Testing and Analysis of Connections for Blast-Loaded Precast Panels

Chuck Oswald[1] and Marlon Bazan[2]

[1]*Protection Engineering Consultants, USA* (coswald@protection-consultants.com)
[2]*Protection Engineering Consultants, USA* (mbazan@protection-consultants.com)

Abstract

This paper presents and discusses response of connections for precast concrete panels subject to blast loads in arena tests and shock tube tests. This analysis, and most of the tests, were sponsored by the U.S. Department of State Bureau of Diplomatic Security. The response of connections for a total of 50 panel spans tested in five arena blast tests and 14 panels tested in a shock tube are presented and analyzed. The connections are analyzed based on the calculated connection capacities, the mechanisms controlling the connection capacities, the calculated dynamic reaction loads applied to the connections, and the observed connection response (e.g. damage level or failure). The peak dynamic reaction loads for many of the test panels were much greater than the connection capacities, which was due in large part to high blast pressures (i.e. in the range 20 to 100 psi (140 – 690 kPa) that were applied to the test panels. Most of the connections resisted the dynamic reaction loads without failing. However, a significant number of connections that did fail that had a capacity controlled by the shear strength of bolts, or punching/pullout failure of concrete around embedded connection plates. There were very few failures of "overloaded" connections with a capacity controlled by weld strength or concrete failure around anchor bolts in shear, indicating that these types of connections may be more ductile under blast loading. These findings and the methods used to analyze the connection response are discussed.

INTRODUCTION

This paper analyzes and discusses response of connections for blast resistant components, primarily precast concrete panels, subject to blast loads in arena tests and shock tube tests. The response of connections for a total of 50 panel spans tested in five arena blast tests and 14 panels tested in a shock tube are presented and analyzed. The tests include single-span panels and two-span continuous panels connected to the test frame with a variety of bolted and welded connections. Many of the test panel connections were designed to be blast resistant using the typical approach for designing connections in UFC 3-340-02 [1] and other blast design documents [2]. In this approach, the static capacity of the connection, including any appropriate dynamic increase factors for material strengths, is designed to be larger than the ultimate static reaction load, V_u, which is the reaction caused by the ultimate dynamic resistance of the tested panel. The applied blast loads, maximum panel deflection, and the observed panel and connection damage levels were reported for all the test panels and the dynamic reaction loads were measured for approximately one-third of the panels. A number of connections failed during the tests, including connections that were designed to resist V_u, and most of the failed connections were bolted connections where the capacity was controlled by the shear strength of the bolts.

The dynamic reaction histories of blast resistant components can be easily calculated as part of the blast design process. However, there are not available methods to calculate the dynamic response of connections or to determine an equivalent static reaction load for connection design based on the dynamic reaction history. Therefore, the connection design of blast resistant components is not typically based on the dynamic reaction history of the component. The dynamic reaction history is used in this paper to analyze the response of connections of all the test panels, and is compared to the calculated connection capacities, the failure mechanisms controlling the connection capacities, and the observed connection damage (e.g. damage level or failure). The peak dynamic reaction loads, $V_d(t)$, for many of the test panels were much greater than V_u and also greater than the calculated connection capacities, V_c. This occurred because very high peak blast pressures (i.e. in the range 20 psi (139 kPa) to 100 psi(690 KPa)) were applied to the test panels, which increased the calculated dynamic reaction loads, but did not affect the reaction load, V_u, used to design many of the panel connections. Figure 1 shows a plot of $V_d(t)/V_c$ for a typical test panel with a high applied blast load. Most of the connections resisted dynamic reaction loads with a short duration peak load much greater than the static connection capacity without failing. Almost all of the "overloaded" connections that did fail had a capacity controlled by the shear strength of bolts, or punching/pullout failure of concrete around embedded connection plates. There were very few failures of "overloaded" connections with a capacity controlled by weld strength or concrete failure around anchor bolts in shear.

