

DESIGN OF SC WALLS AND SLABS FOR IMPULSIVE LOADING
DE-NE0008201
RESEARCH PERFORMANCE PROGRESS REPORT
(FINAL REPORT)

1	COVER PAGE ELEMENTS			
	A	AWARD INFORMATION		
		(1)	Federal Agency	Department of Energy, Idaho Operations Office
		(2)	Award Number	DE-NE0008201
	B	PROJECT INFORMATION		
		(1)	Project Title	Design of SC Walls and Slabs for Impulsive Loading
		(2)	Project Director / Submitting Official	Amit H. Varma, Professor, School of Civil Engineering, Purdue University, West Lafayette, IN 479067, ahvarma@gmail.com , Cell No. (517) 974 0936
	C	REPORT INFORMATION		
		(1)	Date of Report	11/11/2015
		(2)	Recipient Organization	Purdue University, West Lafayette, IN 47907
		(3)	Project / Grant Period	5/1/14 to 11/30/15
		(4)	Report Term	Final
	D	Signature of Submitting Official		
2	ACCOMPLISHMENTS			
	A	MAJOR PROJECT GOALS		
		TASK		PLANNED DATES % COMPLETE
		(1)	Analytical Investigation and Evaluation of SC Slabs	5/1/2014 - 6/30/2014 100%
		(2)	Analytical Investigation of Blast Behavior	5/1/2014 - 8/31/2014 100%
		(3)	Experimental Verification of Analytical Results	6/1/2014 - 9/30/2014 100%
		(4)	Development of Design Guidelines and Recommendations	9/1/2014 - 11/30/2014 100%
	B	GOALS ACCOMPLISHED		
		(1)	Major Activities	Task 1 - Complete. The analytical investigation of SC slabs is complete. Task 2 – Complete. Computational simulation and benchmarking analysis is complete. Task 3 – Complete. Eight SC wall sections were tested in four-point bending and computational simulations from Task (1) were verified. Twelve SC wall sections were subjected to simulated blast loads and computational simulations from Task (2) were verified. Task 4 – Complete. Experimentally validated design recommendations were developed.
		(2)	Significant Results	1. SC slab behavior has been investigated. 2. Static bending tests have been completed.

				3. Idealized resistance function for SC walls and slabs has been developed. 4. Dynamic impulse tests have been completed. 5. Numerical models (single-degree-of-freedom (SDOF) and detailed 3D nonlinear finite element (FE)) have been benchmarked. 6. Analytical parametric studies have been completed. 7. Design guidelines have been prepared and presented. 8. Design tool in the form of a total force vs total impulse (P-I) diagram has been prepared, experimentally validated, and presented.
		(3)	Major Findings / Conclusions	1. Third-scale specimens behavior governed by same mechanics as full-scale specimens. Equations from AISC N690s1 Appendix N9 accurately estimated the strength and stiffness of the scaled specimens. 2. Deformation ductility larger than 10 and curvature ductility larger than 15 were observed for SC wall sections constructed from Grade 50 hot-rolled steel faceplates 3. SC wall ductility was directly related to steel faceplate material ductility and the ratio of net to gross cross-section are of the faceplates. Ductility was indirectly related to steel strength 4. An idealized bi-linear static resistance function was a realistic approximation 5. SC walls designed to be flexure-controlled resisted shear failure under dynamic loads 6. Ductility demand for a given pressure pulse was inversely related to reinforcement ratio, faceplate yield strength, and shear reinforcement ratio. 7. FE model results of blast response of SC walls were sensitive to concrete tensile strength 8. SDOF model results of blast response of SC walls was sensitive to the percent equivalent viscous damping included
		(4)	Key Outcomes	1. Benchmarked numerical models for static and dynamic behavior of SC walls and slabs 2. Procedures to design blast resistant SC Walls and slabs. These procedures expand on the specifications currently included in AISC N690s1-15 Appendix N9. 3. P-I diagram to enable efficient analysis and design of blast resistant SC walls and slabs
	C	OPPORTUNITIES FOR TRAINING AND PROFESSIONAL DEVELOPMENT		
		(1)	Training	None
		(2)	Professional Development	The PI and one graduate student attended (or are scheduled to attend) the following conferences and workshops to present portions of the work from this project and participated in committee meetings related to blast resistant design of structures: (a) Structures Congress 2015, Portland, OR (b) PROTECT 2015, East Lansing, MI (c) SMiRT-23, Manchester, UK

				(d) Geotechnical and Structural Engineering Congress 2016, Phoenix, AZ (d) 2016 Critical Infrastructure Symposium, Charleston, SC
	D	DISSEMINATION OF RESULTS TO COMMUNITIES OF INTEREST		
		Results from this project have been disseminated to the structural engineering community through presentations at conferences and service by the PI and his graduate student on several national technical committees. Journal papers are currently in development to further disseminate the results. Additionally, invited presentations (portions of which describe results from this project) have been made by the PI and his graduate student at universities in the US, Korea, and China.		
	E	PLAN FOR NEXT REPORTING PERIOD		
		This is the final report for this project.		
3	PRODUCTS			
	A	Publications		
		(1)	Journals	The following journal articles are under preparation and will be submitted in 2015. [1] J. C. Bruhl and A. H. Varma, “Resistance Function for Uniformly Loaded Steel-Plate Composite Walls: Part 1, Experimental Development,” planned submission to Journal of Structural Engineering (Federal support will be acknowledged.) [2] J. C. Bruhl and A. H. Varma, “Resistance Function for Uniformly Loaded Steel-Plate Composite Walls: Part 2, Numerical Development and Idealized Function,” planned submission to Journal of Structural Engineering (Federal support will be acknowledged.) [3] J. C. Bruhl and A. H. Varma, “Experimental Evaluation of Blast Response of Steel-Plate Concrete Composite Walls,” planned submission to International Journal of Impact Engineering (Federal support will be acknowledged.) [4] J. C. Bruhl and A. H. Varma, “Performance of Steel-Plate Concrete Composite Walls Subject to Blast Loads,” planned submission to Journal of Constructional Steel Research (Federal support will be acknowledged.)
		(2)	Dissertations	[5] J. C. Bruhl. “Behavior and Design of Steel-Plate Composite (SC) Walls for Blast Loads”, PhD, Purdue University, 2015. (Federal support was acknowledged.)
		(3)	Conferences	[6] J. C. Bruhl and A. H. Varma, “Summary of Blast Tests on Steel-Plate Reinforced Concrete Walls,” in Structures Congress 2015, 2015, pp. 151–159. (Federal support was acknowledged.) [7] J. C. Bruhl and A. H. Varma, “Summary of the Behavior of Steel-Plate Composite (SC) Walls Subjected to Impulsive Loads,” in Transactions of the 23rd International Conference of Structural Mechanics in Reactor Technology (SMiRT-23), 2015. (Federal support was acknowledged.)

				[8] J. C. Bruhl and A. H. Varma, "Protection Using SC Composite Structures: Overview of Blast and Impact Resistance," in Response of Structures Under Extreme Loading [Transactions of the Fifth International Workshop on Performance, Protection, & Strengthening of Structures Under Extreme Loading (PROTECT 2015)], V. K. R. Kodur and N. Banthia, Eds. DEStech Publications, Inc., 2015, pp. 538–545. [9] J. C. Bruhl and A. H. Varma, 14-17 FEB 16, Phoenix, AZ. Paper Title: "Experimentally Validated Analysis Methods for Steel-Plate Composite (SC) Walls Subjected to Blast and Impact Loads." ASCE Geotechnical and Structural Engineering Congress 2016. (accepted for presentation and publication)
	B	Web Site		None
	C	Networks		None
	D	Technologies		None
	E	Inventions		None
	F	Other		[10] J.C. Bruhl and A.H. Varma, "Behavior and Design of Steel-Plate Composite (SC) Walls for Blast Loads," Data Set, Purdue University Research Repository (PURR), https://purrr.purdue.edu/projects/scblast, 2015
4	PARTICIPANTS AND OTHER COLLABORATING ORGANIZATIONS			
	A	INDIVIDUALS		
		(1)	Name:	Amit H. Varma
			Project Role:	P.I.
			Nearest person month worked:	5
			Contribution to Project:	Supervised the development and completion of this project.
			Collaborated with individual in foreign country:	Yes
			Country(ies) of foreign collaborator:	China, Korea
			Travelled to foreign country:	Yes
			If traveled to foreign country(ies), duration of stay:	2 weeks
		(2)	Name:	Jakob C. Bruhl
			Project Role:	PhD Candidate
			Nearest person month worked:	15
			Contribution to Project:	Acquired the materials, constructed and tested the specimens, and completed the analysis for this project. He also coordinated the blast testing at the ERDC testing facilities in Vicksburg, MS.

			Collaborated with individual in foreign country:	No
		(2)	Name:	Thomas G. Bradt
			Project Role:	Research Engineer
			Nearest person month worked:	3
			Contribution to Project:	Assisted in the construction and testing of the specimens. He also assisted in the transport of specimens to the ERDC testing facilities in Vicksburg, MS.
			Collaborated with individual in foreign country:	No
	B	ORGANIZATIONS		
		(1)	Organization Name:	American Institute of Steel Construction (AISC)
			Contributions:	Financial support
5	IMPACT			
	A	IMPACT ON PRINCIPAL DISCIPLINE		
		(1)	The results of this research project provide structural engineers with necessary tools and recommendations to design safety-related nuclear power plant using SC.	
		(2)	Data from this research project contribute to the body of knowledge as the first tests of blast effects on SC structures designed IAW AISC N690s1-15 Appendix N9.	
	B	IMPACT ON OTHER DISCIPLINES		
		(1)	The results and findings are expected to be of interest to the broader protective design community seeking ways to more effectively protect critical infrastructure.	
		(2)	The results and findings are expected to be of interest to the defense industry seeking new and efficient ways to protect equipment and personnel from a variety of threats.	
	C	IMPACT ON DEVELOPMENT OF HUMAN RESOURCES		
		(1)	The project has provided new research opportunities for faculty and graduate students at Purdue University to contribute to protective design structural systems and methodologies.	
		(2)	The project has created opportunities for undergraduate research experiences at Purdue University and the United States Military Academy, exposing these young students to research and concepts of protective design.	
	D	IMPACT ON PHYSICAL, INSTITUTIONAL, AND INFORMATION RESOURCES		
		(1)	None	
	E	IMPACT ON TECHNOLOGY TRANSFER		
		(1)	The design tools and recommendations resulting from this project are expected to be of interest to the U.S. Departments of State, Defense, and Homeland Security, seeking new and efficient ways to protect, infrastructure, equipment and personnel from a variety of threats.	
	F	IMPACT BEYOND SCIENCE, ENGINEERING, AND ACADEMICS		
		(1)	None	
	G	AMOUNT OF AWARD'S BUDGET BEING SPENT IN FOREIGN COUNTRIES		
		(1)	None	
6	CHANGES OR PROBLEMS			
	None.			

7	SPECIAL REPORTING REQUIREMENTS
	None
8	BUDGETARY INFORMATION
	See attachments

PURDUE UNIVERSITY
GRADUATE SCHOOL
Thesis/Dissertation Acceptance

This is to certify that the thesis/dissertation prepared

By Jakob Christian Bruhl

Entitled

BEHAVIOR AND DESIGN OF STEEL-PLATE COMPOSITE (SC) WALLS FOR BLAST LOADS

For the degree of Doctor of Philosophy

Is approved by the final examining committee:

Amit H. Varma

Paul Mlakar

Judy Liu

Pablo D. Zavattieri

Weinong W. Chen

To the best of my knowledge and as understood by the student in the Thesis/Dissertation Agreement, Publication Delay, and Certification/Disclaimer (Graduate School Form 32), this thesis/dissertation adheres to the provisions of Purdue University's "Policy on Integrity in Research" and the use of copyrighted material.

Amit H. Varma

Approved by Major Professor(s): _____

Approved by: Dulcy Abraham

04/20/2015

Head of the Department Graduate Program

Date

BEHAVIOR AND DESIGN OF STEEL-PLATE
COMPOSITE (SC) WALLS FOR BLAST LOADS

A Dissertation

Submitted to the Faculty

of

Purdue University

by

Jakob C. Bruhl

In Partial Fulfillment of the

Requirements for the Degree

of

Doctor of Philosophy

May 2015

Purdue University

West Lafayette, Indiana

To the Engineer Soldiers with whom I have had the pleasure to serve.
Essayons!

ACKNOWLEDGMENTS

This PhD is the result of support, guidance, and encouragement of many. I thank God for the opportunities and blessings he has granted. Thanks are due to my family for their love and support. To my wife, Lesley - thank you for being my partner in this journey and creating a loving home for our family. You and our children bring a special joy to my life. Thank you to my parents, Win and Rita, for the encouragement and guidance you have provided throughout my life.

A special thanks to Professor Amit Varma for taking a chance on this random Army guy. I am extremely grateful for the opportunity you provided, the excellent teaching, patience, and pushing me well beyond what I thought I was capable. Importantly, thank you for showing me that we don't have to choose between being an experimentalist or an analyst - I should strive to do both effectively.

Thank you to my committee: Professors Judy Liu, Wayne Chen, Pablo Zavattieri, and Dr. Paul Mlakar for your suggestions, questions, and interest in making this project a success. Special thanks to Dr. Mlakar for connecting me to many engineers and scientists at ERDC. Help from Dr. Stan Woodson, Dr. Carol Johnson, and Mr. Stephen Robert was essential to this project and my education.

To Tom Bradt, our group's research engineer, thanks for your useful suggestions and ever willing help. To fellow students: Kadir Sener, for showing me the ropes of the research process; Efe Kurt, for introducing me to the joy of LS-DYNA; and many others with whom time was shared at the lab, in classes, and traveling - thanks for making this educational experience an enjoyable one!

Thank you to COL Fred Meyer, COL Joe Hanus, and Dr. Led Klosky for your confidence and encouragement. I am grateful for the opportunity you provided and am excited to serve with you again to inspire and educate the next generation of engineers and leaders for our Army and nation.

The experimental work presented in this dissertation was performed in two facilities: (1) the Robert L. and Terry L. Bowen Laboratory for Large-Scale Civil Engineering Research at the Lyle School of Civil Engineering at Purdue University in West Lafayette, IN and (2) the Blast Load Simulator at the USACE-ERDC Geotechnical and Structures Laboratory in Vicksburg, MS. The research presented was based on work funded by the U.S. Department of Energy (SL-14ID020104) and the American Institute of Steel Construction. All opinions, findings, and conclusions are those of the author and do not necessarily reflect the views of the sponsors.

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SYMBOLS

A_g	Gross cross-section area
A_n	Net cross-section area
A_{sa}	Cross-section area of a single headed stud anchor
A_{st}	Area of steel faceplate
A_{tie}	Cross-section area of a single tie bar
C	Structural damping coefficient
C_C	Compressive force in concrete
C_S	Compressive force in steel plate
E_c	Concrete modulus of elasticity
E_s	Steel modulus of elasticity
EI_{eff}	Effective stiffness of SC section (cracked, transformed section)
$F(t)$	Applied forcing function as a function time
F_u	Ultimate tensile strength
$F_{u,sa}$	Ultimate tensile strength of headed stud anchors
F_y	Yield strength
$F_{y,tie}$	Yield strength of tie bars
G_f	Concrete fracture energy
G_s	Steel shear modulus
I_c	Moment of inertia of concrete portion of SC wall when calculating EI_{eff}
I_s	Moment of inertia of steel plates of SC wall when calculating EI_{eff}
K_{LM}	Load-mass transformation factor
K_M	Mass transformation factor
L	Unsupported length

M	Effective mass of a structural element
M	Moment
M_y	Moment at onset of tension steel plate yielding
M_n	Nominal moment capacity
Q_n	Nominal shear strength of shear stud
R	Resistance
R_y	Resistance at onset of tension steel plate yielding
$R(y)$	Resistance as a function of displacement
T_S	Tensile force in steel plate
T_n	Natural period of structural element
V	Shear force
V_{conc}	Concrete contribution to shear strength
V_n	Nominal shear capacity
V_S	Direct shear demand at support
V_s	Steel tie bar contribution to shear strength
V_u	Ultimate shear demand
V_y	Shear force corresponding to M_y
X_E	Elastic deflection
X_M	Maximum deflection
a	Shear arm length
a	Strain off-set in moment-curvature power-law equation
b	Section width
c_1	Factor for yielding or non-yielding headed stud anchors
c_2	Calibration constant for determining EI_{eff}
d_s	Shear stud diameter
d_t	Tie bar diameter
e_{sh}	Engineering strain at the onset of strain hardening
e_f	Engineering failure (rupture) strain
e_u	Engineering ultimate strain

e_y	Engineering yield strain
f'_c	Concrete compressive strength
f_t	Concrete tensile strength
k_1	Initial (elastic) stiffness of static resistance function
k_2	Post-yield (strain hardening) stiffness of static resistance function
i_r	Peak reflected impulse
m	Mass
n	Modular ratio
n	Exponent in moment-curvature power-law equation
n	Exponent in steel stress-strain power-law equation
$p(t)$	Pressure as a function of time
p_r	Peak reflected pressure
s	Shear stud spacing
s_u	Engineering ultimate stress
s_y	Engineering yield stress
S	Tie bar spacing
t_c	Concrete thickness
t_a	Arrival time of the peak reflected pressure of a pressure pulse
t_d	Duration of the positive phase of a pressure pulse
t_p	Steel plate thickness
t_{sc}	Total thickness of SC section
v_u	Ultimate shear stress demand
w	Concrete crack width
$y(t)$	Displacement as a function of time
$\dot{y}$	Velocity (first derivative of displacement with respect to time)
$\ddot{y}$	Acceleration (second derivative of displacement with respect to time)
x_C	Distance from centroid of concrete compressive stress distribution to neutral axis

x_{NA}	Distance from extreme compressive fiber to neutral axis
α	Pressure exponential decay coefficient
δ_{mid}	Mid-span displacement
ε_o	Concrete strain corresponding to $0.5f'_c$
ε_{slip}	Strain associated with slip due to less than perfect composite action
ε_{St}	Strain in top steel plate
ε_{Sb}	Strain in bottom steel plate
ϕ	Curvature
ϕ_b	Bending strength reduction factor
ϕ_{vo}	Shear strength reduction factor
θ	Rotation
ρ	Reinforcement ratio
μ	Ductility
μ_ϕ	Curvature ductility
μ_Δ	Displacement ductility
ω_n	Natural frequency of structural element
ξ	Factor for yielding or non-yielding tie bars
ζ	Percent of critical damping

ABBREVIATIONS

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BLS	Blast Load Simulator
CFT	Concrete filled tube
CV	Coefficient of variation
DIF	Dynamic increase factor
DLF	Dynamic load factor
DOD	U.S. Department of Defense
DOE	U.S. Department of Energy
DOS	U.S. Department of State
ERDC	Engineer Research and Development Center
FE	Finite element
FRP	Fiber reinforced polymer
FS	Failure strain
GSL	Geotechnical and Structures Laboratory
HSLA	High strength, low alloy
IHQ	Hourglass control type in LS-DYNA
JEAG	Japanese Electric Association Guideline
KEPIC-SNG	Korean Electric Power Industry Code - Structural (Nuclear structures) Guideline
LCSR	Load curve, strain rate (LS-DYNA input parameter)
MDOF	Multi-degree-of-freedom dynamic analysis model
NEET	Nuclear Energy Enabling Technologies

NPP	Nuclear Power Plant
NRC	Nuclear Regulatory Commission
PSSRC	Profiled steel sheeting reinforced concrete
QM	Hourglass coefficient in LS-DYNA
RC	Reinforced concrete
SC	Steel-plate reinforced concrete
SCS	Steel-concrete-steel composite panel
SDOF	Single-degree-of-freedom dynamic analysis model
SEI	Structural Engineering Institute (an institute within ASCE)
SFRC	Steel fiber reinforced concrete
SIF	Static increase factor
SMR	Small modular reactor
SP	Steel cellular stiffened plates
TDOF	Two-degree-of-freedom dynamic analysis model
TM	U.S. Army Technical Manual
UFC	Unified Facilities Code
USACE	United States Corps of Engineers

ABSTRACT

Bruhl, Jakob C. Ph.D., Purdue University, May 2015. Behavior and Design of Steel-Plate Composite (SC) Walls for Blast Loads. Major Professor: Amit H. Varma.

Reinforced concrete (RC) structures have historically been the preferred choice for blast resistant structures because of their mass and the ductility provided by steel reinforcement. Steel-plate composite (SC) walls are a viable alternative to RC for protecting the infrastructure against explosive threats. SC structures consist of two steel faceplates with a plain concrete core between them. The steel faceplates are anchored to the concrete using stud anchors and connected to each other using tie bars. SC structures provide mass from the concrete infill and ductility from the continuous external steel faceplates. This dissertation presents findings and recommendations from experimental and analytical investigations of the performance of SC walls subjected to far-field blast loads.

Twelve SC panels were tested in the U.S. Army Corps of Engineers (USACE) Engineering Research and Development Center (ERDC) Blast Load Simulator (BLS). These panels varied flexure and shear reinforcement ratios, tie bar spacing, and steel faceplate strength. Results from the physical experiments were used to benchmark numerical models which were then used to expand the experimental database and perform a series of parametric studies investigating the influence of blast load, geometric, material, and analysis parameters.

Two benchmarked models were developed: (1) detailed finite element (FE) models using the non-linear FE code LS-DYNA, and (2) idealized single-degree-of-freedom (SDOF) models using experimentally validated bilinear strain hardening static resistance functions and exponential decay or triangular load pulse forcing functions. The idealized static resistance functions were developed from static tests of eight configu-

rations of SC walls which were also used to benchmark the FE modeling method. The results from the static experimental tests and benchmarked models are also provided in this dissertation.

Results from the physical experiments and analytical parametric studies were used to develop design recommendations. A rational method for designing SC walls to resist specific blast loads is presented along with pressure-impulse diagrams for use as design tools or aids.

