BLAST DESIGN OF REINFORCED CONCRETE AND MASONRY COMPONENTS RETROFITTED WITH FRP

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ABSTRACT

Fiber-reinforced polymer (FRP) products are used as an alternative to traditional methods for strengthening and retrofitting concrete and masonry structures to resist blast loads. The development and experimental validation of a methodology for modeling the response of blast loaded concrete and masonry structural components retrofitted with FRP, as well as corresponding response criteria, is important since these types of components often require upgrades in order to provide personnel protection in blast-loaded buildings.

This paper discusses the development of a SDOF-based procedure for designing FRP upgrades to blast loaded masonry and concrete walls by Protection Engineering Consultants for the U.S. Army Corps of Engineers, Protective Design Center. This includes the methodology used to determine the flexural stiffness and ultimate flexural and shear resistance of the upgraded walls. The methodology for estimating the flexural resistance of concrete and masonry components is based on current codes and guidelines (ACI-318 and ACI 440.2R). Experimental data from previous shock tube tests on concrete and masonry walls retrofitted with FRP were used to validate the upgrade design procedure by comparing the observed and calculated response of the tested components. Furthermore, proposed response criteria were developed for flexural and shear response of the walls for damage levels used for DoD antiterrorism design. These damage levels can be correlated to those used in UFC 3-340-02 for explosive safety.

INTRODUCTION

Externally bonded fiber-reinforced polymer (FRP) systems have been used to strengthen and retrofit existing concrete structures around the world since the mid-1980s. The ability and practicality of externally bonded FRP systems for strengthening concrete and masonry walls to resist blast loads have been demonstrated by several testing and research programs to date. However, a simplified design methodology including response criteria is needed so that FRP retrofits can be designed to resist blast loads. This paper presents a SDOF-based procedure for calculating the blast response and resulting damage of concrete and masonry walls retrofitted with FRP systems. Much of the effort was focused on the developing of the resistance-deflection relationships for the retrofitted walls. The calculated and measured responses are compared for validation of the methodology. The derivation of the response limits is based the correlation of observed response and the observed level of damage of the retrofitted walls in the shock tube tests.. This methodology has been developed for the U.S. Army Corps of Engineers, Protective Design Center (PDC) by Protection Engineering Consultants (PEC) to be included into the next version of the SBEDS (Single-Degree-of-Freedom Blast Effects Design Spreadsheets) blast design code.

FIBER REINFORCED POLYMER (FRP) SYSTEMS

Fiber-reinforced polymer (FRP) systems are used as an alternative to traditional methods for strengthening and retrofitting concrete and masonry structures. FRP systems are composite laminates made up of many small diameter, high strength fibers embedded in a polymeric resin matrix. The fibers provide strength and stiffness to the composite, while the resin matrix provides stress transfer between fibers and acts as a bonding agent between the concrete or masonry substrate and the composite laminate. The fibers in the composite laminate can be oriented in one planar direction (unidirectional) or in two or more planar directions (multidirectional), providing enhanced in-plane tensile resistance in only one or more directions, respectively.

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The most widely used type of polymer matrix for applications in building structures is epoxy resins. Other types of polymer matrix include polyesters, vinyl esters, or phenolics resins. The main type of fibers used commercially in civil engineering applications are glass (GFRP), carbon (CFRP) and aramid (AFRP). Table 1 lists the common types of FRP systems used for strengthening building components and corresponding material properties. In general, carbon fiber composites have higher strength and stiffness (elastic modulus), but lower strain capacity, than glass or aramid composites. Carbon fibers have high resistance to alkali or acid attacks, but they can cause galvanic corrosion when in contact with metals (which can be prevented by the resin) and do not have the electrical insulation properties of E-glass fibers. Aramid fibers have good mechanical properties with high strength and stiffness (elastic modulus) in between the corresponding values for glass and carbon fibers. Aramid fiber composites offer the additional advantage of toughness or impact resistance and good electrical and thermal insulation properties. FRP composites made of high-strength steel fibers (SFRP) are also available. The steel fibers consist on steel wires twisted together to form steel cords which, due to the unwinding effects of the cords at high tensile loads, provides high strain capacity (in the range of 0.02 to 0.05). The twisted steel wires also provide adequate bond to the polymer matrix. All the fiber types in Table 1 can be used for blast design and the choice of fiber type is often an economic one.

Table 1. Partial List of Common FRP Systems for Blast Upgrades to CMU and RC Walls

FRP Type	Yield Strength* (ksi)	Modulus of Elasticity (ksi)
Carbon Fibers (CFRP)	100 to 350	15000 to 21000
E-Glass Fibers (GFRP)	75 to 200	3000 to 6000
Aramid Fibers (AFRP)	100 to 250	6000 to 10000
* Manufacturer's recommended value not including any environmental effects or debonding		

The FRP reinforcement is typically applied to a concrete or masonry wall surface as a wet layup system or a pre-cured system. In a wet layup system, a mat with only the fibers is applied to the wall and resin is placed around the fibers that bonds the fibers together and bonds them to the wall. The resin cures against the wall in-situ similar to cast-in-place concrete. In a pre-cured system, the resin is placed around the fibers and cured at the manufacturing plant. The fiber-resin system, which is typically a very thin mat, is attached to the wall with structural adhesive in a process similar to attaching wall paper to a wall. In both cases, the preparation of the wall surface is an extremely important part of achieving a bond that can transfer the full load capacity of the fibers into the wall so that the fibers and wall act as a composite system. However, even in the case of good wall preparation, this bond is typically the weak point of the overall composite system due to environmental degradation of the resin and adhesive. The existing substrate below the concrete or masonry wall surface should have enough tensile and shear strength to transfer the force and develop the strength of the bonded FRP reinforcement.

The durability of the constituent materials of FRP systems can be significantly reduced by prolonged exposure aggressive environment (humidity, salt water, alkalinity, or freezing-and-thawing cycles). Protective coatings can be used to reduce or eliminate the effect of environmental is they are maintained through the life of the FRP systems. ACI 440-2R provides environmental reduction factors for different exposure conditions to be used in the design of FRP retrofits for concrete structures (see Table 2). These values are conservative and do not necessarily cover the different types of FRP systems or explicitly account for the use of protective coatings. Durability test data can be obtained from manufacturers for FRP systems with and without protective coatings. Furthermore, the mechanical properties of FRP materials suffer degradation when subjected to high temperatures. The elastic modulus of the resins and bond properties between the FRP and concrete surface are reduced at temperatures above 140 to 180 ^{o}F (typical range of glass-transition temperature for commercially available FRP systems). However, the mechanical properties of the fibers are affected at higher temperatures (350 ^{o}F for aramid fibers, 530 ^{o}F for glass fibers and 1800 ^{o}F for carbon fibers). Test results have shown that the tensile strength of the overall composites can reduce more than 20% at temperatures above 480 ^{o}F for GFRP and CFRP materials.

Table 2: Environmental Reduction Factor, C_E (ACI 440.2R)

Exposure Conditions	Fiber Type	Environmental reduction factor, C_E
Interior exposure	Carbon	0.95
	Glass	0.75
	Aramid	0.85
Exterior exposure (bridges, piers, and unenclosed parking garages)	Carbon	0.85
	Glass	0.65
	Aramid	0.75
Aggressive environment (chemical plants and wastewater treatment plants)	Carbon	0.80
	Glass	0.50
	Aramid	0.70

EQUIVALENT SDOF MODELING FOR REINFORCED CONCRETE AND MASONRY WALLS RETROFITTED WITH FRP

The Single-Degree-of-Freedom (SDOF) methodology is a widely accepted design approach by the U.S. government and industry for analyzing the dynamic response of structural components subjected to blast load. In the SDOF methodology, the structural component is analyzed as an equivalent spring-mass system. The spring and mass properties of the SDOF model are calculated so that the SDOF system will have the same displacement history as the point of maximum deflection on the component for a given blast load. This methodology is well documented in a number of sources including UFC 3-340-02 and ASCE (1997), and it has been implemented into the SBEDS (Single-Degree-of-Freedom Blast Effects Design Spreadsheets) blast design code distributed by the U.S. Army Corps of Engineers, Protective Design Center (PDC) (PDC-TR 06-01, 2008).

A necessary part of the SDOF methodology for analysis of reinforced concrete (RC) and concrete masonry unit (CMU) walls retrofitted with FRP is the definition of the resistance function, which represents the resisting force developed by the structural component at a given deflection during its dynamic response to the applied blast load. For RC and masonry walls retrofitted with FRP, the resistance function can be defined as a bilinear elastic-plastic function (with very limited plasticity) as shown in Figure 1. The ultimate resistance (R_u) and flexural stiffness (K) are calculated based on static structural analysis relationships as discussed below. The yield deflection, X_y, is also shown in Figure 1 and is defined in Equation 9. The resistance vs. deflection relationship is only valid out to the deflections corresponding to the maximum ductility ratios and/or support rotations allowable for the design.