ANALYSIS OF CONNECTION RESPONSE

The connection response of the test panels is analyzed by comparing the calculated dynamic reaction load for the panel at the support, $V_d(t)$, with the calculated capacity of the connection at the support, V_c, and then comparing this ratio to the observed connection response. The dynamic reaction histories, $V_d(t)$, for the test panels were calculated using dynamic equilibrium equations from Biggs [3], as implemented in the SBEDS V5.0 program [4], based on the measured blast load and test panel properties. Dynamic reactions calculated with this approach compared well with measured dynamic reactions of the test panels where load cells were used. The connection capacities, V_c, were calculated using static LRFD (Load and Resistance Factor Design) methods with enhanced material strengths based on strain-rate effects. This includes a dynamic increase factor of 1.19 for the concrete compression strength, as recommended in UFC 3-340-02 [1]. No increase factors were used for bolts or welds, although some blast design documents (i.e. UFC 3-340-02) recommend a very small increase factor on the order of 5%. The calculated values for V_c include a material strength reduction factor (i.e. ϕ factor). Also, the values for V_c include the effects from any prying forces. The prying forces on welds and bolts of clip angle connections without gussets are based only on the dynamic moment capacity of the angle leg in bending, which was typically a 3/8 inch (9.5 mm) thick by 6 inch (152 mm) wide angle in the available blast tests. The typical assumption that prying is caused by the reaction force applied at the center of the angle leg multiplied by its moment arm to the base of the angle was used when calculating V_c for gusseted clip angles. No prying forces were included for flat strap plate connections.

$V_d(t)$ is compared to V_c for test panel connections using two parameters. First, the dynamic reaction overload is calculated at each time step, equal to $[V_d(t) - V_c]$, and this is divided by V_c to get a Dynamic Reaction Overload Ratio (DROR) at each time step. The DROR is integrated over the panel response time when it has a positive value (i.e. when $V_d(t) > V_c$) to the DROR integral, which indicates the relative magnitude and duration of that part of $V_d(t)$ that exceeds V_c. The units on the right vertical axis of Figure 1 show the DROR and the integral as a yellow area, which has units of msec. The second parameter is V_{d_max}/V_c, which is 5.0 for the example in Figure 1. A non-zero value for the DROR integral and a V_{d_max}/V_c ratio greater than 1.0 both indicate that a connection has been overloaded by the dynamic reaction force for some amount of time. The duration of the connection overload shown in Figure 1 of between 5 and 10 msec is typical for the test panels included in this study with overloaded connections. In addition to these two parameters, this analysis of the test panel connections also considers the specific response mechanism that controls the capacity V_c for each connection (i.e. the failure mechanism). For example, many of the test panels had a bottom connection where clip angles were attached to the test structure with drilled anchor bolts and welded to an embed plate in the test panel. V_c for most of these connections was controlled by the shear strength of the bolts for both inbound and rebound response. Other possible controlling mechanisms for this type of connection are failure of the concrete in the slab around the anchor bolts or pullout failure of the embedded plate from the concrete in the test panel during rebound response.