1. INTRODUCTION

This dissertation describes an experimental and analytical study of the response of one-way steel plate composite (SC) walls to far-field blast loads. Two series of experimental tests were conducted: (1) static experiments to quantify available resistance and ductility and (2) dynamic experiments to confirm behavior under short-duration, high-pressure uniform loads. Results of these experiments were used to benchmark numerical models which were then used to conduct analytical investigations to understand the influence of design parameters on structural response. Results of this study were used to develop methods to design SC walls to resist blast loads.

1.1 Background

Reinforced concrete (RC) structures have often been used for blast resistant structures because of their mass and ductility provided by steel reinforcement. Most experimental work in blast resistant design during the mid-20th century focused on military applications of RC structures. The principal goal of this work was to protect military and national assets from the nuclear threat during the Cold War. Over the past 30 years, researchers have adapted military-focused work for civilian applications [1]–[3].

SC walls are a viable alternative to RC for protecting infrastructure against projectile or explosive threats and could be used as shield walls surrounding important infrastructure. A comparison between RC and SC walls is shown in Figure 1.1. RC walls are reinforced by orthogonal grids of steel reinforcing bars. These reinforcing bars are tied together with transverse ties or stirrups and RC structures require formwork for placing concrete. Alternatively, SC walls are comprised of two steel faceplates which contain a plain concrete core. Composite action is provided by steel

headed stud anchors on the inner surface of each faceplate. Tie bars provide shear reinforcement in-service and stability during construction. Because concrete is contained within the faceplates and no reinforcing bar cages are required for walls, shop fabrication of modules is possible. The plates serve as reinforcement and formwork; no additional work is required to construct or remove formwork as for RC. Construction time may be reduced by up to one-half when compared to RC [4] (see Figure 1.2). Understanding behavior, analysis, and design of SC walls for safety-related nuclear structures has advanced in recent years with the publication of design specifications in the U.S., Japan, and Korea [5]–[7].

Because of modular construction efficiencies, SC walls have recently been used as primary and secondary shield walls in nuclear power plants (NPP) [8], [9]. An SC structure is being considered as a candidate for optimizing the next generation of Small Modular Reactors (SMR) [4]. Bi-Steel, a proprietary SC wall has been used in the United Kingdom in protective barrier and building construction [10].

Widespread use of SC structures for other than NPP applications has not yet occurred. This is due in part to limited guidance to design SC walls for explosive (impulse) and projectile (impact) loads. Design of SC structures for design basis loads (i.e. flexure, in-plane shear, and out-of plane shear) can be accomplished in accordance with Appendix N9 of AISC N690s1 [5] which provides strength requirements to proportion each element of the system. This specification provides limited guidance to design against impulse and impact loads.

1.2 Motivation

The US Department of Energy (DOE) noted a need to advance design methods for steel plate concrete composite construction and minimize risks of terrorism [11]. A body of knowledge exists to design SC structures to resist the effect of impacts, such as from an aircraft or tornado or hurricane-borne projectiles [12]–[16]. Blast loading has been identified as an important concern for future nuclear power plant

designs [17] but limited research on behavior of SC walls subjected to blast exists in the public domain [18]–[21].

SC walls have been shown to have improved impact resistance when compared to RC. Although the concrete may crush, spalling is prevented unless the plates rupture. This improves performance when subjected to projectile impact: an SC wall has the same resistance to perforation by a projectile as a 30% thicker RC wall [22], [23]. Because spalling and scabbing are prevented, there is no secondary debris field on the interior of the structure to injure personnel or damage interior facilities. The characteristics which make SC structures effective against impact are expected to also make them effective against blast loads.

1.3 Research Problem Statement

There is a critical gap in engineering knowledge to understand behavior of SC structures subjected to blast loads. A body of knowledge of the behavior, analysis, and design of SC walls for static loads exists and there is a limited body of knowledge to design SC walls to resist the effect of impact. There is very little prior research on SC behavior under blast loads. This dissertation describes results from experimental and analytical investigations of the blast response of SC walls and recommends guidance for design of SC protective structures.

This project addresses two specific areas of exploration identified in the Nuclear Energy Enabling Technologies (NEET) and Small Modular Reactor (SMR) workshop reports: (1) design codes for steel plate concrete composite construction and (2) mitigation of terrorist hazards [11]. Results of this work are expected to be of interest to the U.S. Departments of State (DOS), Energy (DOE), and Defense (DOD) for mitigation of terrorist hazards against facilities around the globe.

1.4 Research Plan and Overview

1.4.1 Research Goal

This project develops and disseminates new knowledge of the response of SC walls to blast loads. This knowledge is provided by experimental results, benchmarked numerical models, and design recommendations, all of which are made publicly available. These provide nuclear and defense industries and the NRC, DOE, DOS, and DOD regulators references to accelerate design, review, licensing, and construction of small modular reactors and protective structures utilizing composite SC walls.

1.4.2 Research Objectives

The specific project objectives were to:

1. Quantify experimental performance of SC walls subject to blast loads,
2. Benchmark numerical modeling techniques that accurately capture structural behavior, performance, and failure of SC walls subject to blast loads, and
3. Develop modeling guidance, acceptance criteria, and design guidance for SC walls subject to blast loads.

1.4.3 Research Scope

This project includes experimental and analytical investigations of the behavior of SC panels under blast loading. By benchmarking analytical models to experimental results, realistic guidance was developed.

Because limited experimental data were available in the public domain (none of which investigated design parameters of typical SC walls as described by AISC N690s1 Appendix N9) a series of physical experiments was necessary. Quantifying the static resistance function of SC walls was critical to understanding SC post-yield behavior

and available ductility. Dynamic experiments verified dynamic behavior, were used to benchmark finite element (FE) and single-degree-of-freedom (SDOF) numerical models, and supported design guidance recommendations.

This project included two analytical components: SDOF and FEM. SDOF analysis, while limited, is used extensively in blast resistance design as explained in UFC 3-340-02 [24]. More detailed analyses were accomplished using LS-DYNA, a general purpose FE code that can model large deformation static and dynamic response of structures. LS-DYNA is well-suited for modeling explosions and explosive loading on structures. It uses explicit time integration methodology and has a limited implicit solver available [25]. These benchmarked modeling methods were used to complete a series of parametric studies.

The scope was limited:

- It focused solely on member response and did not investigate the influence of simultaneous axial load demand on the member nor did it account for structural system behavior. Current blast resistant design procedures begin by designing individual structural elements, and global structural response is typically evaluated through progressive collapse analysis in which an element is removed from the structure.
- Uniform pressure was assumed across the face of the panel, so findings of this project applied only to far-field blast effects, not to near-field explosions nor those in contact with the wall.
- While ductility of SC wall sections was investigated as part of this project, conclusions about ductility limits for use in design were not drawn.

1.4.4 Research Impact and Original Contributions

By improving the understanding of SC wall behavior when subjected to blast loads, this project enhances the state-of-the-art and knowledge regarding the use of

advanced composite systems. This is an important complement to current understanding of SC wall behavior subjected to impact and will enable designers to more thoroughly design protective structures. There were limited experimental results for blast loaded SC walls available in the public domain; this project expanded the available experimental database and provided benchmarked numerical models, filling a critical void in currently available knowledge.

1.5 Research Method

The research objectives were accomplished through four primary tasks:

1. Prepare single-degree-of-freedom models
2. Quantify SC wall experimental response to blast loads
3. Evaluate design parameter influence on SC wall blast response
4. Prepare design and detailing recommendations

1.5.1 Prepare Single-Degree-of-Freedom Models

Modern blast-resistant design methods are largely based on SDOF modeling of primary and secondary structural elements [26]. It was important that this research provide experimentally validated SDOF modeling guidance. The primary purpose of this task was to develop an idealized experimentally validated static resistance function of one-way SC wall sections.

Idealized resistance functions are cataloged for RC, structural steel, and masonry components. Adding resistance functions for SC walls was critical in linking the findings of this research project to current design practices. As described in Chapter 3, eight four-point bending tests were conducted in Purdue University's Robert L. and Terry L. Bowen Laboratory for Large-Scale Civil Engineering Research (Bowen Laboratory), FE models were benchmarked to experimental results, and the models were

used to develop resistance functions for uniform pressure loads. From these uniform pressure results, an idealized resistance function of an SC panel was developed and validated using dynamic experimental results.

1.5.2 Quantify SC Panel Experimental Response to Blast Loads

Because limited experimental data of the blast response of SC panels existed in the literature, performing physical experiments was critical. The purpose of this task was to validate SDOF analysis and benchmark FE models.

Twelve specimens were tested in the United States Army Corps of Engineers Engineering Research and Development Center (USACE-ERDC) Geotechnical and Structures Laboratory's (GSL) Blast Load Simulator (BLS). The tests included two different pressure-impulse combinations which confirmed elastic and plastic response of SC walls to blast loads. Time-history data from the tests included pressure, acceleration of the panels at mid-span, displacement of the panels at mid-span and at a location other than mid-span, and strains at three points along the tension face and one point on the compression face.

1.5.3 Evaluate Parameter Influence on SC Wall Blast Response

The blast experiments conducted included eight different SC wall designs which varied reinforcement ratio, steel plate strength, tie bar diameter (shear reinforcement ratio), and tie bar spacing. These test results were used to benchmark FE models which were then used to investigate the behavior of the specimens under different blast loads than were tested experimentally. Other numerical studies were completed using the benchmarked FE and SDOF models to quantify the influence of geometric, material, and loading variables to further expand the experimental database. The purpose of this task was to ensure that recommended design and detailing recommendations were valid for a practical range of design and loading variables.

1.5.4 Prepare Design and Detailing Recommendations

Results from the experiments and analysis were combined into recommendations for designing SC panels to resist far-field blast loads. Recommendations were provided in the following forms:

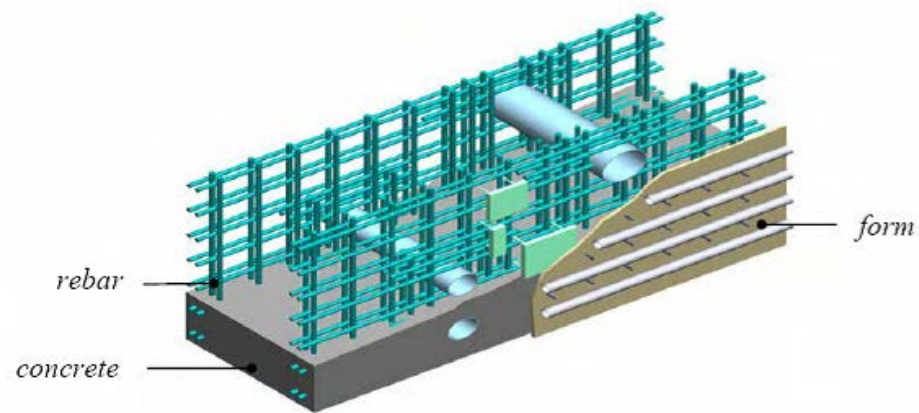
1. Design equations involving common variables: pressure, impulse, concrete thickness, concrete strength, steel strength, steel plate thickness, and tie bar spacing.
2. Elastic and ultimate capacity of the minimum practical wall thickness used in nuclear power plant construction.
3. Modeling guidance for SDOF analysis.
4. Modeling guidance for explicit FEM analysis.

1.6 Organization

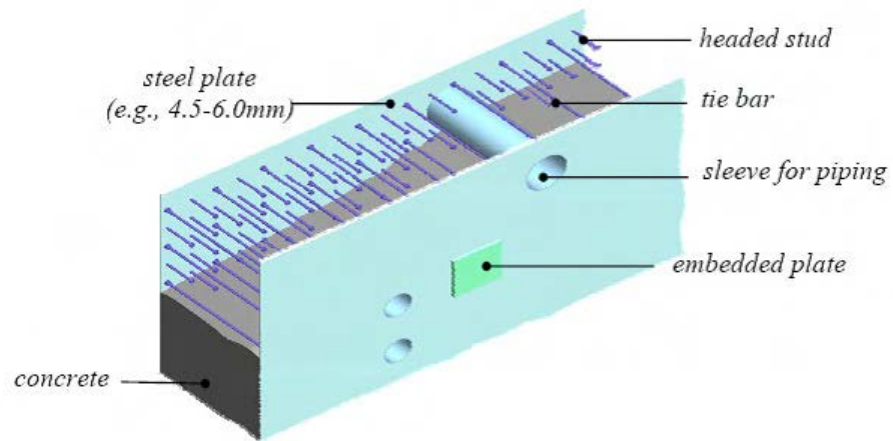
The organization of this dissertation is as follows:

- Chapter 2 is the literature review and addresses state-of-the-art and recent research in three areas: (1) design of SC structures, (2) blast resistant design methodology, and (3) behavior of steel-concrete composite structures under blast loads,
- Chapter 3 describes SDOF design methods, reports results from a series of static experiments, and develops the idealized static resistance function for one-way SC wall sections using benchmarked FE models,
- Chapter 4 reports results from the experimental investigation of blast response of one-way SC wall sections and describes design parameter influence on wall response,
- Chapter 5 describes benchmarked FE and SDOF modeling methods and influence of model parameters on numerical results,

- Chapter 6 describes numerical parametric studies, completed using benchmarked models, which investigated the influence of critical geometric, material, and loading parameters on the blast response of SC walls,
- Chapter 7 provides recommendations to design one-way SC walls to resist blast loads along with an example and design aids, and
- Chapter 8 summarizes the research project, describes the conclusions drawn, and provides recommendations for future work.



(a) RC Wall



(b) SC Wall

Figure 1.1. Comparison of Wall Components (from [4])

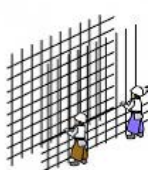
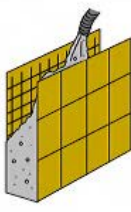
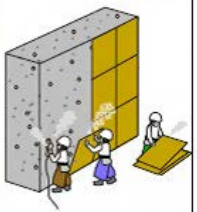
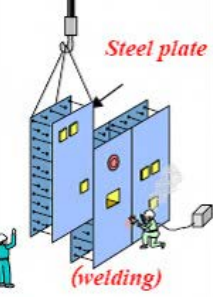
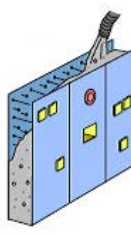
<i>Work Structure</i>	<i>Rebar arrangement</i>	<i>Form work (assembling)</i>	<i>Placing concrete</i>	<i>Form work (removal)</i>
<i>RC</i>		 <i>Wooden form</i>		
<i>28days</i>	<i>13days</i>	<i>7days</i>	<i>4days</i>	<i>4days</i>
<i>SC</i>	—	 <i>Steel plate</i> <i>(welding)</i>		—
<i>14days</i>	—	<i>10days</i>	<i>4days</i>	—

Figure 1.2. Construction Efficiency of SC Compared to RC Walls (from [4])

2. LITERATURE REVIEW

This project focused on understanding behavior of SC walls under blast loads and designing these walls to resist explosions. This literature review is divided into three sections which summarize:

1. State-of-the-art for SC design,
2. State-of-the-art for blast resistant design, and
3. Investigations of blast resistance of steel-concrete composite structures.

2.1 Design of Steel-Plate Reinforced Concrete Structures

2.1.1 Summary of Current SC Design Guidance

Flexure, in-plane shear, and out-of-plane shear behavior of SC walls has been investigated and developed into design guidance as Appendix N9 of AISC N690s1 [5] (a supplement to AISC N690-12 [27]). Bhardwaj et al. [28] provide a useful summary of the contents of the specification.

The specification includes design and detailing requirements for SC walls such as:

- Limitations:
 - SC wall thickness, reinforcement ratio, and material properties,
 - Tie bar and steel anchor spacing to ensure faceplate yielding occurs before faceplate buckling based on [29], and
 - Steel anchor strength and spacing to develop faceplate yield strength and prevent interfacial shear failure based on [29].

- Experimentally validated equations to calculate:
 - Effective flexure and shear stiffness considering the effect of concrete cracking due to accident thermal loading based on [30],
 - In-plane shear strength corresponding to faceplate yielding based on [31] and [32],
 - Flexure and out-of-plane shear strength based on [33],
 - Tensile and compressive strength adapted from AISC 360 [34], and
 - Interaction of in-plane forces and out-of-plane moments based on [35].
- Analysis methods:
 - An equivalent elastic FE model - the specification provides guidance on selecting geometric and material properties for this model,
 - Evaluating performance of SC structures subject to accident thermal loads, and
 - Design SC walls for impact or impulsive loads using simplified methods or FE analysis.
- Detailing requirements for small or large openings in SC walls.

2.1.2 Out-of-Plane Behavior of SC Structures

Sener and Varma [33] compared experimental results from 54 tests to shear design equations from U.S., Japanese, and Korean nuclear power plant design specifications (ACI 349[36], JEAG 4618[6], and KEPIC-SNG[7], respectively). The authors concluded that methods in ACI 349 provided lower bound estimates for shear strength and JEAG and KEPIC-SNG provided upper bound estimates.

Sener et al. [37] conducted a similar comparison of experimental results to design equations for flexural strength. Equations from ACI 349, JEAG 4618, and KEPIC-SNG were all found to provide accurate estimates of flexural strength of SC walls

subjected to out-of-plane loading with those from JEAG 4618 being the most conservative. Based on reliability analysis, the authors recommended strength reduction factors for equations from the three sources.

Leng et al. [38] conducted a series of out-of-plane tests to investigate the influence of shear arm-to-depth ratio focused on understanding shear strength of SC walls. They developed a mechanical model of transverse shear resistance based on internal equilibrium of three effective pieces after the formation of a critical diagonal crack in the shear span. Their model accurately estimated the experimental shear strength while estimates using Eurocode provisions tended to underestimate shear strength.

2.1.3 Composite Action and SC Axial Capacity

Zhang et al. [39] conducted parametric studies to characterize the influence of stud spacing on composite action of SC members and concluded that slenderness ratio (the ratio of stud spacing to plate thickness, s/t_p) was the best measure of expected composite action. Varma et al. [40] examined the influence of slenderness ratio on axial buckling capacity at ambient temperatures and under thermal loading and concluded that for non-slender steel faceplates, the axial capacity is the combined axial strength of the faceplates and concrete. It was demonstrated that local buckling was not expected to occur for non-slender faceplates.

2.1.4 In-Plane Shear Behavior of SC Panels

Ozaki et al. [31] investigated in-plane shear cyclic behavior of SC walls. They studied walls with and without openings and included partitioning webs which increased in-plane stiffness. The authors proposed design equations for an SC wall subject to cyclic in-plane demands.

Using results from Ozaki et al.'s tests and their own additional tests, Varma et al. [32] developed an idealized tri-linear shear force-shear strain response based on a simple mechanics-based model. Their model accounted for composite section behavior

before cracking and the change in behavior after cracking. They concluded that in-plane shear strength corresponded to the limit state of Von Mises yielding of the steel faceplates.

Epackachi et al. conducted a series of experimental and numerical tests to quantify the hysteretic behavior of SC walls [41]. They concluded that faceplate slenderness influenced the rate of strength deterioration under cyclic loads. Strength degradation was found to be reduced by closer spacing of the first row of headed stud anchors and the basemat.

Kurt et al. used benchmarked FE models to investigate the influence of SC wall-to-basemat anchorage on in-plane behavior [42]. They quantified the influence of wall aspect ratio on base moment and base shear. They also demonstrated the influence of the relative tensile strength ratio of the SC wall to basemat on in-plane behavior and provided recommendations to design full-strength connections.

2.1.5 Thermal Effects on SC Behavior

Thermal effects on SC walls were studied by Varma et al. [43]. Results from this investigation demonstrated that flexural and out-of-plane shear stiffness of SC walls in nuclear power plants at operating temperature can be accurately estimated using cracked transformed section properties. The thermal gradient at elevated temperatures led to additional concrete cracking which degraded flexural and out-of-plane shear stiffness. The authors provided equations to account for the effect of elevated temperatures on SC wall stiffness.

2.1.6 Impact Resistance of SC Panels

Research has conclusively demonstrated effectiveness of SC walls to resist local effects of missiles. Mizuno et al. [44] concluded that an SC wall provides the same level of protection as an RC wall which is 30% thicker. Steel faceplates can be accounted for in calculating impact resistance of an SC wall by considering them as

an equivalent thickness of concrete and using this equivalent concrete thickness in accepted equations for evaluating the impact performance of RC walls [15], [22], [23]. Validated against the database of all published tests of missile impact on SC walls available in the literature, Bruhl et al. [16] developed a method to design an SC wall to resist local failure (perforation) of an SC wall subject to missile impact.

Sohel & Liew [45] conducted a series of impact tests to evaluate local and global impact effects on steel-concrete-steel sandwich walls with welded j-hooks serving as shear studs and tie bars. They dropped a 2750-lb hemispherical shaped impacting weight in the center of simply-supported two-way slabs from a height of 9.8-ft (resulting in impact velocities from 21.6 to 24.2-ft/s). The authors presented an energy method to estimate global deflection and impact force using an idealized load-displacement curve. This idealized global resistance function was based on experiments from a previous study of the static performance of this wall configuration in two-way out-of-plane bending. Deflection ductility values on the order of 10 and 15 were observed for normal-weight and light-weight concrete cores, respectively, in these static tests [46]. Results from their method underestimated maximum mid-span displacement by as much as 22% and impact force by as much as 25%.

Johnson et al. [47] suggested that modifying the idealized SDOF method of dynamic analysis of the impact response of RC walls to account for the differences of SC walls was a reasonable way to assess global response of SC walls subjected to impact loads. Through a series of numeric parametric studies, they demonstrated that SC walls exhibit strain hardening after initial yield and recommended an elasto-plastic assumption for the resistance function for two-way bending of SC walls. Ductility ratios of around 10 were observed for fixed-end two-way panels, and slightly smaller ductility was observed for simply-supported two-way panels. Bruhl et al. [48] performed similar analyses and proposed equations to idealize static resistance functions for two-way SC walls with central concentrated loads. The authors recommended use of this resistance function in SDOF analysis of impact loads on SC walls.