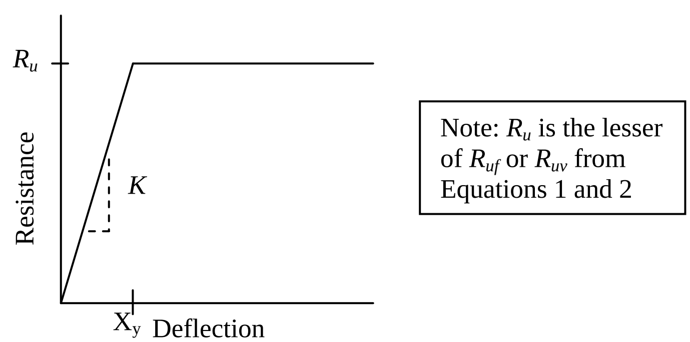

Figure 1. Resistance vs Deflection Curve for RC and CMU wall retrofitted with FRP

The ultimate resistance, R_u is a function of the ultimate moment capacity or the ultimate shear capacity of the retrofitted wall. For a one-way simply supported member with uniform load, the ultimate flexural resistance (R_{uf}) is given in Equation 1. This is the ultimate load capacity of the wall in flexural response. Due to the brittle nature of FRP response and a lack of blast test on walls with indeterminate supports, the wall is not assumed to have sufficient ductility to develop multiple yield locations before failing. If the shear capacity of the wall is less than the flexural capacity, then the ultimate resistance is based on the shear load capacity (i.e. uniform pressure load causing shear failure near the supports) (R_{uv}) of the wall, as shown in Equation 2. In this case, the wall is assumed to responds in flexure up to pressure load causing a shear load at the critical shear section of the wall equal to the shear capacity. Therefore, R_u is based on the controlling load capacity (i.e. lesser of R_{uf} or R_{uv}). The challenge for modeling the blast response of RC and masonry walls reinforced with FRP is to determine appropriate values for the moment capacity, shear capacity, and moment of inertia as discussed below.

$$R_{uf} = \frac{8M_{du}}{L^2} \qquad \textbf{Equation 1}$$

where:

R_{uf} = ultimate flexural resistance
M_{du} = ultimate dynamic positive moment capacity per unit width of the member at midspan
L = member span

$$R_{uv} = \frac{V_{cap}}{(C_v L - \kappa d)} \qquad \textbf{Equation 2}$$

where:

R_{uv} = ultimate shear resistance
V_{cap} = shear capacity of the member per unit width along the supports
L = member span
d = distance from the extreme concrete/masonry fiber in compression to the tension reinforcement (equal to the member thickness for FRP-retrofitted wall)
κ = 1 if critical shear location is at a distance d from support, otherwise κ = 0
C_v = shear span coefficient $V/(R_u L)$ from PDC-TR 06-01 Table 4-3 and 4-4
(C_v = 0.5 for simply supported uniformly loaded members)

The ultimate dynamic moment capacity of a RC or masonry member reinforced with FRP composites is equal to the static moment capacity calculated using standard flexure theory and analysis methods from ACI 440.2R and ACI 318-05 and dynamic material strengths as explained below. Although ACI 440.2R does not explicitly address masonry structures, its methods for calculating the ultimate moment capacity of a rectangular section can be applied to masonry wall members upgraded with FRP. The ultimate moment capacity is therefore calculated as shown in Equation 3. The stress and force terms in are based on strain compatibility (i.e. linear distribution of axial strain along the component cross section), equilibrium of internal forces, and the controlling mode of failure (i.e. FRP tension failure or debonding, or concrete/masonry crushing).

$$M_u = A_f f_f (h - c) + A_s f_s (d - c) + A_s' f_s' (c - d') + C_c \left(c - \frac{k_2 c}{2} \right) \qquad \textbf{Equation 3}$$

where:

M_u = ultimate moment capacity per unit width
A_f = area of FRP reinforcement per unit width
A_s = area of tension steel reinforcement per unit width
A'_s = area of compression steel reinforcement per unit width
f_f = stress in the FRP laminate
f_s = stress in the tension steel reinforcement (typically equal to yield strength)
f'_s = stress in the compression steel reinforcement
h = distance from the extreme concrete/masonry fiber in compression to the FRP laminate (i.e. typically total thickness of member)
d = distance from the extreme concrete/masonry fiber in compression to the tension steel
d' = distance from the extreme concrete/masonry fiber in compression to the compression steel
c = depth of the neutral axis from the extreme concrete/masonry fiber
k_2 = factor defining location of the resultant compression force C_c relative to the extreme compression fiber
C_c = resultant compression force in concrete/masonry based on strain compatibility and enhanced dynamic compression strength. C_c and k_2 may be calculated using the rectangular Whitney stress block or based on a simplified concrete stress-strain curve.

The wall material dynamic compression strength and reinforcing steel dynamic yield strength in Equation 3 include an increase factor in the range of 1.2 to account for strain-rate effects, as discussed in UFC 3-340-02. A static increase factor of 1.1 is also included for the reinforcing steel yield strength to account for typical variation between the actual yield strengths and minimum specified value. No static or dynamic increase factor is assumed for FRP. Concrete or masonry crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain (ε_{cu} = 0.003 for RC or ε_{cu} = 0.0025 for masonry). Rupture or debonding of the externally bonded FRP is assumed to occur if its strain reaches the design rupture strain (ε_{fu}) or debonding strain (ε_{fd}) before the concrete reaches its maximum usable strain. The FRP stress is assumed proportional to the strain based on the modulus of elasticity for the FRP reported by the manufacturer. Any conventional reinforcing steel is assumed ductile enough that it will yield and will not strain to failure prior to the concrete/masonry or FRP.

The design limit strength of the FRP for blast loading (f_{fb}) is defined in Equation 4. The environmental reduction factor (C_E) in Equation 4 varies from 0.95 to 0.50 as defined in ACI 440.2R-02 and accounts for possible degradation of FRP properties caused by long-term environmental exposure. The bond-dependent coefficient for blast loading, Km_b, is calculated as shown in Equation 5. Based on comparisons to blast test data that are discussed later in this paper, K_{mb} can be based on the procedures in ACI 440.2R-02 rather than the more conservative criteria in ACI 440.2R-08. However, both versions of ACI 440.2R are developed for static loading and improved modeling will be possible if there is future test data that will allow development of new equations for K_{mb} and f_{fb} specifically for strain-rates consistent with response of retrofitted walls to blast loading.

$$f_{fb} = K_{mb} f_{fu} \qquad \textit{where} \qquad f_{fu} = C_E f_{fu}^* \qquad \textbf{Equation 4}$$

where:

f_{fb} = blast design limit stress in the FRP (considering environmental and debonding effects)
f_{fu} = ultimate strength at failure accounting for environmental conditions
C_E = the environmental reduction factor (see Table 2)
f_{fu}^* = the ultimate strength of the FRP reinforcement reported by the manufacturer
K_{mb} = bond-dependent coefficient for blast response

$$K_{mb} = \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{nE_f t_f}{2000000}\right) \le 0.9 \qquad \text{for } nE_f t_f < 1000000$$

Equation 5

$$K_{mb} = \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{500000}{nE_f t_f}\right) \le 0.9 \qquad \text{for } nE_f t_f > 1000000$$

where:

K_{mb} = bond-dependent coefficient for blast response
ε_{fu} = design rupture strain of FRP reinforcement = f_{fu} / E_f
n = number of plies of FRP reinforcement
E_f = tensile modulus of elasticity of FRP (psi)
t_f = nominal thickness of one ply of FRP reinforcement (in)

The stiffness (K) in Figure 1 is defined for different support and loading conditions as a function of the concrete or masonry modulus of elasticity, the component span length, and the component moment of inertia. For the case of one-way uniformly loaded members with simple supports (i.e. the most common condition for walls), the flexural stiffness is determined by Equation 6. The resistances and stiffnesses for other boundary conditions are determined in a similar manner as shown in Equation 1, Equation 2, and Equation 6 except that different constants are used and the negative moment capacity of components with fixed boundary conditions is used (PDC-TR 06-01).

$$K = \frac{384}{5}\frac{EI_{cr}}{L^4}$$ **Equation 6**

where:

L = clear span of the member
E = concrete modulus of elasticity
I_{cr}= fully cracked moment of inertia per unit width

As stated in Equation 6, the fully cracked moment of inertia is used to determine the stiffness of the equivalent SDOF system for a RC or masonry component reinforced with FRP. In reality, the moment of inertia changes during response to blast load from the uncracked, or partially cracked moment of inertia if there is cracking from previous loading, to a fully cracked moment of inertia. Comparisons between the calculated response of the equivalent SDOF system and the measured response from a number of blast tests shows that the equivalent SDOF system can provide results that are sufficiently accurate for blast design if the stiffness is based on a single moment of inertia value from the fully cracked cross section.

Based on the current state-of-the-knowledge, FRP applied to the surface of walls is not assumed to increase the wall shear capacity against lateral blast loads. The shear capacity in Equation 2, V_{cap}, is therefore equal to the dynamic diagonal shear capacity of the concrete or masonry wall. For RC walls, the dynamic diagonal shear capacity of concrete per unit width, V_c, can be calculated as shown in Equation 7 (PDC TR 06-01).

$$V_c = 2\sqrt{f'_{dc}}\,d$$ **Equation 7**

where:

V_c = concrete dynamic diagonal shear capacity per unit width (lb/in)
f'_{dc} = concrete dynamic compressive strength (psi)
d = distance from the extreme concrete fiber in compression to the tension steel (in)

The dynamic diagonal shear capacity per unit width of masonry walls, V_m, can be calculated using Equation 8. Conservatively the masonry static compressive strength, f'_m, is used instead of the dynamic compressive strength. The net shear area per unit width, A_{net}, is the sum of the CMU block area and grouted area through the wall thickness. This area can be calculated based on minimum dimensions for the web and face shell for standard CMU blocks. The use of net area for shear capacity is based on a similar approach in ACI 530-02 (2002). For fully grouted CMU, the net area is based on the whole cross section of the wall. This approach agrees well with limited blast test data for masonry walls controlled by shear response, as discussed later in this paper. The shear strength of ungrouted CMU walls can be increased significantly by grouting the voids of the walls as part of the FRP retrofit. In all cases, the wall connections to the supporting frame and/or foundation must transfer the calculated reaction force from the retrofitted wall into the supports.

$$V_m = 2\sqrt{f'_m}\,A_{net}$$ **Equation 8**

where:

V_m = masonry dynamic diagonal shear capacity per unit width (lb/in)
f'_m = masonry static compressive strength (psi)
A_{net} = net shear area per unit width (in^2/in)

The yield deflection, X_y, in Figure 1 is calculated as shown in Equation 9. This represents the deflection causing the ultimate dynamic moment capacity, M_{du}, in the retrofitted wall panel, or shear capacity, V_{cap}, depending on the controlling failure mode. In the case that the moment capacity controls the ultimate resistance, the yield deflection is the deflection where either; 1) rupture or debonding of the FRP occurs, or 2) crushing in the masonry or concrete occurs, depending on the mode that controls the ultimate moment capacity of the wall.

$$X_y = \frac{R_u}{K}$$ **Equation 9**

COMPARISON OF BLAST DESIGN METHODOLOGY TO BLAST TEST DATA

The blast design methodology described in the previous sections was used to calculate the maximum response of concrete and CMU walls upgraded with FRP and subjected to shock tube tests. The design methodology was incorporated into a beta version of the SBEDS blast design code and used to calculate the maximum deflections and maximum dynamic reaction forces, which were compared to measured values. Measured blast loads were used in the SDOF analyses and the FRP properties were based on manufacturers' published material property information. The applied shock loads shown in the following tables represent a fairly wide range of blast loads from accidental industrial explosions. This range of shock loads is considered sufficiently "dynamic" to cause similar wall response (i.e. similar high strain-rates) as high explosive loading.