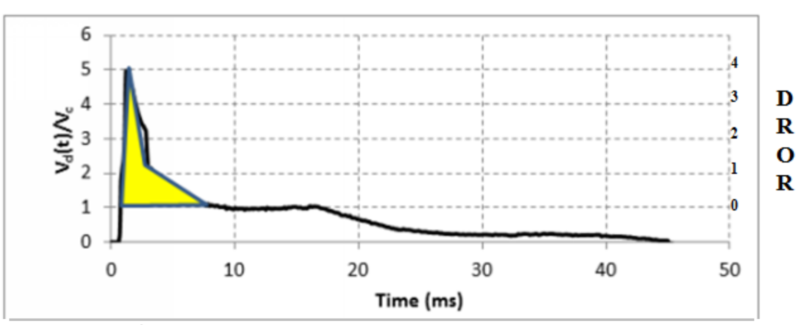

Figure 1. Ratio of Dynamic Reaction History to Connection Capacity and DROR

CONNECTION DATA FROM BLAST TESTS

There have been several recent blast test programs on blast-resistant precast concrete panels that were connected to the supporting test frame with connections similar to those typically used for precast construction. These tests include shock tube tests sponsored by the Department of State (DoS) [5], high explosive arena tests sponsored by PCA/PCI at the Air Force Research Laboratory (AFRL) [6] and high explosive arena tests sponsored by DoS at AFRL [7]. These tests generally had high applied blast loads (i.e. up to 100 psi (690 KPa) peak pressures and 350 psi-msec (2413 KPa-msec)) and medium to high levels of panel response (i.e. 2 degrees to 10 degrees of support rotation). In some of these tests, the dynamic reaction forces were also measured. There were also high explosive tests at AFRL where very large blast loads were applied to heavy steel stud walls that were heavily damaged, but did not fail, and were connected to the support frame with concrete anchor bolts [8]. The load path for all of these test panels included the connections, so that either the panels or the connections could fail from the applied blast loads. All tests where the panel failed are excluded from this analysis of the panel connections. Table 2 shows the applied blast loads and test results related to connection response from the DoS shock tube tests, including one test with a connection failure. These tests were conducted with load cells that measured the reaction load transmitted through the panel connections to a bottom support beam. These test panels were 5 to 6 inches (127 to 152 mm) thick with an 8 ft. (2.44 m) span and simple supports. They had damage levels ranging from low to heavy damage with peak measured support rotations ranging from 1 degree to 9 degrees. Table 1 shows a summary of the connections for the test panels in Table 2 with a calculated non-zero DROR integral and/or V_{d_max}/V_c ratio greater than 1.0. There were no cases where the test panel connections had a ratio of V_{d_max}/V_c in rebound that exceeded 1.0. The rebound capacity of all the test panels was controlled by pullout of the embed plates from the surrounding concrete. Note that Table 2 and Table 4 shows peak measured and calculated dynamic reaction loads, respectively. The bolted connection in Test 6 failed very early in the response while the measured dynamic reaction history was still increasing and the calculated peak dynamic reaction for this connection was greater than the measured peak reaction. Figure 2 shows typical comparisons between calculated and measured dynamic reaction histories from these tests.

Table 1. Calculated Dynamic Overload Parameters for Panel Connections in DoS Shock Tube Tests

Connection Type	Failure Mode	Panel	Top Connection[1]		Bottom Connection[1]	
			Integral of DROR (msec)	V_{d_max}/V_c	Integral of DROR (msec)	V_{d_max}/V_c
Clip angle with A325 or A490 bolt	Bolt Failure	1A	0.46	1.40	3.08	1.13
		6	0.0	0.97	1.33	1.25
		10	7.07	1.55	2.51	1.25
Gusseted angle with A325 bolt	Bolt Failure	5A	8.00	1.70	8.00	1.70
Gusseted angle with welds	Weld failure	8	14.20	1.93	14.20	1.93
Note 1: Based on inbound response. Red values indicate connection failure. V_{d_max} is maximum value from calculated dynamic reaction load.						

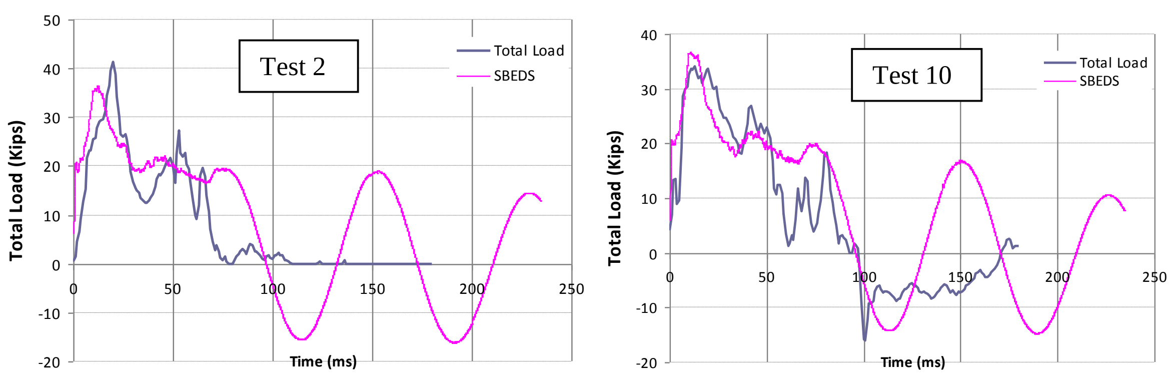

Figure 2. Comparison of Measured and Calculated Reaction Loads for Two Shock Tube Tests

