Testing and Analysis of Precast Concrete Wall Panels

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Abstract

This paper presents and discusses results from recent blast tests on non-load bearing precast concrete wall panels, and compares observed dynamic response and damage against single-degree-of-freedom (SDOF) analysis results. This analysis, and most of the tests, was sponsored by the U.S. Department of State Bureau of Diplomatic Security. A total of 50 panel spans were tested in five arena tests with large high explosive charges and 14 panels were tested in a shock tube. The tests include single-span panels and two-span continuous panels with conventional and pre-stressed reinforcement, and also include both solid and insulated ("sandwich") panels. The measured dynamic deflections are compared to the corresponding values calculated with the SDOF analysis method commonly used for blast resistant design. Also, the observed panel damage levels at measured levels of response are compared to current response criteria for reinforced concrete panels. These comparisons are used to evaluate the accuracy and adequacy of the simplified SDOF methodology and the degree of conservatism in the current response limits for blast design of precast concrete wall panels. In particular, the current and a proposed new methodology for modeling the rebound response of concrete walls panels and the amount of composite action for sandwich panels in SDOF analysis are evaluated. This study will provide test and analysis data that will be used to evaluate and update current guidelines for blast-resistant design of precast concrete wall panels.

Keywords: Blast Design; Precast Concrete; Wall Panels; Blast Testing; SDOF Analysis

INTRODUCTION

During the last decade, the use of precast panels for blast resistant design has gained acceptance and it is often chosen for industrial and institutional buildings subject to moderate to high blast loads. Recent blast tests conducted on different types of precast concrete panels have shown that solid and insulated (sandwich) precast concrete panel cross sections with pre-stressed and conventional steel reinforcement can resist high design blast loads, with peak pressures in the range of 100 psi (690 kPa) and impulses over 300 psi-msec (2068 kPa-msec). Typically, traditional single-degree-of-freedom (SDOF) blast design methods are used for analysis of these components. However, the traditional SDOF approach does not explicitly consider the different types of reinforcement and cross section configurations that are commonly used for precast concrete panels. Recent blast tests have demonstrated that blast-resistant precast panels can be designed using the traditional SDOF-based design approach with some modifications to methods used to calculate the properties for the equivalent SDOF system and the response limits based on the panel reinforcement cross section type. Connections are also a critical part of the design of precast components. This part of the design is addressed elsewhere [1].

This paper analyzes and discusses the response of non-load bearing precast concrete panels subject to blast loads in two series of arena tests and one set of shock tube tests. These test data included approximately 60 panels, including solid and sandwich cross sections, as well as conventional and prestressed reinforcing for each section type. The tests also included single-span panels and two-span continuous panels. All three test programs included blast loads with large impulses exceeding 300 psi-msec (2068 kPa-msec). This testing was used to validate results of SDOF analyses of the different types of precast concrete panels and to recommend less conservative response criteria for design. The measured dynamic responses of the test panels (i.e. deflection and reaction histories) are compared to the calculated response using the SBEDS V5.0 program [2]. The SBEDS analyses used the existing methodology to calculate the dynamic response of test panels, with some minor modifications for sandwich panels to account for the low mass of the interior insulation. Also, the reinforcement index of all the test panels was calculated as shown in Equation 1. This parameter is a measure of the tensile reinforcement ratio relative to the balanced reinforcement ratio, where crushing of the concrete occurs simultaneously (i.e. at the same midspan deflection) as yielding of the reinforcing steel. The reinforcement index, the observed damage level, and the measured maximum response of the test panels are used to verify and recommend improvements for existing response criteria.

$$\omega_i = \frac{A_s}{bd}\frac{f_{ds}}{f'_{dc}} \qquad (1)$$

where:

ω_i = reinforcement index
A_s = tensile reinforcing steel area
b = spacing of tensile reinforcing steel
d = depth of tensile steel reinforcement
f_{ds} = dynamic yield strength of reinforcing steel
= 77 ksi for conventional rebar
= 265 ksi for Grade 270 prestressing strands
f'_{dc} = dynamic compression strength of concrete = 1.19 f'_c

