Comparison of SDOF Analysis Results to Test Data for Different Types of Blast Loaded Components

Chuck Oswald, Marlon Bazan

Protection Engineering Consultants, 4203 Gardendale, Suite C112, San Antonio, TX 78229, coswald@protection-consultants.com; mbazan@protection-consultants.com

ABSTRACT

SDOF analysis is widely accepted for blast design, but many designers are not aware of the large database of available comparisons between results from SDOF analysis and blast test data. This paper collects a large amount of this data, excluding test data that is proprietary or restricted from public distribution, and presents comparisons of SDOF analysis results with test results for different types of structural components. This comparison includes typical structural components included in the SDOF-based blast design tool SBEDS from the U.S. Army Corps of Engineers, Protective Design Center (PDC). This comparison provides a comprehensive view of the degree of accuracy and conservatism of SDOF as a design method. This information is valuable for both researchers and designers of structural components subject to blast effects.

INTRODUCTION

Single-degree-of-freedom (SDOF) analysis is a widely used dynamic analysis procedure for designing and analyzing structural components subject to blast loads that combines a design level simplicity with the ability to explicitly consider dynamic response. The structural component is idealized as an equivalent spring-mass system with a single degree of freedom subject to the applied blast load history and the response of this system is calculated by solving the equation of motion for the equivalent system. Usually, the equation of motion is solved with a time-stepping numerical integration approach. The details of this analysis approach are presented in many available references (UFC 3-340-02, 2008; ASCE, 2010; Biggs, 1964).

The SDOF analysis method is based on simplifying assumptions for both the structural component and the applied blast load. Although these assumptions do not all strictly apply in many design cases, the resulting calculations are usually assumed to be a conservative estimation of the dynamic response of the component. In general, the SDOF methodology is most typically used for modeling flexural response of structural components to dynamic loading. However, other modes of response including rigid arching (compression membrane), catenary action (tension membrane), combined loading (bending and axial loading), and P-delta effects can be included in the formulation of the SDOF methodology. Although shear deformations are not typically included in SDOF analysis, shear force demand and capacity can be accounted for using an equivalent static load approach. SDOF methods are widely accepted for blast design, but they have important limitations that must be well understood and properly accounted for in the design of blast loaded components.

This paper collects a large amount of test data on blast loaded components, excluding test data that is proprietary or restricted from public distribution, and presents a comprehensive comparison of SDOF analysis results for different types of structural components by comparing calculated with measured maximum deflections. This is a practical approach for the comparison since most blast design is based on comparing the calculated component maximum deflection from a SDOF analysis to response criteria.

Blast Test Data

The comparisons in this paper are limited to component blast tests with a complete set of data, including sufficient information to completely define the equivalent SDOF system for the component, the applied blast load, and the measured maximum dynamic deflection. This excludes tests where only the general damage level is reported, tests where complete blast load and component information is not provided, and tests where the component fails.

The comparisons do include some tests where some of the basic assumptions of SDOF analysis are not met very well. SDOF analysis is most applicable when a spatially uniform blast load is applied to the component, the component has rigid supports, and the component responds in a flexural mode. Therefore, the best comparisons between analytical and measured deflections are expected for these cases. However, SDOF analysis is used to design components with non-uniform blast loads, usually after the blast load is averaged (UFC 3-340-02, 2008) or converted to an "equivalent" uniform blast load (UFC 3-340-01, 2002) that has the same work energy as the actual blast load. Also, SDOF analysis techniques have been developed to account for non-flexural component response modes such as compression and tension membrane (PDC-TR 06-01, 2006; Salim, 2003; UFC 3-340-01, 2002). Some simplifying assumptions are required for these cases, but the SDOF analysis procedure is still usually considered accurate enough for design purposes. Therefore, these procedures are used to analyze blast tests that have non-uniform loads and non-flexural response modes that are included in this study.

Much of the available blast test data for structural components has been compiled in the CEDAW Methodology Report (PDC-TR 05-02, 2005) that was developed for the U.S. Army Corps of Engineers, Protective Design Center (PDC). Additional test data is also included from a wide variety of reports, papers, and conference proceedings. The comparisons in this paper were made several years ago and more recent data is not included in this paper. Therefore, this paper includes a considerable number of comparisons in one comprehensive paper, but the comparison of the SDOF methodology to test data is an ongoing effort as more data becomes available.

SDOF Analyses

The test data is compared to SDOF analyses performed with the SBEDS V4.1, which was distributed by the U.S. Army Corps of Engineers, Protective Design Center (PDC). SBEDS (SDOF Blast Effects Design Spreadsheets) is an Excel® based tool for design of structural components subjected to blast loads using SDOF methodology. SBEDS has been widely used for blast design over the past 10 years, so it is a well-accepted tool for design and analysis of blast-loaded structural components. It calculates all the properties of the equivalent SDOF system from input parameters defining the cross section and geometry of different types of structural components, solves the equation of motion for the equivalent SDOF system, and produces a wide range of output plots and summary information. The output includes information on deflection, support rotation, ductility, dynamic and equivalent static reactions, and a shear check. The structural components available in SBEDS include reinforced and unreinforced masonry, reinforced and pre-stressed concrete, steel, cold-formed steel, and wood components. The latest version of SBEDS (V5) also includes concrete and masonry retrofit components such as fiberreinforced polymer (FRP), bonded polymer and membrane catch systems. Dropdown menus with common member sizes, material properties, boundary conditions, and other inputs allow for quick model setup. Blast loads can be input in pressure-time pairs, as charge-weight and standoff (side-on and reflected blast loads with and without clearing), or read from a file. More information on SBEDS is available in PDC-TR 06-01 (2012) and PDC-TR 06-02 (2012). Previous versions of these documents were unlimited distribution, but the current versions are limited distribution documents,

SBEDS generally follows the guidance contained in Unified Facilities Criteria (UFC) 3-340-02, "Structures to Resist the Effects of Accidental Explosions", and UFC 3-340-01 (FOUO), "Design and Analysis of Hardened Structures to Conventional Weapons Effects", as applicable. Response criteria in SBEDS are based on PDC-TR 06-08 Revision 1 from the U.S. Army Corps of Engineers. SBEDS is used primarily to analyze structural components that respond to blast load with ductile flexural response. It also will analyze other response modes as applicable for different component types, including compression membrane and/or tension membrane, P-delta effects from static and dynamic (i.e., dynamic shear histories from supported components) axial loads, shear-controlled response, and axial load arching on unreinforced masonry components.