Table 2. Summary of Connection Information for DoS Precast Concrete Panel Tests

Test No [1]	Peak Pressure (psi)	Impulse (psi-msec)	Connection	Connection Damage	Ultimate Static Reaction V_u (kip)	Inbound			Rebound		
						Max. Measured Reaction V_{max} (kips)[2]	Conn. Capacity V_c (kip)[3]	V_{max}/V_c	Max. Measured Reaction V_{max-r} (kips)[2]	Conn. Capacity V_{c-r}(kip)[3]	V_{max-r}/V_{c-r}
1A	16.8	232	Bolted Angles		24	35	31 (bot) 25 (top)	1.15(bot) 1.4 (top)	12	21	0.6
3	10.3	141	Bolted Angles		28	22	36	0.61	9	21	0.43
3A	20.6	277	Bolted Angles		28	31	36	0.87	N/A		N/A
5	17.7	119	Bolted, gusseted Angles		24	22	22	1.0	5	21	0.23
5A	20.1	277	Bolted, gusseted Angles		24	38	22	1.73	11	21	0.5
6	20.9	255	Bolted Angle (Bot) Halfen (Top)	Bot. bolt fail (inbound)	21	28	31 (bot) 40 (top)	0.9 (bot) 0.7 (top)	N/A		N/A
7	17.8	121	Welded angles		15	25	40	0.65	3.5	21	0.17
7A	20.3	303	Welded angles		15	31	40	0.8	N/A		N/A
8	20.4	311	Welded, gusseted angles		24	41	20	2.1	11	21	0.5
9	7.6	105	Welded angles (in tension from load)	Concrete damage[4]	24	19	21	0.95	8	48	0.2
10	20.8	350	Bolted angles	Concrete damage [4]	24	37	31 (bot) 25 (top)	1.2 (bot) 1.5 (top)	9	21	0.43

Note 1: All panels were 5 inch to 6 inch thick and reinforced to have dynamic ultimate resistance of approximately 5 psi. All panels are nominally 8 ft x 8 ft with an 8 ft vertical simply supported span. Panels had measured peak support rotations ranging from 1 degree to 9 degrees.
Note 2: Total measured reaction force at bottom of panels summing the two load cells (one load cell near each connection).
Note 3: LRFD capacity (i.e. including phi factor) for two connections at each support.
Note 4: Connection damage due to tension load on connection. Test 9 was loaded in rebound direction by initial shock tube load. Test10 has connection damage during rebound response.
Note 5: Blue text indicates measured peak reaction force exceeds connection capacity. Red text indicates connection failure (bolt shear failure in Test 6

The PCA/AFRL test panels consisted of single span M panels with a nominal 10 ft. (3.05 m) span and simple supports, and continuous two-span F 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 insulated sandwich panels. With two exceptions, the shear connectors for the test panels were designed so that that the panels had 100% composite flexural response. The connections of M-panels had load cells and were designed not to fail. The connections of the F-panels were only designed to resist conventional loads (i.e. were not designed to be blast resistant).

Table 3 summarizes the F- panels, applied blast loads, the measured maximum dynamic panel deflections, and measured peak support rotations. The test panels constructed as shown in Table 3 were subjected to high explosive blast loads in Test 1 and 2. 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), where larger blast loads were applied during Test 2. The test panels had moderate to heavy panel damage with peak support rotations up to 6 degrees, and heavy damage to connections of the F-panels including some failures. Table 4 shows calculated dynamic overload parameters for these panel connections and cases where connections failed or were damaged. Some of the panels had connection damage that obviously occurred during rebound (e.g. tension failure around studs of embed plates). These cases are not included in Table 4. Also cases of Halfen strap buckling were not included in Table 4 since the buckling length was very difficult to determine. However, this observed failure mode should be considered during blast design of slotted anchors.