The environmental factor, C_E, in Equation 4 was set equal to 1.0 in all comparisons to test data since the tests were done relatively quickly after the FRP was applied to the wall and there was no reason to assume any environmental degradation. In all cases, the test walls spanned 8 ft one-way in the vertical direction between simple supports and all FRP shown in the test summary tables was installed on the unloaded face in the direction of the span of the walls. FRP was also applied to the loaded side of all CMU walls to resist rebound response. Dynamic reaction forces were calculated from the applied load and calculated resistance in the equivalent SDOF systems for each test wall as described in PDC-TR 06-01. The calculated properties of the equivalent SDOF systems for all the test walls and more details on the test walls and test data are provided elsewhere (PEC, 2009). The masonry prism compression strength, f'_m, is a variable of some importance for determining the equivalent SDOF system, particularly regarding the wall shear strength. The compressive strengths of lightweight CMU prisms were measured in one of the test series as shown in Table 3. The strengths for ungrouted and 3000 psi grout were used to determine f'_m for the other test series where there were no measured values. In cases where the walls were partially grouted, f'_m was determined by interpolation between the no grout and 3000 psi grout cases based on the percentage of grouted void space.

Table 3: Measured Prism Compressive Strengths

Grout Strength	Compressive Strength of CMU Prisms
No grout	1950 psi
3000 psi	2100 psi
5000 psi	2650 psi

Tests on Concrete Wall Panels Upgraded with FRP Composites

A total of 11 tests were performed on six reinforced concrete (RC) walls upgraded with Carbon FRP composites as summarized in Table 4 (BakerRisk, 2008). Walls showing minor damage after the first test were re-tested. The 4-inch thick RC walls had one layer of steel reinforcement for rebound response at the loaded face that reinforced the walls during rebound response. The moment resistance against inbound response (i.e. the predominate response to the shock loads) was provided by the FRP composite attached to the unloaded face of the walls. Figure 2 shows a picture of test specimen SP1 installed in the shock tube test. Table 4 summarizes the comparison between the measured response and the analytical response. This comparison shows that the predicted values are within 10% of the measured values on the average. The calculated deflections of non-failing walls were less than the yield deflections except for one test (SP7). Some delamination between the FRP and concrete was reported for Tests SP2, SP3, and SP5 near the edges of strips, but the deflections were predicted accurately without any consideration for this delamination in the analysis.

Unreinforced and Reinforced Concrete Masonry Walls Upgraded with FRP Composites

Table 5 shows summaries of shock tube tests on unreinforced and reinforced CMU walls retrofitted with E-glass FRP (BakerRisk 2003, BakerRisk 2004). The test walls were constructed of nominal 6-inch thick CMU blocks that were unreinforced except for two walls summarized in Table 5. Only the reinforced cells in these walls were grouted. One of the unreinforced test walls was fully grouted and two were ungrouted. Figure 2 shows a picture of one of these specimens (specimen No. 9) installed in the shock tube test. The comparisons in Table 5 show that the predicted maximum deflection values (X_{max}) are within 10% of the measured values, on the average, and the predicted maximum dynamic reactions (V_{max}) are within 20% of measured values, on the average, for tests with measured dynamic reactions. The information in Table 5 shows that both shear and flexural response modes were observed in the tests. The methodology was accurate in predicting the observed response mode in all cases except one.

Also, strain gages were placed on the FRP in these tests and very high strains (i.e. more than twice the calculated debonding strain and more than the manufacturer's reported static yield strain) were measured in one test where there was obvious debonding, but the wall did not fail. This indicates that the FRP can debond in the highest strain region (i.e. maximum moment region) during response to blast load, where the high strain demand only occurs for a limited amount of time (typically milliseconds or tens of milliseconds) until rebound occurs and reduces the stresses. More testing in high response ranges is needed to understand the maximum dynamic strain that the FRP can attain. Conservatively, the dynamic strains are not assumed to be enhanced compared to static values until there is more testing to demonstrate this.

Figure 2. Test Walls Upgraded with FRP Composite in Shock Tube (BakerRisk, 2008; BakerRisk, 2004)

Table 6 shows summaries of shock tube tests on unreinforced CMU walls retrofitted with Aramid FRP (i.e. Kevlar®) (WBE, 1999). The test walls in this series were all nominal 8-inch thick lightweight CMU blocks with no steel reinforcement. One of the walls was ungrouted while all other walls were fully grouted. The comparisons show that the predicted deflection values are conservatively calculated as 1.34 times greater than the measured values, on the average, and the predicted maximum dynamic reactions are within a few percent of measured values. The methodology was accurate at predicting the observed response modes (i.e. shear or flexural response) shown in Table 6.

RESPONSE CRITERIA

Response criteria specify quantitative response limits corresponding to different blast damage levels. Typically response criteria are developed based on test data and therefore the available test data from the previously described shock tube tests were used for this purpose. Response criteria for brittle components, such as concrete and masonry components reinforced with FRP, are typically based on the ductility ratios, as opposed to support rotations. The maximum ductility ratio (μ) was calculated for all test specimens using the measured maximum mid-span deflection and the calculated elastic yield deflection (X_y=R_u/K). The yield deflection was calculated using the controlling ultimate resistance value (i.e. based on Equation 1or Equation 2 depending on whether wall capacity was controlled by flexure or shear load capacity).

The first step towards developing blast response criteria for concrete and masonry walls reinforced with FRP was to establish, a set of damage levels shown in Table 8. This table also shows how these damage categories would map into the Damage Levels used by the U.S. Department of Defense (PDC-TR 06-08, 2008). Figure 3 shows examples of observed flexural damage levels. Figure 4 shows an example of shear failure observed in a CMU panel retrofitted with E-glass FRP.

Table 4: Comparison of Calculated and Measured Response of Reinforced Concrete Walls Upgraded with Carbon FRP

Test No.[1]	Blast Load		FRP Layers[3]		SDOF Properties			X_{max}		V_{max}		μ	Controlling Mode
	P	I	Type	Layers	Mass	R_u	K	Calc	Calc/Meas	Calc	Calc/Meas		
	psi	psi-ms			psi-ms²/in	psi	psi/in	in		lb/in			
SP1	9.5	79	Carbon FRP	0.5	899	8.03	2.27	1.49	1.19	145	0.87	0.35	Flexure
SP2[2]	13.8	107	Carbon FRP	0.5	899	8.03	2.27	1.85	1.06	183	1.03	0.49	Flexure
SP3[2]	15	110	Carbon FRP	0.5	899	8.03	2.27	1.93	0.86	183	0.96	0.64	Flexure
SP5[2]	18	146	Carbon FRP	0.5	899	8.03	2.27	2.88	0.89	265	1.08	0.92	Flexure
SP6	16	161	Carbon FRP	1	899	10.91	4.17	2.42	0.97	377	1.32	0.95	Flexure
SP7[2]	20	188	Carbon FRP	1	899	10.91	4.17	2.72	0.84	425	1.29	1.24	Flexure
SP8	37.5	304	Carbon FRP	2	899	12.32	7.45	3.13	Fail	522	0.89	1.32	Flexure
SP9	16	145	Hardwire 3x2-20-12	1	899	11.67	4.82	1.91	1.09	348	0.88	0.72	Flexure
SP10[2]	16.5	177	Hardwire 3x2-20-12	1	899	11.67	4.82	2.22	0.89	424	1.05	1.03	Flexure
SP11	6.9	381	Carbon FRP	1	899	10.91	4.17	2.73	1.21	461	1.44	0.86	Flexure
SP12[2]	9	383	Carbon FRP	1	899	10.91	4.17	2.73	Fail	461	1.59	1.34	Flexure
Avg.									**1.0**		**1.1**		

P = applied blast pressure
I = blast load impulse
X_{max} = maximum midspan deflection.
V_{max} = maximum dynamic reaction force along supports
R_u = ultimate resistance of test wall in controlling response mode (i.e. shear or flexure)
K = flexural stiffness of test wall
Mass = mass per unit area of test wall
$\mu = X_{max} / X_y$ = ductility ratio calculated using the measured maximum mid-span deflection and the calculated elastic yield deflection ($X_y = R_u/K$).

Notes:
1. All walls had a concrete compressive strength of 4000 psi are simply supported at top and bottom, 8 ft clear span and 8 ft wide, 4 in thick. Loaded face reinforcement: #3 rebar @ 10 in O.C. with 0.5 in clear cover provides strength in rebound. No reinforcement on unloaded face except for FRP.
2. Denotes retest
3. See Table 7 for FRP properties

Table 5: Comparison of Calculated and Measured Response of CMU Walls Upgraded with E-Glass FRP

Test No.[1]	Blast Load		f'_m	Percentage of Voids Grouted	FRP Layers[2,4]	SDOF Properties			X_{max}		V_{max}[5]		μ	Controlling Mode
	P	I				Mass	R_u	K	Calc	Calc/Meas	Calc	Calc/Meas		
	psi	psi-ms	psi			psi-ms²/in	psi	psi/in	in		lb/in			
6	4.75	135	2000	100	1	893	8.57	4.27	1.38	1.53	244	N/A	0.45	Flexure
6A	7.5	193	2000	100	1	893	8.57	4.27	2.13	0.93	355	N/A	1.14	Flexure
7	4.5	120	1700	0	1	439	5.21	4.19	1.37	0.91	225	N/A	1.21	Flexure
7A	7	179	1700	0	1	439	5.21	4.19	3.3	Fail	241	N/A	4.84	Shear
8	3.25	102	1700	0	0.69	439	6.69	3.02	1.5	1.20	193	1.02	0.56	Flexure
8A	4.5	107	1700	0	0.69	439	6.69	3.02	1.67	0.95	214	0.87	0.79	Flexure
8B	5	134	1700	0	0.69	439	5.21	3.02	2.12	Fail	227	0.87	1.30	Shear
9[3]	4.3	128	2000	33	0.75	589	7.36	4.08	1.5	1.20	257	1.36	0.69	Flexure
9A[3]	5.15	139	2000	33	0.75	589	7.36	4.08	1.65	1.02	279	1.39	0.90	Flexure
9B[3]	7	197	2000	33	0.75	589	7.36	4.08	2.28	0.83	321	N/A	1.53	Flexure
10[3]	5	124	2000	25	0.75	552	7.40	3.9	1.65	1.20	268	1.33	0.72	Flexure
Avg.										**1.1**		**1.2**		

See Table 4 for definition of terms.