COMPARISON OF TEST DATA TO SDOF ANALYSES

The comparison of test data to SDOF analysis is done by comparing calculated and measured maximum deflections for different types of structural components, as discussed previously. The comparisons are reported separately for each component type, so that trends in the accuracy of the SDOF analysis that are component type dependent can be noted. Also, a brief overview of each test program is presented and any key assumptions that were required to model the tests in SBEDS are discussed. More complete information on each test is available in the referenced documents. Statistical information on the comparisons for each component type is included.

Corrugated Steel Panels

Table 1 shows statistical information for ratios of maximum deflections calculated with SDOF analyses to corresponding measured maximum deflections from twenty-two tests on corrugated steel panels that do not develop significant tension membrane response. Table 2 shows detailed information for these comparisons. The data from Blast Capacity Evaluation of Pre-Engineered Building (Stea et al., 1979), which is abbreviated as (BCEPB) in Table 1, was from a pre-engineered building that was subject to high explosive loads. The same building was subjected to six tests, which all caused relatively minor damage, and was repaired as necessary between tests. No tension membrane was assumed because the panel supports did not provide the in-plane restraint necessary to develop tensile force in the panels. These conditions are typical of most pre-engineered building construction. The blast loads are based on measured reflected blast loads from 2000 lbs of high explosive at relatively large standoffs.

The data from Blast-Resistant Capacities of Cold-Formed Steel Panels (Stea et al., 1981), which is abbreviated as (BCCSP) in Table 2, is from high explosive loads applied to test panels attached to support frames. Relatively strong panels were tested that would be suitable for a blast resistant building. The researchers provided the properties of the equivalent SDOF systems for the test panels as shown in Table 2. These values were used in the SBEDS analyses, except that the resistances were multiplied by 0.9 to be consistent with the SBEDS methodology for corrugated panels where this factor is applied to the ultimate dynamic moment capacity of the panels. Since almost all the measured support rotations were less than 4 degrees, no significant tension membrane was assumed for these tests. The reported deflections for these tests, which are compared to the calculated maximum deflections, are the sum of the measured permanent deflections and the calculated elastic deflections. This probably reduces the accuracy of the measured maximum dynamic deflections, but it is very difficult to quantify this effect.

The last set of data in Table 2 is from tests conducted during a blast design short course held by Protection Engineering Consultants (PEC) on two nested 24 gage corrugated steel panels. The panels had a 1.5 inch rib height and spanned 8’-3” with simple supports that provided no tensile restraint. The panels were assumed to deflect together in a non-composite manner, so that the total cross sectional properties equaled the sum of those for the two panels. The blast loads were applied with a shock tube by ABS Consulting, Inc. as part of a class demonstration project for the course. The panel maximum deflections were measured with a dynamic displacement gage. The measured pressure histories were saved to a file that was directly read into SBEDS, so that the exact measured pressure histories were used in the SDOF analyses. There was a significant negative phase impulse in the applied shock loads that affected the response of the test panels.

The statistical information in Table 1 shows that the four BCEPB tests have the highest average and coefficient of variation (i.e., the standard deviation divided by the average) for the ratio of measured to calculated maximum deflections from the three test series, while the four PEC short course tests have the lowest coefficient of variation. Electronic files of the measured blast loads were available for the SBEDS analyses of the short course tests and very accurate, fast-responding equipment was used to measure the maximum deflections. Neither factor was the case in the other two tests series, which helps to explain why they had higher coefficients of variation. The very high coefficient of variation for the BCEPB may be due to some inaccuracies in measured values for these tests. The overall average ratio for corrugated steel panels is consistent with the overall ratio for other component types discussed herein.

Table 1. Statistical Information for Ratios of Calculated Maximum Deflections to Measured Maximum Deflections for Corrugated Steel Panels

Test Series	Average	Standard Deviation	Coefficient of Variation	No. of Tests
BCEPB Tests	1.52	1.03	0.68	4
BCCSP Tests	1.15	0.27	0.23	14
PEC Short Course Tests	1.25	0.09	0.07	4
All Corrugated Steel Panel Tests	**1.24**	**0.47**	**0.38**	**22**

Table 2. Comparison of Maximum Deflections for Corrugated Steel Panels without Significant Tension Membrane