Table 3. AFRL Panel Sandwich Test Panels

Panel	**Wythe** (inches)	**Reinforce-ment**	**Connections[1]**	**Concrete Strength** (psi)	**Conventional** A_s^+ (in^2)	**Conventional** A_s^- (in^2)	**Prestress** A_s^+ (in^2)	**Prestress** A_s^- (in^2)
F1	3-2-3	Prestressed	(B) Drilled Anchor; (M) Weld plate; (T) Weld plate	8723	0.116	0.116	0.51	0.51
F2	3-2-3	Prestressed	(B) Drilled Anchor; (M) Weld plate; (T) Weld plate	7907	0	0	0.51	0.51
F3	3-2-3	Conventional	(B) Drilled Anchor; (M) Weld plate; (T) Weld plate	4828	1.2	1.2	0	0
F4	3-2-6	Conventional	(B) Drilled Anchor (M and T) Halfen slotted anchor	4828	1.2	0.425	0	0
Note: (T) top connection, (M) middle connection of two-span panel, (B) bottom connection								

Table 4. Calculated Dynamic Overload Parameters for Panel Connections in AFRL/PCA Tests

Failure Mode	**Panel**	**Connection**	**Connection Capacity V_c (kips)**	**Test 1[1]** Integral of DROR (msec)	**Test 1[1]** V_{d_max}/V_c	**Test 2[1]** Integral of DROR (msec)	**Test 2[1]** V_{d_max}/V_c
Concrete failure of anchor bolts in shear	F1	Drilled anchor bolt at bottom connection	37.5	0.26	1.20	5.96	2.8
	F2		37.5	0.20	1.17	5.77	2.8
	F3		37.5	0.05	1.08	5.14	2.8
	F4		37.5	0.19	1.21	5.59	2.8
Concrete punching failure[2]	F1-Top	Weld plate connections at top and middle supports	59	0.00	0.79	1.37	1.62
	F2-Top		59	0.00	0.77	1.31	1.62
	F3-Top		62	0.00	0.68	0.85	1.54
	F1-Mid		59	13.75	2.93	22.68	7.83
	F2-Mid		59	12.05	2.99	21.56	7.83
	F3-Mid		62	6.46	2.84	16.23	7.45
Weld failure[2]	F1-Top	Weld plate connections at top and middle supports	60	0.00	0.79	1.37	1.62
	F2-Top		60	0.00	0.77	1.31	1.62
	F3-Top		60	0.00	0.68	0.85	1.54
	F1-Mid		60	N/A	N/A	22.68	7.83
	F2-Mid		60	12.05	2.99	21.56	7.83

Note 1: All red values indicate connections that failed, or possibly failed, during inbound response. Blue values indicate reported connection damage. Definite rebound connection failures are not included.
Note 2:Dynamic reactions for top and middle supports are sum from spans above (cantilever for top) and below.

Table 5 shows a summary of the DoS/AFRL test panels from Test A and B. These test panels were similar to the PCA/AFRL test panels except that the connections were designed to resist the blast loads. The connection strengths ranged from those designed to resist the ultimate equivalent static reaction load (V_u), up to connections designed to resist $3.5V_u$. Reaction loads were measured at many of the connections with load cell(s) that were connected to the panel support with a system of rods and clevises. Table 5 shows the observed damage levels of the test panels and connections, including several cases of connection failures. Table 6 shows calculated connection overload parameters for these test panels and the cases where connections failed during inbound (red text). The table also notes several test panels with connection failure (welded strap plates) during rebound response. The strap plates had approximately equal capacity during inbound and rebound, but much lower dynamic reaction loads during rebound. Therefore, inbound response is assumed to be the controlling case and used to calculate the overload factors in Table 6.

A factor that very possibly caused the rebound connection failures was that many unfailed weld plates of these test panels had very significant upward plastic deformation, as shown in Figure 3. High speed video clips also show the gap between the bottom of the test panels and the test frame slab increased during the panel response, indicating that the panels were moving upward as they deflected inward. This was apparently due to a slight misalignment of clevis rods in the reaction load measuring system at the bottom support. Figure 3 also shows a connection from these test panels with bolt shear failure.