ANALYSIS OF BLAST TEST DATA

Three test series on precast panels were used in this study: shock tube tests sponsored by the Department of State (DoS) [3], high explosive arena tests sponsored by PCA/PCI at the Air Force Research Laboratory (AFRL) [4] and high explosive arena tests sponsored by DoS at AFRL [5]. The DoS shock tube test panels were 5 to 6 inches (127 to 152 mm) thick with an 8 ft. (2.44 m) span and simple supports. All the test panels were solid concrete panels reinforced with conventional steel reinforcement. The panels had damage levels ranging from low to heavy damage with peak measured support rotations ranging from 1 degree to 9 degrees. The peak pressures in these tests were in the range of 10 to 20 psi (69 to 138 kPa) and impulses ranged from 100 to 350 psi-msec (689 to 2413 kPa-msec). The PCA/AFRL test panels consisted of single-span panels with a nominal 10 ft. (3.05 m) span and simple supports, and continuous two-span panels with spans in the range of 11 ft. (3.35 m) to 13.5 ft. (4.1 m) and a 3 ft. (0.9 m) cantilever above the top support. All the test panels were sandwich panels. Half of the test panels were reinforced with conventional reinforcement, while the other half were reinforced with prestressed reinforcement. Some of the prestressed panels also had a relatively small amount of conventional reinforcement. With two exceptions (not included in this study), the shear connectors for the test panels were designed so that that the panels had 100% composite flexural response. The peak pressures in these tests were in the range of 30 to 90 psi (206 to 618 kPa) and impulses ranged from 170 to 300 psi-msec (1170 to 2068 kPa-msec). The test panels had moderate to heavy panel damage with peak support rotations up to 6 degrees. The DoS/AFRL test panels were similar to the PCA/AFRL panels except that they included both solid and sandwich panels.

All test panels were analyzed with SBEDS V5.0 using the measured blast loads for each test panel and using the default values for static and dynamic increase factors (SIF and DIF) for conventional and prestressed reinforcement. The average ratio of the calculated maximum inbound deflection to the measured values for the solid precast panels was 1.22 with a standard deviation value of 0.33. This included solid panels reinforced with conventional reinforcement and panels with prestressed reinforcement. This deflection ratio is consistent with the general degree of accuracy and conservatism of the SDOF method for calculation of the maximum inbound deflection of blast loaded components. A much lower average ratio was calculated for non-solid test panels (i.e. sandwich panels), as discussed in the next section. A small number of test panels with calculated support rotations less than 1 degree were excluded from this study. These panels had ratios significantly lower (i.e. more than 20% lower on average) than similar panels with larger support rotations. The reason for this is not known. It may involve simple connections for the test panels that initially provide some fixity. However, small deflections causing support rotations less than 1 degree are not of interest for reinforced concrete panels from a design standpoint.

COMPOSITE ACTION OF SANDWICH PRECAST PANELS

The average ratio of the calculated peak deflection to the measured deflection for sandwich test panels was equal to 0.92, which is much lower than that for solid panels. Sandwich panels were analyzed in SBEDS assuming they were fully composite for flexural response (i.e. have the same moment capacity as a solid panel of the same thickness). The cross section was input in the same manner as a solid panel of the same thickness with a negative supported weight that accounted for the reduced weight of the interior insulation layer. SBEDS calculated the average moment of inertia (i.e. the average of the gross and fully cracked moments of inertia) in the same manner as a solid panel, which is considered acceptable since the gross moment of inertia is typically within 5% of that for a solid panel of the same thickness and the cracked moment of inertia is considered to be similar for both types of cross section. The sandwich panels were assumed to yield ductilely with fully composite action and to maintain the maximum moment capacity between the yield deflection and the maximum calculated deflection. The much lower ratio of calculated to measured peak deflections for the sandwich panels, relative to solid panels, indicates this analysis approach requires modification.

The SDOF analysis methodology was modified by sub-categorizing the sandwich panels into panels with "ductile" and "non-ductile" shear connectors based on a small scale static testing by Naito et al. [6]. This testing showed that truss-type shear connectors with composite material exhibited an elastic brittle response in shear, while composite FRP pins produced an elastic-plastic response with higher deformation capacity. The average ratios of the calculated to measured peak deflection for sandwich panels with ductile and non-ductile shear connectors were 1.0 and 0.83, respectively. Based on these observations, the sandwich test panels were reanalyzed with SDOF analyses where the ultimate resistances calculated using a fully composite cross section were reduced by a factor C_s in Equation 2. The use of this reduction factor for the ultimate resistance of the sandwich panels caused the deflection ratios for all sandwich panels to be 1.2, on the average, for both types of shear connectors (see Table 1). The reduction factors in Equation 2 are generally consistent with static test results on sandwich panels by Naito et al. [7] where uniformly loaded sandwich panels had failure loads that corresponded to 80% to 90% of the calculated ultimate moment capacities of the panels for the two types of shear connectors.

$$R_u = R_{uc} C_s \qquad (2)$$

where:

R_u = ultimate resistance for calculating ductile response of sandwich panel to blast load
R_{uc} = ultimate resistance of panel assuming 100% composite flexural response
C_s = reduction factor based on ductility of shear connector between wythes
= 0.85 for ductile shear connectors
= 0.75 for non-ductile shear connectors