Component [1,2]	L (in)	B (in)	F_{dy} (psi)	E (psi)	Z (in^3)	I (in^4)	Self Weight (psf)	M (lb-in)	R_u (psi)	K (psi /in)	Mass (psi-ms^2/in)	P (psi)	I (psi-ms)	μ	θ (deg)	Measured Max Defl (in)	Calc Max Defl (in)	Ratio Calc/Meas.
BCEPB Test No.1	49	12	8.4e4	2.9e7	0.05	0.045	1.3	4.8e3	1.6	3.0	22.5	0.5	10	1.2	1.4	0.6	0.25	0.42
BCEPB Test No.3	49	12	8.4e4	2.9e7	0.05	0.045	1.3	4.8e3	1.6	3.0	22.5	1.4	25	2.0	2.4	1.0	1.04	1.04
BCEPB Test No.4	49	12	8.4e4	2.9e7	0.05	0.045	1.3	4.8e3	1.6	3.0	22.5	2.1	32	3.5	4.3	1.8	3.22	1.79
BCEPB Test No.5	49	12	8.4e4	2.9e7	0.05	0.045	1.3	4.8e3	1.6	3.0	22.5	2.5	44	3.7	4.5	1.9	5.35	2.82
BCCSP Str.A Roof Test 5	60						Note 3		2.7	2.6	32.0	2.0	30	1.4	2.8	1.5	1.40	0.93
BCCSP Str.B Roof Test 5	60						Note 3		5.3	11	60.0	4.5	59	1.9	1.8	0.9	0.97	1.08
BCCSP Str.C Roof Test 5	60						Note 3		7.7	17	80.9	5.6	76	1.4	1.3	0.7	0.66	0.94
BCCSP Str.D Roof Test 5	60						Note 3		7.0	30	51.2	7.0	91	4.1	1.8	1.0	1.29	1.29
BCCSP Str.A Roof Test 6	60						Note 3		3.3	6	32.7	3.2	48	2.9	3.2	1.7	1.39	0.82
BCCSP Str.B Roof Test 6	60						Note 3		6.8	14	66.8	4.5	63	1.4	1.3	0.7	0.60	0.86
BCCSP Str.D Roof Test 6	60						Note 3		8.3	36	60.8	9.5	114	7.4	3.3	1.7	2.50	1.47
BCCSP Str.A Roof Test 7	60						Note 3		5.0	8	45.7	4.5	65	2.3	2.6	1.4	2.47	1.76
BCCSP Str.B Roof Test 7	60						Note 3		5.2	9	47.6	5.6	76	3.9	4.4	2.3	2.78	1.21
BCCSP Str.C Roof Test 7	60						Note 3		10.3	22	108.2	11	138	3.6	3.1	1.6	2.09	1.31
BCCSP Str.D Roof Test 7	60						Note 3		15.8	72	104.7	15	158	3.1	1.3	0.7	0.85	1.21
BCCSP Str.B Wall Test 5	60						Note 3		8.3	19	111.6	11.5	46	1.8	1.5	0.8	1.07	1.34
BCCSP Str.B Wall Test 6	60						Note 3		5.5	12	77.7	11.5	46	6.1	5.5	2.9	2.55	0.88
BCCSP Str.B Wall Test 7	60						Note 3		8.0	13	81.1	13.5	54	3.4	4.0	2.1	2.20	1.05
PEC Short Course 1	99	12	6.70e4	2.90e6	0.2074	0.1836	2.12	1035	0.84	0.35	38	1.3	2	0.11	0.31	0.27	0.36	1.33
PEC Short Course 2	99	12	6.70e4	2.90e6	0.2074	0.1836	2.12	1035	0.84	0.35	38	1.1	3	0.20	0.56	0.48	0.61	1.27
PEC Short Course 3	99	12	6.70e4	2.90e6	0.2074	0.1836	2.12	1035	0.84	0.35	38	1.4	3	0.24	0.67	0.58	0.74	1.28
PEC Short Course 4	99	12	6.70e4	2.90e6	0.2074	0.1836	2.12	1035	0.84	0.35	38	2.9	12	1.23	3.41	2.95	3.33	1.13

Note 1: BCEPB are tests from Blast Capacity Evaluation of Pre-Engineering Building report (Stea et al., 1979)
Note 2: BCCSP are tests from Blast-Resistant Capacities of Cold-Formed Steel Panels report (Stea et al., 1981)
Note 3: R_u, K, and Mass properties calculated by researchers

Reinforced Concrete Walls

Table 3 shows statistical information for ratios of maximum deflections calculated with SDOF analyses to corresponding measured maximum deflections from 76 tests on one-way spanning reinforced concrete slabs from eight different test programs. Table 4 shows detailed information for these comparisons for the first five test programs. Detailed information for the last three test programs and analysis is included in Oswald and Bazan (2014) and is not reproduced in this paper.

The data from Scaled Testing and Analysis of Building Components (Wright, 1993) was conducted at one-quarter scale with high explosive cylindrical charges. The data in Table 4 is converted into equivalent full-scale values. This caused all the dimensional data to be increased by the inverse of the scale factor raised to the same power as the dimensional units and the impulse to be increased by the inverse of the scale factor. It caused no change in the concrete and reinforcing steel material yield strength, mass density, and modulus values. The measured blast loads were also scaled up (i.e., no change to peak pressure but impulse increased by the inverse of the scale factor) to the values shown in the table. Full measured blast load histories were reported and these were used to generate simplified point-wise linear pressure histories that preserved the impulse and shape of the applied blast loads for use in the SDOF analyses.

The data from Airblast Loading on Wall Panels (Forsèn, 1985) is from full-scale high explosive tests on wall panels. Measured blast loads are based on a separate testing program with the same blast loading configurations and range of charge weights, where the blast loads were measured (Forsèn, 1989). Reinforcing data and concrete strength information was obtained by private communication with Mr. Forsèn based on information in a more detailed Swedish version of the referenced test report. The blast loads were applied with a suspended spherical charge against a test structure that allowed clearing of the reflected blast. The blast loads in the SDOF analyses were generated by input charge weights at the reported standoffs, where the charge weight was varied from the reported value to cause a calculated positive phase impulse in SBEDS within 5% of the test impulse values. The negative phase blast load was also included in the SBEDS analysis. This test series includes multiple identical tests with measured deflections that vary by up to 30%.

The data from WES Semi-Hardened Facility Design Criteria Tests (Colthorp et al, 1985) is from tests on the wall of a box culvert type structure with two opposite sides open and one-way spanning wall, roof, and floor slabs that was subject to blast loads and fragments from a close-in fragmenting explosive. All tests were exposed to the same explosive loading, which was spatially non-uniform. An equivalent, spatially uniform blast load that included the measured impulse of the fragments was developed by the researchers and was used in the SDOF analyses. The tested wall slabs correspond to a one-half scale model of a relatively thick, heavily reinforced concrete wall. The data in Table 4 for this test series is shown in terms of the actual measured and tested dimensions.