Bruhl et al. [49] proposed a two-degree-of-freedom (TDOF) model to account for local and global deformation due to impact loading of two-way SC walls. This method consisted of a cascaded system of two effective masses and resistance functions. One considered global flexure behavior and the other accounted for local deformation due to a dislodged concrete conical plug. Their work was not prescriptive but described the model components and method to extract the local resistance function from an analytically derived resistance function.

2.2 Blast Resistant Design Methodology

Until 2011, when *Blast Protection of Buildings* (ASCE/SEI 59-11) [26] was published, there was no industry standard for structural engineers to reference when designing structures to withstand explosions. This is not to suggest that information was not available. Guides, handbooks, and manuals have existed for the past sixty years to assist engineers in designing structures to withstand the effects of explosions but no specification existed. The most common reference was a military manual referred to as the Tri-Service Manual or by its Army designation, Technical Manual (TM) 5-1300 [50]. The need for transferring military knowledge for civilian applications was recognized shortly after the Second World War when *Design of Protective Structures (A New Concept of Structural Behavior)* was published and presented at the 1950 annual meeting of ASCE [51].

After the Cold War interest in blast-resistant design waned in the U.S. until the high profile terrorist attacks of the 1990s. Interest surged after the attacks of September 11, 2001. Updated literature includes commercially published references such as the *Handbook for Blast-Resistant Design of Buildings* [52], which compiled a wide range of sources into a single source for blast consultants. Other recently published books on the topic of blast resistant design include two published by the American Society of Engineers [53], [54] and one published in the UK [55]. *Modern Protective Structures* [56] is a comprehensive reference for designing blast- and impact-resistant

structures and provides extensive background on weapons effects. Government publications have been regularly updated over the past five decades, most notably the Unified Facilities Code (UFC) 3-340-02, *Structures to Resist the Effects of Accidental Explosions* [24] which updated and replaced TM 5-1300.

Most research about structural response to blast loads focused on RC and structural steel. Historically, RC has been the material of choice for protective structures because of its mass and the extensive experimental database upon which to validate design procedures. Tests of structural steel components demonstrated benefits of steel ductility to dissipate energy from an explosive event. Because of its ubiquitous use throughout the world, a large database of blast tests of reinforced and unreinforced masonry walls also exists. ASCE 59-11 permits structural components of reinforced concrete, structural steel, masonry, and composite slabs. The standard permits strengthening of RC and masonry components with fiber reinforced polymers (FRP). UFC 3-340-02 contains chapters to design blast-resistant structural elements of RC, structural steel, and reinforced masonry. SC structures are not addressed by ASCE 59-11 or UFC 3-340-02.

Regardless of the material chosen, the most common design procedures include SDOF analysis of structural elements to estimate maximum deflection and end rotation. These calculated values are compared to ductility and rotation limits. If the element is within the limit, it is accepted as surviving the blast pressure wave. If not, it is considered destroyed and analysis proceeds with progressive collapse analysis with the destroyed element removed from the structure.

The pressure on a structural element due to an explosion depends on several factors including:

- The weight of the explosive, typically expressed as equivalent weight of TNT,
- The distance from the explosion to the element (referred to as “stand-off”),
- The angle of incidence,
- Surrounding structures, and

- Whether the explosion occurred in air or on a surface.

As shown in Figure 2.1, pressure on a structure varies with time as the blast wave moves through space. Typically, it begins with a very short rise time (often assumed to be zero), decays quickly, and may have a negative phase in which the pressure drops below atmospheric pressure.

There are several methods to estimate pressure distribution across the face of a structure and along the length of structural members. Formulae and charts exist to determine the characteristic parameters of a blast wave for a given charge weight and stand-off distance [24]. Empirical methods are incorporated into various software programs such as AT Blast [57] or CONWEP [58].

In addition to SDOF analysis, ASCE 59-11 describes pressure-impulse charts, a simpler method of somewhat limited use. More complex methods include multi-degree-of-freedom (MDOF) and explicit linear or non-linear finite element analysis (FEA) for use when simpler methods are judged to reduce the reliability of the results or where greater reliability is desired. LS-DYNA includes several methods to model blast pressures including fluid-structure interaction and simpler methods based on the same blast equations implemented by CONWEP[59].

2.3 Blast Loaded Steel-Concrete Composite Structures

2.3.1 Retrofitting RC Components to Improve Blast Resistance

The need to retrofit critical infrastructure to better resist potential explosive attacks led to studies of jacketing existing RC and masonry structures with steel plates. Orton explained that attaching steel plates eventually gave way to the use of fiber reinforced polymer (FRP) because of the ease of application[60]. However, current research continues into the benefits of encasing existing RC columns and bridge piers with steel plates. Among the recommendations that Winget et al. [61] provided to retrofit reinforced concrete bridge piers is the addition of steel jackets. Problems were

noted at connections, however, because of discontinuous steel plates which increased flexural resistance but also increased chances of direct shear failure of the RC member.

2.3.2 Blast Resistance of Concrete Filled Tubes

Tests demonstrated the improved blast resistance of concrete filled tubes (CFT) bridge piers when compared to jacketed reinforced concrete columns [62], [63]. These experiments demonstrated ductile behavior of CFTs and reduced the risk of direct shear failure if the foundation connection was properly detailed. If steel plates were continuous, as they are in SC construction, direct shear failure was prevented and blast-resistance was better than RC structures.

2.3.3 Blast Resistance of SC Structures

Heng et al. [18] reported results of static and blast tests of one-third scale fully enclosed steel-concrete-steel sandwich panels. Their panel design contained no internal components (shear studs or tie bars) and the concrete core was enclosed by steel plates on all sides as shown in Figure 2.2. Static tests exhibited initial stiffness prior to first cracking, reduced stiffness to yield, and nearly perfect plasticity post-yield as shown in Figure 2.3. The four explosive tests varied stand-off distance for a constant charge weight. The charges were placed no further than 17.3-in away from the panels resulting in near field effects. They used an energy balance method equating kinetic energy from the explosion to the area under the resistance function to estimate maximum deflection. Their results ranged from underestimating the deflection by 31% for the closest charge to overestimating the deflection by 55% for the furthest charge. They concluded that the energy method was generally conservative and that this structural system was a viable way to provide expedient construction of protective structures.

Lan et al. [64] described test results from a series of blast tests of three different configurations of steel-concrete composite panels: (1) steel fiber reinforced

concrete (SFRC), (2) profiled steel sheeting reinforced concrete (PSSRC), and (3) steel-concrete-steel composite (SCS) panels similar to those in [18]. Of the configurations they tested, SCS was most similar to the SC structures discussed in this dissertation. Tests of the SCS panels investigated the influence of concrete fill (they tested some panels with no concrete), concrete thickness, and steel plate thickness for a variety of charge weights at a constant stand-off distance. Results indicated that panels filled with concrete provided much higher resistance than those without and that as total thickness and steel plate thickness increased so did the resistance of the panel.

Liew and Wang [19] conducted blast tests on the same configuration of SCS as tested for impact resistance by [45] and a type of panel they called cellular stiffened plate (SP). The SCS panels had j-hooks in place of headed stud anchors and tie bars as shown in Figure 2.4(a) and the SP configuration connected the steel faceplates with internal steel stiffeners as shown in Figure 2.4(b). They tested panels with different steel plate thicknesses subjected to blasts of the same charge weight and stand-off distance. They concluded that increased steel faceplate thickness improved blast resistance of the structure and the type of concrete infill influenced the type of failure experienced by the panel. For specimens with lightweight concrete, failure was dominated by shear. For those with normal-weight concrete, flexure dominated the response. They also concluded that the use of ultra high strength concrete does not improve the performance of SCS panels because of concrete brittleness.

Hulton [20] provided a qualitative summary of tests conducted by the United Kingdom Defence Research Agency on steel-backed reinforced concrete panels subjected to blast loads. This test series included eight panels of varying dimensions, some with steel plates on the back-face alone and others with steel plates on both front- and back-faces. The author concluded that panels with plates on both sides performed better than those with only back-plates but that the front-plate was inefficient because of fragment damage or buckling.

Hulton & Gough [21] discussed the development of a proprietary form of SC walls by British Steel, referred to as Bi-Steel™ (see Figure 2.5). The authors described various configurations of SCS composite sections and explained how the Bi-Steel™ design overcame weaknesses of earlier designs. Bi-Steel™ consisted of two steel plates separated by transverse bar connectors friction welded to the plates. These modular sections could be cast with concrete in the factory and shipped to their intended location or set in place, welded to each other, and filled with concrete on site. The authors described a truss analogy for the static behavior of Bi-Steel™ panels and reported general behavior of the panels in blast tests but provided no details on test specimens, blast loads, or observed damage.

Coyle & Cormie [55] described blast resistant design of SCS composite sections, addressed similarities and differences between RC and SCS, and discussed computation of strength and stiffness. They focused on Bi-Steel™ and the information was not general enough to be useful for other SC configurations.

Of the different configurations of steel-concrete composite wall sections that have been tested, Bi-Steel™ is the most similar to the general SC system described in this dissertation but it is proprietary and has some key differences from SC structures designed in accordance with AISC N690s1 Appendix N9. For example, Bi-Steel™ does not include headed stud anchors and its flexural reinforcement is often larger than the 5% maximum permitted by AISC N690s1 Appendix N9.

2.4 Chapter Summary

This chapter provided a review of literature relating to the project reported in this dissertation. A discussion of research related to in-plane, out-of-plane, and combined loading of SC walls has been provided. A summary of the current state-of-the-practice regarding blast resistant design was also provided. Finally, a summary of experimental investigations of the blast response of various configurations of steel-plate concrete composite panels was included highlighting the fact that these tests consisted of

unique configurations, none of which were covered by the provisions of AISC N690s1 Appendix N9.

The next chapter reports results from static out-of-plane bending experiments of eight different SC wall designs and describes the development of an idealized static resistance function for one-way SC walls for use in dynamic SDOF analysis. Development of FE and fiber models are also described in the following chapter.

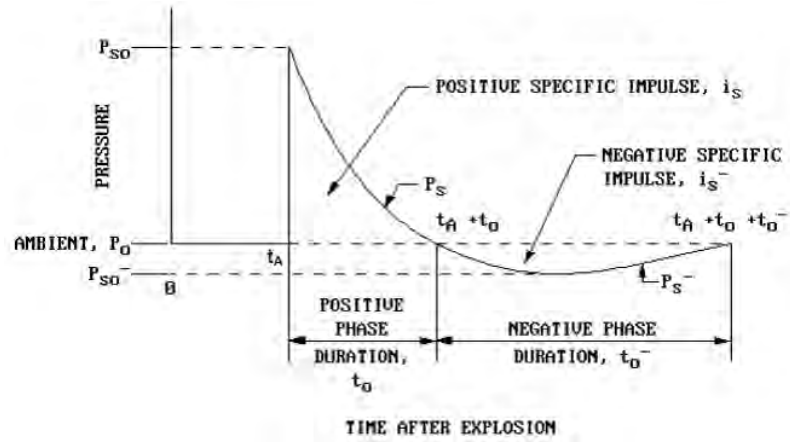
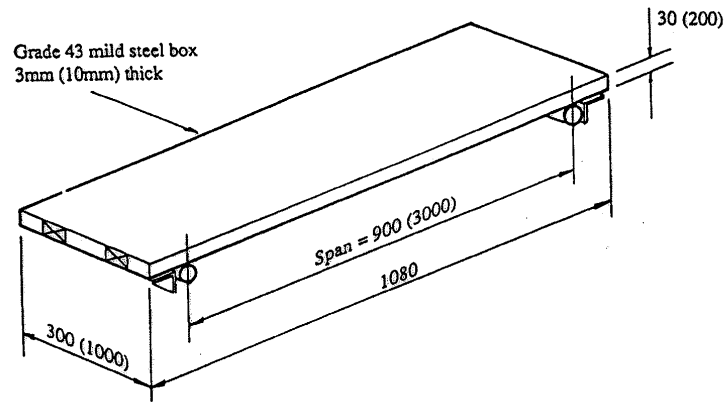


Figure 2.1. Realistic Pressure Curve (from [24])



Figures in brackets denote dimensions of N. Ireland trial panels

Figure 2.2. Configuration of Heng et al.'s SCS Panels (from [18])

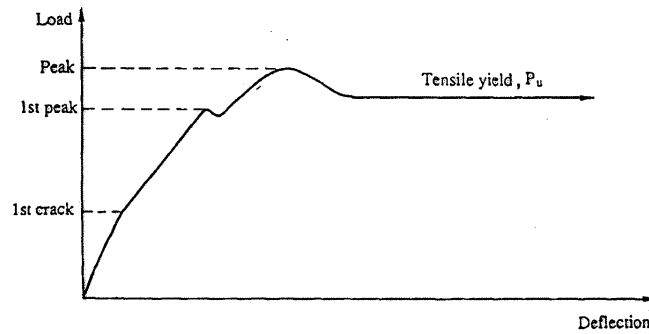


Figure 2.3. Static Load-Displacement Curve of Heng et al.'s SCS Panels (from [18])

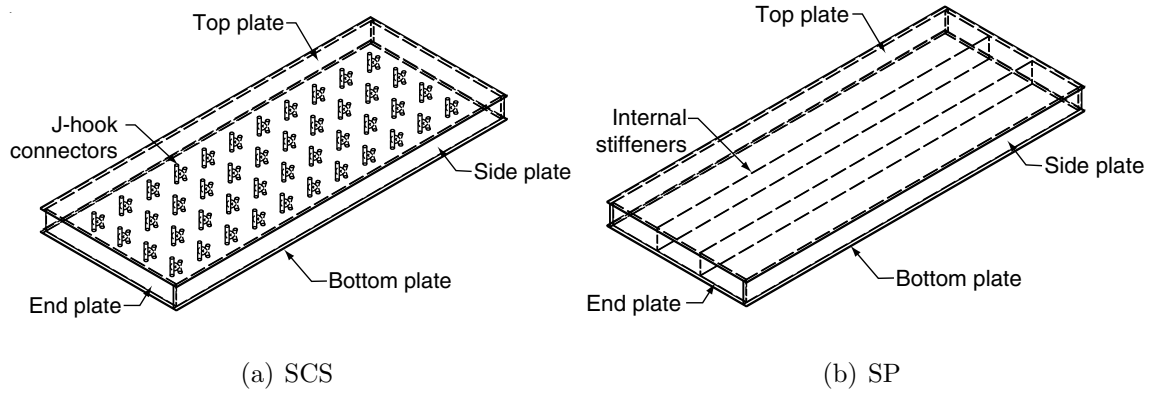


Figure 2.4. Configuration of Liew and Wang's Panels (from [19])

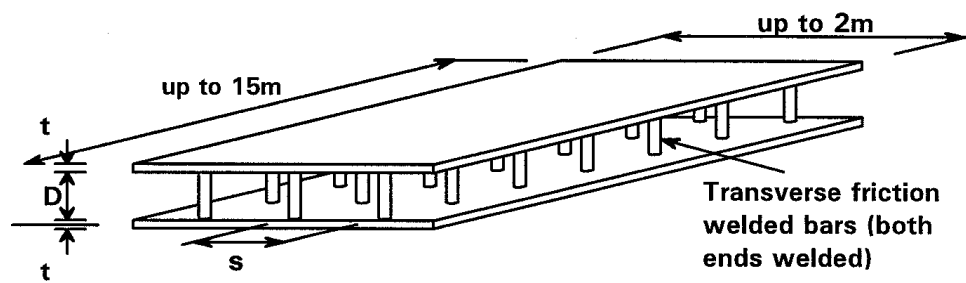


Figure 2.5. Configuration of Bi-Steel™ Panels (from [21])

3. STATIC RESISTANCE OF ONE-WAY SC WALL SECTIONS

The idealized lumped mass SDOF system is a common analysis tool used in blast resistant design. SDOF evaluation methodology to estimate the response of structural elements to impact and impulse loads is well established as described by Biggs [65], ASCE Manual of Practice No. 58 [66] and UFC 3-340-02 [24].

The static resistance function for the member being analyzed is a critical input to an SDOF model. Development of this function is the focus of this chapter. Four-point bending static tests of eight specimens were completed. These specimens varied design parameters including flexural and shear reinforcement ratios, steel faceplate strength, and tie bar spacing. Benchmarked numerical models were used to analyze behavior of SC walls subject to uniform loads and from these results an idealized bi-linear static resistance function was developed.

3.1 Single-Degree-of-Freedom Analysis Method

Consider a generalized SDOF equation of equilibrium:

$$M \cdot \ddot{y}(t) + C \cdot \dot{y}(t) + R(y) \cdot y(t) = F(t) \quad (3.1)$$

In which M is the effective mass of the structure, C is the effective structural damping, $R(y)$ is the resistance as a function of the displacement $y(t)$, $F(t)$ is the applied forcing function, and $\dot{y}$ and $\ddot{y}$ are the first and second derivatives of the displacement of the mass with respect to time (the velocity and acceleration, respectively). Typically, the peak response of a structural element to an impact or impulse load occurs in the first cycle and structural damping, C , is conservatively neglected [24]. The equation of motion to solve for the displacement time history, $y(t)$, is most often Equation (3.2).

$$M \cdot \ddot{y}(t) + R(y) \cdot y(t) = F(t) \quad (3.2)$$

3.1.1 Effective Mass

The effective mass, M , of a structural element is calculated by multiplying the total mass by the mass transformation factor, K_M , based on member geometry, support conditions, and expected response (elastic, elastic-plastic, or plastic). Values of mass factors for uniformly loaded one-way slabs are tabulated in a variety of sources [24], [65], [66].

3.1.2 Forcing Function

The forcing function, $F(t)$, for blast loads is computed by multiplying the pressure at a given time, $p(t)$, by the wall area. This assumes a uniform pressure distribution over the wall surface: a reasonable approximation for far-field explosions. There are several methods to idealize $p(t)$: multi-linear, triangular with finite rise time, triangular with zero rise time, or constant pressure for a finite time. Each of these idealizations are suited for particular categories of explosions as UFC 3-340-02 explains.

Other options to idealize $p(t)$ include best fit curve equations. One of the most widely accepted for explosion-generated pressure waves is the modified-Friedlander waveform (Equation (3.3)) which assumes exponential decay of the pressure and includes the negative phase of the pressure-time history. In Equation (3.3), t_d is the duration of the positive phase and α is the decay coefficient. This formulation includes the arrival time, t_a , and assumes a linear rise from 0 to p_r but can be modified to assume zero rise time (i.e. $t_a = t_o$).

$$p(t) = \begin{cases} p_r \frac{t}{t_a}, & \text{if } t \leq t_a \\ p_r \left[1 - \frac{t-t_a}{t_d} \right] e^{-\alpha(t-t_a)/t_d}, & \text{otherwise} \end{cases} \quad (3.3)$$

Figure 3.1 depicts the two idealizations used in this study. The Friedlander waveform (solid line) was used to idealize the blast load for FE and SDOF benchmarking models in Chapter 4. In these cases, the measured pressure time-history was idealized with the best fit of the Friedlander waveform. The triangular idealization (dashed

line) was used for the parametric study in Chapter 6 to reduce the variables considered and focus on the influence of design parameters rather than on pressure form variables. For both idealized forms, t_a can be set equal to t_o to model zero rise time.

3.1.3 Resistance Function

The resistance function, $R(y)$, for one-way RC walls is often conservatively assumed to be elastic-perfectly plastic. When coupled with a simplified forcing function, chart solutions are available in UFC 3-340-02. These same chart solutions would be available for one-way SC walls if the same simplifications were made. In order to determine the suitability of the elastic-perfectly plastic idealization for $R(y)$ for one-way SC walls, static behavior was experimentally investigated. The rest of this chapter describes laboratory and numerical investigations of one-way SC specimens.

Resistance functions for specific wall designs subjected to uniform pressure have been experimentally developed using several methods summarized by Salim et al[67]:

- a positive pressure air bladder reacts against the test specimen and a rigid frame,
- a water chamber pressurized to apply a uniform pressure to the test specimen,
- a loading tree distributes the load as evenly as possible through a series of spreader beams and suspenders, or
- a vacuum chamber applies pressure to the test specimen by pulling a vacuum on one side of the chamber as the other side remains at ambient pressure.

Each of the methods above have benefits and limitations. The air bladder, water chamber, and vacuum chamber apply a uniform load over the surface area of the test specimens but are limited by the amount of pressure that can be generated in each system. For example, the largest pressure that can be applied in the vacuum chamber is ambient pressure. Tests conducted using the vacuum chamber at the

University of Missouri - Columbia by Salim et al[67] on full-scale cold-rolled steel stud walls and CMU walls required less than 3 psi to reach full capacity and observe available ductility of the systems. The water chamber constructed at the USACE-ERDC has been used to develop resistance functions of one-third scale RC and CMU walls and can reach pressures of 35 psi[68]. Controlling the pressure in these chambers as yielding begins or sudden drops in capacity occur requires careful observation and responsive controls. Better control for these situations is achieved with the loading tree [69].

Each of the methods described require specialized facilities not available in most university structural engineering laboratories. For researchers at these institutions, additional cost must be incurred to conduct tests at other facilities or construct their own specialized equipment. An alternative exists: use a conventional bending test such as four-point bending, benchmark numerical models to the results, and use the benchmarked model to perform numerical analysis for uniform pressure distribution. The result of this analysis is the desired resistance function without the use of specialized testing facilities. This method was the approach used for this research.

3.2 Static Tests of SC Wall Sections

3.2.1 Specimen Design and Material Properties

Eight SC wall sections were tested in four-point bending at Bowen Laboratory. Geometric and material details of each specimen are provided in Table 3.1. These specimens were of the same design as those tested dynamically (see Chapter 4). Each had identical global dimensions: 4-in total thickness, t_{sc} ; 11-in width, b ; and 64-in total length.