REBOUND RESPONSE OF PRECAST PANELS

A study on rebound response of the precast concrete test panels [8] showed that the resistance-deflection curves used to model SDOF response of reinforced concrete panels must account for significantly reduced stiffness during rebound response. Current SDOF-based design assumes that reinforced concrete panels have stiffness in rebound that is equal to the inbound stiffness. However, panel response typically results in significant concrete cracking during inbound response such that the panel has a much lower stiffness during rebound. Also, the study showed that the stiffness reduction is much greater in sandwich panels compared to solid panels. Empirical equations for the stiffness of solid and sandwich panels during rebound (i.e. after peak inbound deflection) were developed by trial and error, based on equations that have been developed for rebound response of reinforced concrete components to earthquake loading. Equation 3 shows the equation that was developed for the stiffness during unloading (i.e. during rebound while the deflection is still positive), K_{ur}. Equation 4 shows the equation for stiffness during rebound when the deflection is negative, K_r. When used in SDOF analysis, these equations caused a best fit to the measured rebound response from test panels, including the test panels discussed herein. Figure 1 shows an illustration of the resistance-deflection curves calculated with Equations 3 and 4 for each type of cross section.

$$K_{ur} = K_e \left(\frac{1}{\mu} \right)^{\gamma} \qquad (3)$$

$$K_r = K_e \left(\frac{1}{1 + \left(\frac{R_u}{R_{ur}} \right) (\mu - \mu^{\gamma})} \right) \qquad (4)$$

where:

- K_{ur} = unloading stiffness (i.e. rebound stiffness during positive deflections)
- K_r = rebound stiffness (i.e. rebound stiffness during negative deflections)
- K_e = inbound elastic stiffness (equivalent elastic stiffness for indeterminate components)
- μ = ductility ratio during inbound response
- R_u = ultimate resistance during inbound response
- R_{ur} = ultimate resistance during rebound response
- γ = empirical constant = 0.5 for solid cross section, 1 for sandwich cross section

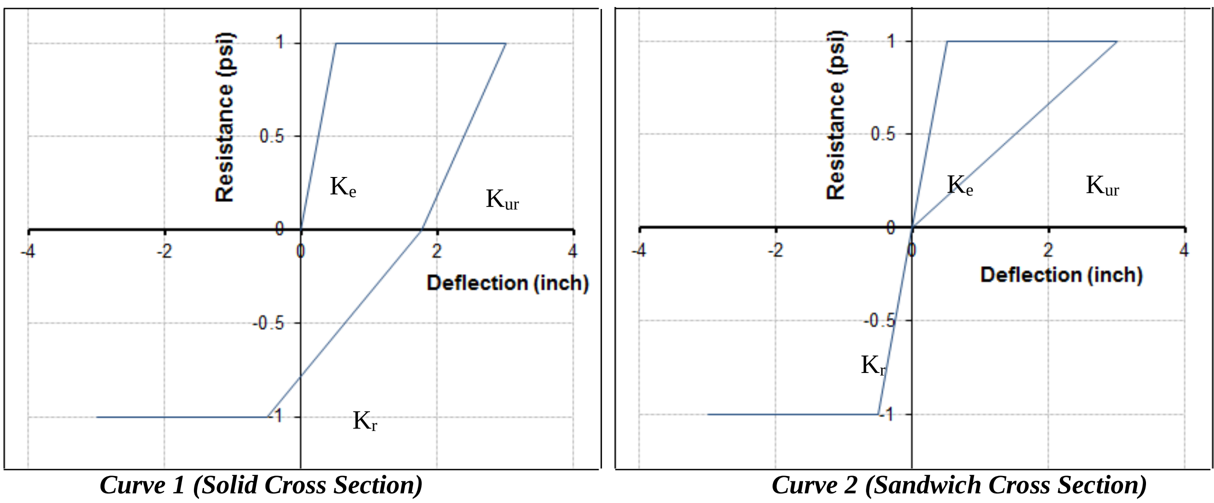

Figure 1. Illustrations of resistance-deflection curves for SDOF analysis of precast panels modeling stiffness degradation during rebound

Figure 2 shows a comparison of calculated resistance-deflection curves and deflection histories for a sandwich panel from the AFRL/DoS test series using the standard rebound approach and the modified approach with Equations 3 and 4. This sandwich panel had ductile shear connectors and its ultimate resistance was modified per Equation 2 to account for reduced composite action. Figure 2 shows how the reduced rebound stiffness causes a much better match between the calculated and measured rebound response for the test panel. This figure is representative of the typical difference in the calculated rebound response of the other test panels (including solid and sandwich panels) in this study using the standard approach of symmetric inbound and rebound properties and using a modified (reduced) rebound stiffness calculated with Equation 3 and 4.