The data from Analytical Assessment of the Blast Resistance of Precast, Prestressed Concrete Components (Cramsey and Naito, 2007) are from tests by the Air Force Research Laboratory (AFRL) at Tyndall AFB, where 6 inch reinforced precast concrete slabs were tested against high explosive blast. The wall panels spanned 30 feet with simple supports at each end. Simplified pressure-histories that matched the shape and impulse of the measured blast loads on the panels were used in the SDOF analyses, including the measured negative phase blast load. The first two of these tests in Table 4 were conducted on one test panel and the last three tests in the table were conducted on a second test panel. The comparisons to SDOF analyses do not indicate that the retesting significantly affected the panel response since any such effect would cause the SDOF analysis to underpredict damage in the retests compared to initial tests, since the retested damage would deflect more than expected.

Finally, the NCEL Tests in Table 4 (Tancreto, 1988) are approximately one-half scale concrete slabs subject to close-in explosions at scaled standoffs less than 1.0 ft/lb$^{1/3}$. This causes non-uniform blast load over the slab area, which cannot be accounted for explicitly in the SBEDS methodology. Therefore, the CONWEP computer program (CONWEP V2.1.0.3) was used to calculate the equivalent uniform peak pressure and impulse, based on conservation of work energy between the equivalent uniform load and the actual load distributions. The deflected shape function used in the conservation of energy equations is based on the yield line pattern of the slab. The reported equivalent TNT charge weight from the tests (76.8 lbs) was used in SBEDS at various standoffs to produce a positive phase impulse equal to the equivalent uniform impulse from CONWEP for each test. The negative phase blast load from this charge weight-standoff was included in the SBEDS analysis. The slabs were heavily reinforced with lacing and/or stirrups due to the close-in standoffs, and they developed tension membrane response due the large deflections and their square dimensions (i.e., allowing a compression ring to form around the perimeter of the slabs). The slabs were clamped with large bolts at the supports to prevent rotations, but the bolt holes were oversized so that compression membrane response could not develop. Therefore, they were modeled in SBEDS with flexure and tension membrane response. Only a Type I cross section was used, since inclusion of a Type II cross section made the calculated results significantly more conservative than shown in the table. The heavy lacing and shear steel was assumed to preclude shear failure, as indicated in the test report. The slab in Test 5 had some shear cracks, but this did not significantly affect its tension membrane response according to the researchers. Test 1, 2, and 4 are nearly identical tests with measured deflections that vary from each other by about 20%.

The data in Table 3 from shock tube tests on solid precast panels sponsored by U.S. State Department Bureau of Diplomatic Security Physical Security Division's Research and Development Branch (USSD) is from a test program designed by Protection Engineering Consultants (PEC) and conducted by Baker Engineering and Risk Consultants (BakerRisk) Lowak et al, 2012). The data in this paper includes 9 out of 17 tests where the panel did not fail and maximum deflections were measured. These panels were connected to the shock tube frame using conventional precast connections so that the connections were tested with the panels. The panels were tested to sustain heavy damage or failure. The connections were designed to resist loads based on the ultimate resistance of the test panels, as they would be in a typical blast design. The blast loads, acceleration of the panel at midspan, and dynamic reactions of the test panels were measured in the tests. Also, a high speed video camera recorded the midspan movement of the panels against a background grid. This was used to verify the panel displacement history that was determined from double integration of the measured acceleration histories (Oswald and Bazan, 2014).

The data from AFRL Tests on Insulated Precast Panels in Table 3 is from a recent testing program on insulated concrete sandwich panels conducted at the Air Force Research Laboratory (AFRL) (Naito et al, 2011; Naito et al, 2013). In the main phase of the testing, blast loads were applied in two separate tests to eight test panels. Four panels were single-span panels with simple supports, and four panels were two-span continuous over an interior support with a 3 ft. cantilever above the top support. Each span was considered independently for the SDOF analyses and comparisons. The panels were constructed to meet the design standards used for the Precast/Prestressed Concrete Industry (PCI) to resist typical conventional design loads. In no case were the panels or connections to the test frame designed for blast. The design of the reinforcement for these panels was controlled by loads during lifting and handling, so the panels were considerably stronger than required only to resist lateral wind loads. The insulated sandwich panels were modeled in SBEDS assuming they were 100% composite, including two panels in each test that were only designed to be 50% composite. The insulated panels were modeled as monolithic panels with the full panel thickness and an input negative supported mass to account for the lightweight insulating panel. However, the SDOF analysis results did not indicate a difference in behavior between the panels designed to be fully composite and those designed to be only 50% composite (Oswald and Bazan, 2014).

Statistical information for additional blast tests by BakerRisk on 5.5 inch thick reinforced concrete slabs with a single layer of #5 bars at 11.25 inch spacing at mid-thickness, spanning 8 ft. with simple supports, is shown in Table 3 (Holland and Wesevich, 2010). This table shows that analyses with SBEDS also predicted the measured deflection for these solid conventionally reinforced concrete panels well (Oswald and Bazan, 2014).

Table 3 shows the statistical information on the maximum deflection ratios for the eight test programs. This component type has the most different tests all for the same component type. The WES tests have the highest average and standard deviation, but it is not clear that this represents an important trend.

Table 3. Statistical Information for Ratios of Calculated Maximum Deflections to Measured Maximum Deflections for One-Way Reinforced Concrete Slabs

Test Series	Average	Standard Deviation	Coefficient of Variation	No. of Tests
Scaled Testing, Analysis of Building Components	1.25	0.36	0.29	11
Airblast Loading on Wall Panels	1.40	0.36	0.25	11
WES Tests	1.27	0.60	0.47	5
AFRL Tests	0.98	0.40	0.41	5
NCEL Tests	1.41	0.34	0.24	6
USSD Shock Tube Tests on Solid Precast Panels	1.19	0.47	0.39	9
AFRL Tests on Insulated Precast Panels	1.06	0.21	0.20	19
BakerRisk Tests	1.04	0.11	0.10	6
All Reinforced Concrete Slab Tests	**1.19**	**0.36**	**0.30**	**76**