As shown in Figure 3.2, each specimen consisted of A1011 hot-rolled steel sheets on top and bottom with normal-weight concrete infill. Steel sheet grade and gage varied between the specimens (12 gage HSLA grades 50 and 80 and 14 gage HSLA grades 50 and 65). Concrete aggregate was pea gravel with a maximum diameter of 0.375-in. Concrete mix details are provided in Table 3.2 and had day-of-test strengths between 6300- and 6600-psi. Threaded rods provided transverse shear reinforcement. Diameter of the rods varied from 0.138-in to 5/16-in. Holes were drilled in the faceplates 1/32-in larger than the tie bar diameter. Tie bars extended through the holes and were secured with hex nuts on both sides of the faceplate. Tie bar spacing varied: 2-in ($t_{sc}/2$) or 4-in (t_{sc}). All specimens included 0.25-in diameter, 1.125-in long, headed shear stud anchors spaced at 2-in. Specimen names provide basic information about the design: 1-2-3-4(5) where 1 denotes the approximate reinforcement ratio as a percent, 2 is the tie bar spacing in inches, 3 is the grade of the steel sheet, 4 is the nominal concrete strength, and 5 is the specimen number of that type. Multiple beams were constructed of each type to allow for static and blast tests of the same configurations.

Specimens were designed in accordance with provisions of AISC N690, Appendix N9 [5]. Because they were one-third scale and faceplate strength varied, the specimens were not in accordance with every aspect of the specification:

- The smallest plate thickness permitted is 0.25-in but these specimens use 12 and 14 gage sheet (nominally 0.105-in or 0.075-in respectively) and
- Grade 80 steel is not permitted.
- Flexure reinforcement ratio for three specimens exceeded the 5% limit because actual plate thickness of the 12 gage HSLA Grade 50 sheet was slightly larger than nominal thickness.

Because these designs were to be subjected to blast loads, provisions to design RC structural members to withstand blast loads (described in UFC 3-340-02, Chapter 4, [24]) were applied. A few modifications to the procedure were necessary to account for geometric differences between RC and SC (i.e. different equations to calculate moment capacity). The most important change to the design in accordance with AISC N690 was the design for dynamic shear capacity. Blast loads generate high strain rates and moment capacity increases in larger proportion than shear capacity. This results in the need to increase the shear strength of a structural member subjected to dynamic loads. This was achieved by increasing the tie bar diameter to meet dynamic shear demands. Sample calculations are provided in Appendix A. This initial design method formed the basis for the recommended assessment and design methods provided in Chapter 7.

Material properties of the steel sheet used in design calculations were provided by the supplier, MST Steel (see Table 3.3). To obtain full stress-strain curves for use in analysis, uniaxial tension tests were conducted at Bowen Laboratory. Sheet-type standard coupons were machined and tested in accordance with provisions of ASTM E8 [70]. Tension tests were completed using a Baldwin uniaxial testing machine equipped with an Instron ISRS-R-120-BTE system. Results for yield and ultimate strength were within 6.5% of those reported by the supplier (see Table 3.4). Chemical composition of the sheets was reported by the supplier (see Table 3.5).

Figure 3.3 - 3.6 provide engineering stress-strain curves for each of the four varieties of A1011 steel sheet used. Overlaid on each stress-strain curve is the idealized power-law model developed by Varma [71] (Equation (3.4)).

$$s = s_u - (s_u - s_y) \left(\frac{e_u - e}{e_u - e_{sh}} \right)^n \quad (3.4)$$

This was used as input for moment-curvature calculations and FE models. In Equation (3.4), based on engineering stress and strain, s_y is yield stress, s_u and e_u are the ultimate stress and strain, and e_{sh} is the strain at the onset of strain hardening (the end of the yield plateau). Values for variables which provided the best fit for each variety of steel sheet are provided in Table 3.4.

Material properties of the threaded rods used as tie bars were obtained using the Baldwin/Instron system at Bowen Laboratory. The properties of all varieties were $\pm 10\%$ of the average values: $E_s=24000$ -ksi, $F_y=58.3$ -ksi, and $F_u=63.2$ -ksi. Figure 3.7 shows a representative engineering stress-strain curve for the threaded rod overlaid with the power-law model used to idealize the behavior in numerical models. Values for the power-law model variables for the threaded rod are provided in Table 3.4. Material properties and chemical composition of the headed stud anchors were provided by the supplier (see Tables 3.6 and 3.7). All eight of the static specimens were constructed using studs from heat 526630.

Concrete compressive strength was obtained using a Forney F-60C-DFM/I uniaxial testing machine at Bowen Laboratory. This system did not provide stress-strain data; only the maximum force and calculated stress were recorded. Three cylinders (4-in diameter) were tested on the day of each test and average strength reported. Day-of-test strength data ranged between 6300 and 6600 psi as listed in Table 3.1.

3.2.2 Test Setup

Specimens were tested with simple supports. Figure 3.8 depicts the test setup and sensor layout. The unsupported span, L , for all tests was 52-in with a 6-in overhang beyond each support to allow development of the faceplate. All specimens

were tested in four-point bending with an 18-in shear arm. An ENERPAC RCH206 actuator applied load to a W-shape spreader beam which in turn applied the load to the specimen through two 1-in diameter steel rods extending across the width of the specimen. A 2-in steel rod placed on a 1.5-in thick steel plate served as the roller support and a 1-in steel rod between two steel plates machined to fit the rod served as the pin support. 0.5-in thick steel plates, with holes drilled to fit the tie rod ends, were placed between the specimen and supports applying the reaction across a 6-in wide surface the full width of the specimen. Both supports reacted against 2-ft thick concrete reaction blocks. The steel rods and plates used in the supports were cleaned and lubricated with white lithium grease before each test.

The layout and type of sensors was the same for each of the eight tests. Strains were measured using Tokyo Sokki Kenkyujo YEFLA-5-5LT 120 Ω 5-mm strain gages. One strain gage (SG1) was centered at mid-span on the compression (top) face of the specimen. Three strain gages (SG2, SG3, and SG4) were placed at the quarter-points of the span centered on the tension (bottom) face of the specimen. Displacements at quarter points were measured by UniMeasure PA-10-TS cable extension linear position transducers. Rotation at the supports and directly under the two load points were measured by Seika NG2u $\pm 10^\circ$ inclinometers attached at mid-thickness of the specimen.

Each test began using load control with 5-kip load increments until yield was reached. Post yield, displacement control was applied in 0.5-in increments until 3-in of mid-span displacement at which point increments were increased to 1-in. Data from the actuator and all sensors was recorded continuously at 10-second increments using a National Instruments PXI-1052 chassis, PXI-PCI 8331 converter, and two modules: SCXI-1521-B for strain measurement and SCXI-1102 for voltage measurement. At the end of each load or displacement increment concrete cracks were marked and photographs of the specimen were taken.

3.2.3 Static Test Results

Each specimen followed the same progression, as shown for beam 5-2-50(3) in Figure 3.9. Flexure cracks first appeared near the center of the constant moment region (Figure 3.9(a)). Additional flexure cracks appeared throughout the constant moment region and shear spans as load increased (Figure 3.9(b)). Shear cracks first appeared at approximately the same load for each specimen (around 20-kips of total load; 10-kips shear force) corresponding to average concrete shear strength of $2.97\sqrt{f'_c}$. All specimens ultimately failed due to net section rupture of the tension steel plate (Figures 3.9d and e). This rupture occurred suddenly, without audible warning or gradual drop in load, along the line of tie bars under the loading point closest to the pin support.

The fundamental moment-curvature and load-case specific load-displacement behavior of all eight static tests were similar as seen in Figures 3.10(a) and 3.10(b). Elastic behavior was nearly linear until the bottom steel plate yielded. All beams exhibited deformation ductility larger than 7.1 with the largest being 13.1 for 5-2-50-5(3). Table 3.8 provides a summary of experimentally measured flexural rigidity, yield and ultimate strengths, and curvature and deformation ductility values. Statistical evaluation of the behavior of all eight specimens demonstrated that flexural stiffness and yield capacity were close to the expected values calculated from AISC N690s1 N9. Experimental stiffness was calculated at $0.5M_y$. The average ratio of experimental stiffness to calculated stiffness was 0.989 (CV = 0.091). The ratio of yield moment to calculated yield moment was 1.022 (CV = 0.021). All eight specimens had approximately $1/3$ additional capacity beyond yield as shown by the average ratio of experimental ultimate moment to calculated yield moment of 1.333 (CV = 0.016). Results of each test are described in the following sections.

3.2.3.1 3-x-50-5 Specimens

Two configurations built with 14 gage A1011 HSLA Gr50 steel sheet as compression and tension steel were tested: one with minimum tie bar diameter (3-2-50-5) and another with tie bars one size larger (3-2b-50-5). The shear force-displacement responses in Figures 3.11 indicated that behavior was similar for these two specimens. In this figure, shear force was calculated as half of the total applied load from the single actuator. Larger tie bars did not influence stiffness or yield strength but did reduce failure displacement. Larger diameter holes to accommodate larger tie bars resulted in reduced net section and led to earlier fracture of the tension sheet.

Figure 3.12 indicated that fundamental moment-curvature behavior was nearly identical for these two specimens and yield moment was slightly higher than estimated using AISC N690s1 Appendix N9 provisions. In this figure, the moment was calculated as the shear force, V , multiplied by the shear arm, a , and the curvature was calculated from the measured rotations at each load point (CM4 and CM12) and the arc length, $L - 2a$ (16-in), between load points.

3.2.3.1.1. Specimen 3-2-50-5

Figure 3.13 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.14 (S2 and S4 were nearly identical in the elastic region). Unsymmetric displacement began to occur after yield was reached. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Strain profiles for various load and displacement points are shown in Figure 3.15. The neutral axis remained approximately 2.75-in from the bottom of the beam in the elastic region ($1/3^{rd}$ of the depth from the top surface) and shifted up to a location about $1/5^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 3.09-in of mid-span displacement; this is the last point shown in Figure 3.15 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.16. The symmetry of displacement in the elastic region is evident in this figure. CM 5 malfunctioned at just over 4° of rotation.

Photographs of the test progression are provided in Figure 3.17. Figure 3.17(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 20-kips of total load, shear cracks were visible as seen in Figure 3.17(b) corresponding to an average concrete shear strength of $2.93\sqrt{f'_c}$. Shear cracks were most evident in the shear span closest to the pin support (note that Figure 3.17(a) is a photo of the front of the beam and Figure 3.17(b) is of the rear because shear cracks were first evident on the rear face.) Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.17(c).

3.2.3.1.2. Specimen 3-2b-50-5

Figure 3.18 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.19 (S2 and S4 were nearly identical in the elastic region). Unsymmetric displacement began to occur after yield was reached. As the beam

moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Maximum displacement at failure was less than for specimen 3-2-50-5 because the net section was smaller for specimen 3-2b-50-5 ($0.929A_g$ and $0.915A_g$ respectively) because it required larger holes to accommodate larger tie bars. This smaller net section had negligible influence on stiffness or yield strength.

Strain profiles for various load and displacement points are shown in Figure 3.20. The neutral axis remained approximately 2.65-in from the bottom of the beam in the elastic region ($1/3^{rd}$ of the depth from the top surface) and shifted up to a location about $1/5^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 2.02-in of mid-span displacement; this is the last point shown in Figure 3.20 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.21. Symmetry of displacement in the elastic region is evident in this figure. Smaller maximum rotations were observed in this specimen than for 3-2-50-5 because less total deflection was reached when failure occurred.

Photographs of the test progression are provided in Figure 3.22. Figure 3.22(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 20-kips of total load, shear cracks were visible as seen in Figure 3.22(b) corresponding to an average concrete shear strength of $2.97\sqrt{f'_c}$. Shear cracks were most evident in the shear span closest to the pin support. Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.22(c).

3.2.3.2 3-x-65-5 Specimens

Two configurations built with 14 gage A1011 HSLA Gr65 steel sheet as compression and tension steel were tested: one with minimum tie bar diameter (3-2-65-5) and another with tie bars one size larger (3-2b-65-5). The shear force-displacement responses in Figures 3.23 indicated that behavior was similar for these two specimens. In this figure, shear force was calculated as half of the total applied load from the single actuator used in each test. Larger tie bars did not influence yield strength or failure displacement but decreased the initial stiffness by about 10%.

Figure 3.24 indicates that fundamental moment-curvature behavior was nearly identical for these two specimens and yield moment was slightly higher than estimated using AISC N690 provisions. In this figure, moment was calculated as the shear force, V , multiplied by the shear arm, a , and the curvature was calculated from measured rotations at each load point (CM4 and CM12) and the arc length of 16-in between load points.

3.2.3.2.1. Specimen 3-2-65-5

Figure 3.25 indicates that displacement was slightly non-symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) differ slightly. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.26 (S2 and S4 differed slightly in the elastic region). Unsymmetric displacement continued and was amplified after yield was reached. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Strain profiles for various load and displacement points are shown in Figure 3.27. The neutral axis remained approximately 2.90-in from the bottom of the beam in the

elastic region ($1/4^{th}$ of the depth from the top surface) and shifted up to a location about $1/5^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 1.59-in of mid-span displacement; this is the last point shown in Figure 3.27 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.28. Non-symmetry of displacement in the elastic region is evident in this figure as the difference between CM3 and CM4 is larger than the difference between CM12 and CM5. The final negative rotation measured by CM4 is because of the way the specimen failed and the portion of the cracked concrete to which CM4 was affixed. Because this was post failure behavior, it is unimportant data but was included for completeness.

Photographs of the test progression are provided in Figure 3.29. Figure 3.29(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 20-kips of total load, shear cracks were visible as seen in Figure 3.29(b) corresponding to average concrete shear strength of $2.97\sqrt{f'_c}$. Shear cracks were most evident in the shear span closest to the pin support (note that Figure 3.29(a) is a photo of the front of the beam and Figure 3.29(b) is of the rear because shear cracks were first evident on the rear face.) Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.29(c).

3.2.3.2.2. Specimen 3-2b-65-5

Figure 3.30 indicates that, similarly to specimen 3-2-65-6, displacement of 3-2b-65-5 was non-symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) differ slightly. This same conclusion was not supported by measured strains in the tensile plate as shown in Figure 3.31 (S2 and S4 are nearly identical in the elastic region). Unsymmetric displacement was amplified

after yield was reached. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and increased deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Maximum displacement at failure was nearly the same as specimen 3-2-65-5 but the load at maximum displacement was less for specimen 3-2b-65-5. The smaller net section had negligible influence on stiffness or yield strength.

Strain profiles for various load and displacement points are shown in Figure 3.32. The neutral axis remained approximately 3.0-in from the bottom of the beam in the elastic region ($1/4^{th}$ of the depth from the top surface) and shifted up as additional displacement was imposed to a location about $1/5^{th}$ of the depth from the top surface. SG3 exhausted at 1.44-in of mid-span displacement; this is the last point shown in Figure 3.32 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.33. Non-symmetry of displacement in the elastic region is evident in this figure as the difference between CM3 and CM4 is larger than the difference between CM12 and CM5. Similar maximum rotations were observed in this specimen as in 3-2-65-5 with the exception of CM5.

Photographs of the test progression are provided in Figure 3.34. Figure 3.34(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 20-kips of total load, shear cracks were visible as seen in Figure 3.34(b) corresponding to an average concrete shear strength of $2.91\sqrt{f'_c}$. Shear cracks were most evident in the shear span closest to the pin support (note that Figure 3.34(a) is a photo of the front of the beam and Figure 3.34(b) is of the rear because shear cracks were first evident on the rear face). Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension

plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.34(c).

3.2.3.3 5-x-50-5 Specimens

Three configurations built with 12 gage A1011 HSLA Gr50 steel sheet as compression and tension steel were tested: one with tie bars spaced at $t_{sc}/2$ optimized in accordance with AISC N690 (5-2-50-5), another with tie bars one size larger (5-2b-50-5), and a third with tie bars spaced at t_{sc} optimized in accordance with AISC N690 (5-4-50-5). The shear force-displacement responses in Figures 3.35 indicated that the behavior was similar for the three specimens and all exceeded their expected capacity. In this figure, shear force was calculated as half of the total applied load from the single actuator used in each test. Because tie bar spacing for specimen 5-4-50-5 is greater than $t_{sc}/2$ AISC N690s1 Appendix N9 provisions did not permit inclusion of the contribution of shear reinforcement, thus the capacity for 5-4-50-5 is about 70% of that for 5-2-50-5 and 5-2b-50-5. As shown, flexure controlled 5-4-50-5 response as it did for the other two specimens. Specimen 5-2b-50-5 had less stiffness than 5-2-50-5 and 5-4-50-5 had even less. All three had approximately the same yield strength but different maximum strength: 5-2-50-5 had the highest strength, then 5-4-50-5, and 5-2b-50-5 had the lowest maximum strength. Specimen 5-2b-50-5 had the smallest net section area of the three specimens ($0.872A_g$ compared to $0.915A_g$ for specimen 5-2-50-5 and $0.906A_g$ for specimen 5-4-50-5.) Specimens 5-2-50-5 and 5-4-50-5 had similar failure displacement and 5-2b-50-5 had slightly less. Figure 3.36 indicates that the fundamental moment-curvature behavior was similar for these three specimens, and the yield moment was close to that estimated using AISC N690 provisions. In this figure, the moment was calculated as the shear force multiplied by the shear arm of 18-in, and the curvature was calculated from the measured rotations at each load point (CM4 and CM12) and the arc length of 16-in between load points.

3.2.3.3.1. Specimen 5-2-50-5

Figure 3.37 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.38 (S2 and S4 were nearly identical in the elastic region). Unsymmetric displacement began to occur after yield was reached. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Strain profiles for various load and displacement points are shown in Figure 3.39. The neutral axis remained approximately 2.50-in from the bottom of the beam in the elastic region ($2/5^{th}$ of the depth from the top surface) and shifted up to a location about $1/5^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 1.78-in of mid-span displacement; this is the last point shown in Figure 3.39 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.40. Symmetry of displacement in the elastic region is evident in this figure. CM 5 malfunctioned during this test so the data it provided was discarded.

Photographs of the test progression are provided in Figure 3.41. Figure 3.41(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 15-kips of total load, shear cracks were visible as seen in Figure 3.41(b) corresponding to an average concrete shear strength of $2.27\sqrt{f'_c}$. Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the right of the right load point as seen in Figure 3.41(c).

3.2.3.3.2. Specimen 5-2b-50-5

Figure 3.42 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was not supported by measured strains in the tensile plate as shown in Figure 3.43 (S2 and S4 diverged early in the test). Unsymmetric displacement and yielding of the bottom steel plate was amplified after yield was reached. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Maximum displacement at failure was less than for specimen 5-2-50-5. The net section was smaller for specimen 5-2b-50-5 because it required larger holes to accommodate larger tie bars. This smaller net section had negligible influence on yield strength or stiffness.

Strain profiles for various load and displacement points are shown in Figure 3.44. The neutral axis remained approximately 2.60-in from the bottom of the beam in the elastic region ($1/3^{rd}$ of the depth from the top surface) and shifted up to a location about $1/5^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 1.78-in of mid-span displacement; this is the last point shown in Figure 3.44 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.45. Symmetry of displacement in the elastic region is evident in this figure. Similar maximum rotations were observed in this specimen as in 5-2-50-5.

Photographs depicting the progression of the tests are provided in Figure 3.46. Figure 3.46(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 20-kips of total load, shear cracks were visible as seen in Figure 3.46(b) corresponding to an average concrete shear strength

of $2.97\sqrt{f'_c}$. Shear cracks were most evident in the shear span closest to the pin support. Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the right of the right load point as seen in Figure 3.46(c).

3.2.3.3.3. Specimen 5-4-50-5

Figure 3.47 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.48 (S2 and S4 were nearly identical in the elastic region). Unsymmetric displacement began to occur after yield was reached. As the beam moved on the roller support, the left shear arm was reduced which increased the relative moment in the right shear arm thus increasing deflection in that portion. The horizontal spikes in the lower left portion of Figure 3.47 were due to malfunctioning of SP22. The specimen was unloaded after reaching approximately 3-kips total load, connections to SP22 were repaired, and the specimen was reloaded.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure but yield was reached at a smaller strain than observed for the tension plate.

Strain profiles for various load and displacement points are shown in Figure 3.49. The neutral axis remained approximately 2.50-in from the bottom of the beam in the elastic region ($2/5^{th}$ of the depth from the top surface) and shifted up to a location about $1/10^{th}$ of the depth from the top surface as additional displacement was imposed. SG3 exhausted at 1.78-in of mid-span displacement; this is the last point shown in Figure 3.49 rather than the state immediately prior to failure.

End and load point rotations are shown in Figure 3.50. These rotations suggest non-symmetry of displacement in the elastic region as the difference between CM3 and CM4 is larger than that between CM12 and CM5.

Photographs of the test progression are provided in Figure 3.51. Figure 3.51(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 10-kips of total load, shear cracks were visible as seen in Figure 3.51(b) corresponding to an average concrete shear strength of $2.97\sqrt{f'_c}$. Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.51(c).

3.2.3.4 5-2-80-5 Specimen

A single configuration was tested with 12 gage A1011 HSLA Gr80 steel sheet as compression and tension steel and tie bars spaced at $t_{sc}/2$ optimized in accordance with AISC N690s1: (5-2-80-5). The shear force-displacement is shown in Figure 3.52. In this figure, the shear force was calculated as half of the total applied load from the single actuator used in each test. Displacement drifted as load was held constant at 5-kips, 10-kips and 12.5-kips of total load. All displacement sensors measured increasing displacement at constant load suggesting support settlement occurred. This specimen was the first of the eight specimens tested. The specimen was unloaded after 12.5-kips of total load and no more displacement anomalies were noted as the specimen was reloaded. The data in Figure 3.52 was corrected to re-zero displacement prior to reloading. Figure 3.53 shows the fundamental moment-curvature behavior was similar to that estimated using AISC N690 provisions. In this figure, moment was calculated as the shear force multiplied by the shear arm of 18-in and the curvature was calculated from the measured rotations at each load point (CM4 and CM12) and the arc length of 16-in between load points.

