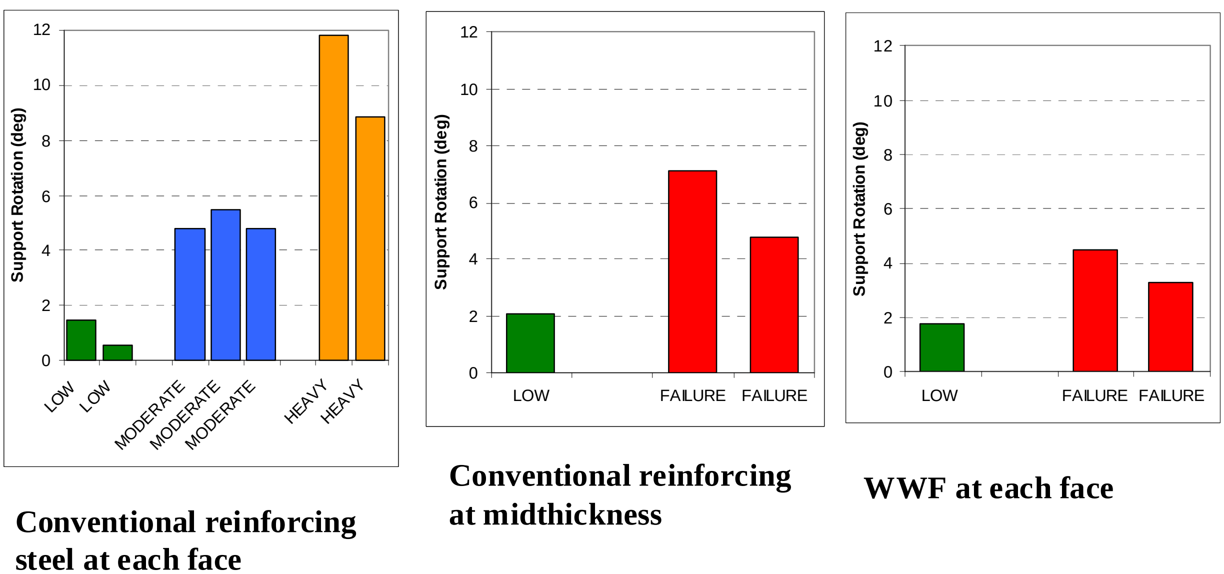

Figure 3. Measured Support Rotations vs. Damage Levels for Precast Test Panels from Shock Tube Tests with Different Types of Steel Reinforcement

The information in Figure 3 indicates that the type of conventional reinforcing steel affects the response limits. The test data for panels with conventional steel reinforcement at each face, shown at the left of Figure 3, indicates that these panels can undergo support rotations up to 12 degrees without failing. There is no shear reinforcement to laterally support the compression face steel after concrete crushing in any of these panels, as required for higher response limits in reinforced concrete components in many existing sets of response criteria (e.g. ASCE, 2011;

PDC-TR 06-08 Rev1, 2008). Conversely, panels with the other two types of conventional reinforcement fail at much lower support rotations, particularly panels with welded wire fabric (WWF). Failure of panels with WWF was initiated by tensile rupture of the WWF, which is not nearly as ductile as conventional steel reinforcement. The failure of test panels with a single layer of reinforcement was initiated by concrete crushing in the maximum moment region and there was no reinforcing steel at the compression face to take the compression force after concrete failure. It should be noted that some test panels in Table 6 with one layer of conventional reinforcement did exceed 6 degrees of support rotation in shock tube tests without failing. Therefore, the method used to determine the failure deflection of panels for Figure 3 may be somewhat conservative (i.e. underpredicts the amount of deflection that the panels could undergo prior to failure).

The AFRL blast tests on precast insulated sandwich panels, which are summarized in Table 5, show these panels with conventional and pre-stressing steel can deflect 5 to 6 degrees of support rotation with medium damage. This is the same relationship between support rotation and damage observed from the shock tube tests for solid panels with conventional reinforcement in Figure 3. Each wythe of the insulated panels had reinforcement, so these panels can be considered to have two layers of reinforcement. The SDOF analyses indicate that the insulated sandwich test panels in this test series acted as fully composite under blast loading, even though one of the test panels was designed to be only 50% composite. Corresponding static testing of these tests panels in Table 3 showed that they did not generally act as 100% composite after yielding (Naito, 2011a), so that the panels evidently act compositely in more ductile manner under highly dynamic loading than under static loading. This observation was also made by Naito et al (2013).

Additionally, there was no apparent difference in the relationship between support rotation and observed damage level for the AFRL test panels reinforced with pre-stressed steel and with conventional reinforcing steel (one-half of the test panels had each type of reinforcement). It should be noted that the AFRL test panels had a very low pre-stressed steel reinforcement index, ω_p in Equation 1, equal to 0.03.

$$\omega_p = \frac{A_{ps}}{bd_p}\frac{f_{ps}}{f'_c} \tag{1}$$

where:

A_{ps} = area of pre-stressing reinforcement in tension zone
f_{ps} = calculated stress is pre-stressing steel in design load
b = member width
d_p = depth to center of pre-stressing steel
f'_c = concrete compressive strength

The test data discussed in this paper was used to develop recommended response criteria in Table 7 for precast and cast-in-place concrete panels that are non-load bearing panels with equal layers of reinforcing steel at each face. The test data is for precast panels, but cast-in-place panels are not expected to have lower response limits since they are very similar except that they have a monolithic connection to their supports. The damage levels are described in Table 2. These criteria assume that the panel ultimate load capacity is controlled by flexural response, not by the shear strength or connection strength of the panel. This requirement is typically considered satisfied for blast resistant design if the panel shear strength and connection strength are each equal, or greater than, the equivalent static reaction load from the ultimate dynamic resistance of the panel.