Table 1 shows a summary of the ratios of the calculated peak inbound and rebound response compared to measured peak response for all the test panels. The calculated values are based on SDOF analysis that include the modification of the ultimate resistance for sandwich panels per Equation 2 and the modification of the rebound stiffness for all panels per Equations 3 and 4. The rebound deflection ratio in Table 1 compares the amount of calculated and measured rebound deflection that occurs after peak inbound response (see Equation 5). This removes some of the bias on calculated rebound response caused by error calculating peak inbound response. Table 1 shows that the modified analysis approach calculates the peak inbound and rebound response well, on the average (i.e. in a conservative manner within 20%) for all types of test panels. The large standard deviations for rebound response ratios in Table 1 indicate that it may be possible to refine the method used to calculate rebound stiffness of precast panels.

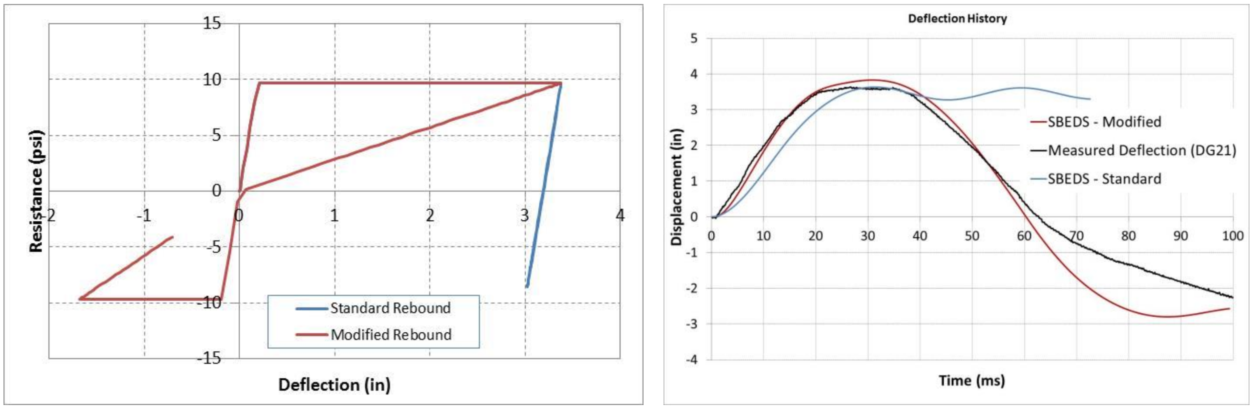

Figure 2. Calculated dynamic response of sandwich panel (AFRL/DoS) compared to measured response

Table 1. Table for ratios of calculated peak response with modified SDOF Analysis to measured response

Test Series	Reinforcing Type	Panel Type	Inbound[1] Maximum Deflection Ratio		Rebound Relative Maximum Deflection Ratio[2]	
			Avg.	**Std. Dev.[3]**	**Avg.**	**Std. Dev.[3]**
DoS Shock Tube	Conventional	Solid	1.24	0.44	1.00	0.74
AFRL/DoS	Conventional	Solid	1.13	0.17	1.12	N/A
AFRL/DoS	Conventional	Sandwich	1.22	0.22	1.21	0.23
AFRL/PCA	Conventional	Sandwich	1.15	0.22	1.23	0.70
AFRL/PCA	Prestressed	Sandwich	1.24	0.25	1.01	0.35
AFRL/DoS	Prestressed	Solid	1.32	N/A	Limited data	
AFRL/DoS	Prestressed	Sandwich	1.14	0.17	Limited data	
All solid panels			**1.22**	0.33	**1.01**	0.46
All sandwich panels (ductile shear connector)			**1.18**	0.22	**1.23**	0.61
All sandwich panels (brittle shear connector)			**1.20**	0.24	**0.91**	0.20

Note 1: Tests where calculated maximum support rotation is less than 1 degree are not included.
Note 2: See Equation 5 for a definition of this ratio for rebound response.
Note 3: No Standard Deviation (Std. Dev.) data is provided when there are less than 3 tests for a given category (N/A).

$$R_{dif} = \frac{X_{mi} - X_{ri}}{x_{mi} - x_{ri}} \qquad (5)$$

where:

R_{dif} = relative maximum rebound deflection ratio
X_{mi} = calculated maximum inbound deflection
X_{mr} = calculated maximum rebound deflection
x_{mi} = measured maximum inbound deflection
x_{mr} = measured maximum rebound deflection

RESPONE CRITERIA FOR REINFORCED CONCRETE PANELS

Table 2 shows the PDC (U.S. Army Corps of Engineers, Protective Design Center) response criteria correlating the peak response of concrete components with conventional and prestressing reinforcement to damage levels as given in PDC-TR 06-08 [9]. The PDC response criteria were developed specifically for anti-terrorism design based on available blast test data mostly prior to 2000. More recent test data, including the test series presented herein, were not available. Furthermore, the blast test data available to the PDC was especially limited for prestressed concrete. The AFRL tests and DoS shock tube tests discussed herein were used to develop more accurate response criteria for reinforced concrete components responding in flexure.