The last test series included in this paper are blast tests conducted on heavy steel stud wall systems at AFRL with very large blast loads (i.e. peak pressures greater than 100 psi (689 kPa)) [7]. The test walls were attached to the concrete support slabs with 5/8 in. (16 mm) drilled anchors with 4 inch (100 mm) embedment and there were no connection failures during the tests. The test walls were heavily damaged. The combined tension and shear reaction loads on the anchor bolts were calculated with dynamic finite element analyses of the stud wall systems using the measured blast test loads and were compared to the calculated nominal capacities of the anchor bolts in combined shear and tension. This comparison showed that the shear overload factor, V_{d_max}/V_c for the bolts ranged between 2.0 and 5. The bolt tension overload factors ranged from 1.0 to 2.0. In both cases, the anchor bolt capacities were controlled by concrete failure

ANALYSIS OF CONNECTION TEST DATA

Table 7 shows a summary of the results from this analyses of the test panel connections in terms of calculated values for the two overload parameters, the mechanisms controlling the connection capacity (i.e. failure mode), and the observed connection damage level. The table shows the total number of connections and number of failed connections for each type of failure mode controlling the connection capacity. The table also shows the total number of connections and number of failed connections for given magnitudes of each overload parameter. Failure of any connection at a support is considered as a connection failure for the whole support in this table. Table 7 shows that dynamically overloaded connections were much more likely to fail if their connection capacity was controlled by bolt shear or concrete punching strength compared to the other failure modes. Table 7 also shows that overloaded connections where the capacity is controlled by weld failure and concrete failure around anchor bolts in shear are very unlikely to fail even when they have high calculated overload parameters. There was also an attempt to more directly correlate the magnitude of the two overload parameters to the probability of connection failure for each connection failure mode, but these correlations have very significant scatter. This is probably caused by an insufficient number of test data points to develop equations correlating the magnitude of the overload parameters to probability of connection failure and possibly due to the fact there are other contributing factors to connection response not included in these two overload factors.

Based on these observations, bolt shear and punching concrete failure are apparently more brittle failure mechanisms for blast-loaded connections. Also, weld failure and concrete failure around anchor bolts in shear are apparently more ductile failure mechanisms when the connections are “overloaded” by the dynamic reaction force. By extension of the results for punching shear failure in Table 7, embed plate pullout in tension from surrounding concrete can also be considered as a more brittle failure mode for blast-loaded components. This is consistent with observed concrete damage around embed plates that were in tension for test panels in the DoS Shock Tube tests shown in Table 2. More test data to confirm these trends would be very helpful.

Table 5. Test Panel Design for Series B

Test Series	Panel	Panel Type	Peak Pressure (psi)	Impulse (psi-msec)	Reinforcement	Connection Type	Damage Level	
							Panel	Connection
A	1S and 2S	Monolithic	26.2	109	#4@16"o.c	(T): Load cells (B): Drilled anchors	Low	Shear failure of anchor bolts at bottom supports.
	1C and 4C	Monolithic	24.4	140	3/8" prestress strand @13"o.c.	(T) PSA with strap (M) Welded plate (B) Load cells	Low	No Failures
	2C and 3C	Sandwich	26.7	191	#4@16"o.c. (2C) #5@13"o.c. (3C)	(T) Welded angles (M) Load cells (B) Load cells	Low	No Failures
B	3S	Sandwich	120	302	#5@10"o.c. plus W2.5 (12 wires) Total = 2.16in^2	(T) Welded angle (B) Drilled anchors	Heavy	No Failures
	4S	Sandwich	112	303	#5@10"o.c. (6#5 = 1.86 in^2) [9]	(T) Steel plate (B) Drilled anchors	Moderate	Shear failure of anchor bolts at top and bottom supports.
	5C	Monolithic	84 (top span) 82 (bot span)	326 (top span) 316 (bot span)	#4@16"o.c. (6#4 = 1.2 in^2)	(T) Welded angle (M) Welded plate (B) Load cells	Heavy	No Failures
	6C	Sandwich			#5@13"o.c. (7#5 = 2.17 in^2)	(T) PSA with strap (M) Steel plate (B) Load cells	Heavy	No Failures
	7C	Monolithic			3/8 trand@13"o.c. (0.60 in^2) [9]	(T) Welded angle (M) Welded plate (B) Load cells	Moderate	Failure of welds on mid-height connection during rebound.
	8C	Sandwich			3/8 trand@13"o.c. (0.60 in^2) [9]	(T) PSA with strap (M) Welded plate (B) Angle with drilled anchor	Moderate	<u>Top</u> – Pull-out failure of the PSAs in rebound – failure of the insert (right side) and concrete failure (left side) <u>Middle</u> – Weld failure of tie plate to panel (one side) and failure of tie plate weld to test structure (other side) <u>Bottom</u> - Shear failure of anchor bolts (right side), failure of clip angle weld to panel embed (left side)