The response criteria in Table 7 have some conservatism compared to the test data summarized in this report. Figure 3 shows that conventionally reinforced solid panels with reinforcement at each face can have moderate damage at support rotations at least equal to 5 degrees and heavy damage at support rotations greater than 10 degrees. Also, the response criteria in Table 7 for panels with pre-stressed steel and insulated sandwich panels are conservative compared to the AFRL test data, since these types of panels had only moderate damage at 5 degrees of support rotation. This conservatism is primarily due to the limited blast test database. It is possible additional testing that causes heavy damage will show that these response criteria can be increased. The test walls used to develop the criteria in Table 7 all have relatively low levels of reinforcement, so limits are placed on the wall pre-stressing index and conventional reinforcement index in the table. Existing response criteria (e.g. from PDC-TR 06-08 Rev. 1) can be used for walls with higher levels of reinforcement.

Table 7. Recommended Response Criteria for Non-Load Bearing Reinforced Concrete Wall Panels with Flexural Response and Existing Criteria

Damage Level	Proposed New Response Criteria[1]			PDC-TR 06-08 Rev. 1	
	Conventional Reinforcing[2]	Pre-stressed Reinforcing[3]	Insulated Sandwich Panels [2,3,4]	Conventional Reinforcing[5]	Pre-stressed Reinforcing[6]
Low	2	1	1	N/A	N/A
Moderate	5	2	2	2 (4)	1
Heavy	10	5	5	5 (6)	2

Note 1: All response criteria are valid only for non-load bearing panels that have equal reinforcing steel capacity at each face and the panel load capacity is controlled flexural response.
Note 2: Reinforcing steel conforming to ASTM 615 or ASTM 706. The maximum support rotation for panels reinforced with welded wire fabric should be limited to 2 degrees. The ratio of the tensile steel reinforcement ratio to the balanced steel ratio should not exceed 0.4.
Note 3: Pre-stressing steel strands conforming to ASTM 416. The pre-stressing index using Equation 1 should not exceed 0.05 since this is the upper bound range of current blast test data. Refer to criteria from PDC-TR 06-08 Rev. 1 for response criteria applicable at higher pre-stressing indexes.
Note 4: Panels are statically designed to act 100% compositely per criteria from the PCI Design Handbook (2010).
Note 5: Criteria for panels with shear ties connecting the front and back face of flexural reinforcement shown in parentheses.
Note 6: Criteria for pre-stressing steel are shown here only for the case where the pre-stressing index using Equation 1 is less than 0.15. More restrictive criteria apply for higher pre-stressing indices.

The recommended response limits in Table 7 are compared in the table to existing response criteria for concrete panel components reinforced with conventional and pre-stressing steel in PDC-TR 06-08 Rev 1 (2008). The PDC response criteria are specifically for anti-terrorism design, but they are also used for other types of blast resistant design. ASCE (2011) has very similar response limits for conventionally reinforced and pre-stressed concrete panels that are used to design against accidental industrial explosions. UFC 3-340-02 (2008) has more conservative response criteria. All of these response criteria are based on test data available at the time these criteria were developed. The information in Table 7 indicates that that the PDC response criteria can be updated with less conservative criteria for concrete panels and slabs reinforced with conventional reinforcing steel at each face. The current PDC response criteria is appropriate for panels and slabs reinforced with a single layer of reinforcing steel (see Figure 3) and it is possibly unconservative for slabs and panels reinforced with welded wire fabric. It should be noted that Figure 3 is based on only three tests each with WWF and conventional reinforcement at mid-thickness. Also, the response limits for panels with pre-stressed reinforcement in Table 7 are for a much lower pre-stressing index than the PDC response criteria. Therefore, the recommended values in Table 7 for concrete panels with pre-stressed reinforcement can be added as a separate category of response criteria that only apply to wall panels with very low pre-stressing indices.

Summary

This paper recommends that reinforced concrete non-load bearing panels, including solid concrete panels and insulated sandwich panels with conventional and pre-stressed reinforcement, can be designed to resist blast loads using the SDOF design methodology with new recommended response criteria as described in the paper. The recommendations are based on tests of precast panels described in the paper, but are considered equally applicable for cast-in-place concrete panels.

The shock tube tests summarized in this paper show that solid precast concrete wall panels with two layers of conventional reinforcing steel and a relatively low reinforcing steel ratio (i.e. less than 40% of the balanced steel ratio) respond with more than 10 degrees of support rotation without failing. These tests also show that concrete wall panels reinforced with two layers of conventional reinforcing steel obtain a much larger support rotation prior to failure (i.e. larger ductility) than similar wall panels reinforced with one layer of conventional reinforcement at mid-thickness and panels reinforced with WWF. Panels reinforced with WWF were the least ductile and therefore should not typically be used for blast resistant design, if possible. Also, the precast wall panels in these tests had connections designed to resist an equivalent static reaction load from the ultimate resistance of the panel that allowed response in flexure with large support rotations (i.e. up to 10 degrees) without connections failure. Finally, the blast tests summarized in this paper showed that concrete sandwich panels reinforced with low reinforcement ratios of pre-stressing steel, as required for typical blast design loads, are much more ductile during response to blast load than indicated by current response criteria.

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