Table 2. Response Criteria for Reinforced Concrete Panels from PDC TR-08-01 Rev 1 (2008) [9]

<table>
<tr><th rowspan="2">Reinforce-ment Type</th><th rowspan="2">Construction</th><th colspan="2">Superficial [1]</th><th colspan="2">Moderate [1]</th><th colspan="2">Heavy [1]</th></tr>
<tr><th>μ_{max}</th><th>θ_{max} (deg)</th><th>μ_{max}</th><th>θ_{max} (deg)</th><th>μ_{max}</th><th>θ_{max} (deg)</th></tr>
<tr><td rowspan="3">Conventional Steel Rein-forcement</td><td>Flexure</td><td>1</td><td>N/A</td><td>N/A</td><td>2</td><td>N/A</td><td>5</td></tr>
<tr><td>Flexure with compression face reinforcement and shear reinforcing</td><td>1</td><td>N/A</td><td>N/A</td><td>4</td><td>N/A</td><td>6</td></tr>
<tr><td>Flexure with tension membrane action, L/h>=5</td><td>1</td><td>N/A</td><td>N/A</td><td>6</td><td>N/A</td><td>12</td></tr>
<tr><td rowspan="4">Prestressing Steel [2]</td><td>Flexure with $\omega_i > 0.3$ (Over-reinforced)</td><td>0.7</td><td>N/A</td><td>0.8</td><td>N/A</td><td>0.9</td><td>N/A</td></tr>
<tr><td>Flexure with $0.15 < \omega_i < 0.3$</td><td>0.8</td><td>N/A</td><td>$0.25/w_i$</td><td>1</td><td>$0.25/w_i$</td><td>1.5</td></tr>
<tr><td>Flexure with $\omega_i < 0.15$</td><td>1</td><td>N/A</td><td>N/A</td><td>1</td><td>N/A</td><td>2</td></tr>
<tr><td>Flexure with tension membrane action, L/h>=5</td><td>1</td><td>N/A</td><td>N/A</td><td>1</td><td>N/A</td><td>6</td></tr>
<tr><td colspan="8">1. Damage level description: Superficial = no visible permanent damage; Moderate = some permanent deflection, generally repairable, if necessary, although replacement may be more economical and aesthetic; Heavy = component has significant permanent deflections causing it to be unrepairable.
2. ω_i = reinforcement index calculate as shown in Equation 1for prestressed concrete.</td></tr>
</table>

Figure 3 shows the observed damage levels for solid precast panels from the DoS shock tube tests 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 time of failure estimated using the measured dynamic reaction loads. Panels with connection failure (one panel) and over-reinforced test panels from this test series are not included in Figure 3. Therefore, all the panels in Figure 3 had sufficient connection strength and were not over-reinforced.

The information in Figure 3 indicates that the type and number of layers of conventional reinforcing steel (i.e. rebar) affects the correlation between maximum support rotation and damage level. The test panels with rebar at each face can undergo support rotations up to 12 degrees without failing. The reinforcing steel at the compression face in these panels was able to provide compression resistance after concrete crushing in the maximum moment region. Figure 3 shows that much lower support rotations caused failure of the test panels with only a single layer of rebar at mid-thickness, which was initiated by concrete crushing in the maximum moment region. It is also notable that there is no shear reinforcement to laterally support the compression face steel after concrete crushing in any of these panels, which is required in Table 2 for higher response limits in reinforced concrete components.

The test panels in Figure 3 with rebar at both faces also have a much lower reinforcement index than the test panels with a single layer at mid-thickness (0.028 compared to values between 0.15 and 0.2) and this may also be important. Figure 3 shows that test panels with welded wire fabric (WWF) clearly have the lowest support rotations at failure. Failure of these panels was initiated by tensile rupture of the WWF, which is not nearly as ductile as rebar because of the manufacturing process for WWF.