Table 4. Comparison for Reinforced Concrete Wall Slabs

Test Series[1]	Test No.	L (in)	Thick (inch)	Depth (inch)	f'_{dc} (psi)	f_{dy} (psi)	Reinf. Ratio (%)	Support	Weight (psi)	I_{eff} (in^4/in)	E (psi)	M (lb-in/in)	R_u (psi)	K (psi/in)	Mass (psi-ms^2/in)	P (psi)	I (psi-ms)	Max. Meas. Defl. (inch)	θ (deg)	Calc Max Defl (in)	Ratio Calc/Meas
Scaled Testing, Analysis of Building Components	F1	250	7.9	6.7	8000	8.5e4	0.66	Simple	0.68	20.3	5.1e6	2.4e4	3.0	2.0	1753	42	212	5.2	2.4	4.7	0.90
	F3	250	7.9	6.7	8000	8.5e4	0.66	Simple	0.68	20.3	5.1e6	2.4e4	3.0	2.0	1753	15	140	2.5	1.2	2.1	0.84
	F4	250	7.9	6.7	8000	8.5e4	0.66	Simple	0.68	20.3	5.1e6	2.4e4	3.0	2.0	1753	7	72	0.8	0.4	1.1	1.38
	F5	250	7.9	6.7	8000	8.5e4	0.66	Simple	0.68	20.3	5.1e6	2.4e4	3.0	2.0	1753	166	350	7.9	3.6	11.9	1.50
	F6	250	7.9	6.7	8000	8.5e4	0.66	Simple	0.68	20.3	5.1e6	2.4e4	3.0	2.0	1753	4	32	0.3	0.1	0.5	1.73
	P1-shot 1	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	15	72	2.4	1.1	2.3	0.97
	P1-shot 2	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	167	350	13.4	6.1	23.0	1.71
	P2	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	8	76	2.4	1.1	2.7	1.10
	P3	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	56	224	11.8	5.4	13.0	1.10
	P5	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	15	124	4.9	2.2	3.8	0.77
	P6	250	5.3	4.5	8000	8.5e4	0.66	Simple	0.46	6.2	5.1e6	6.8e3	0.9	0.6	1183	3	36	0.6	0.3	1.0	1.73
Airblast Loading on Wall Panels	150-1	94	5.9	5.0	6000	7.7e4	0.21	Simple	0.51	8.6	4.4e6	3.9e3	3.6	37.4	1316	62	116	1.8	2.2	1.63	0.91
	150-2	94	5.9	5.0	6000	7.7e4	0.21	Simple	0.51	8.6	4.4e6	3.9e3	3.6	37.5	1317	234	227	4.5	5.4	4.16	0.92
	200-1	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.9	4.7	5.50	1.41
	200-2	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e33	7.9	75.7	1753	1008	528	3.6	4.4	5.50	1.53
	200-3	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.8	4.6	5.50	1.45
	200-4	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	219	227	2.1	2.5	1.78	0.85
	200-5	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	529	311	2.2	2.6	3.24	1.47
	200-6	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.1	3.8	5.50	1.77
	200-7	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.0	3.7	5.50	1.83
	200-8	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.1	3.7	5.50	1.77
	200-9	94	7.9	7.0	4400	7.7e4	0.26	Simple	0.68	20.3	3.8e6	8.7e3	7.9	75.7	1753	1008	528	3.7	4.5	5.50	1.49
WES Semi - Hardened Facility Design Criteria Tests	Test I-1	65	12.8	11.0	6000	8.5e4	1.00	Fixed	1.10	87.3	4.4e6	9.5e4	360	6644	2851	2960	916	0.6	1.0	0.95	1.58
	Test I-2	65	12.8	11.0	6000	8.5e4	0.50	Fixed	1.10	87.3	4.4e6	5.0e4	188	6644	2851	2960	916	1.0	1.8	1.84	1.84
	Test I-3	65	12.8	11.0	6000	8.5e4	0.50	Fixed	1.10	87.3	4.4e6	5.0e4	188	6644	2851	2960	916	3.0	5.3	1.84	0.61
	Test I-4	65	12.8	11.0	6000	8.5e4	0.25	Fixed	1.10	87.3	4.4e6	2.5e4	96	6644	2851	2960	916	2.2	4.0	3.73	1.70
	Test I-6	65	12.8	11.0	6000	8.5e4	0.25	Fixed	1.10	87.3	4.4e6	2.5e4	96	6644	2851	2960	916	1.5	2.6	0.95	1.58
AFRL/ Lehigh Tests	1	360	6	3.1	8900	7.7e4	0.0009	Simple	0.52	9.6	5.2e6	5273	2.3	0.33	1347	8	64	2.1	0.7	1.77	0.84
	2	360	6	3.1	8900	7.7e4	0.0009	Simple	0.52	9.6	5.2e6	5273	2.3	0.33	1347	13	95	3.6	1.1	3.8	1.06
	3	360	6	3.1	8900	7.7e4	0.0009	Simple	0.52	9.6	5.2e6	5273	2.3	0.33	1347	21	129	5.4	1.7	3.5	0.65
	4	360	6	3.1	8900	7.7e4	0.0009	Simple	0.52	9.6	5.2e6	5273	2.3	0.33	1347	25	164	6.9	2.2	4.9	0.71
	5	360	6	3.1	8900	7.7e4	0.0009	Simple	0.52	9.6	5.2e6	5273	2.3	0.33	1347	67	260	17.6	5.6	28.8	1.64
NCEL Tests[2]	1	90	4.5	3.1	4760	8.7e4	0.0106	Fixed	0.39	4.5	3.8e6	7833	41.1	139	1011	4489	933	8	10.1	11.9	1.49
	2	90	4.5	3.1	4760	8.7e4	0.0106	Fixed	0.39	4.5	3.8e6	7833	41.1	139	1011	4489	933	7.4	9.3	11.9	1.61
	3	90	4.5	3.3	4760	7.7e4	0.0133	Fixed	0.39	4.8	3.8e6	9785	76.5	221	1011	4657	959	8.4	10.6	10.77	1.28
	4	90	4.5	3.1	4760	8.7e4	0.0106	Fixed	0.39	4.5	3.8e6	7833	41.1	139	1011	4489	933	9.8	12.3	11.9	1.21
	5	90	6	4	4760	8.7e4	0.0031	Fixed	0.52	9.5	3.8e6	4211	22.1	295	1348	2529	637	8.3	10.5	7.78	0.94
	6	90	4.5	3.3	4760	7.7e4	0.0222	Fixed	0.39	5.2	3.8e6	14724	104	220	1011	4657	959	3.8	4.8	7.31	1.92

Note 1: All spans one-way with only flexural response except NCEL tests that are two way with 90 in x 90 in loaded area, fixed on all sides with tension membrane.
Note 2: Tests at scaled standoffs <1 ft/$lb^{1/3}$. CONWEP computer program was used to calculate blast loads with same equivalent uniform impulse as CONWEP that are shown in table.