Notes:

1. All walls are simply supported at top and bottom, 8 ft clear span and 8 ft wide and 5.625 in thick. All walls with A, B, or C after test number designate retests.
2. All FRP was Fyfe Tyfo SEH-51A except No. 9, 9A, 9B, 10 with 1 layer of SEH-25A equal to 0.5 layers of Fyfe Tyfo SEH-51A
3. Vertical steel reinforcement in these tests: #4@24" (Tests No. 9, 9A, 9B); #4@32" (Test No. 10). These walls were also reinforced with Dur-A-Wall horizontal reinforcement at 16 inch spacing.
4. See Table 7 for FRP properties.
5. Tests with dynamic reaction gages that did not function properly are marked with N/A.

Table 6: Comparison of Calculated and Measured Response of CMU Walls Upgraded with Aramid FRP

Test No.[1]	Blast Load		f'_m	Percentage of Voids Grouted	FRP Layers[2,3]	SDOF Properties			X_{max}		V_{max}		μ	Controlling Mode
	P	I				Mass	R_u	K	Calc	Calc/Meas	Calc	Calc/Meas		
	psi	psi-ms	psi			psi-ms²/in	psi	psi/in	in		lb/in			
1	2	66	1950	0	1	527	13.4	7.73	0.38	1.27	127	0.85	0.17	Flexure
1A	4	123	1950	0	1	527	13.4	7.73	0.7	1.75	236	1.01	0.23	Flexure
1B	7.5	64	1950	0	1	527	7.83	7.73	1.06	1.32	294	1.25	0.79	Shear
2	10	328	2100	100	1	1227	14.0	7.79	1.59	1.22	529	1.06	0.72	Flexure
2A	16	178	2100	100	1	1227	14.0	7.79	1.64	0.91	464	1.09	1.00	Flexure
3	8.8	270	2650	100	1	1227	16.0	7.96	1.45	1.61	494	1.19	0.45	Flexure
3A	19	668	2650	100	1	1227	16.0	7.96	4.48	Fail	745	0.93	n/a	Shear
4	20	146	2100	100	1.5	822[4]	11.1	7.49	1.83	1.83	429	1.08	0.68	Flexure
4A	17	226	2100	100	1.5	822[4]	11.1	7.49	2.86	Fail	507	0.92	n/a	Flexure
5	10.5	79	2100	100	1.41	822[4]	11.5	7.52	1.13	1.88	307	1.34	0.39	Flexure
8	12	275	2650	100	1	1227	16.0	7.96	1.55	0.82	514	0.86	0.95	Flexure
10	7.5	170	2650	100	1.5	822[4]	12.7	7.69	1.11	1.11	355	0.93	0.61	Flexure
10A	9	243	2650	100	1.5	822[4]	12.7	7.69	1.53	1.02	127	0.85	0.91	Flexure
Avg.										**1.34**		**1.05**		

See Table 4 for definition of terms

Notes:

1. All walls built with 7.625 inch thick CMU spanning 8 ft vertically with simple supports and are 8 ft wide. All walls with A, B, or C after test number designate retests.
2. All retrofits were DuPont AK-60 FRP. Tests 4, 5, 10,10A have window openings where a "replacement area" of FRP was placed on each side of opening. This additional FRP was averaged over the solid wall area. The opening had plywood cover that was not blast resistant.
3. See Table 7 for FRP properties.
4. Reduced mass due to window opening at center of wall

Table 7: **FRP Material Properties for Wall Tests**

FRP Type	Manufacturer's Ultimate Tensile Strength	Young's Modulus	Thickness
	psi	psi	in
Carbon (Table 4)	120000	7000000	0.055
Hardwire 3x2-20-12 (Table 4)[1]	144000	9700000	0.047
E-glass (Table 5)	83400	3790000	0.05
Aramid (Table 6)	110000	6000000	0.03
Note 1: Hardwire material has steel wire rather than fibers, but the design theory applies to this material.			

Table 8: Damage Categories for Flexural Response of RC and CMU panels Retrofitted with FRP

Observed Damage Level Categories	Wall Damage Included in Category	Corresponding PDC Damage Level
Minor	Minor cracking in wall and epoxy. Minor FRP delamination	Moderate
Moderate	Significant delamination of FRP	Moderate
Severe	Severe cracking of wall	Severe
Collapse	Wall failed within 10 ft of test structure	Hazardous Failure
Blow Out	Parts of wall blown 10 ft away from test structure	Collapse

Next, the tests exhibiting flexural response were plotted in terms of their ductility ratios and observed damage levels as shown in Figure 5 and Figure 6. CMU tests that showed predominantly shear damage, or failed in shear, were categorized separately in terms of Shear Damage or Shear Failure and plotted in terms of their ductility ratios and two observed damage levels, as shown in Figure 7. This figure shows the ductility ratio relative to the deflection when the wall yields in shear. Figure 5 through Figure 7 also show proposed response limits in terms of ductility ratios defining the upper and lower bounds of each damage level. The intent of the response limits, or response criteria, is to generally err on the conservative side compared to the scatter in the data, but not to require that all outliers must be on the conservative side.

Figure 3. Damage levels of CMU walls upgraded with FRP (BakerRisk, 2004; BakerRisk, 2008)

Figure 4. Shear damage to CMU walls upgraded with FRP (BakerRisk, 2003)

Table 9 and Table 10 summarize the ductility ratios bounding the damage levels in Figure 5 through Figure 7. A comparison of Figure 5 and Figure 6 indicate that separate response criteria are not needed for concrete and masonry walls upgraded with FRP. Given the brittle nature of shear failure and the data shown in Figure 7, a ductility limit of 1.0 is proposed as the lower bound for Shear Failure damage level. The proposed ductility ratio limits in Table 9 and Table 10 are in the range typically associated with brittle component response to blast loads. For example, they are consistent with the range of ductility ratios for concrete walls controlled by shear response in Design of Blast Resistant Buildings for Petrochemical Facilities (ASCE, 1997).

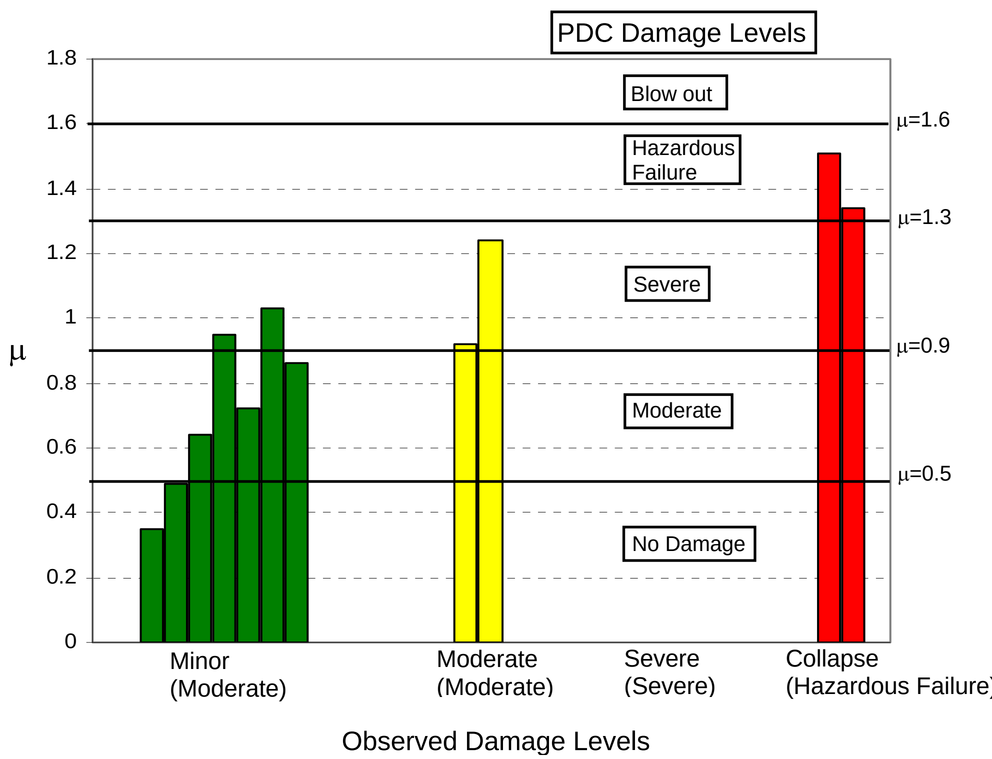

Figure 5. Observed Damage and Maximum Ductility Ratios for RC panels (Flexural Response)

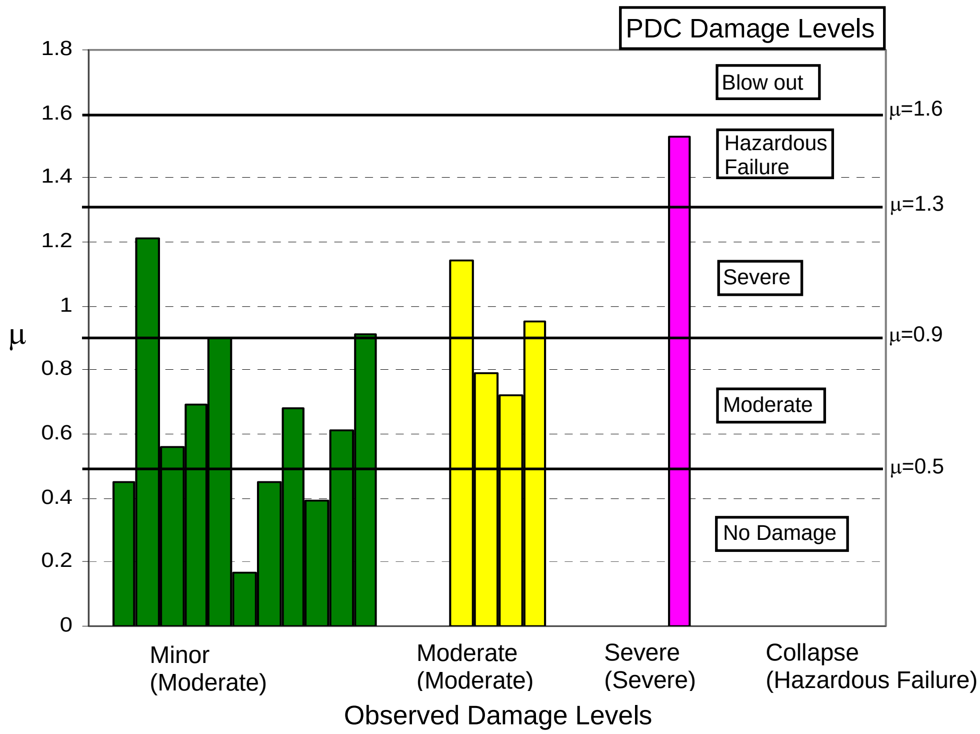

Figure 6. Observed Damage and Maximum Ductility Ratios for CMU panels (Flexural Response)