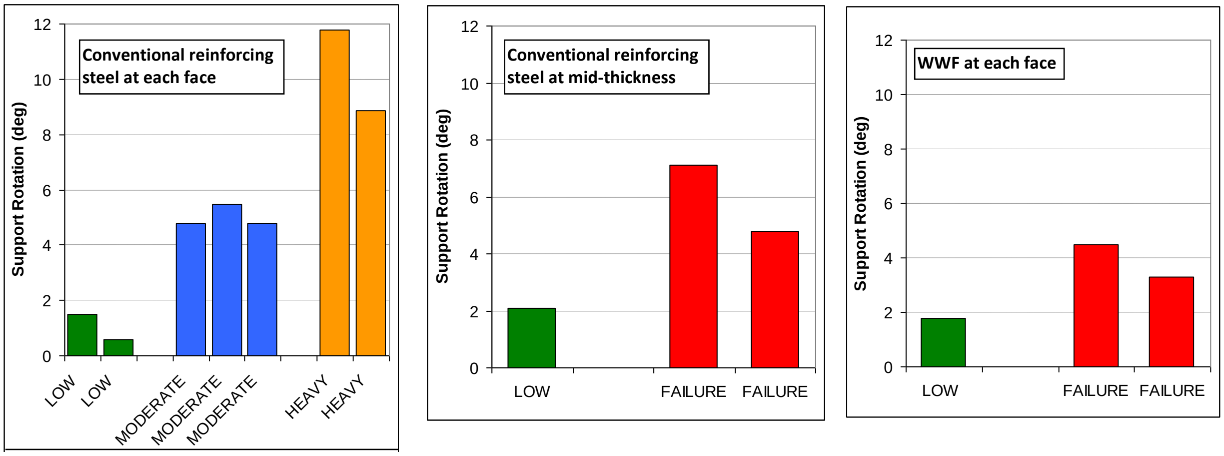

Figure 3. Measured Support Rotations vs. Damage Levels for Precast Test Panels from DoS Shock Tube Tests with Different Types of Steel Reinforcement

The AFRL/PCA blast tests on precast sandwich panels showed that panels with conventional and prestressing steel can have support rotation up to 5 to 6 degrees with moderate to heavy panel damage. Each wythe of the sandwich panels had reinforcement, so these panels can be considered to have two layers of reinforcement. There was no apparent difference in the relationship between support rotation and observed damage level for these test panels reinforced with prestressed steel and with conventional reinforcing steel (one-half of the test panels had each type of reinforcement). This is not consistent with the much lower response criteria for prestressed concrete compared to conventionally reinforced concrete in Table 2. The prestressed AFRL/PCA test panels had very low reinforcement index values, ω_i, which were all less than 0.03. The response criteria for concrete reinforced with prestressed steel in Table 2 increases with decreasing values of ω_i to a minimum value of ω_i equal to 0.15 Therefore, the AFRL prestressed panels have values of ω_i that are far below the minimum value considered in Table 2. A study by Au et al. [10] shows that ductility of prestressed concrete beams in static tests decreases significantly as the reinforcement index increases from values in the range of 0.05 to 0.15 and higher. This indicates that the AFRL prestressed concrete panels with a reinforcement index of 0.03 can be expected to have a much higher ductility, and therefore much higher response limits, than a component with the minimum value of 0.15 considered in Table 2.

The importance of reinforcement index on response criteria for all types of reinforced concrete components was investigated. The test panels with each observed damage level (i.e. Low, Moderate, Heavy, or Failure) were plotted in terms of measured support rotation vs. panel reinforcement index (ω_i), which was calculated using Equation 1. Panels with a given cross section type (i.e. solid and sandwich cross section) and reinforcement type (i.e. conventional rebar or prestressed reinforcement) were plotted separately. Figure 4 shows this plot for solid test panels with rebar. It also shows proposed response criteria curves for each damage level as a function of reinforcement index based on the data points. This figure indicates that the test panels with low ω_i values (and also two layers of rebar) will undergo less damage at a given support rotation than test panels with a higher ω_i (and one layer of rebar).

Figure 5a shows a plot of ω_i values vs. maximum measured support rotations for conventionally reinforced sandwich panels with each damage level (i.e. data points). This plot also has proposed response criteria curves for each damage level as a function of reinforcement index based on the data points. All the test panels have reinforcement in each wythe (i.e. at each face), as is typical for sandwich panel construction. A comparison of Figure 4 and Figure 5a shows that solid panels with conventional reinforcement can rotate to larger support rotations with less damage than sandwich panels with similar reinforcement indices.

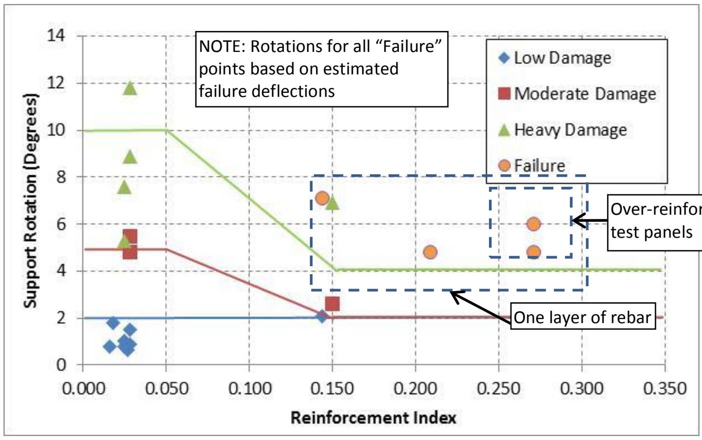

Figure 4. Reinforcement index vs. measured support rotation of conventionally reinforced solid concrete panels with three observed damage levels