Figure 3.54 indicates that displacement was symmetric in the elastic region because values of displacement at symmetric quarter points (SP29 and SP22) were nearly identical. This same conclusion was supported by measured strains in the tensile plate as shown in Figure 3.55 (S2 and S4 were nearly identical in the elastic region). Unsymmetric displacement began to occur after yield. As the beam moved on the roller support, the left shear arm length reduced, which increased the relative moment in the right shear arm and deflection in that portion.

The tensile steel plate first yielded at mid-span and spread toward the supports. The compression steel plate yielded well before failure at a similar strain as the tension plate.

Strain profiles for various load and displacement points are shown in Figure 3.56. The neutral axis remained approximately 2.50-in from the bottom of the beam in the elastic region ($2/5^{th}$ of the depth from the top surface) and shifted up as additional displacement was imposed to a location about $1/4^{th}$ of the depth from the top surface. The final strain profile at 2.5-in of mid-span displacement was the state immediately prior to failure. No strain gage exhausted its capacity during this test.

End and load point rotations are shown in Figure 3.57. The symmetry of displacement in the elastic region is evident in this figure.

Photographs of the test progression are provided in Figure 3.58. Figure 3.58(a) shows that flexure cracks first appeared in the region of constant moment and then in the shear span. At 25-kips of total load, shear cracks were visible as seen in Figure 3.58(b) corresponding to an average concrete shear strength of $2.97\sqrt{f'_c}$. Flexure cracks in the constant moment region extended to within 0.5-in of the compression plate and no concrete crushing was observed. Failure occurred due to net-section fracture of the tension plate at the line of tie bars immediately to the left of the right load point as seen in Figure 3.58(c).

3.3 Normalized Comparison of Static Tests

To compare all tests, the results were normalized. Experimental moment was divided by calculated yield moment and experimental curvature was divided by curvature associated with bottom plate yield. Experimental load (shear force) was divided by the yield load (M_y/a) and experimental mid-span displacement at yield was divided by theoretical beam deflection using the cracked, transformed flexural stiffness, EI_{eff} , calculated using provisions of AISC N690s1. Normalized moment-curvature and load-displacement curves are shown in Figure 3.59.

3.3.1 Normalized Moment-Curvature Response

As Figure 3.59(a) shows, all eight specimens had similar flexural rigidity until $0.80M_y$ at which point the 3-x-50-5 specimens reduced to about 90% of initial rigidity and the other 6 specimens softened more. All began to display rapid loss of stiffness (corresponding to yielding of the bottom steel plate) at approximately M_y . All had similar maximum normalized moment (around $1.33M_y$) but the ductility at which this maximum was realized differed. Specimens built from the most ductile steel (5-x-50-5) achieved maximum moment at larger curvatures than those comprised of less ductile steels. The specimen built from the least ductile steel (5-2-80-5) achieved maximum moment at the smallest curvature. Curvature ductility ranged from 7.9 to 20.4. Results from each specimen are provided in Table 3.8. Specimen ductility was directly related to steel plate material ductility (e_f/e_y , where e_f is strain at failure or rupture) and the ratio of net to gross steel plate area. The reduction from gross steel plate area is due to holes drilled for tie bars.

The moment-curvature relationship for each of these eight specimens can be idealized with a power law model similar to the Ramberg-Osgood model for stress-strain[72]. In this model (Equation (3.5)) the first term accounts for elastic behavior and the second accounts for plastic behavior.

$$\phi(x) = \frac{M(x)}{EI_{eff}} + a \left(\frac{M_y}{EI_{eff}} \right) \left(\frac{M(x)}{M_y} \right)^n \quad (3.5)$$

In this equation, $M(x)$ is the moment along the beam length, M_y was calculated from equilibrium at first yield of the tension plate, and EI_{eff} was calculated from AISC N690s1 Equation A-N9-8. Figure 3.60 graphically depicts the method by which constants a and n were determined. In practical terms, a accounted for softening prior to yield (smaller values of a indicate less softening) and n accounted for curvature ductility (larger values of n indicated higher ductility). The equation to calculate n , shown in the figure, is taken from the description of the Ramberg-Osgood model in [73]. The values of a and n which provided the best fit for the normalized moment curvature diagrams are provided in Table 3.9. Figures 3.61-3.68 show the best fit for each of the eight specimens. There is large variation in the values of a ($\bar{x} = 0.393$, $s = 0.188$, $CV = 0.493$). The power law constant n is close in value to the experimental curvature ductility.

3.3.2 Normalized Load-Displacement Response

Comparison of normalized load-displacement curves in Figure 3.59(b) leads to similar findings as for normalized moment-curvature. All specimens had flexural stiffness close to theoretical until $0.50V_y$. At larger loads, 5-4-50-5 experienced the most rapid loss of stiffness, 5-2-80-5 retained the highest stiffness, and the other six specimens had similar stiffness (about 80% of initial stiffness). This loss of stiffness is due to the reduction in net section caused by tie bar holes in the plates. Maximum load was reached at the displacement related to curvature ductility by an average factor of 0.73 ($CV=20\%$). Displacement ductility is provided in Table 3.8.

Load-displacement relationships could be idealized with a power law model similar to Equation (3.5) for moment-curvature. The usefulness of such a model is limited, however, because load-displacement curves were generated for specific load conditions and geometry. The power law model for moment-curvature is more useful as it provides an idealization of fundamental behavior that can then be used to estimate load-displacement relationships for a variety of load conditions and geometries. To

estimate the load-displacement relationship using the moment-area method and the power law model for moment-curvature, the displacement at mid-span for a given load can be calculated using Equation (3.6).

$$\delta_{mid} = \int_0^{\frac{L}{2}} x \cdot \phi(x) dx \quad (3.6)$$

Because $\phi(x)$ includes the function $M(x)$ raised to the power n , a closed form solution for δ_{mid} does not exist but with all calculated inputs for a specific case, the solution can be determined.

3.4 Numerical Modeling of Statically Loaded SC Wall Sections

Two benchmarked models were developed: (1) a fiber model and (2) an FE model. This section provides details of both.

3.4.1 Fiber Model

Moment-curvature relationships for each specimen using a fiber model were benchmarked to experimental results. Equilibrium was computed using strain compatibility assuming perfect composite action (no slip) between the steel plates and concrete infill. To account for slip, an artificial “slip strain” was calculated as described below (see Equation (3.13) and Figure 3.69). Concrete stress was calculated using the Hognestad parabola (Equation (3.7)) relating concrete stress to strain [74].

$$f_c = f'_c \left(\frac{2\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o} \right)^2 \right) \quad (3.7)$$

The value ε_o was computed by evaluating Equation (3.7) at $0.5f'_c$ and a corresponding strain value of $0.5f'_c/E_c$. E_c was calculated from Equation (3.8). The resulting equation for ε_o is Equation (3.9).

$$E_c = 57,000\sqrt{f'_c} \quad (3.8)$$

$$\varepsilon_o = \frac{f'_c}{E_c} \left(1 + \sqrt{\frac{1}{2}} \right) \quad (3.9)$$

Integrating the concrete stress distribution over the depth of the concrete compressive section led to Equation (3.10) for total concrete compressive force, C_C . The centroid of the concrete stress distribution is located a distance x_C from the neutral axis and calculated from Equation (3.11) and shown in Figure 3.69.

$$C_C = f'_c \cdot b \frac{\phi}{\varepsilon_o} \cdot x_{NA}^2 \left(1 - \frac{\phi x_{NA}}{3\varepsilon_o} \right) \quad (3.10)$$

$$x_C = x_{NA} \left(\frac{8\varepsilon_o - 3\phi x_{NA}}{12\varepsilon_o - 4\phi x_{NA}} \right) \quad (3.11)$$

The material power-law model defined by Equation (3.4) was used for steel stress-strain relationships. Because the thickness of the steel plates was small compared to total depth, constant strain was assumed through the thickness of the steel plates and the resulting force was assumed to act at mid-thickness (i.e. a single fiber was used for each steel plate). The compressive and tensile forces in the steel plates (C_S and T_S respectively) were computed by multiplying the stress at each plate mid-thickness by the area of the plate. To account for section loss due to holes drilled in the plate to accommodate the tie bars, the total width of holes in a section was subtracted from the specimen width, b .

Depth from the top of the section to the neutral axis, x_{NA} was computed by equating total compressive force, $C_C + C_S$, to tensile force T_S . Concrete tensile stresses were neglected. Because concrete and steel material behavior was modeled as nonlinear, solving for x_{NA} was an iterative process and Microsoft Excel was used to perform the calculations for concrete compressive strains up to a maximum strain of 0.004-in/in.

The resulting moment-curvature relationship was used to estimate load-displacement behavior of the specimens by calculating the moment of the area beneath the curvature diagram (between one support and mid-span) about mid-span. Figure 3.70 depicts this process. Because moment-curvature is nonlinear, the beam was discretized along its length, curvature associated with the moment in each segment was determined from the moment-curvature relationship, area under the curvature over the length of each segment was computed and moments of each incremental

area about the beam end were summed to estimate the deflection at mid-span (see Equation (3.12)). For completeness, shear deformation was added to the deflection calculated using the moment-area method.

$$\Delta_{mid} = \sum \phi_i x_i dx \quad (3.12)$$

To account for partial composite action, slip strain was calculated for each equilibrium state along the moment-curvature curve using Equation (3.13). This assumes that the shear between the beam end to the point of maximum moment was carried equally by all shear studs within the shear arm. Effective shear area of the studs was used in this calculation (thus the $5/6$ term in the denominator of Equation (3.13)). Because tie bars were not fixed to the steel plate they were not included in this calculation. Curvature was modified to include ε_{slip} using Equation (3.14).

$$\varepsilon_{slip} = \frac{M/a}{5/6 A_{studs} \cdot G_s} \quad (3.13)$$

$$\phi = \frac{-\varepsilon_{St} + \varepsilon_{slip} + \varepsilon_{Sb}}{t_{sc} - t_p} \quad (3.14)$$

Figures 3.71 - 3.78 compare results from moment-curvature and load-displacement using the methods described above. In all cases, flexural rigidity was close to experimental results. This same was true for flexural stiffness of the load-displacement diagrams. Load-displacement calculated from moment-curvature maintains a higher stiffness as loading progresses than observed experimentally and does not capture softening at yield. The fiber model indicated a more distinct yield point than shown experimentally.

For most of the tests, the load at which yield occurred was close to the calculated value but ultimate load was higher in the tests than calculations. In only one case (5-2b-50-5) did the calculated curve suggest higher strength than observed experimentally. The difference was less than 10%.

3.4.2 FE Model Overview

All eight static tests were modeled numerically using LS-DYNA to serve as benchmarked models for which the loading would later be changed to develop static resistance functions for uniform loads. An overview of the model is shown in Figure 3.79 and a list of all keywords used in the model is provided in Appendix B.

Material models available in the LS-DYNA library were used. Winfrith concrete model (MAT_084/085) was used for the concrete infill. The piecewise linear plasticity material model (MAT_024) was used for all steel components. Supports and load application components were modeled with rigid (MAT_020) solid elements. All material input values or equations used to compute properties are provided in Table 3.10.

Reduced integration solid elements (SOLID ELFORM 1) were used for the concrete infill. Concrete mesh size was approximately 0.50-in (7 elements through the specimen thickness). Reduced integration elements are known to generate spurious hourglass energy, especially under large deformation, and require strain stabilization. For this purpose, a method developed by Belytschko and Bindeman [75] and integrated into LS-DYNA as hourglass control type (IHQ) 6 was used with an hourglass coefficient (QM) of 0.10. A sensitivity study to assess the influence IHQ and QM on numerical results is described in Section 3.4.5. Sensitivity of numerical results to mesh size is described in Section 3.4.6.

Fully integrated solid elements developed specifically for elements with poor aspect ratios (SOLID ELFORM -1) were used for the steel sheets. Mesh size was 0.50-in with a single element through the thickness. Hughes-Liu with cross section integration elements (BEAM ELFORM 1) were used for tie bars and shear studs. Tie bars consisted of four 1-in beam elements. Shear studs were modeled with a single element and were connected to the steel plate using zero-length discrete beam elements which modelled the load-slip behavior of these components using an elastic spring discrete beam material model (MAT_074). Tie bar ends shared nodes with the steel plate.

Tie bars and studs were mathematically embedded in the concrete using a penalty coupling mechanism (CONSTRAINED_LAGRANGE_IN_SOLID) assuming perfect

bond. Contact between steel faceplates and concrete infill was defined using a penalty mechanism (AUTOMATIC_SURFACE_TO_SURFACE) and included friction. Contact between the specimen and supports and load application was modeled with the same penalty mechanism. Displacement was applied to the specimen by assigning a constant velocity to the load applicators. Strain rate effects were not included in the material model and the displacement was applied at 0.50-in/sec. Mass scaling was applied by setting the minimum time step in the *CONTROL_TIMESTEP card. The minimum time step was computed based on the wave speed of concrete and element effective length: results did not differ from the model without mass scaling but the time required to run the model was reduced.

3.4.3 Material Modeling Details

The Winfrith concrete model (MAT_084/085) was developed in response to the nuclear industry's need to model accidental impact and blast loads on reinforced concrete structures and was validated for a variety of impact and blast tests [76]. It is a smeared crack model that accounts for tension softening due to crack opening. The measured day-of-test concrete compressive strengths as reported in Section 3.2.1 were used in the models. The Winfrith concrete model is based on the compressive strength of a cube rather than a cylinder; the input property for compressive strength was 25% larger than that obtained from a cylinder test [77]. The rest of the input parameters were estimated using equations as listed in Table 3.10. Tensile strength of concrete in an SC wall is about half that of an RC wall because there are no reinforcing bars to limit shrinkage cracking thus f_t differs from that typically used for RC structures [78]: $3.0\sqrt{f'_c}$ was used in these models because it corresponds to the average stress at the onset of shear cracking in the experiments as described in Section 3.2.3.

Strain rate effects were not included in these models so the tension softening (post-crack) behavior was linear as shown in Figure 3.80. The crack width, w , at

which tensile strength is zero was computed using Equation (3.15) in which G_f and f_t are the fracture energy and tensile strength, respectively computed as shown in Table 3.10.

$$w = \frac{2G_f}{f_t} \quad (3.15)$$

A piecewise linear material model (MAT_024) was used for steel components. Stress-strain curves shown in Figures 3.3 - 3.7 were input for the steel sheet and tie rods although the input curves were converted from engineering stress and engineering strain to true stress and plastic strain as required by the material model. A stress-strain curve for the shear studs was not available, so typical Grade 60 steel properties were input. The failure strain at which tie bar elements eroded was set at 0.05 - slightly less than the measured failure strain. This was selected as a conservative value. No failure strain was defined for stud elements. No failure strain was defined for the steel plate elements in order to estimate at what equivalent plastic strain fracture of the tensile steel plates occurred by determining the elemental strain at displacement corresponding to experimental failure.

Headed stud anchors can be modeled accurately and efficiently using beam and connector elements to account for the interfacial (push-out) force-slip displacement relationship [29]. The stud was modeled with a single beam element and MAT_024. The stud beam element was embedded in the concrete solid elements, and connected to the steel plate with a zero-length discrete beam element. The idealized force-slip displacement relationship developed by Olgaard et al. [79], confirmed experimentally by Anderson & Meinheit [80] and Shim et al. [81], and implemented analytically by Zhang et al. [29] was used as input to MAT_074 for these zero-length discrete beam elements. As shown by Zhang et al., this modeling approach accounts for force-slip displacement interaction between the steel plates and concrete infill of SC walls and interaction between the embedded stud and surrounding concrete. Equations to characterize this load-slip behavior are provided in Table 3.10.

3.4.4 Modeling Results

Load-displacement and moment-curvature from the numerical model compared well to experimental results as shown in Figures 3.71-3.78. Unlike the fiber model, the FE numerical model captures decreasing stiffness in the elastic region and softening at yield. In three cases (3-2-50-5, 3-2b-50-5, and 5-2-80-5) the numerical model suggested lower post-yield strength than observed experimentally (within 10%). In one case the numerical model suggested higher yield and post-yield strength than was experimentally observed (5-2b-50-5, the same model for which the fiber model suggested higher strength). For the remaining four cases (3-2-65-5, 3-2b-65-5, 5-2-50-5, and 5-4-50-5) numerical model results were nearly identical to experimental results.

Experimentally observed crack patterns were compared to those obtained from numerical analysis in Figures 3.71 - 3.78. The experimental crack maps were generated from photographs taken at the displacement step immediately prior to tension plate fracture. The numerical crack patterns were taken at the same level of displacement. These images provide two outputs from the Winfrith concrete model within the FE results: (1) the lines denote crack locations and (2) the fringe colors denote the level of crack damage output variable on a scale of 0 to 3 (0, red, is uncracked; 3, blue, is fully cracked). As observed, the FE model results in a similar crack pattern for all specimens.

3.4.5 Sensitivity to Hourglass Control Choices

To investigate sensitivity of results to hourglass type (IHQ) and coefficient (QM), two studies were completed. The model for 3-2b-65-5 was modified for these studies. Everything within the keyword file remained unchanged for these additional models except for values of the input parameters QM and IHQ in the keywords *CONTROLHOURGLASS and *HOURGLASS.

Results from four models investigating QM are shown in Figure 3.81 with experimental results (3-2b-65-5) and numerical results from the baseline model (QM

$= 0.10$). This figure shows the influence of changing QM while keeping IHQ the same (type 6). From the moment-curvature relationship in Figure 3.81(a), QM had negligible influence on flexural rigidity but did influence post-yield behavior. An increase in QM resulted in a small increase in post-yield strength. The influence of QM was more evident on the load-displacement relationship in Figure 3.81(b). As QM increased from 0.01 to 0.10, the flexural stiffness increased but additional increases in QM had negligible influence on stiffness. Similar to the observation for moment-curvature, as QM increased the post-yield strength also increased a small amount. The influence on hourglass energy is shown in Figure 3.82. The largest influence of QM on hourglass energy was in the elastic region but no clear trend was evident. These results support the recommendation that $QM = 0.10$ is appropriate for most cases [59].

Erhart [82] recommends IHQ 6 for most situations when reduced integration solid elements are employed. Results from six models investigating IHQ are shown with the experimental results (3-2b-65-5) and numerical results from the baseline model (IHQ = 6) in Figure 3.83. This figure shows the influence of changing IHQ while keeping QM constant (0.10). Results for moment-curvature and load-displacement indicated that IHQ types 1, 2, and 3 had little effect on controlling hourglass energy compared to the case with no hourglass control and led to inaccurate results. Types 4, 5, and 6 provided required control of hourglass effects and results from these three were similar. Hourglass energy was smallest for types 4 and 5 as seen in Figure 3.84. This energy increased for types 4, 5, and 6 after yield of the steel plates. Hourglass energy for the models with type 1, 2, and 3 exceeded 50% of the total energy at very low load making these unsuitable. From these results, types 4, 5, or 6 were acceptable for the models in this study.

3.4.6 Sensitivity to Mesh Size

Mesh size for the benchmarked models was chosen for practical reasons. To allow for co-location of tie bar and shear stud end nodes with nodes on the steel plates a uniform mesh had to be 0.5-in. Because thickness of the plates was much less than the other dimensions of the steel elements, only one element through the thickness was chosen and a formulation developed for elements with poor aspect ratios was selected. There needed to be at least four elements through the concrete thickness in order to reasonably capture the effect of bending: 0.5-in mesh was chosen for the concrete to match the steel mesh and provide seven elements through the thickness.

To investigate the influence of changing the mesh size, five additional models were analyzed. All other input parameters for these models remained identical to that for 3-2b-65-5. The results for moment-curvature and load-displacement for each of these variations are shown with experimental results (3-2b-65-5) and the baseline model (7 c; 1 s) in Figure 3.85. The names of each model provide the number of elements through the concrete (c) and steel (s).

Models with four or seven elements through the concrete thickness had similar moment-curvature response (Figure 3.85(a)). As the number of elements through the steel thickness increased, post yield strength was less than the model with a single steel element through the thickness. The model with four concrete elements through the thickness (4 c; 1 s) did not exhibit loss in flexural stiffness which was observed experimentally and captured by the models with seven elements through the concrete thickness (Figure 3.85(b)). As the number of concrete elements through the thickness increased to 14 and 28, the flexural rigidity and stiffness decreased as did the strength. Both of these provided results that were inaccurate. For accuracy and computational efficiency, seven reduced integration constant stress elements (ELFORM 1) through the concrete thickness and one full integration element for elements with poor aspect ratios (ELFORM -1) through the steel thickness was reasonable.

3.5 Numerical Models of Uniformly Loaded SC Wall Sections

As discussed at the beginning of this chapter, the primary purpose of static experiments and benchmarked numerical models was to develop resistance functions for specimens under uniform pressure loading. Benchmark numerical models were altered by changing the loading to a uniform pressure across the face between supports.

3.5.1 Fiber Model of SC Wall Section with Uniform Load

The fiber model as described in Section 3.4.1 was modified for uniform load. Because the load case was different, the way in which the slip strain was calculated was different. Equation (3.13) was modified to Equation (3.16) which accounts for the shear due to a uniform load and assumes this is resisted evenly by each shear stud between the support and the point of maximum moment - mid-span for the uniform loading case.

$$\varepsilon_{slip} = \frac{2M/L}{5/6 A_{studs} \cdot G_s} \quad (3.16)$$

Using the moment-area method to compute deflection at mid-span, the revised moment-curvature relationship for each specimen was used to calculate the pressure-deflection relationship as shown in Figure 3.86. Load-displacement curves generated from the moment-curvature relationships are shown as dashed lines in Figures 3.87 to 3.94 and are similar to those obtained from FE models. Both methods estimate nearly identical initial stiffness and yield point. For all eight cases, the FE models show reduction in stiffness before yield. This softening is due to spreading of flexure cracks which first appear at mid-span and appear closer to the supports as the load increases. Both methods suggest that resistance functions of simply supported one-way SC walls subject to uniform load are approximately linearly until yield with reduced linear post-yield stiffness. A bi-linear resistance function is a realistic approximation.