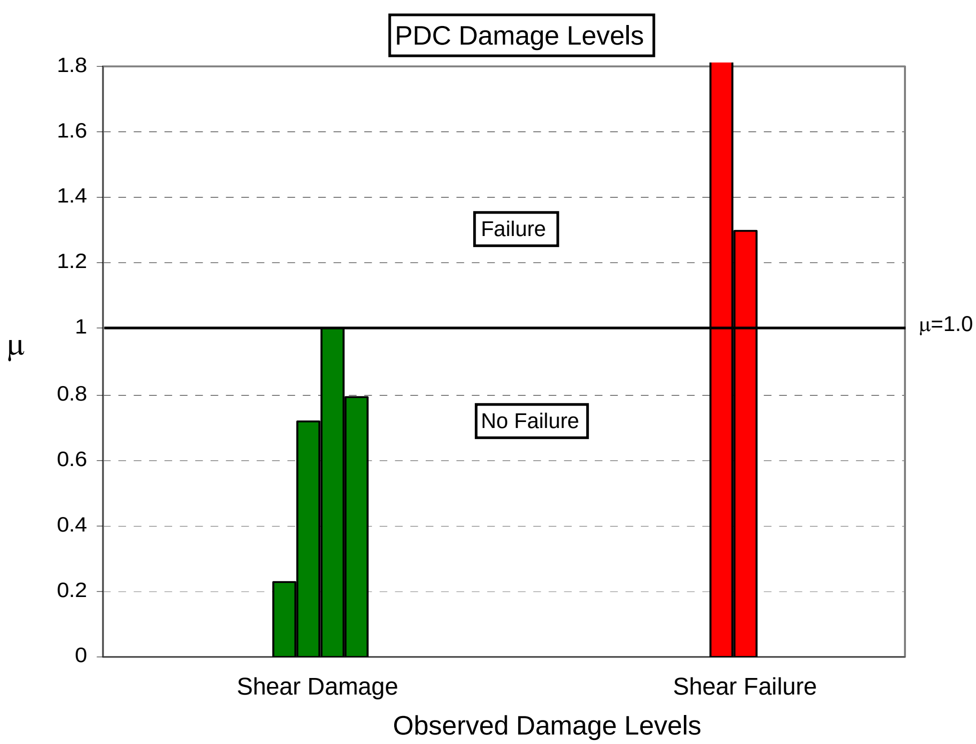

Figure 7. Observed Damage and Maximum Ductility Ratios in CMU Panels (Shear Response)

Table 9: Proposed Ductility Ratio Limits for Flexural Response of RC and CMU panels Retrofitted with FRP

PDC Damage Level	**Observed Damage Level**	**Maximum Ductility**
No Damage	N/A	$\mu \leq 0.5$
Moderate	Minor and Moderate	$0.5 < \mu \leq 0.9$
Severe	Severe	$0.9 < \mu \leq 1.3$
Hazardous Failure	Collapse	$1.3 < \mu \leq 1.6$
Blow Out	Blow Out	$\mu > 1.6$

Table 10: Proposed Ductility Ratio Limits for Shear Response of RC and CMU panels Retrofitted with FRP

PDC Damage Level*	**Observed Damage Level**	**Maximum Ductility**
No Failure	Moderate shear damage	$\mu \leq 1.0$
Failure	Shear failure	$\mu > 1.0$
∗ The PDC does not have explicitly defined damage levels for shear, but it required prevention of shear failure.		

SUMMARY AND CONCLUSIONS

This paper describes a procedure to design concrete and masonry walls upgraded with FRP responding in flexure against blast loads. The upgraded wall is modeled as an equivalent SDOF system, which is a commonly used approach for design of other types of blast-loaded components. The wall is assumed to respond in flexure up the ultimate moment capacity or ultimate diagonal shear capacity, whichever controls the overall lateral load capacity of the wall. The ultimate moment capacity, shear capacity, and flexural stiffness of upgraded walls are calculated using approaches very similar to those used for static design of walls upgraded with FRP with increased dynamic material strengths. Response criteria that correlate the maximum calculated wall response to the expected damage level are also presented.

Also, comparisons are presented between measured maximum deflections and maximum dynamic reaction forces and corresponding values calculated with the design methodology for approximately 35 shock tube tests of reinforced concrete and CMU walls upgraded with a variety of different FRP products. On the average, the calculated maximum deflections and maximum reaction forces are within 10% of the maximum measured values. The test data is also used to develop response criteria for upgraded concrete and masonry walls that correlate the ductility ratio of upgraded walls to observed wall damage levels. The ductility ratio is determined directly from the measured maximum dynamic deflection and properties of the wall including span length and calculated yield deflection. The proposed ductility ratio limits are in the range typically associated with brittle component response to blast loads.

ACKNOWLEDGEMENTS

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BLAST DESIGN OF REINFORCED CONCRETE AND MASONRY COMPONENTS RETROFITTED WITH FRP

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Protection Engineering Consultants

34th DDESB Seminar · Portland, OR

July 13-15, 2010

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Overview

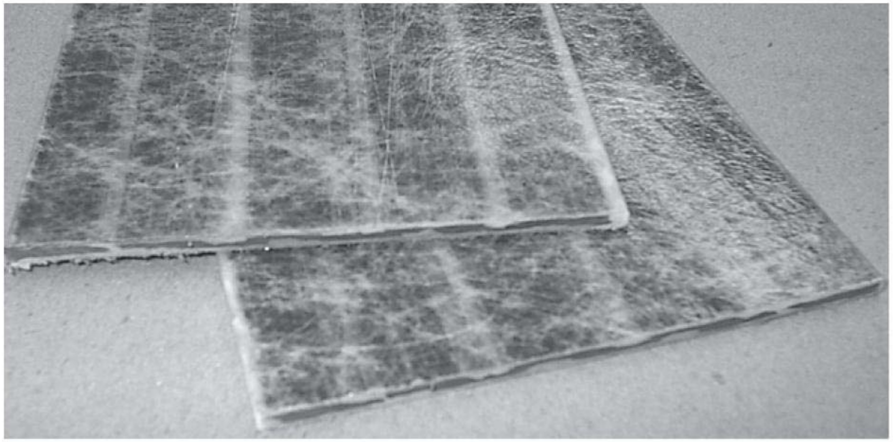
- SBEDS (Single-Degree-of-Freedom Blast Effects Design Spreadsheet) version 5.0 will include concrete and masonry components retrofitted with FRP
  - SBEDS development funded by U.S. Army Corps of Engineers, Protective Design Center
- Methodology was developed to calculate resistance function (strength and stiffness) for equivalent SDOF system representing these retrofitted components
- Comparison of SDOF results to test data
- Development of Response Criteria based on test data

Fiber Reinforced Polymers (FRP)

- FRP composites: small diameter high strength fibers embedded in a resin matrix
- Fibers provide strength and stiffness (unidirectional or multidirectional)
- Resin matrix provide stress transfer between fibers and between laminate and substrate
- Structural properties:
  - High strength
  - Limited ductility (Brittle)

Fiber Reinforced Polymers (FRP)

- Applied as a wet layup or pre-cured system



Fiber Reinforced Polymers (FRP)

- Common FRP systems for blast upgrades to RC and masonry walls

FRP Type	Yield Strength* (ksi)	Modulus of Elasticity (ksi)
Carbon Fibers (CFRP)	100 to 350	15000 to 21000
E-Glass Fibers (GFRP)	75 to 200	3000 to 6000
Aramid Fibers (AFRP)	100 to 250	6000 to 10000
* Manufacturer's recommended value not including any environmental effects or debonding		

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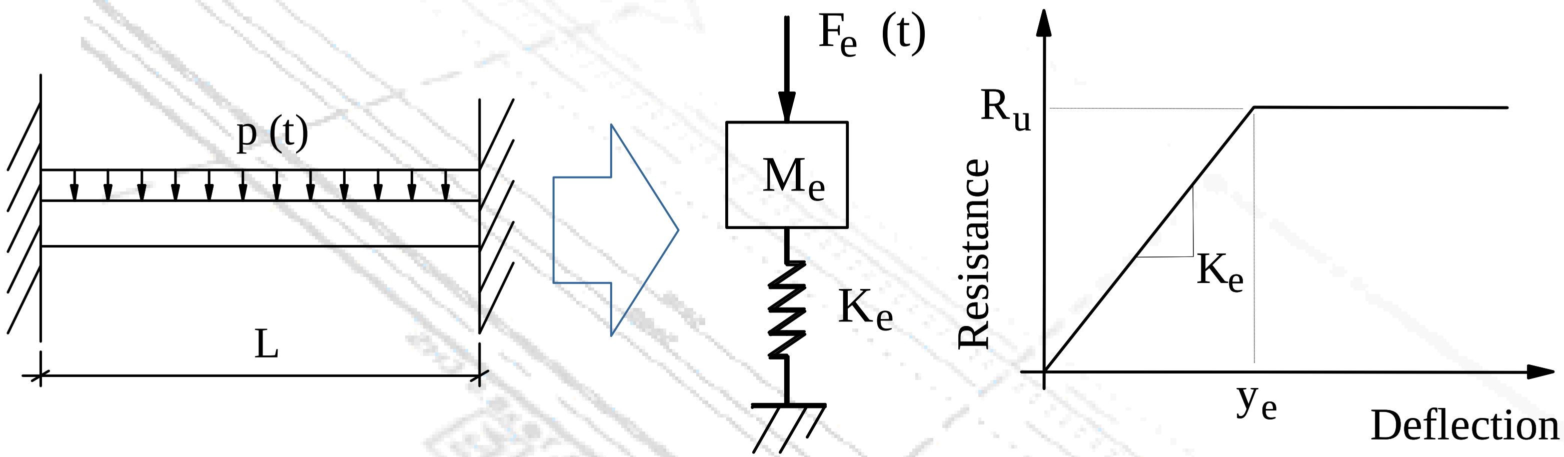
SDOF Analysis