Figure 5b shows a plot of ω_i values vs. maximum measured support rotations for prestressed solid and sandwich panels with each damage level. This plot also has proposed response criteria curves for each damage level based on the data points. These curves are defined in Equation 6, which is loosely based on the relationship found between reinforcement index and ductility ratio of prestressed beams by Au et al. [10]. Since there are not any data points with heavy damage, the response criteria curve for this damage level is conservatively estimated. All of the data points in Figure 5b except four are from sandwich panels. These four data points for prestressed solid panels overlie data points from sandwich panels with the same damage levels (i.e. Low and Moderate). Therefore, the available data points do not indicate a significant difference in the response criteria for prestressed solid and sandwich panels.

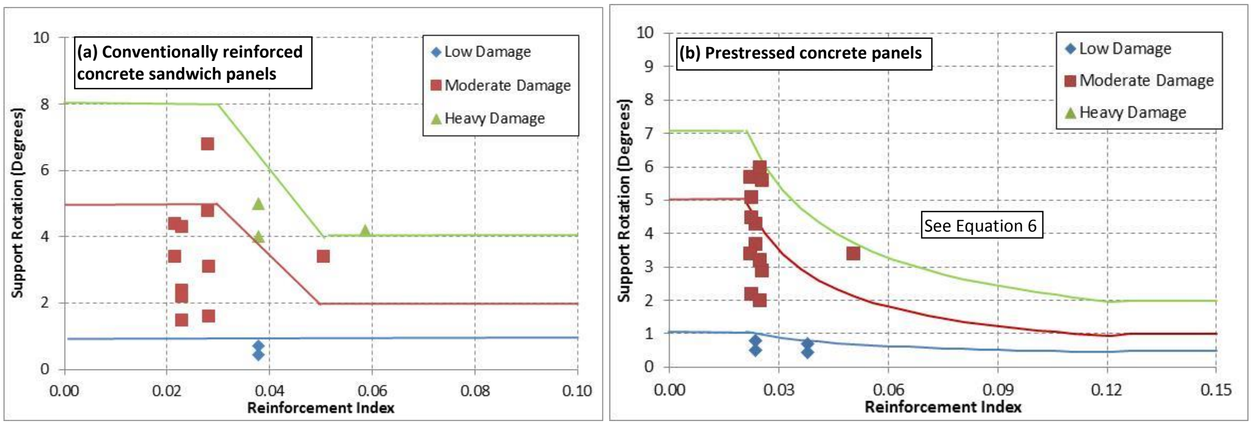

Figure 5. Reinforcement index vs. measured support rotation of (a) conventionally reinforced concrete sandwich panels and (b) prestressed concrete panels with three observed damage levels

The response criteria curves in Figure 4 and Figure 5 can be conservatively simplified as shown in Table 4. The current PDC response criteria requires shear reinforcement in maximum moment regions to laterally restrain the compression steel (i.e. prevent buckling, assuming that this steel provides the compression force of the resisting moment because the surrounding concrete has crushed) at support rotations greater than 5 degrees. The maximum deflections calculated with SDOF analyses for the test panels in this study, which were in good agreement with measured deflections, assumed the full resisting moment for test panels, even with support rotations as large as 10 degrees and no shear reinforcement. This indicates that the compression reinforcing steel in these test panels, which has a maximum reinforcing bar size of 0.5 inch (12.5 mm) diameter, was able to provide the required compression force after concrete crushing without shear reinforcement. Therefore, shear reinforcement is not required by the response criteria in Table 4 when the reinforcing bar diameter is equal or less than 5/8 inch (16 mm). This is a small extrapolation from the maximum 0.5 inch (12.5 mm) bar diameter in the test panels, which is considered acceptable since there was no evidence of any loss of moment resistance in the panels with 0.5 inch (12.5 mm) diameter reinforcing bars.