One-Way Reinforced Masonry Walls

Table 5 shows data from 17 shock tube tests on one-way reinforced masonry walls by WBE (Wilfred Baker Engineering) for the PDC (Oswald et al., 2006). The walls were full-scale simply-supported CMU walls reinforced with rebar with a minimum static yield strength of 60,000 psi and an assumed dynamic yield strength of 77,000 psi. As reported in Oswald et al. (2006), the blast load was fit with a simplified, piece-wise linear representation of the blast load that preserved the impulse and shape of the actual blast load in SDOF analyses that modeled each test. The reported average of the ratio of calculated to measured maximum wall deflection was 0.99 with a standard deviation of 0.3. The exact pressure histories for all the tests were not available for this study, so a simple right triangular shaped blast load was developed using the measured peak pressure and impulse from the tests for these cases. No negative phase blast loads were included in the SBEDS analyses, but the maximum wall deflections occurred prior to the small amount of negative phase blast load applied in the shock tube tests (Oswald et al, 2006).

Tests in Table 5 with an “A” or “B” in the test number indicate repeat tests on the same wall. Although these walls were slightly to moderately damaged, the wall properties were not reduced in the SBEDS analyses. There is no obvious trend in Table 5 that the SBEDS analyses were more or less accurate compared to the measured response for the repeat tests. Therefore, the damage did not affect the properties of the walls more than the test-to-test variations in wall properties. Table 7 shows a statistical summary of the ratios of calculated maximum deflections to measured maximum deflections in Table 5.

Two-Way Unreinforced Masonry Walls

Table 6 shows data from 14 two-way spanning unreinforced masonry walls tested by the Indian Ministry of Defense (MOD) with high explosive loading (Varma et al, 1997). All the data is for brick walls simply supported on four sides. All the brick walls greater than 9 inches thick were multi-wythe walls that were mortared together and assumed to act compositely as a single wall. The brick walls were attached to a strong surrounding concrete frame with dowels, grooved connections in the brick fitting into recesses in the frame, and plain bonding using mortar in the joint between the walls and frame. The different attachments to the frame did not seem influence the results except for a relatively small difference at the lowest damage level. The wall deflections were measured at the center point of the walls with an LVDT type deflectometer.

The walls were modeled in SBEDS as responding with brittle flexure and axial load arching from self-weight above midspan. The peak pressure and impulses on the wall were reported in the test data, as well as standoffs for the suspended spherical charges of varying composition (TNT or Comp B) used in the tests. The test structure also allowed some clearing of the reflected blast load to occur. The positive and negative phase blast load for each test was calculated in SBEDS using the reported charge standoff and a TNT charge weight that produced a positive phase impulse matching the reported impulse on the test walls within 5%. Evidently, the surrounding concrete frame did not provide sufficient lateral confinement for compression membrane response to occur in these tests because several trial analyses with SBEDS assuming compression membrane response calculated less than 50% of the reported deflections and did not reach peak compression membrane response. However, the researchers reported major cracks and dislocations of brickworks for these tests. Unfortunately, no details on the reinforced concrete frame, or the frame response, are provided.

Table 5. Comparison for One-Way Reinforced Masonry Walls

Test No.	L (inch)	Rebar Spacing (inch)	Thick (inch)	f'_m (psi)	d (inch)	steel ratio	W psi	I (in^4)	E psi	M* in-lb	R_u (psi)	K (psi/in)	Mass (psi-ms^2/in)	P (psi)	I (psi-ms)	Max. Defl. (inch)	θ (deg)	Calc Max Defl (in)	Ratio Calc/Meas
1	96	88	5.6	2.0e3	2.8	0.002	0.25	609	2.0e6	9.2e4	0.9	12.5	645	5	30	1.25	1.19	0.76	0.61
1A	96	88	5.6	2.0e3	2.8	0.002	0.25	31	2.0e6	9.2e4	0.9	0.6	645	7.8	68	6.00	1.19	5.58	0.93
2	96	88	5.6	2.0e3	2.8	0.007	0.39	704	2.0e6	3.2e5	3.2	14.5	1012	4	52	0.63	1.19	0.49	0.78
2A	96	88	5.6	2.0e3	2.8	0.007	0.39	98	2.0e6	3.2e5	3.2	2.0	1012	6.6	85	1.50	1.19	2.18	1.45
2B	96	88	5.6	2.0e3	2.8	0.007	0.39	98	2.0e6	3.2e5	3.2	2.0	1012	9.9	119	3.25	1.19	3.60	1.11
3	96	88	5.6	2.0e3	2.8	0.009	0.39	713	2.0e6	3.8e5	3.8	14.7	1012	9.8	131	2.88	1.19	2.50	0.87
3A	96	88	5.6	2.0e3	2.8	0.009	0.39	117	2.0e6	3.8e5	3.8	2.4	1012	10.2	122	4.75	1.19	3.15	0.66
4	96	88	5.6	2.0e3	2.8	0.005	0.29	647	2.0e6	2.3e5	2.3	13.3	750	7	201	10.35	1.19	10.82	1.05
6	96	88	7.6	2.0e3	3.8	0.002	0.40	1507	2.0e6	1.9e5	1.8	31.0	1032	2.9	127	1.25	1.19	1.25	1.00
7	96	88	7.6	2.0e3	3.8	0.004	0.40	1535	2.0e6	3.3e5	3.2	31.6	1032	4.5	212	1.63	1.19	0.94	0.58
7A	96	88	7.6	2.0e3	3.8	0.004	0.40	141	2.0e6	3.3e5	3.2	2.9	1032	6.4	296	13.33	1.19	8.74	0.66
9	96	88	7.6	2.0e3	3.8	0.002	0.35	1449	2.0e6	2.2e5	2.2	29.8	896	3.8	163	3.50	1.19	3.27	0.93
9A	96	88	7.6	2.0e3	3.8	0.002	0.35	100	2.0e6	2.2e5	2.2	2.1	896	4.5	200	9.92	1.19	8.88	0.90
10	96	88	7.6	2.0e3	3.8	0.002	0.35	1449	2.0e6	2.2e5	2.2	29.8	896	4.6	216	6.71	1.19	7.76	1.16
10A	96	88	7.6	2.0e3	3.8	0.002	0.35	100	2.0e6	2.2e5	2.2	2.1	896	4.6	218	8.46	1.19	10.55	1.25
11	96	88	7.6	2.0e3	3.8	0.004	0.40	1535	2.0e6	3.3e5	3.2	31.6	1032	6.5	345	7.13	1.19	11.90	1.67
12	96	88	7.6	2.0e3	3.8	0.004	0.40	1535	2.0e6	3.3e5	3.2	31.6	1032	5.7	310	6.25	1.19	7.22	1.15