3.5.2 FE Model of SC Wall Section With Uniform Load

As shown in Figure 3.95, no pressure was applied to the overhang beyond the centerline of the supports to prevent influence of negative moment over a continuous support. Failure strain (FS) for the steel faceplates was defined as the maximum effective plastic strain which, when exceeded, resulted in removal of the element from the mesh. This value was reduced in the FE models of uniform load to predict when steel plate rupture would occur. From benchmarked static test model results, maximum effective plastic strain, $\varepsilon_{p,max}$, in the bottom steel plate at the experimentally observed failure displacement value was input as FS for the uniform load FE models. These strains for each model are provided in Table 3.11. The values of FS were related to the steel plate material ductility and the net section reduction of the plates. All other aspects of the uniform load models remained identical to the benchmarked models.

Pressure-displacement and moment-curvature (at mid-span) results for each of the test specimens subjected to uniform pressure are shown as the solid lines in Figures 3.87 to 3.94. These are the experimentally developed static resistance functions, $R(y)$.

3.6 Idealized Model of SC Wall Static Resistance Function

3.6.1 Continuous Static Resistance Function

The method described in Section 3.4.1 could be used to calculate theoretical pressure-displacement relationships for SC wall sections using the idealized power-law model for moment-curvature developed in Section 3.3. The solution could be completed analytically rather than numerically (discretizing the beam) because the equation for $\phi(x)$ is continuous for the case of a uniformly loaded beam and displacement at mid-span can be calculated by integrating of this function (Equation (3.6)).

3.6.2 Bi-Linear Static Resistance Function

It has been shown that the post-yield portion of the resistance function of one-way, simply-supported, SC wall sections exhibits strain hardening. Thus, a bi-linear function with strain hardening (Figure 3.96) is a realistic approximation of the resistance function. This section describes a bi-linear model computed from section properties.

The yield moment, M_y , can be computed from section equilibrium setting the strain in the tension steel plate equal to the yield strain of the material or from design equations such as Equation (3.17) (AISC N690s1 Equation A-N9-18[5]). The yield resistance, R_y , is calculated from M_y (see Equation (3.18)).

$$M_y = 0.9F_y A_{st} t_{sc} \quad (3.17)$$

$$R_y = \frac{8M_y}{L} \quad (3.18)$$

The initial stiffness, k_1 (Equation (3.19)), can be calculated from the cracked transformed flexural stiffness, EI_{eff} , using Equation (3.20) (AISC N690s1 Equation (A-N9-8) [5]).

$$k_1 = \frac{384EI_{eff}}{5L^3} \quad (3.19)$$

$$EI_{eff} = E_s I_s + c_2 E_c I_c \quad (3.20)$$

$$I_s = \frac{bt_p(t_{sc} - t_p)^2}{2} \quad (3.21)$$

$$I_c = \frac{bt_c^3}{12} \quad (3.22)$$

$$c_2 = 0.48 \frac{2t_p}{t_{sc}} \frac{E_s}{E_c} + 0.10 \quad (3.23)$$

As described in Section 3.3, flexural rigidity matched well between theory and the static experiments but the experimental flexural stiffness was less than calculated. The ratio between the experimental (FE models in the case of uniform load) and the calculated stiffness using Equation (3.19) are provided in Table 3.12. The initial stiffness was close to theoretical up to approximately half of the yield load. After this point, the stiffness reduced in proportion to the reduction net section.

For a conservative solution, perfect plasticity could be assumed. From the experimental results, however, there is post-yield stiffness due to strain hardening of the

steel plates. The ratio between post-yield and initial stiffness is provided for each specimen in Table 3.12. The post-yield stiffness is larger for higher reinforcement ratios: for k_2 , it is recommended to use 2.0% of k_1 for beams with 3% reinforcement and 2.5% of k_1 for beams with 5% reinforcement.

3.7 Chapter Summary

In this chapter, results from four-point bending quasi-static tests of eight configurations of SC wall sections were described. The primary purpose of these tests was to benchmark FE models which were then subjected to uniform pressure loading in order to determine the static resistance function for each. The static resistance function was required to complete SDOF dynamic analysis to estimate specimen response to dynamic loads.

Fundamental behavior was confirmed through comparison of experimental results to those from FE analysis, a fiber model, and equations from current specifications. These comparisons highlighted the fact that behavior of these small-scale specimens follows the same fundamental mechanics as full-scale specimens. Another important finding was the influence of slip on flexural stiffness and post-yield behavior. This knowledge was used to generate models to idealize the static resistance function of a one-way SC wall: a power-law fit of the moment-curvature relationship or a bi-linear approximation including post-yield stiffness.

The following two chapters report results from blast experiments of these SC wall designs and develop benchmarked numerical models (FE and SDOF) based on the numerical models developed in this chapter.

Table 3.1. Details of Specimen Designs

Panel	t_p (in)	ρ (%)	Sheet	s/t_p	d_s/t_p	d_t (in)	S/t_{sc}	ρ_t (%)	$\frac{M_n}{V_n t_{sc}}$	f'_c (ksi)
3-2-50-5	0.079	4.0	14ga, HSLA 50	25.3	3.16	0.1380	0.50	0.37	3.24	6.5
3-2b-50-5	0.079	4.0	14ga, HSLA 50	25.3	3.16	0.1640	0.50	0.53	2.58	6.3
3-2-65-5	0.069	3.5	14ga, HSLA 65	29.0	3.62	0.1380	0.50	0.37	4.10	6.3
3-2b-65-5	0.069	3.5	14ga, HSLA 65	29.0	3.62	0.1640	0.50	0.53	3.26	6.6
5-4-50-5	0.112	5.6	12ga, HSLA 50	17.9	2.23	0.3125	1.00	0.48	5.71	6.4
5-2-50-5	0.112	5.6	12ga, HSLA 50	17.9	2.23	0.1640	0.50	0.53	3.68	6.3
5-2b-50-5	0.112	5.6	12ga, HSLA 50	17.9	2.23	0.2500	0.50	1.23	2.09	6.5
5-2-80-5	0.099	5.0	12ga, HSLA 80	20.2	2.53	0.2500	0.50	1.23	3.31	6.4

Table 3.2. Concrete Mix Design for Static Specimens (As Received from Supplier; 6 CY order)

Component	Type	Total Amount (lb)	Amount per CY (lb)	% Moisture	Water Adjust. (lb)
Gravel	Pea	11220	1870	1.5	167
Sand	SAND-23	9020	1503	3.9	350
Cement	Type I	3390	565		
Water		944	157		
Admixtures	GLEN 3030	68 oz	11 oz		
Water (added on site)					0
Total Water (lb)					1460
Water : Cement Ratio					0.43

Table 3.3. A1011 Steel Sheet Material Properties (Reported by Supplier)

Gage	Grade	Heat	F_y (ksi)	F_u (ksi)	ϵ_f (%)
12	HSLA 50	842X36160	58.1	66.5	31.5
12	HSLA 80	0130734	91.7	97.8	18.9
14	HSLA 50	177875	55.6	67.9	29.6
14	HSLA 65	831J12710	72.7	79.8	25.4

Table 3.4. Power Law Variables Providing Best Fit Steel Materials

Material	E_s (ksi)	s_y (ksi)	s_u (ksi)	e_{sh} (in/in)	e_u (in/in)	n
A1011 HSLA 50, 12 ga	29000	57.7	68.4	0.0139	0.183	3.6
A1011 HSLA 80, 12 ga	28000	86.6	102.1	0.0105	0.131	2.8
A1011 HSLA 50, 14 ga	26000	57.9	64.8	0.0267	0.169	3.6
A1011 HSLA 65, 14 ga	27000	68.0	84.4	0.0108	0.159	3.2
Threaded Tie Bars	21000	59.5	66.0	0.0028	0.091	7.0

Table 3.5. A1011 Steel Sheet Chemical Composition (Reported by Supplier)

	12ga, HSLA 50	12ga, HSLA 80	14ga, HSLA 50	14ga, HSLA 65
	842X36160	0130734	177875	831J12710
C	0.0600	0.0600	0.0500	0.0700
Mn	0.3900	1.1700	0.5700	0.8700
P	0.0090	0.0120	0.0120	0.0130
S	0.0060	0.0020	0.0050	0.0040
Si	0.0490	0.0500	0.0200	0.0550
Al	0.0240	0.0950	0.0300	0.0360
Cb	0.0170	-	-	0.0370
Ni	0.0200	0.0200	0.0800	0.0100
Cr	0.0300	0.0300	0.0500	0.0300
Cu	0.0300	0.0400	0.1100	0.0140
Nb	-	0.0880	0.0090	-
Mo	0.0060	-	0.0100	0.0060
Sn	-	0.0020	-	-
V	0.0010	0.0050	-	0.0010
Ca	-	0.0045	0.0010	-
Ti	0.0020	-	0.0010	0.0020
N	0.0060	0.0081	0.0060	0.0050
B	0.0002	-	0.0001	0.0002

Table 3.6. C1010 Headed Stud Anchor Material Properties (Reported by Supplier)

Heat	F_y (ksi)	F_u (ksi)	Reduction of Area (%)	ϵ_{4D} (%)	ϵ_{5D} (%)
526630	61.6	76.1	71.0	20.0	16.0
10324140	81.1	81.7	72.0	28.0	25.0

Table 3.7. C1010 Headed Shear Stud Chemical Composition (Reported by Supplier)

	526630	10324140
C	0.090	0.090
Mn	0.380	0.470
P	0.007	0.009
S	0.014	0.013

Table 3.8. Static Test Experimental Results Summary

Panel	$\frac{EI_{test}}{EI_{N690}}$	$\frac{M_{y,test}}{M_{y,N690}}$	$\frac{M_{u,test}}{M_{y,N690}}$	Failure Type	μ_{Δ}	μ_{ϕ}
3-2-50-5(3)	1.001	1.040	1.351	Flexure	11.2	16.3
3-2b-50-5(3)	1.096	1.057	1.371	Flexure	8.6	14.2
3-2-65-5(2)	1.124	1.011	1.330	Flexure	8.2	7.9
3-2b-65-5(2)	0.985	1.026	1.315	Flexure	7.9	10.6
5-4-50-5(3)	0.878	1.003	1.318	Flexure	10.7	16.5
5-2-50-5(3)	0.908	1.009	1.318	Flexure	13.1	20.4
5-2b-50-5(2)	0.998	0.994	1.343	Flexure	10.5	16.7
5-2-80-5(2)	0.924	1.034	1.317	Flexure	7.1	8.6
$\bar{x}$	0.989	1.022	1.333		9.7	13.9
σ	0.087	0.021	0.020		2.0	4.4
CV	0.091	0.021	0.016		0.217	0.329

Table 3.9. Moment-Curvature Power Law Best Fit Constants

Panel	a	n
3-2-50-5(3)	0.119	16.5
3-2b-50-5(3)	0.114	15.4
3-2-65-5(2)	0.539	10.5
3-2b-65-5(2)	0.421	12.7
5-4-50-5(3)	0.628	13.4
5-2-50-5(3)	0.378	15.4
5-2b-50-5(2)	0.518	13.0
5-2-80-5(2)	0.425	11.8

Table 3.10. Material Input for LS-DYNA Models of Static Experiments

<i>Input for MAT_084/085 (Concrete)</i>	<i>Value or Equation Used</i>
Mass Density, RO ($lbf \cdot sec^2/in$)	2.36×10^{-4}
Initial Tangent Modulus, TM (psi)	$57000\sqrt{f'_c}$ [83]
Poisson's Ratio, PR	0.15
Uniaxial Compressive Strength, UCS (psi)	$1.25f'_c$
Uniaxial Tensile Strength, UTS (psi)	$3.0\sqrt{f'_c}$
Crack Width at Zero Tensile Strength, FE (in)	$2G_f/UTS$, $G_f = 0.948lb \cdot in/in^2$
Aggregate Radius, ASIZE (in)	0.1875
Rate Effects (0=ON, 1=OFF)	1
Unit Conversion, CONM	-1 = $lbf \cdot sec^2/in, in, sec$
<i>Input for MAT_024 (Steel Plate, Tie Bars, and Shear Studs)</i>	
Mass Density, RO ($lbf \cdot sec^2/in$)	7.33×10^{-4}
Young's Modulus, E (psi)	E_s from Table 3.4
Poisson's Ratio, PR	0.30
Plastic Strain at Failure, FAIL	0.15 (plates), 0.05 (ties)
Load Curve, LCSS ($\sigma_{true} - \varepsilon_{plastic}$)	see Table 3.4
<i>Input for MAT_074 (Connector Elements for Tie Bars and Shear Studs)</i>	
Mass Density, RO ($lbf \cdot sec^2/in$)	7.33×10^{-4}
Displacement at Failure, TDF	$\Delta_{max} = (0.48 - 0.029 \cdot f'_c)d_s$ [81]
Load Curve, FLCID (Load-Slip)	$Q = Q_n(1 - e^{-18\Delta})^{2/5}$ [79] $Q_n = 0.65A_sF_u$ [34]
<i>Input for MAT_020 (Supports and Load Plates)</i>	
Mass Density, RO ($lbf \cdot sec^2/in$)	7.33×10^{-4}
Young's Modulus, E (psi)	29×10^6
Poisson's Ratio, PR	0.30
Constraints, CMO	1.0
Translation Constraints	4, 7
Rotation Constraints	7

Table 3.11. Maximum Plastic Strain in Bottom Steel Plate of FE Models at Experimentally Observed Failure Displacement

Specimen	$\varepsilon_{p,max}$
3-2-50-5	0.063
3-2b-50-5	0.055
3-2-65-5	0.045
3-2b-65-5	0.046
5-2-50-5	0.080
5-2b-50-5	0.078
5-4-50-5	0.075
5-2-80-5	0.068

Table 3.12. Ratios of Elastic and Strain Hardening Slope for Uniform Load Case

Specimen	$k_{1,FEM}/k_{1,calc}$	$k_{2,FEM}/k_{1,FEM}$
3-2-50-5	0.929	0.0161
3-2b-50-5	0.894	0.0174
3-2-65-5	0.895	0.0246
3-2b-65-5	0.913	0.0217
5-2-50-5	0.745	0.0277
5-2b-50-5	0.817	0.0218
5-4-50-5	0.862	0.0204
5-2-80-5	0.734	0.0363

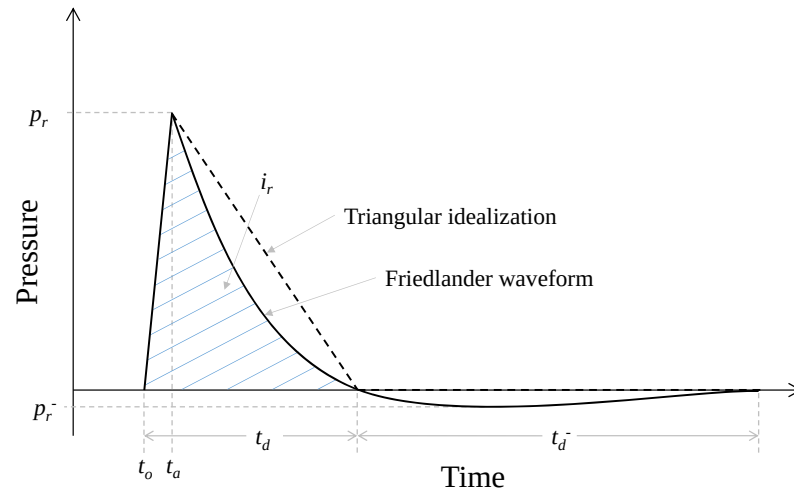


Figure 3.1. Pressure Time-History Idealizations Used in this Study

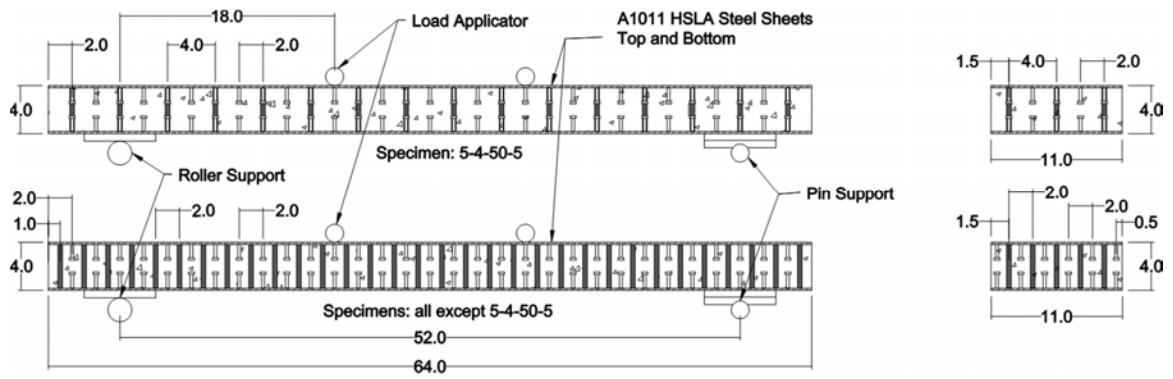


Figure 3.2. Design Drawing of Test Specimens (dimensions in inches)