The test data in Figure 4 and Figure 5 generally indicate that the reinforcement index, which is a measure of the tension reinforcement ratio compared to the balanced reinforcement ratio for a reinforced concrete component, is an important parameter that affects the response criteria. More blast tests with test panels that have a higher reinforcement index would possibly confirm this apparent trend. However, from a practical standpoint, test panels with the lower reinforcement index values in Figure 4 and Figure 5 and reinforcement at each face were able to resist very high blast loads without failing.

$$\theta_{i,\min} \le \theta_i \le \theta_{i,\max} \quad \text{where} \quad \theta_i = A_i\left(\frac{1}{\omega_i}\right) \tag{6}$$

where:

θ_i = allowable support rotation for ith damage level
$\theta_{i,min}$ = minimum allowable support rotation for ith damage level defined in Table 3
$\theta_{i,max}$ = maximum allowable support rotation for ith damage level defined in Table 3
ω_i = tension reinforcement index based on prestressed steel calculated using Equation 1
A_i and B_i = constants for i$_{th}$ damage level defined in Table 3
Note: Use response criteria for prestressed concrete components in Table 3 for ω_i > 0.15

Table 3. Constants for Damage Levels to Prestressed Components

Parameter	Damage Level		
	Low	Moderate	Heavy
$\theta_{i,min}$	0.5	5	7
$\theta_{i,max}$	1.5	1	2
A	0.13	0.125	0.42
B	0.63	0.95	0.73

Table 4. Response criteria for non-load bearing reinforced concrete components with flexural response

<table>
<tr><th rowspan="4">Damage Level [1]</th><th colspan="7">Maximum Support Rotation [1,2,3] (degrees)</th></tr>
<tr><th colspan="5">Conventional steel reinforcement [4]</th><th colspan="2" rowspan="3">Prestressed steel Reinforcement [5,6]</th></tr>
<tr><th colspan="3">Solid cross section</th><th colspan="2" rowspan="2">Insulated sandwich panels [5]</th></tr>
<tr><th colspan="2">Reinforcement at each face</th><th>Single layer reinf.</th></tr>
<tr><th></th><th>$\omega_i \le 0.05$</th><th>$\omega_i > 0.05$</th><th>All ω_i</th><th>$\omega_i \le 0.03$</th><th>$\omega_i > 0.03$</th><th>$\omega_i \le 0.02$</th><th>$\omega_i > 0.02$</th></tr>
<tr><td>Low</td><td>2</td><td>1.5</td><td>1.5</td><td>1</td><td>1</td><td>1</td><td rowspan="3">See Equation 6</td></tr>
<tr><td>Moderate</td><td>5</td><td>2</td><td>2</td><td>5</td><td>2</td><td>5</td></tr>
<tr><td>Heavy</td><td>10</td><td>5</td><td>5</td><td>8</td><td>4</td><td>7</td></tr>
<tr><td colspan="8">1. Damage level description: Low = hairline cracking on both faces, no or very small permanent deflection; Moderate = significant cracking on non-loaded face, light spalling on the loaded face, small but noticeable permanent deflection; Heavy = significant cracking and spalling on both faces, significant permanent deflection.
2. ω_i = reinforcement index calculate as shown in Equation 1
3. Stirrups are required in the maximum moment regions for all framing components (i.e. beams) and for slab and panels with reinforcing bars greater than 0.625 inch diameter.
4. 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.
5. These criteria are intended for sandwich panels that are designed to act 100% compositely in flexure.
6. Solid or sandwich panels with prestressing steel strands conforming to ASTM 416.</td></tr>
</table>

SUMMARY AND CONCLUSIONS

Blast test data from recent blast tests on approximately 60 non-load bearing concrete precast panels subject to high blast loads were analyzed to verify SDOF-based design methodology. The test data included both solid and insulated (sandwich) precast panels, as well as conventional and prestressed reinforcing. This analysis showed that a modified SDOF methodology, as discussed in this paper, calculated peak dynamic response within 20% of the measured response, on the average, for all panel cross section and reinforcement types and was generally design-conservative. The SDOF analyses for sandwich panels included an empirical reduction factor based on the type of shear connectors (ductile or non-ductile) for the ultimate resistance calculated assuming a fully composite cross section. This assumes the panels are designed with sufficient shear connectors to fully develop the static yield strength of the reinforcing steel at the maximum moment locations. The modified SDOF analysis also included equations to calculate a reduced stiffness for rebound, which are based on equations developed for reinforced concrete framing components subject to earthquake loads. These modifications caused the calculated rebound response of the test panels to compare much better to measured rebound response than SDOF analyses using the standard assumption of equal inbound and rebound properties. The calculated peak rebound response was within 20% of the peak measured rebound response, on the average, although there was much more scatter in these comparisons, compared to the calculated and measured peak inbound response. Correlations between the observed damage, measured maximum support rotations, reinforcement and cross section types, and reinforcement index values of the test panels were used to develop improved response criteria for reinforced concrete components. The response limits are approximately twice as high, and more in some cases, compared to the current response criteria in PDC TR 06-8 Rev 1 for conventionally reinforced and prestressed concrete components with two layers of reinforcement and maximum limit values for reinforcement index as noted in Table 4. These response criteria are based on the above test data that was not available when the response criteria in PDC TR 06-8 Rev 1 were developed. These proposed criteria also include more requirements for components with the largest response criteria to help ensure more ductility.

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