Table 6. Comparison for Two-Way Unreinforced Masonry Walls

Test No.	Lx (in)	Ly (in)	h (in)	f_m (psi)	f_t (psi)	W (psi)	Wall Type	I_{eff} (in^4/in)	S in^3/in)	E (psi)	M (lb-in/in)	R_A (psi)	Ru (psi)	K (psi/in)	Mass (psi-ms^2/in)	R_A/R_u	P (psi)	I (psi-ms)	Meas Max Defl (in)	Calc Max Defl (in)	Ratio Calc/Meas
1	117	117	9	1775	178	0.63	brick	61.87	13.67	1.8e06	2426	0.19	4.3	148	2426	0.046	188.5	162	5.0	3.9	0.78
11	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	414.8	324	3.7	3.80	1.03
12	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	58.0	113	0.9	1.07	1.19
13	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	121.0	143	1.6	0.71	0.44
14	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	132.0	164	1.9	1.60	0.84
15	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	344.6	221	4.6	2.50	0.54
17	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	414.8	324	4.1	3.82	0.93
20	117	117	13.6	1775	178	0.95	brick	205	30.38	1.8e06	3634	0.29	9.5	489	5392	0.046	111.7	176	1.0	2.05	2.05
24	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	480.2	345	1.2	2.53	2.11
25	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	480.2	345	1.2	2.53	2.11
28	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	183.3	185	1.6	1.14	0.71
30	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	473.1	377	3.6	2.66	0.74
32	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	480.2	345	1.2	2.53	2.11
33	117	117	18	1775	178	1.26	brick	486	54	1.8e06	4846	0.39	17	1160	9585	0.046	480.2	345	2.3	2.53	1.10

Table 6 has several examples where the same blast tests were conducted on identical walls. Test 32 and 33 are identical tests with measured deflections that vary by a factor of 2. On the other hand, Tests 11 and 17 are identical tests with measured deflections that vary by only about 10% and Tests 24 and 25 are identical tests with the same maximum deflection. This shows the variation in test-to-test variation that is possible in blast tests. Unreinforced masonry may have higher test-to-test variations because of the variations in mortar batches, applications of the mortar, and bonding between the mortar and blocks during construction. Table 7 shows a statistical summary of the ratios of calculated maximum deflections to measured maximum deflections in Table 6.

SUMMARY AND CONCLUSIONS

Table 7 has statistical information for the ratio of maximum calculated deflection from SDOF analyses using the SBEDS program to the corresponding measured maximum deflections from blast tests on structural components. The average values in Table 7 indicate that the SDOF methodology typically overpredicts measured maximum dynamic deflections in blast tests by 20% to 30%. The standard deviations in Table 7 indicate significant variation in the ratio of the calculated-to-measured maximum deflections for individual tests.

Table 7. Statistical Information for Ratios of Calculated Maximum Deflections

Component Type	Average	Std. Dev.	Coeff. of Variation	No. of Tests	No. of Test Programs
Corrugated Steel Panels	1.24	0.47	0.38	22	3
One-Way Reinforced Concrete Walls	1.19	0.36	0.30	76	8
One-Way Reinforced Masonry Walls	0.99	0.30	0.30	17	1
Two-Way Unreinforced Masonry Walls	1.19	0.63	0.53	14	1
All Tests	**1.17**	**0.41**	**0.35**	**129**	**13**

However, there are considerable uncertainties in the parameters for the test walls and blast loads that were input into the SDOF calculations. Material properties were measured in only a few tests, and only a few test reports provided definitions of the full applied blast pressure history. Calculated negative phase blast loads were included in the SDOF analyses of all tests where the calculated maximum deflection occurred after the end of the positive phase blast load, but there was not any reported information on the measured negative phase blast load in most of the tests.

Standard values and assumptions intended to reflect average conditions were used in SDOF analyses for cases lacking complete information. This is not ideal for purposes of this study, but it is representative of the level of information that is commonly available for blast designers. For example, blast design is usually based on standard component cross section information rather than specific measured values, on assumed material strengths taken from specified minimum static strength information, and on blast load information for a basic TNT surface burst with some possible simplified consideration of blast wave clearing or spatial non-uniformity.

Factors of special note from the data that cause variations between measured and calculated component deflections include the use of an “equivalent” uniform impulse from the CONWEP computer program for one group of tests, where the applied blast load varied over the span of the component, and maximum “measured” dynamic deflections that were based on measured permanent deflections for a group of corrugated steel panel tests.