Table 6. Calculated Overload Parameters for Connections of AFRL/DoS Test Panels

Failure Mode	Test	Panel	Connection	Connection Capacity V_c (kips)[1]	Integral of DROR at Support (msec)[1,2]	V_{d_max}/V_c (Note 1,2)	Comment
Shear Failure of Anchor Bolt	A	1S	Drilled anchor bolt at bottom connection (UNO)	18	0.9	1.46	
	A	2S		16	1.5	1.5	
	A	4C		29	0.4	1.2	
	AA	4C		29	9.2	4.0	
	B	3S		38	2.1	2.0	
	B	4S		33	4.0	2.7	Drilled anchors at top and bottom
	B	8C		43	5.2	2.3	Equal concrete and bolt shear capacity
Concrete Around Anchor bolt	B	3S	Drilled anchor bolt at top connection	114	1.06	2.0	Bolt is in shear
Weld failure	A	1C	(T) PSA strap (M)Weld plate	(T) 17 (M) 54	(T) 2.4 (M) 2.8	(T) 1.6 (M) 1.8	PSA weld failure possibly caused by panel uplift
	A	2C	(T) Welded angle	(T) 37	(T) 0.35	(T) 1.2	
	A	3C	(T) Welded angle	(T) 36	(T) 0.0	(T) 1.0	
	A	4C	(T) PSA strap (M)Weld plate	(T) 33 (M) 103	(T) 0.12 (M) 0.1	(T) 1.1 (M) 1.1	
	B	5C	(T) Welded angle (M) Weld plate	(T) 60 (M) 91	(T) 0.83 (M) 8.07	(T) 1.5 (M) 4.3	
	B	6C	(T) PSA strap (M) Steel plate	(T)62 (M) 193	(T) 1.35 (M) 1.87	(T) 1.7 (M) 2.1	
	B	7C	(T) Welded angle (M)Weld plate	(T) 61 (M) 224	(T) 1.41 (M) 1.41	(T) 1.6 (M) 1.8	Rebound failure of middle connection[3]
	B	8C	(T) PSA strap (M)Weld plate	(T)50 (M)103	(T) 3.16 (M) 8.7	(T) 2.1 (M) 4.0	Connections failed during rebound. [3]

Note 1: (T) top connection, (M) middle connection, (B) bottom connection (for continuous panel spans only)
Note 2: All red values indicate connections that most possibly failed during inbound response.
Note 3: Uplift of panel during inbound contributed to rebound failure. The dynamic reactions compared to connection strength are calculated for inbound (controlling case vs. rebound).

Upward panel movement acting on weld *Failed bolted Connection*

Figure 3. Photographs of Connections

Table 7. Summary of Connection Responses Based on Failure Modes and Overload Parameters

Overload Parameter	Parameter Range	Connection Failure Mode							
		Shear Failure of Bolt		Weld failure[1]		Concrete Failure Anchor Bolts in Shear		Concrete Punching Shear	
		Total	Failed	Total	Failed	Total	Failed	Total	Failed
DROR Integral	***0 to 2 msec***	***5***	***3***	***13***	***1***	***5***	***0***	***3***	***0***
	2 to 4 msec	***4***	***1***	***4***	***1***	***0***	***0***		
	4 to 20 msec	***5***	***2***	***7***	***0***	***4***	***0***	***6***	***3***
V_{d_max}/V_c	*1 to 2*	*11*	*3*	*12*	*2*	*5*	*0*	*3*	*0*
	2 to 6	*3*	*3*	*9*	*0*	*9*	*0*	*6*	*3*
Total[2]		15	6 (43%)	21	2 (10%)	13	0 (0%)	9	3(33%)

Note 1: Both failed weld connections were probably affected by large support movements or upward panel deflections caused by load cell measuring system.
Note 2: Total cases are shown twice in table, once for DROR Integral and again V_{d_max}/V_c ratio.