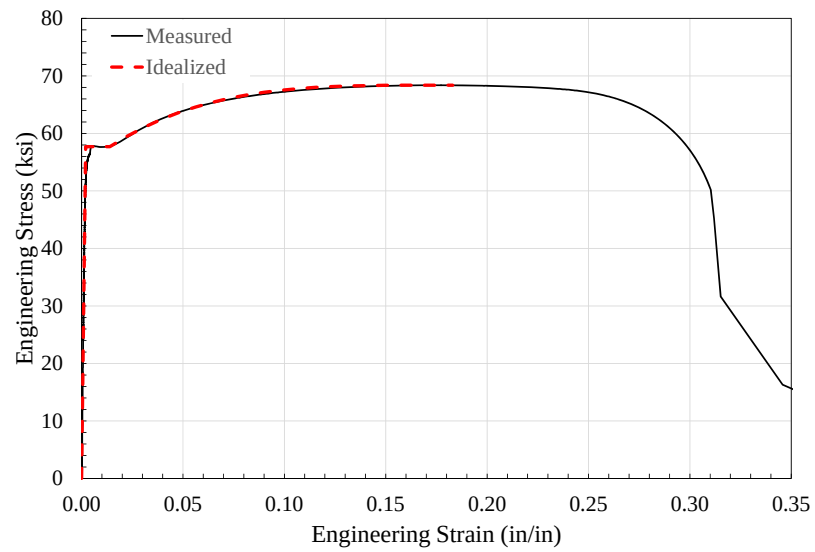


Figure 3.3. A1011 HLSA 50, 12 Gage Steel Sheet Tensile Stress-Strain Curves

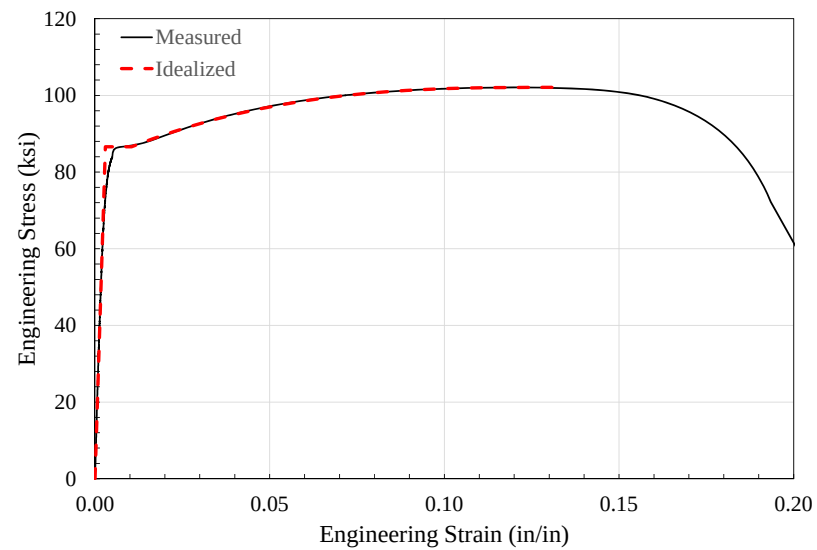


Figure 3.4. A1011 HLSA 80, 12 Gage Steel Sheet Tensile Stress-Strain Curves

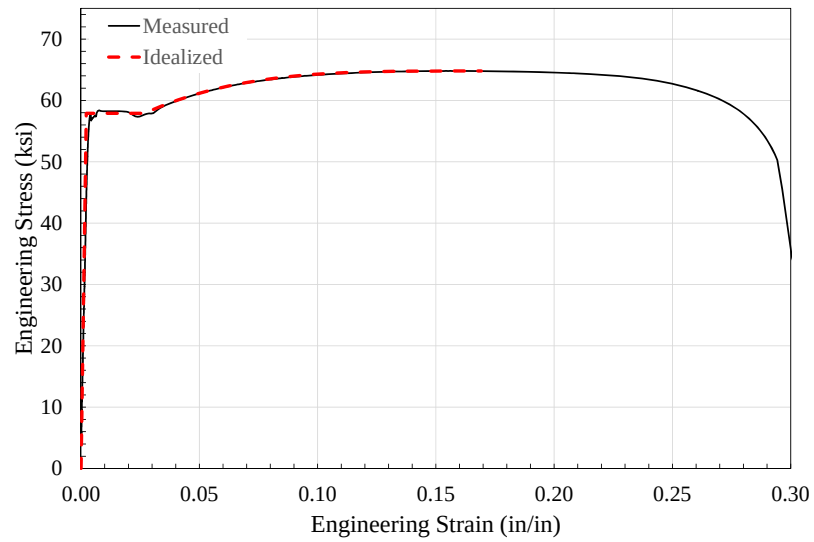


Figure 3.5. A1011 HLSA 50, 14 Gage Steel Sheet Tensile Stress-Strain Curves

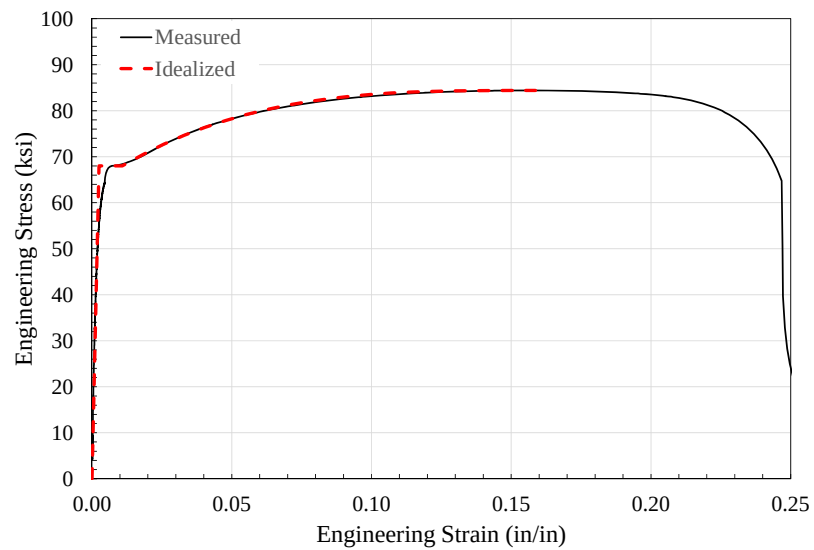


Figure 3.6. A1011 HLSA 65, 14 Gage Steel Sheet Tensile Stress-Strain Curves

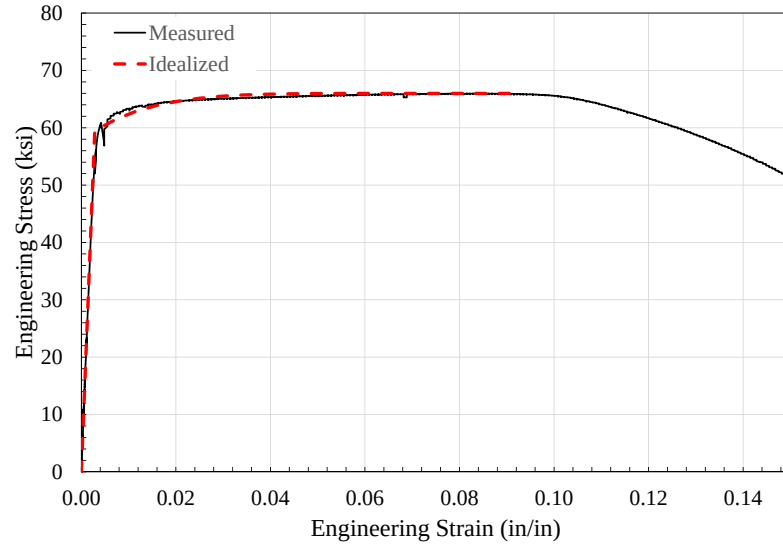


Figure 3.7. Threaded Rod Tensile Stress-Strain

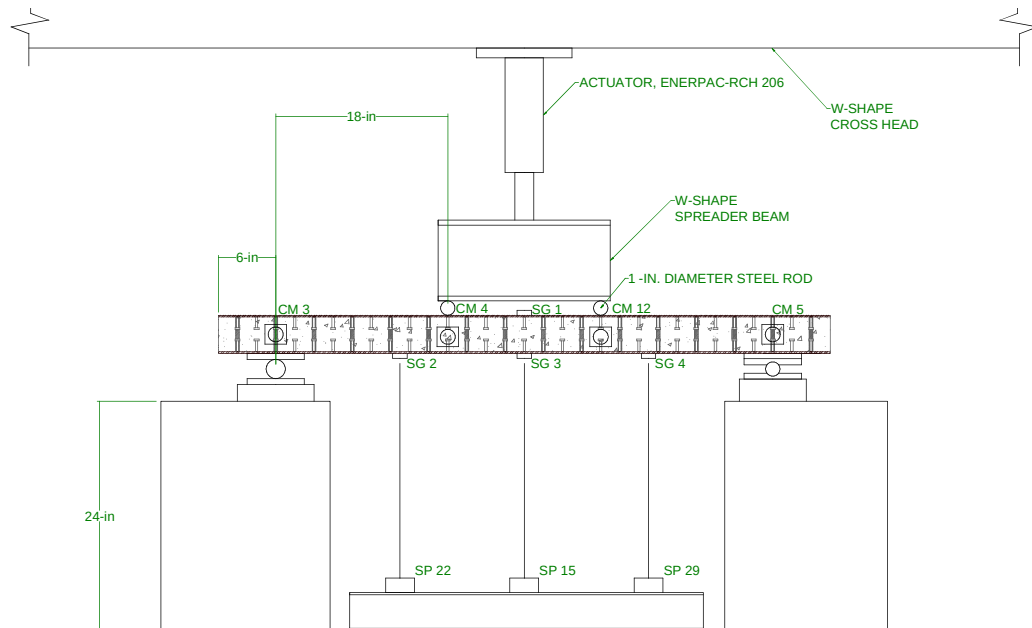


Figure 3.8. Static Four-Point Bending Test Setup

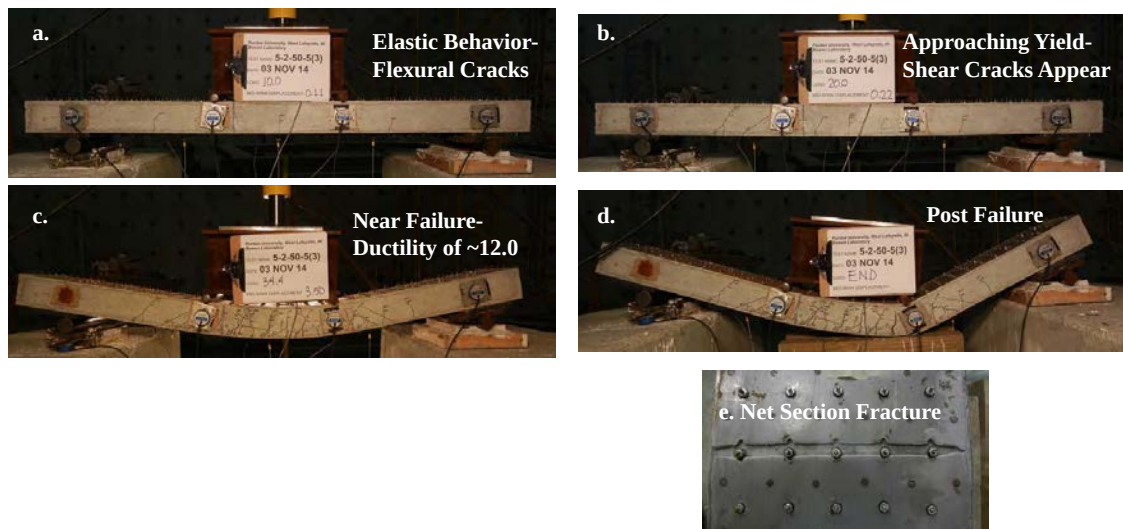
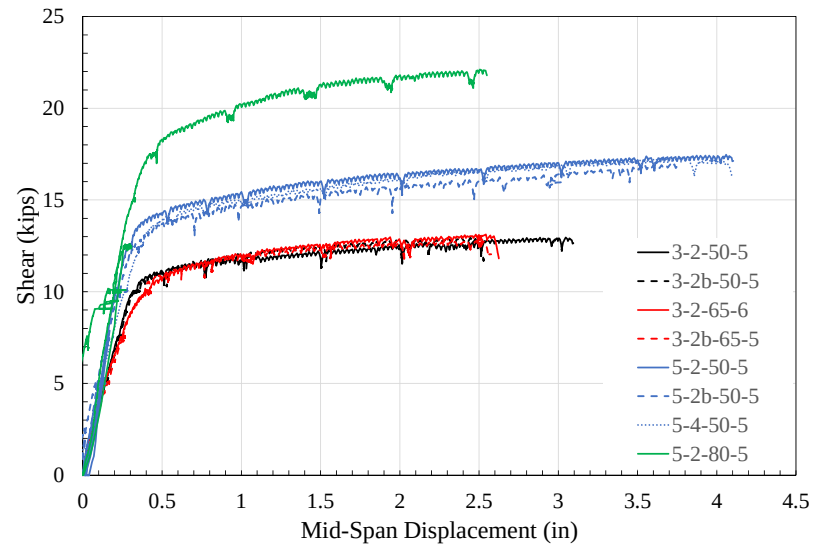
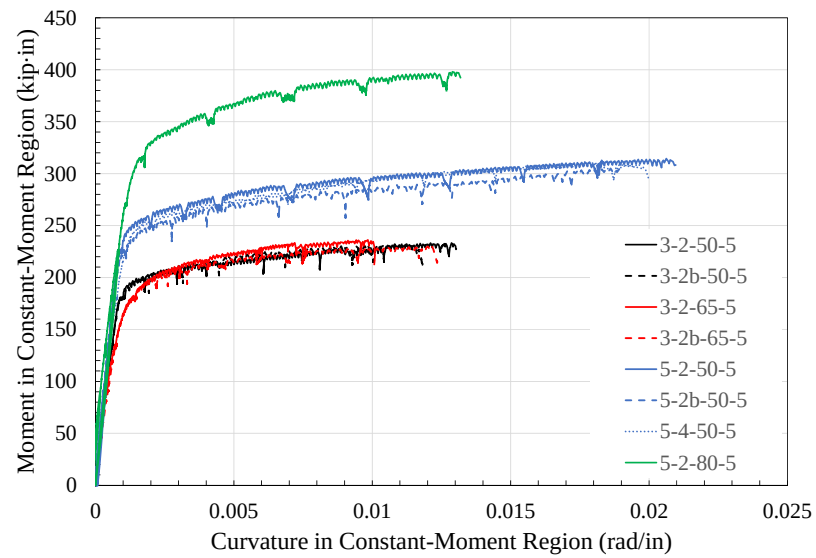


Figure 3.9. Sequence of Events for Static Tests



(a) Load-Displacement



(b) Moment-Curvature

Figure 3.10. Results from Static Tests

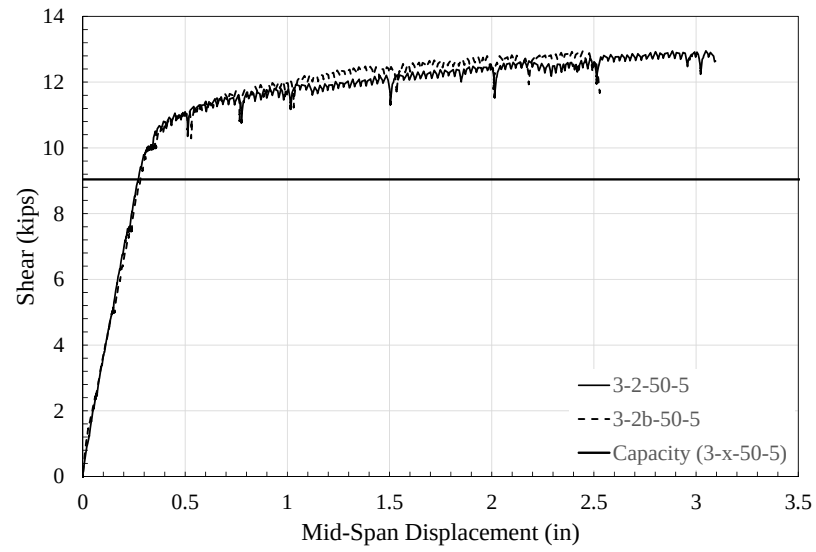


Figure 3.11. Shear-Displacement Curves and Capacity for 3-x-50-5 Beams

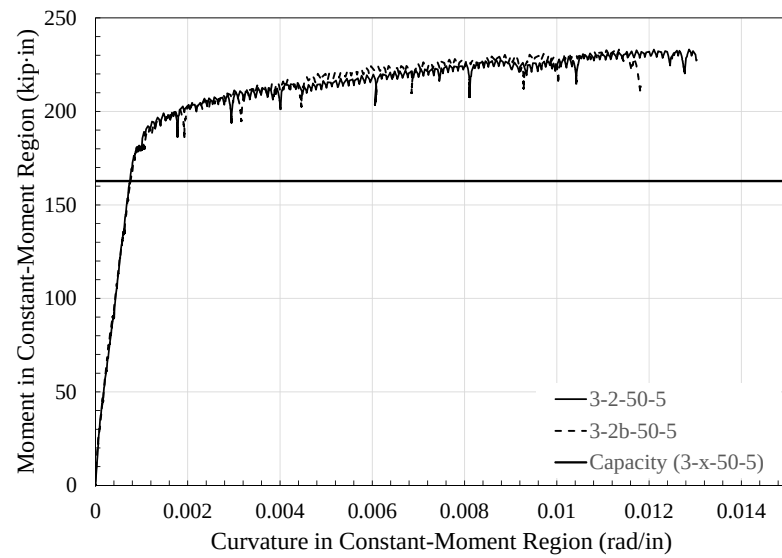


Figure 3.12. Moment-Curvature Curves and Capacity for 3-x-50-5 Beams

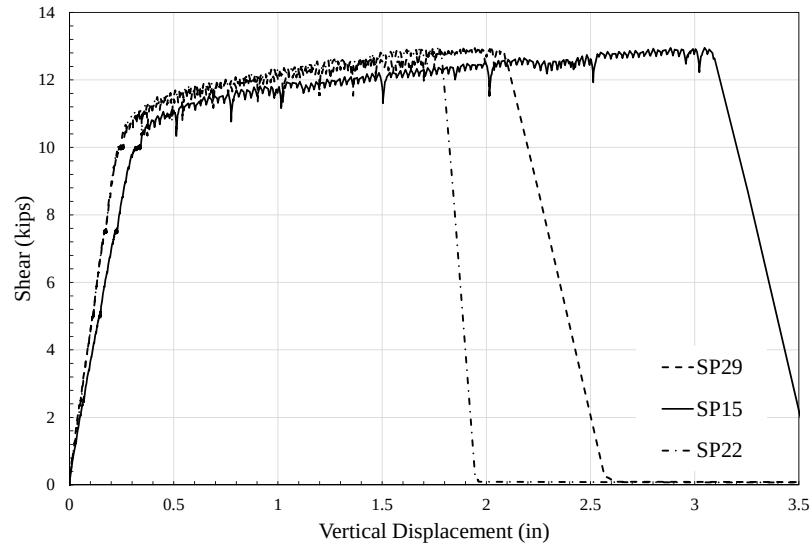


Figure 3.13. Shear Force - Vertical Deflection Response of 3-2-50-5

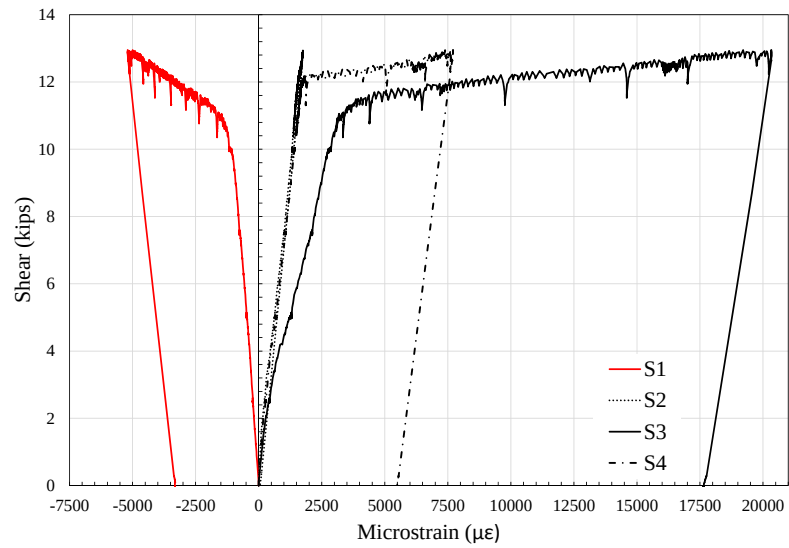


Figure 3.14. Shear vs Longitudinal Faceplate Strains of 3-2-50-5

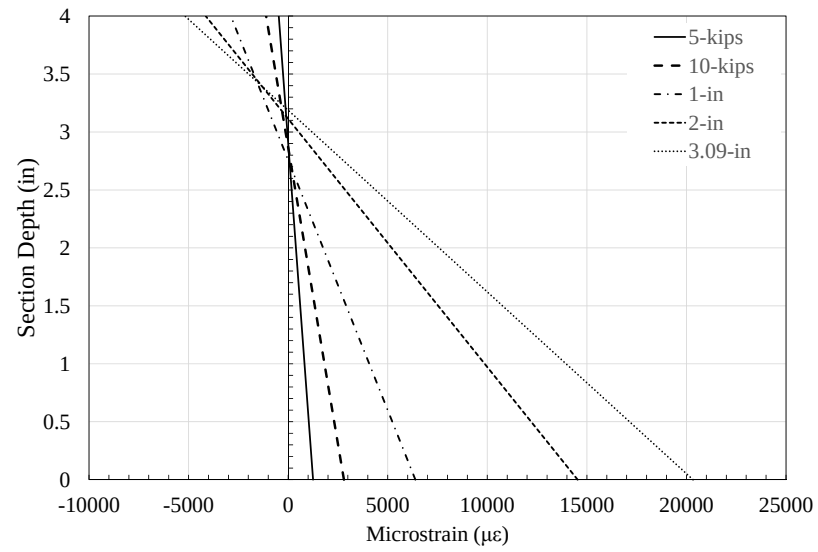


Figure 3.15. Strain Profile at Mid-Span Section of 3-2-50-5

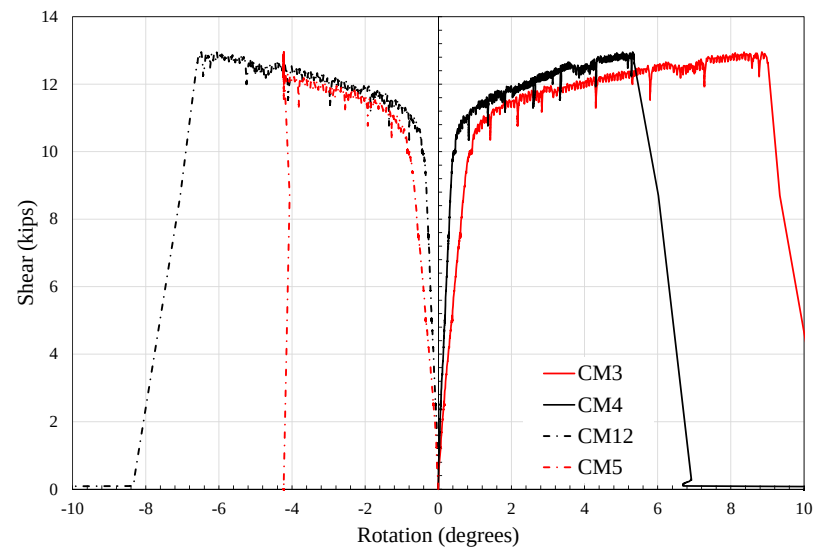


Figure 3.16. Measured Rotations of 3-2-50-5



(a) flexure cracks at Total Load of 15-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.17. Images of Progression of Static Test of 3-2-50-5