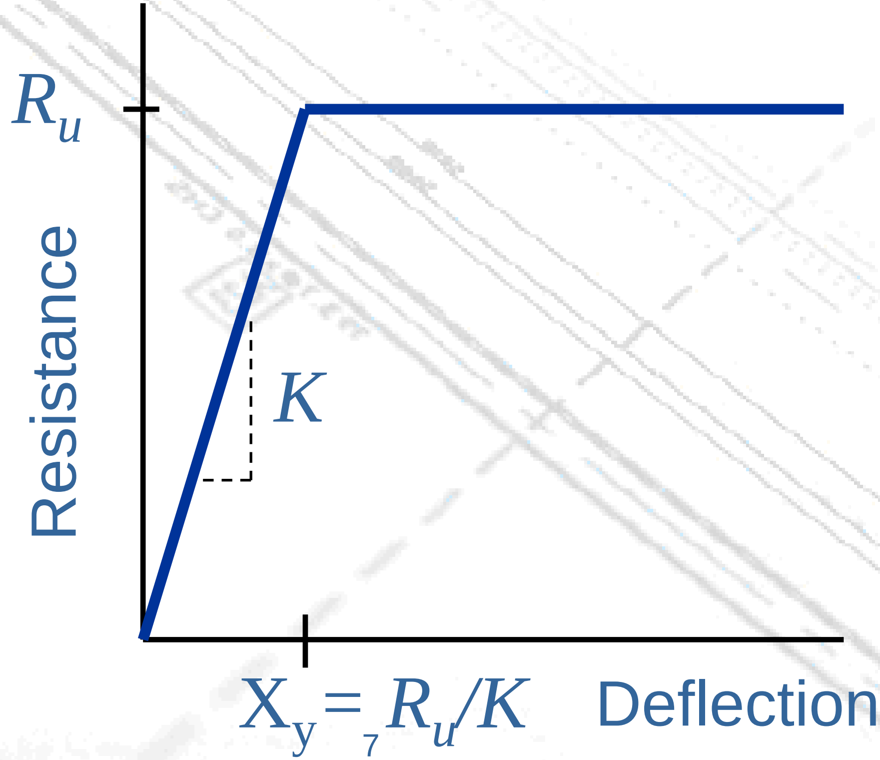
- SDOF methodology for blast analysis and design of RC and masonry components (UFC 3-340-02, PDC-TR 06-01)



- Need to determine the resistance function of the equivalent SDOF system for RC and masonry wall retrofitted with FRP

Resistance Function for RC & Masonry walls

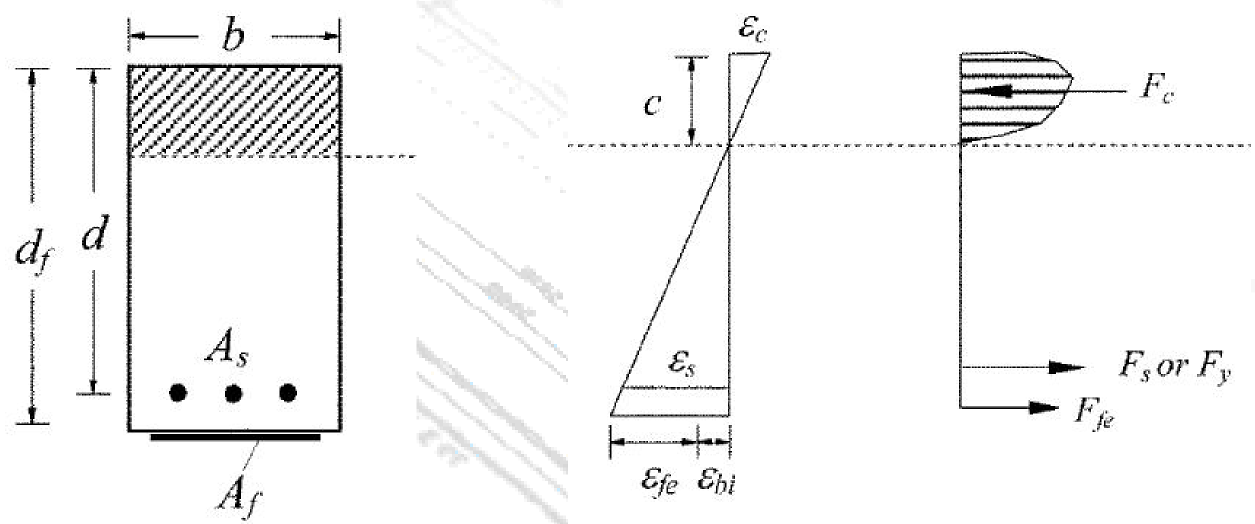
- Bilinear elastic-plastic with very limited plasticity
- Ultimate resistance (R_u) controlled by flexure (R_{uf}) or shear capacity (R_{uv})
- Stiffness (K) function of E, L and I



Ultimate Flexural Resistance

- Simply supported component : $R_{uf} = \dfrac{8M_{du}}{L^2}$

- Ultimate dynamic moment capacity M_{du}:



$$M_{du} = A_f f_f (h - c) + A_s f_s (d - c) + A_s' f_s' (c - d') + C_c \left(c - \frac{k_2 c}{2} \right)$$

Ultimate Flexural Resistance (Cont'd)

- Calculation of ultimate moment capacity based on ACI 440.2R and ACI 318-05
- Include steel and concrete SIF and DIF for blast design (UFC 3-340-02)
- No SIF or DIF factors for FRP
- Controlling modes:
  - Crushing: ε_{cu}= 0.003 (RC) or 0.0025 (Masonry)
  - Rupture or debonding of FRP laminate

  Reinforcing steel yields but will not strain to failure (ductile)
- FRP is elastic-brittle and strength limited by:
  - Environmental reduction factor (C_e)
  - Bond coefficient for blast loading (K_{mb})

Ultimate Flexural Resistance (Cont'd)

- FRP design limit strength for blast loading:

$$\boxed{f_{fb} = K_{mb} f_{fu}}$$

$f_{fu} = C_E f^{*}_{fu}$; environmental reduction factor (C_E): 0.5~0.95

$$K_{mb} = \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{nE_f t_f}{2000000}\right) \leq 0.9 \qquad \text{for} \qquad nE_f t_f < 10^6$$

$$K_{mb} = \frac{1}{60\varepsilon_{fu}}\left(1 - \frac{500000}{nE_f t_f}\right) \leq 0.9 \qquad \text{for} \qquad nE_f t_f > 10^6$$

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Ultimate Shear Resistance

$$R_{uv} = \frac{V_{cap}}{(C_v L - \kappa d)}$$

C_v : shear span coefficient
κ : critical shear location coefficient

- Shear capacity per unit width, $V_{cap} = V_c \text{ or } V_m$:

$$\text{RC}: \quad V_c = 2\sqrt{f'_{dc}}\, d$$

$$\text{Masonry}: \quad V_m = 2\sqrt{f'_m}\, A_{net}$$

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Flexural Stiffness

- For one-way uniformly loaded members with simple supports (most common conditions for walls)

$$K = \frac{384}{5}\frac{EI_{cr}}{L^4}$$

- Different constants for different boundary conditions (PDC-TR 06-01)
- Moment of inertia changes from uncracked (I_g) to fully cracked (I_{cr}) during blast response
  - I_{cr} based on FRP and any steel reinforcement
- Comparison between SDOF results and blast test data showed that I_{cr} provide significantly more accurate results

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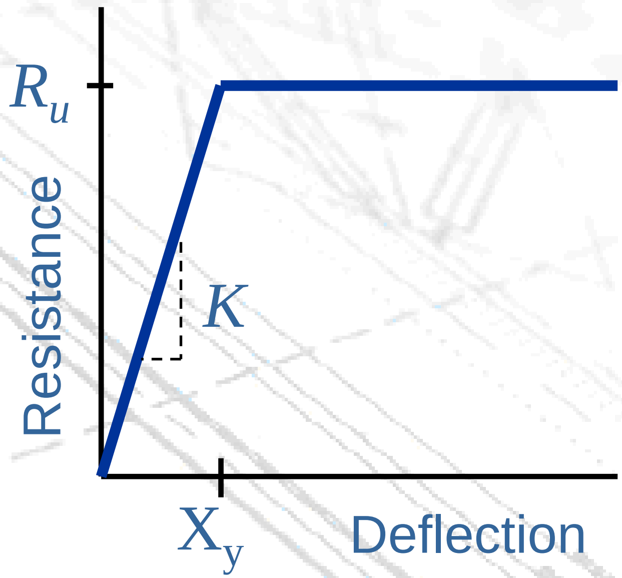
Resistance Function: Summary for Simply Supported Wall



Note: R_u is the lesser of R_{uf} or R_{uv}

$$R_{uf} = \frac{8M_{du}}{L^2} \qquad R_{uv} = \frac{V_{cap}}{(C_v L - \kappa d)}$$

$$K = \frac{384}{5}\frac{EI_{cr}}{L^4} \qquad X_y = \frac{R_u}{K}$$

$$M_{du} = A_f f_f (h-c) + A_s f_s (d-c) + A_s' f_s' (c-d') + C_c \left(c - \frac{k_2 c}{2} \right)$$

$$f_{fb} = K_{mb} C_E f_{fu}^*$$

$$V_c = 2\sqrt{f_{dc}'}\, d \qquad V_m = 2\sqrt{f_m'}\, A_{net}$$

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Comparison to Blast Test Data

Test walls in BakerRisk shock tube

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Comparison to Blast Test Data

- Shock tube tests on RC and Masonry walls upgraded with FRP composites
  - Test data performed by BakerRisk and made available by test sponsors
  - 11 tests on concrete walls upgraded with Carbon FRP
  - 24 test on reinforced and unreinforced masonry walls upgraded with E-glass FRP and Aramid FRP (Kevlar)
- Design methodology was incorporated beta version of SBEDS and used to calculate response of walls
- Measured blast loads were used in the analysis
- Environmental factor C_E was set to 1.0
- Measured and calculated maximum deflection, peak dynamic reactions and response mode (flexural or shear) were compared