Also, two seemingly identical blast tests can often have different measured results tests. Although there were not many identical test setups in the data, a limited number of such these tests showed that differences of 10% to 30% are possible. SBEDS will calculate identical results for identical setups, so this indicates that variations in the ratios of calculated-to-measured maximum deflections of +/-10% to +/-30% can be attributed to inherent variability in test parameters that cannot be accounted for in a deterministic analysis approach.

In summary, this comparison shows that the maximum dynamic deflections calculated by the SDOF methodology in SBEDS are conservative compared to measured values in blast tests by 20% to 30%, on average. This is a reasonable result for a simplified design procedure, where some conservative simplifications and assumptions in the analysis are necessary. There is considerable variability in the comparisons between calculated and measured maximum deflections for individual tests, which is due to a number of factors, including the inherent variability between identical blast tests and the limited measurements of many specific component and blast load parameters for the tests. Much less variability is expected for a database of tests where all the pertinent component and blast load parameters are measured. Therefore, for future comparisons, it would be helpful if any measured material properties in blast tests are documented and along with electronic files containing the applied blast loads.

REFERENCES

ASCE Task Committee on Blast-Resistant Design, *Design of Blast-Resistant Buildings in Petrochemical Facilities*, 2nd ed., American Society of Civil Engineers, Reston, VA, 2010.

Biggs, J. D., Introduction to Structural Dynamics, McGraw Hill Publishing Company, New York, NY, 1964.

Colthorp, D.R., Vitayaudom, K.P., and Kiger, S.A., "Semi-Hardened Facility Design Criteria Improvement," Report No. ESL-TR-85-32 by U.S. Army Waterways Experiment Station for Engineering and Services Laboratory at Tyndall AFB, September, 1985.

CONWEP V2.1.0.3 Computer Program, Distributed by USACE Engineer Research & Development Center (ERDC), Vicksburg, Mississippi.

Forsèn, R., "Airblast Loading of Wall Panels," Swedish Defense Research Institute (FOA) Report C-20586-D6, October, 1985.

Forsèn, R., "Increase of In-Plane Compressive Forces Due to Inertia in Wall Panels Subjected to Blast Loading," Swedish Defense Research Institute (FOA) Report C-20769-2.3,2.6, October, 1989.

Holland, T., and Wesevich, J., "In-situ Blast Testing of Shear-Screw Mechanical Couplers," presented at the 34th DoD Explosives Safety Board Seminar, July, 2010.

Lowak, M., and Montoya, J., "Shock Tube Testing of Precast Concrete Panels," prepared for Protection Engineering Consultants, Inc., BakerRisk Project No. 01-03471-001-11, March, 2012.

Naito, C., Beacraft, M., Hoemann, J., Shull, J., Bewick, B., and Hammons, M., Dynamic Performance of Insulated Concrete Sandwich Panels Subjected to External Explosions (Volume II), Air Force Research Laboratory Report, AFRL-RX-TY-TR-2011-0039, April 2011.

Naito, C., Beacraft, M., Hoemann, J., Shull, J., Salim, Hani, Bewick, B., “Blast Performance of Single-span Precast Concrete Sandwich Wall Panels,” Accepted for publication in ASCE Journal of Structural Engineering, 2013.

Oswald, C. J., and Bazan, M., “Performance and Blast Design for Non-Load Bearing Precast Concrete Panels,” SEI Structures Congress, Boston, MA, 2014.

Oswald, C., Nebuda, D., Holgado, D., and Diaz, M., “Shock Tube Testing on Reinforced Masonry Walls,” 32nd DoD Explosives Safety Seminar Proceedings, Las Vegas, NV, August 2006.

PDC-TR 05-02, Component Explosive Damage Assessment Workbook (CEDAW), U.S. Army Corps of Engineers, Protective Design Center, July, 2005.

PDC-TR 06-01, Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS), U.S. Army Corps of Engineers, Protective Design Center, September, 2006.

PDC-TR 06-01, Rev. 2, Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS) (Distribution C), U.S. Army Corps of Engineers, Protective Design Center, December, 2012.

PDC-TR 06-02, Rev. 2, User’s Guide for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS) (Distribution C), U.S. Army Corps of Engineers, Protective Design Center, December, 2012.

PDC-TR 06-08, Single Degree of Freedom Structural Response Limits for Antiterrorism Design, U.S. Army Corps of Engineers, Protective Design Center, Revision 1, 2008.

Salim, H.A., Dinan, R., Kiger, S.A., Townsend, P.T., and Shull, J., “Blast Retrofit Wall Systems Using Cold-Formed Steel Studs,” Presented at the 16th ASCE Engineering Mechanics Conference, University of Washington, July 16-18, 2003.

Stea, W., Dobbs, N., Weissman, S., Price, P., and Caltagirone, J., “Blast Capacity Evaluation of Pre-Engineered Building,” US Army Armament Research and Development Command, Weapon Systems Laboratory, Contractor Report ARLCD-CR-79004, March, 1979.

Stea, W., Sock, F.E. and Caltagirone, J., “Blast-Resistant Capacities of Cold-Formed Steel Panels,” US Army Armament Research and Development Command, Weapon Systems Laboratory, Contractor Report ARLCD-CR-81001, May 1981.

Tancreto, J., “Dynamic Tests on Reinforced Concrete Slabs,” 23rd DoD Explosives Safety Seminar Proceedings, Atlanta, GA, August, 1988.

UFC 3-340-01, Design and Analysis of Hardened Structures to Conventional Weapons Effects (FOUO), Unified Facilities Criteria Program, U.S. Department of Defense, 2002.

UFC 3-340-02, Structures to Resist the Effects of Accidental Explosions, Unified Facilities Criteria Program, U.S. Department of Defense, 2008.

Varma, R.K., Tomar, C.P.S., Parkash, S., and Sethi, V.S., “Damage to Brick Masonry Wall Panels Under High Explosive Detonations,” PVP-Vol. 351, Stuctures Under Extreme Loading Conditions, ASME, 1977.

Wright, S.J., “Scaled Testing and Analysis of Concrete Buildings and Components,” Naval Air Warfare Center Weapons Division Report No. NAWCWPNS TM 7554, April, 1993.