It is interesting that bolt shear failure in this analysis of connection response is considered to be less ductile than concrete response around anchor bolts in shear. This is counter to the assumption in ACI 318 [9] for design of anchor bolt connections against earthquake loads, where connections with a failure mode in the concrete are considered to be less ductile than connections where the capacity is controlled by bolt strength. This can be explained by the fact that connection loads caused by earthquakes have numerous, longer duration cycles of response, whereas connection loads caused by blast loads have only one significant load cycle with a very high peak magnitude, short duration, and high strain-rate. A high strain-rate causes an increase in strength to steels that is generally inversely related to yield strength [1] and a reduced ductility. The strain rate effects are very low (i.e. in the range of 5%) for typical high strength steel used for bolts. Since welds are typically lower strength than bolts, the welds will have a higher dynamic strength increase factor. However, this does not explain the large difference in the observed connection response of bolts and welds. Other researchers have noted cases where welds of open web steel joints were highly overloaded by calculated reaction forces from blast loading and did not fail [10].

High strain-rates cause a very significant increase in concrete strength, especially concrete response involving tensile stress [1] (e.g. concrete pullout or punching cone failure around a connection). Therefore, it is expected that connections strengths controlled by concrete failure, including punching failure and concrete failure around embeds and anchor bolts, will be greater under blast loading than static loading. Also, all connections involving concrete failure around anchor bolts or embeds benefit from the fact that concrete failure will not cause connection failure if the cracked concrete can remain in-place long enough to provide a support for the connection before rebound response relieves the loading. This is particularly true for concrete resisting shear from an anchor bolt because the cracked concrete must be displaced laterally and upward before the bolt can fail.

SUMMARY AND CONCLUSIONS

Blast resistant connection are typically designed to resist the equivalent static reaction load from the ultimate resistance of the panels, V_u, without consideration of the dynamic reaction load. High applied peak blast pressures on blast-resistant components, including the test panels discussed in this paper, will very often cause the peak dynamic reaction loads to exceed V_u, and therefore the connection capacity, for a limited duration on the order of 5 to 10 msec. The calculated peak dynamic loads exceeded the LRFD-based connection capacities in approximately 60 connections of the test panels discussed in this paper, and they exceeded by a factor greater than 2 (and as high as 5) in approximately one-half of these cases. These dynamic “overload” cases caused approximately 20% of the connections to fail. Test panel connections with a capacity controlled by bolt shear and punching/pullout failure of concrete around the connection were much more likely to fail (i.e. 30% to 50% of connections failed) when they were overloaded by the peak dynamic reaction load compared to connections with a capacity controlled by weld failure and concrete failure around anchor bolts in shear (i.e. 10% or less of the connections failed).

Therefore, welded connections and anchor bolt connections in shear controlled by concrete capacity responded in a much more “ductile” manner to reaction loads from blast than bolted connections and connections controlled by concrete punching shear or embed pullout. Some possible explanations for this observed trend between connection response mode, or failure mechanism, and apparent ductility are discussed in the paper. Based on these observations, the traditional approach of designing connections to resist the reaction load from the ultimate resistance of the blast-loaded component (V_u), without regard for the dynamic reaction load, can be considered sufficient for connections controlled by a "ductile" response mechanism against dynamic reactions from blast loads. However, connections with their capacity controlled by other response modes (e.g. bolt failure and punching or pullout failure of the concrete around the connection) should also be designed to resist the peak calculated dynamic reaction load, V_{d_max}. This additional consideration of the maximum dynamic reaction force will probably be the more stringent design requirement whenever the peak applied blast pressure exceeds approximately 1.5 times the ultimate resistance of the connected component. These observations assume the connection capacity is calculated using a LRFD (Load and Resistance Factor Design) method including a material strength reduction factor (i.e. ϕ factor) and a load factor of 1.0 for the design connection load. The connection capacity also includes a dynamic increase factor of 1.19 applied to the concrete compression strength for calculation of the connection capacity.

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