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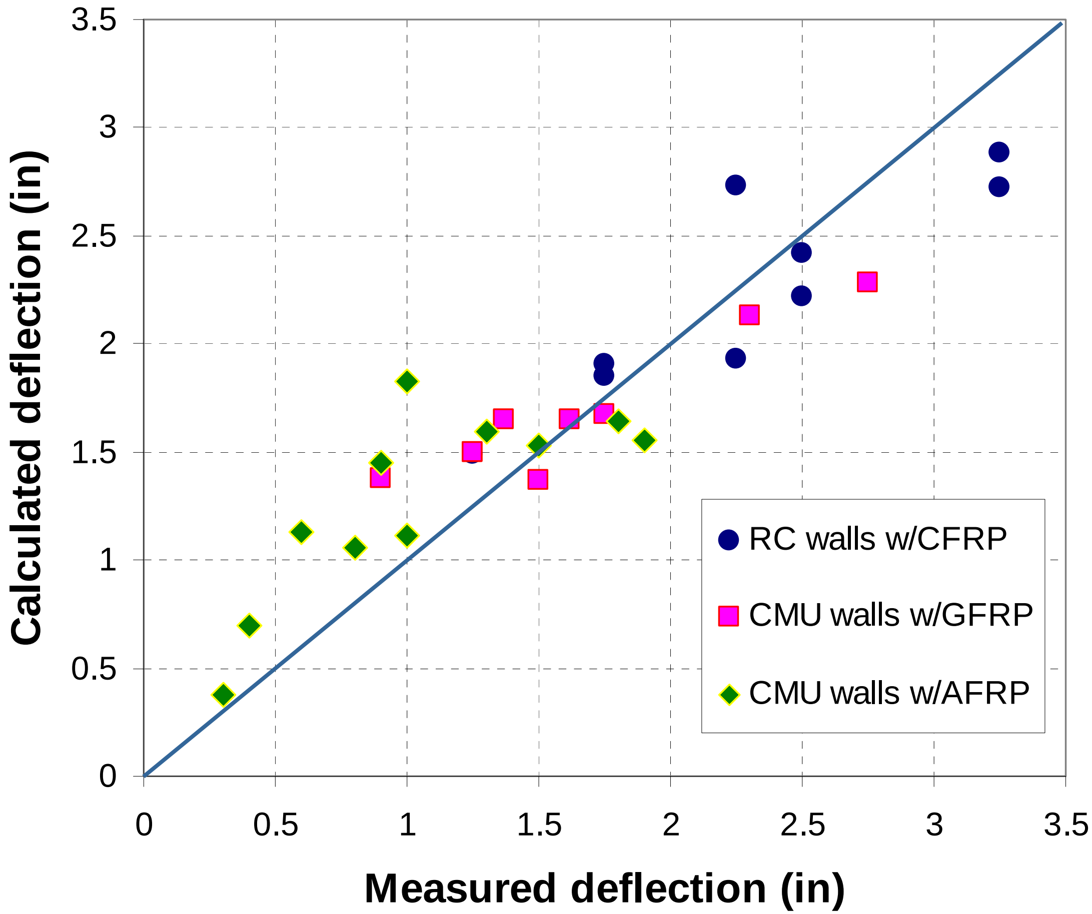
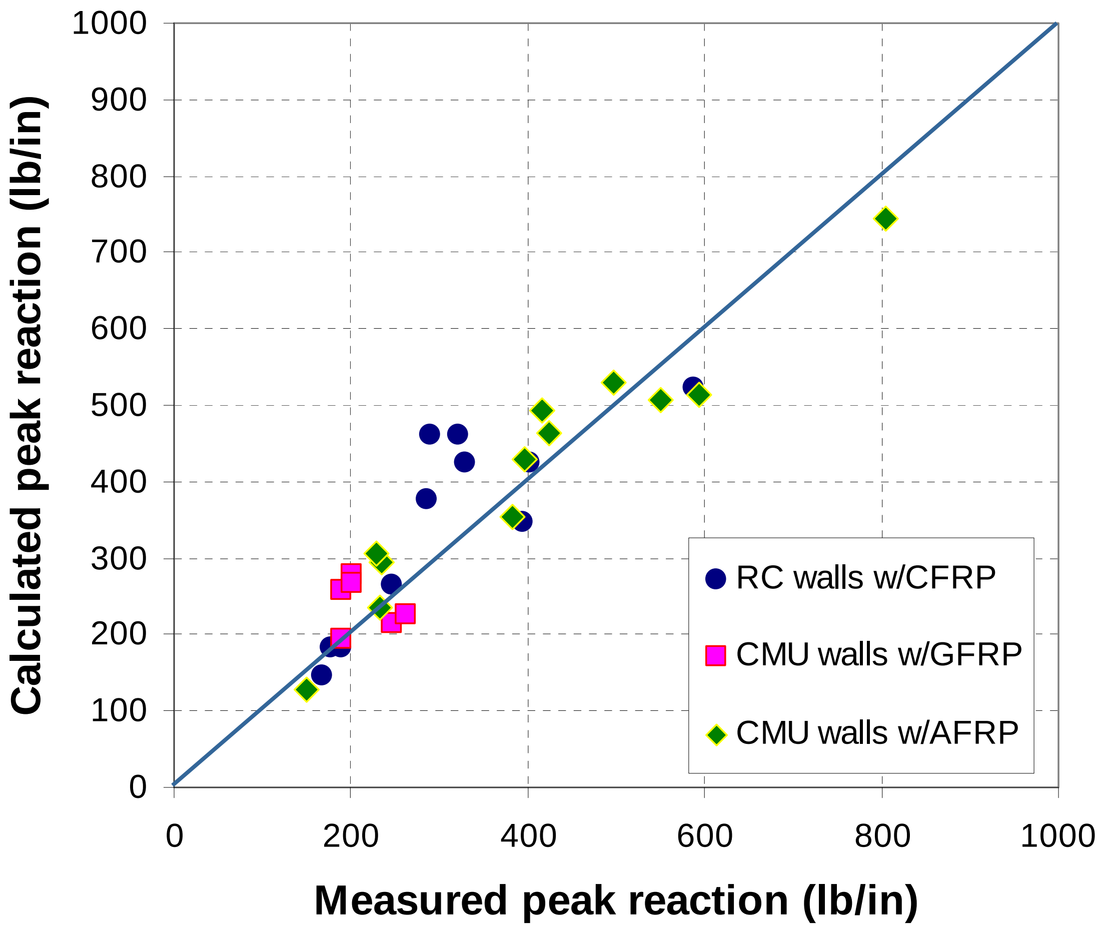
Comparison to Blast Test Data

- Maximum Deflection: Calc/Meas = 1.16 (average)



Comparison to Blast Test Data

- Peak Dyn. Reactions: Calc/Meas = 1.09 (average)



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Response Criteria

- Response criteria for brittle components are typically based on Ductility Ratios (μ)
- Ductility ratio was calculated for tests specimens
  (Meas. max. defl.)/(Calc. yield defl.)
- Separate response criteria developed for flexural and shear response:
  1) Established set of Damage Levels
  2) Plot and correlate observed Ductility Ratios and Damage Levels
  3) Establish upper and lower bound limits for Ductility Ratios defining each Damage Level
- Response limits err on the conservative side compared to scatter in the data but not necessarily conservative for all outliers

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Response Criteria: Damage Levels

Observed Damage Level Categories	Wall Damage Included in Category	Corresponding PDC Damage Level
Minor	Minor cracking in wall and epoxy. Minor FRP delamination	Moderate
Moderate	Significant delamination of FRP	Moderate
Severe	Severe cracking of wall	Severe
Collapse	Wall failed within 10 ft of test structure	Hazardous Failure
Blow Out	Parts of wall blown 10 ft away from test structure	Collapse

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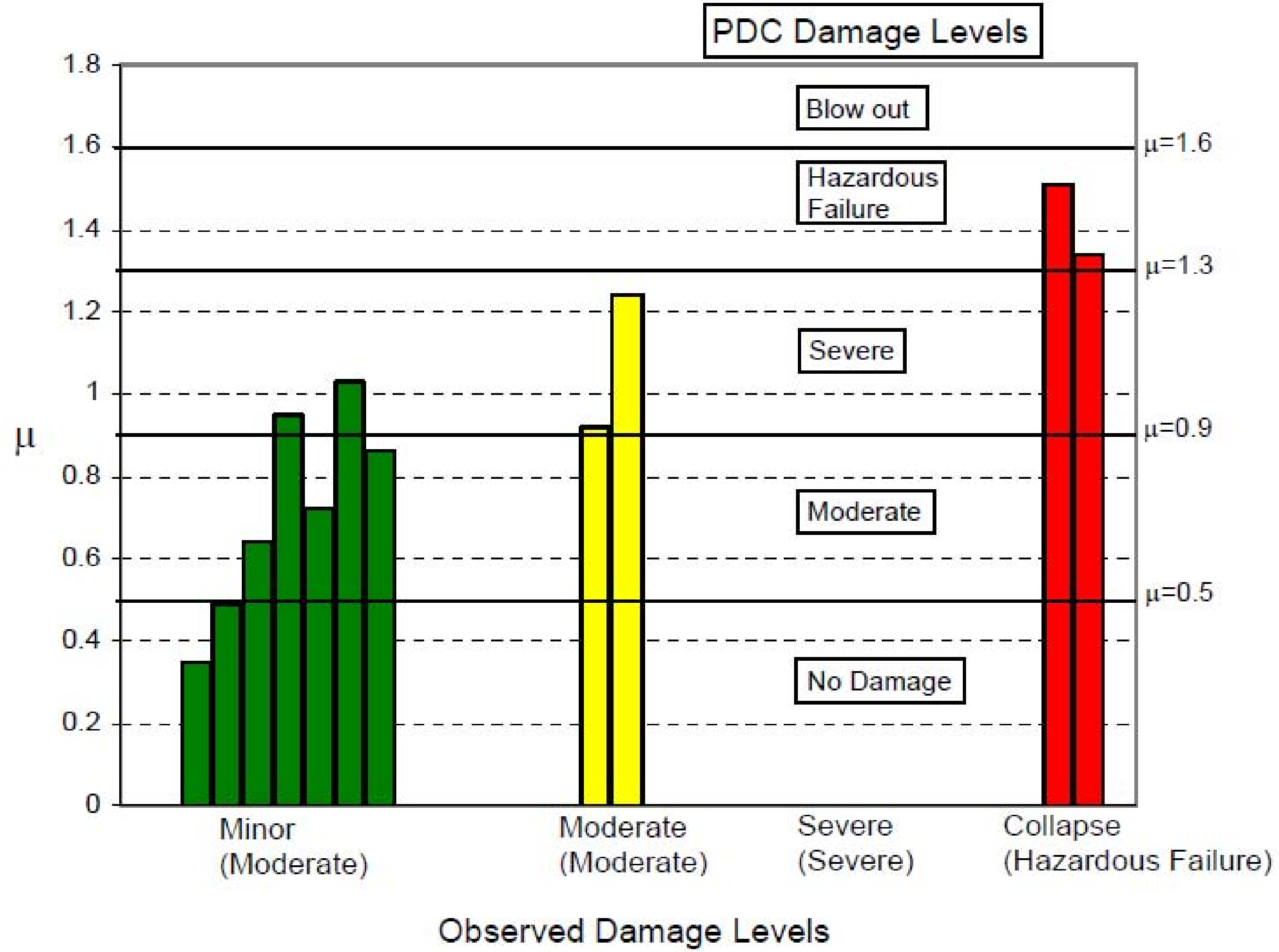
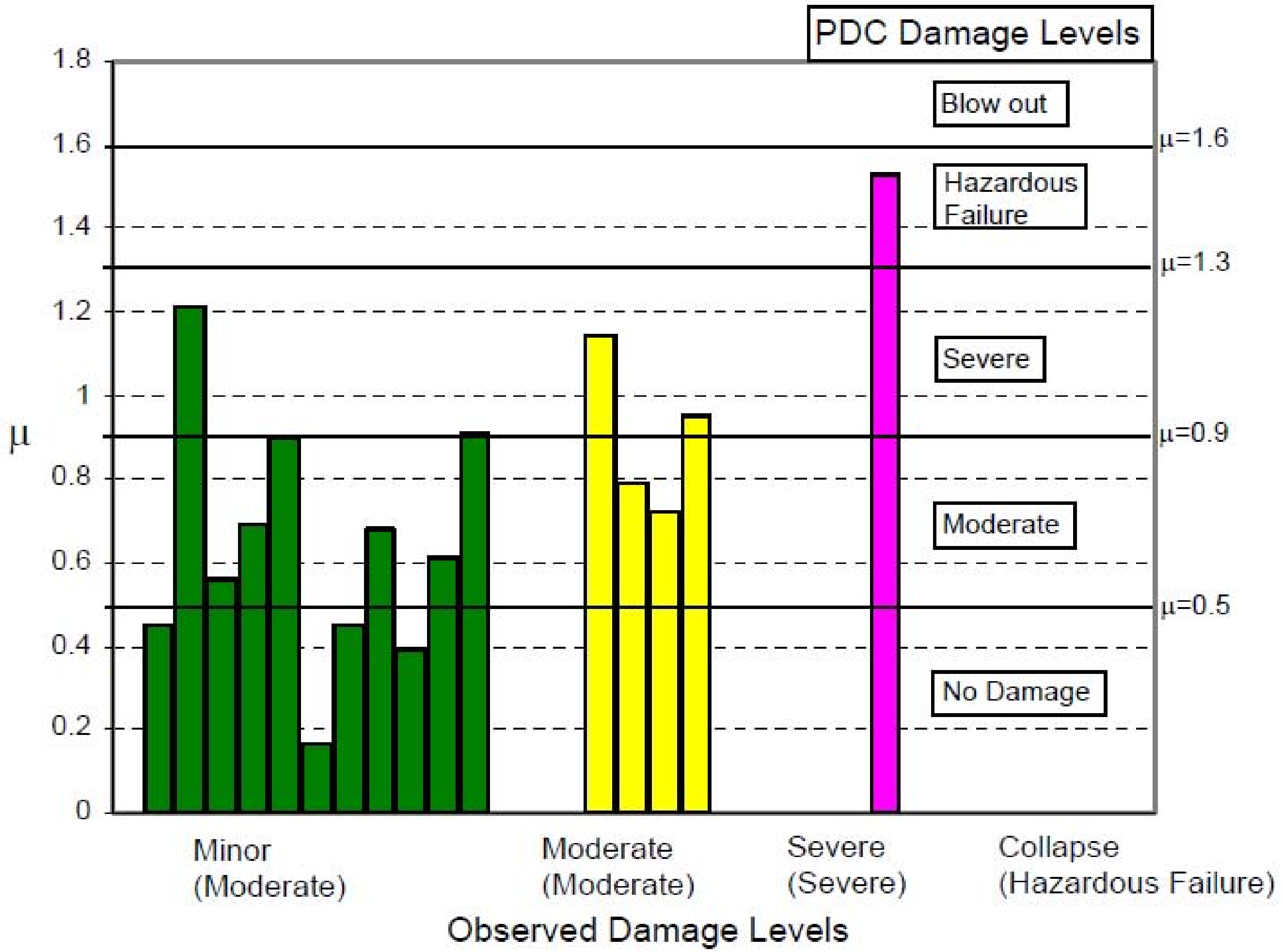
Response Criteria: Flexural Response

- Observed damage and ductility ratios: RC Walls



Response Criteria: Flexural Response

- Observed damage and ductility ratios: Masonry Walls



Response Criteria: Flexural Response

- Proposed Ductility Ratio Limits for flexural response of RC and Masonry walls upgraded with FRP

PDC Damage Level	Observed Damage Level	Ductility Ratio
No Damage	N/A	$\mu \leq 0.5$
Moderate	Minor and Moderate	$0.5 < \mu \leq 0.9$
Severe	Severe	$0.9 < \mu \leq 1.3$
Hazardous Failure	Collapse	$1.3 < \mu \leq 1.6$
Blow Out	Blow Out	$\mu > 1.6$

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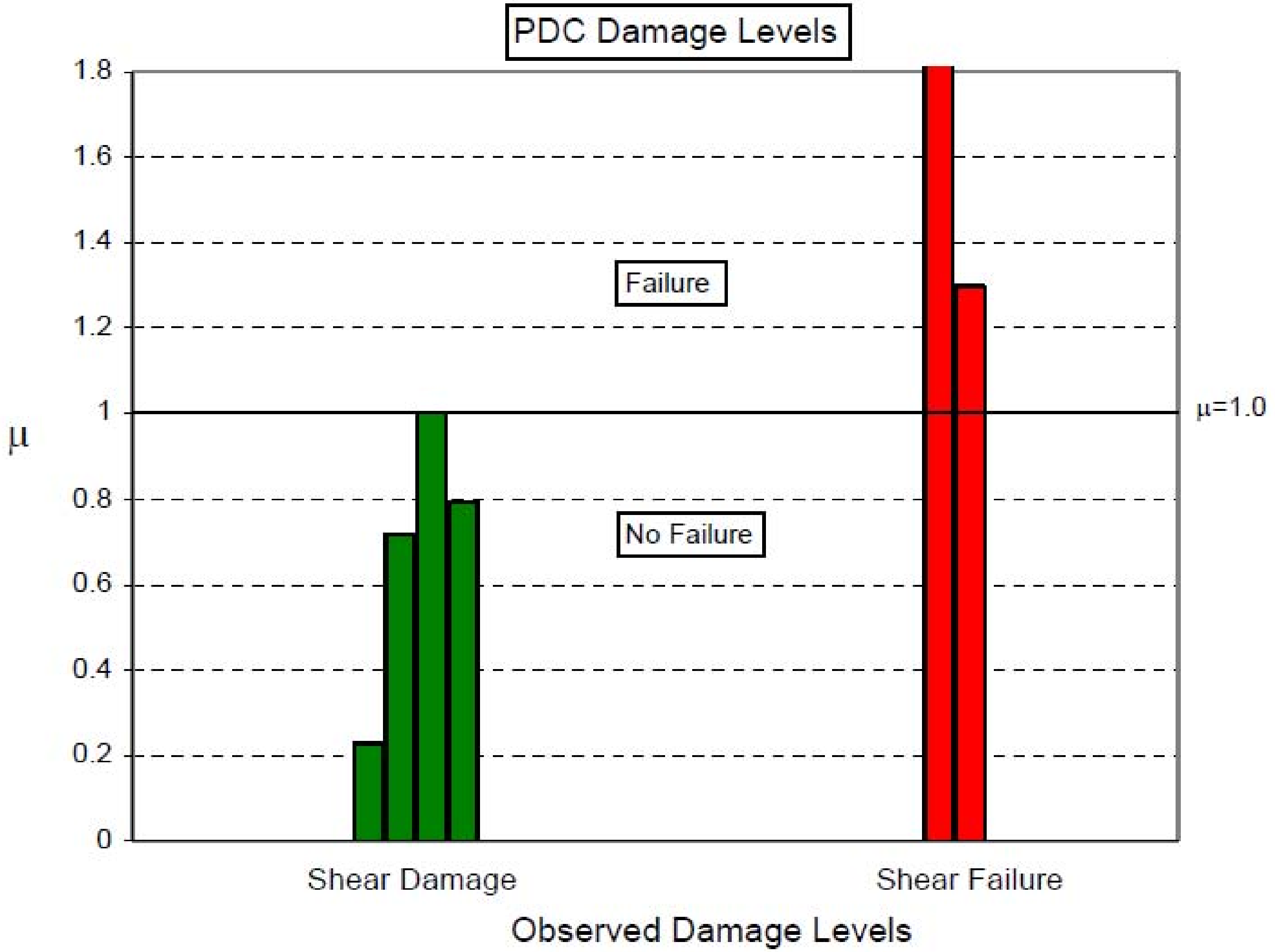
Response Criteria: Shear Response

- Observed damage and ductility ratios: Masonry Walls



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Response Criteria: Flexural Response

- Proposed Ductility Ratio Limits for Shear Response of RC and Masonry walls upgraded with FRP

PDC Damage Level *	Observed Damage Level	Maximum Ductility
No Failure	Moderate shear damage	$\mu \leq 1.0$
Failure	Shear Failure	$\mu > 1.0$
* The PDC does not have explicitly defined damage levels for shear, but it required prevention of shear failure.		

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Summary and Conclusions

- Methodology was developed for SDOF-based blast design of RC and masonry walls with FRP retrofitting
- Calculated maximum deflections and reaction forces are within 10% of measured values on average from 35 tests
- Based on comparison with blast test data, f_{fb} can be based on ACI 440.2R criteria (static)
- Modeling could be improved with test data allowing development of new equations for max FRP stress at blast loading strain-rates
  - Also more tests causing higher ductility ratios are needed
- Proposed response criteria based on ductility ratio are in range typically associated with brittle components response to blast loads

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