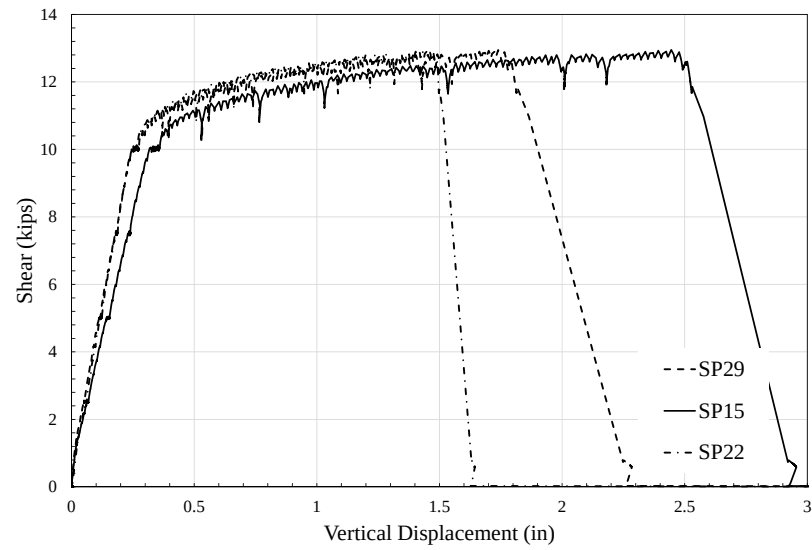


Figure 3.18. Shear Force - Vertical Deflection Response of 3-2b-50-5

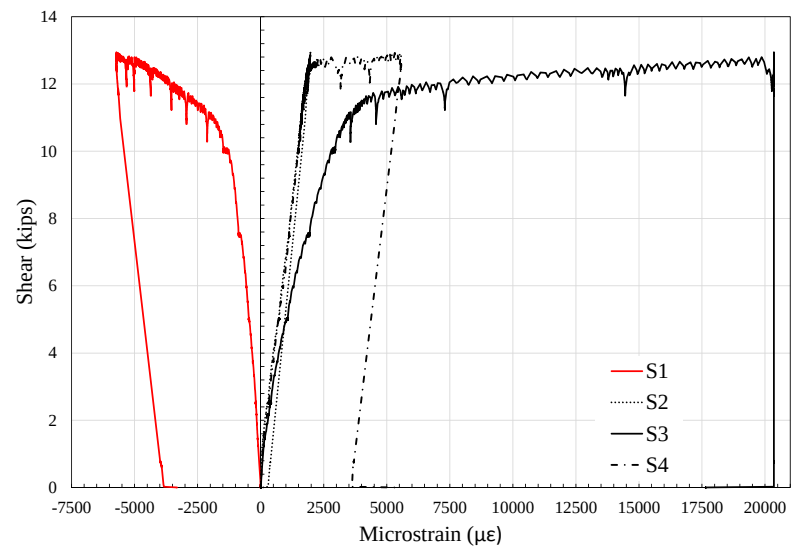


Figure 3.19. Shear vs Longitudinal Faceplate Strains of 3-2b-50-5

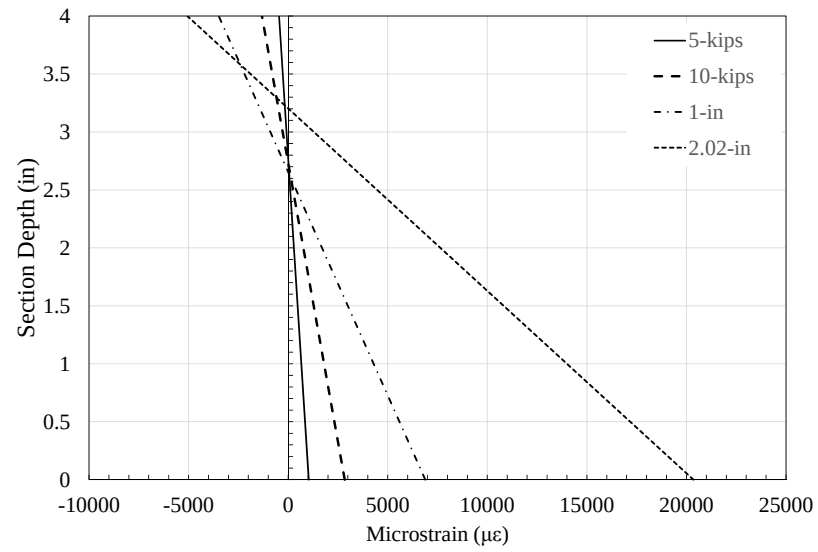


Figure 3.20. Strain Profile at Mid-Span Section of 3-2b-50-5

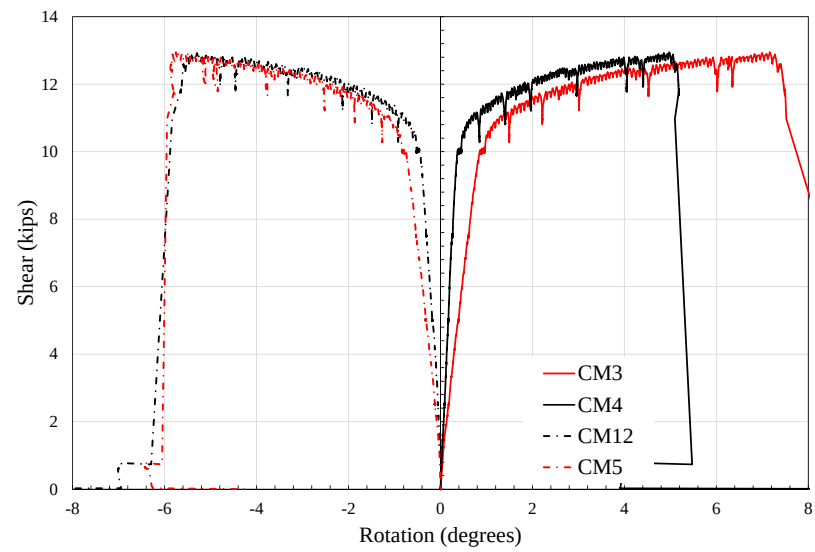
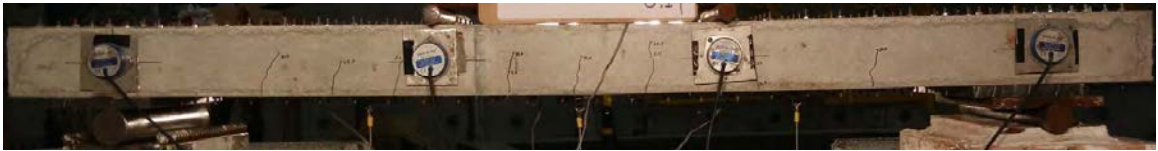


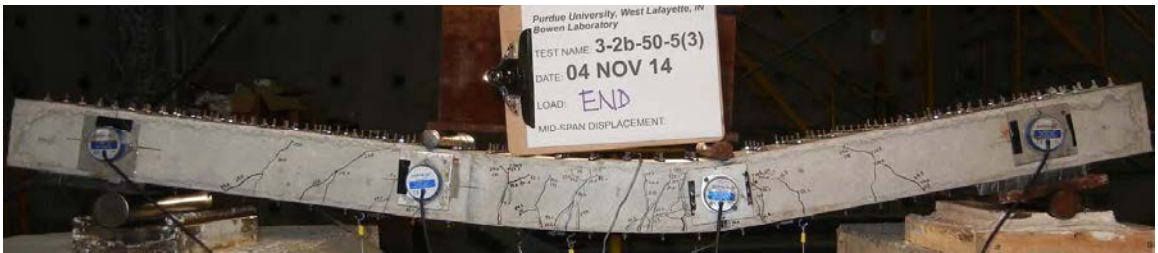
Figure 3.21. Measured Rotations of 3-2b-50-5



(a) flexure cracks at Total Load of 10-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.22. Images of Progression of Static Test of 3-2b-50-5

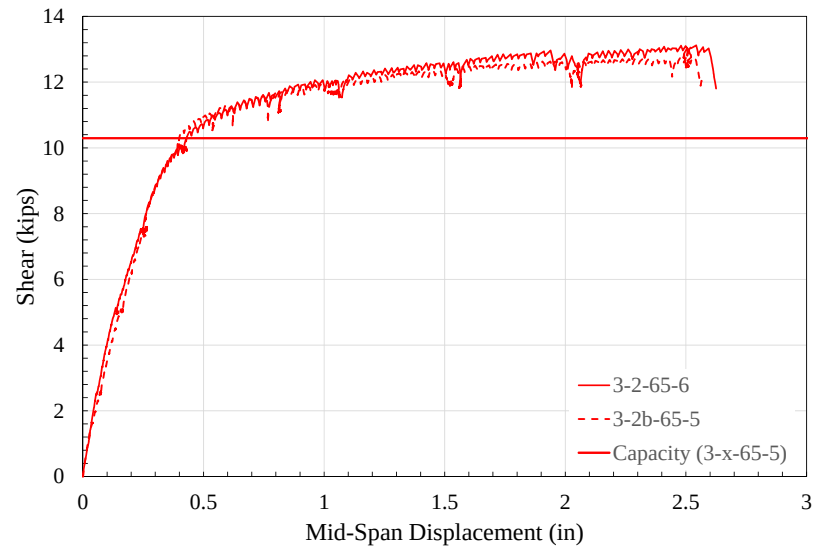


Figure 3.23. Shear-Displacement Curves and Capacity for 3-x-65-5 Beams

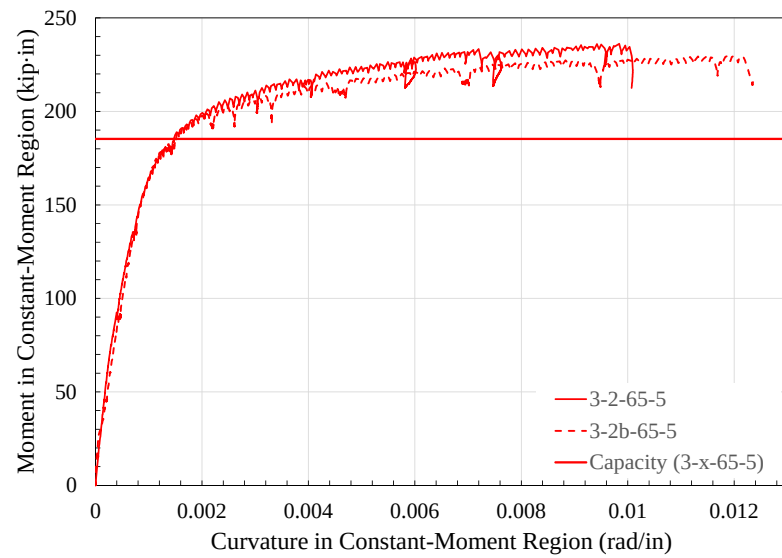


Figure 3.24. Moment-Curvature Curves and Capacity for 3-x-65-5 Beams

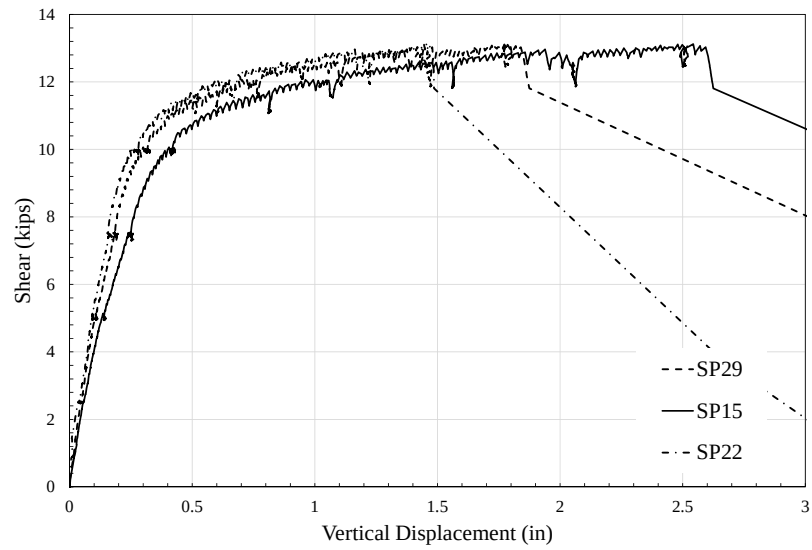


Figure 3.25. Shear Force - Vertical Deflection Response of 3-2-65-5

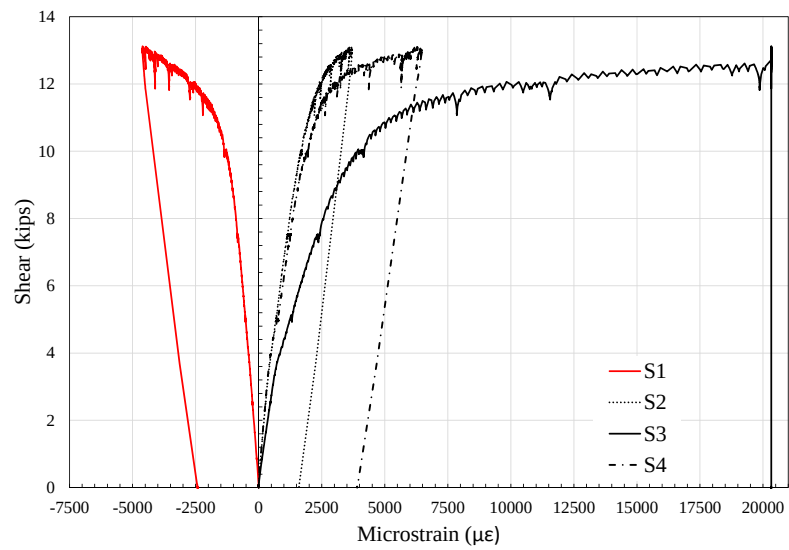


Figure 3.26. Shear vs Longitudinal Faceplate Strains of 3-2-65-5

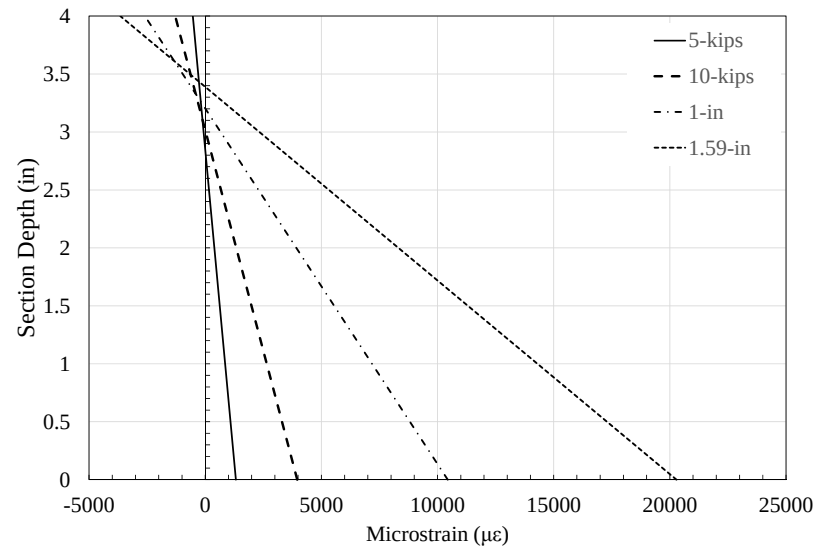


Figure 3.27. Strain Profile at Mid-Span Section of 3-2-65-5

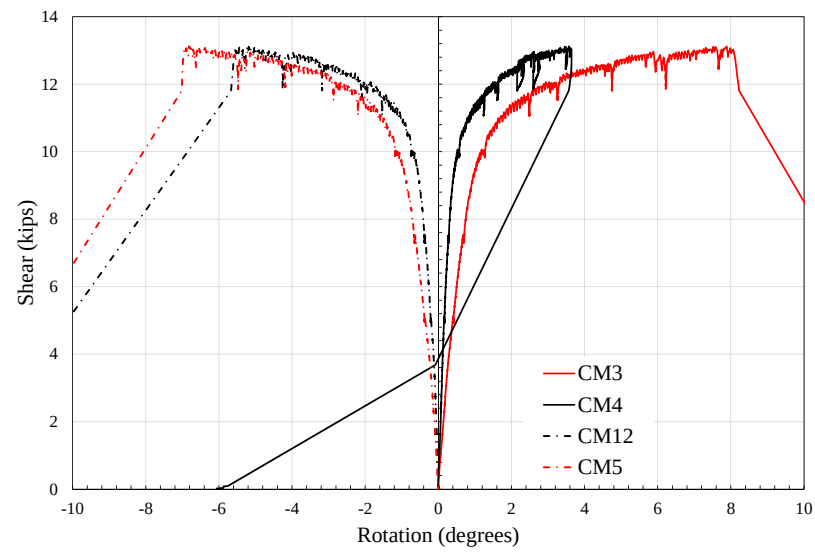


Figure 3.28. Measured Rotations of 3-2-65-5



(a) flexure cracks at Total Load of 15-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.29. Images of Progression of Static Test of 3-2-65-5

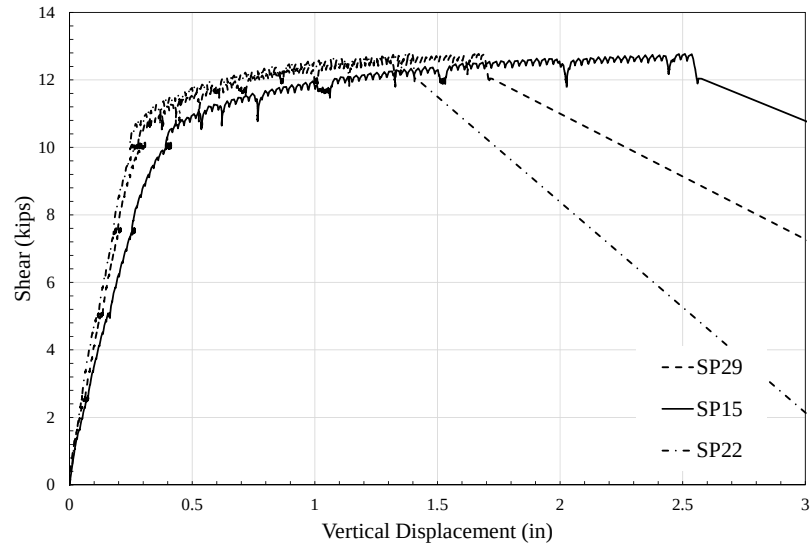


Figure 3.30. Shear Force - Vertical Deflection Response of 3-2b-65-5

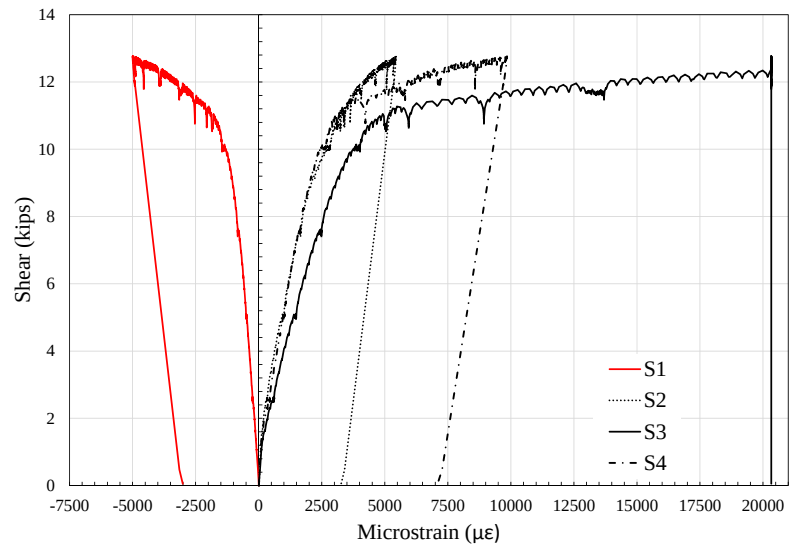


Figure 3.31. Shear vs Longitudinal Faceplate Strains of 3-2b-65-5

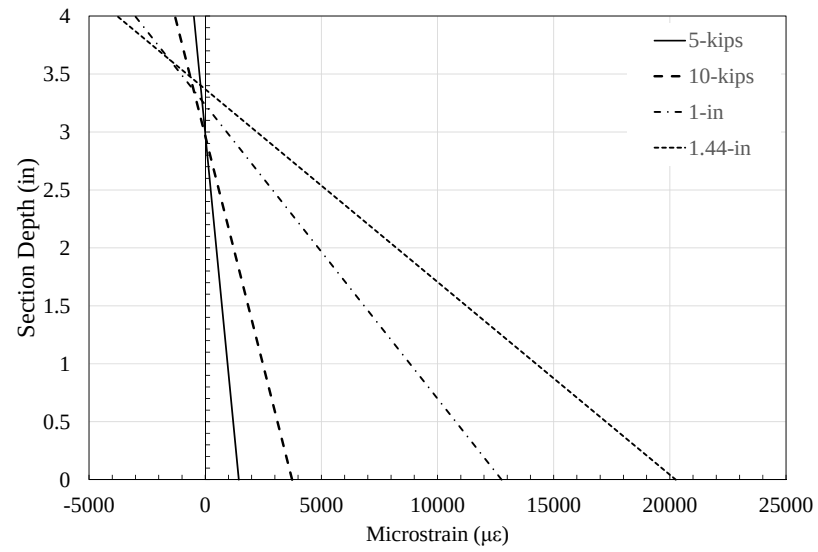


Figure 3.32. Strain Profile at Mid-Span Section of 3-2b-65-5

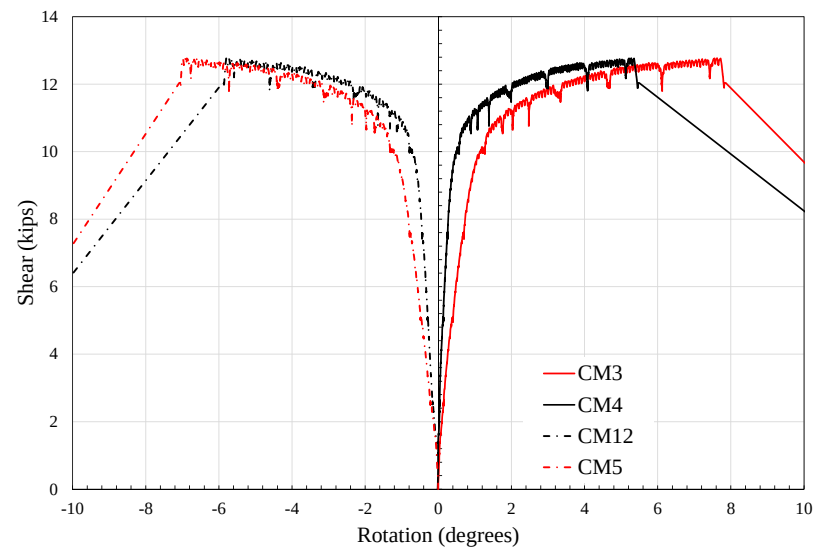
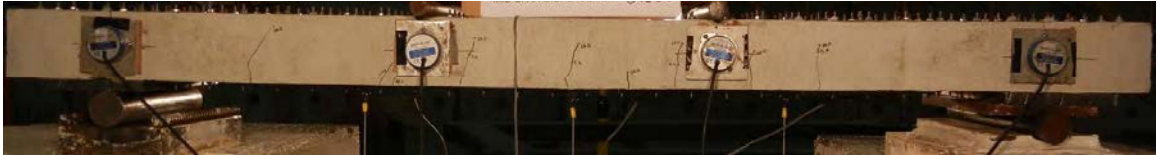
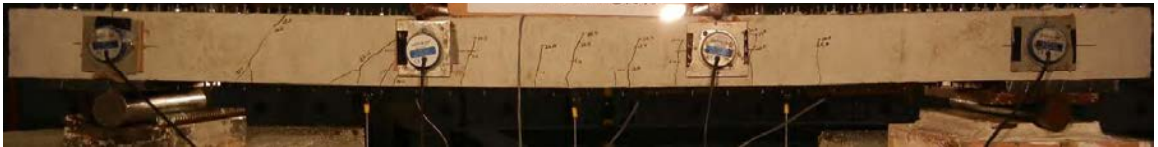


Figure 3.33. Measured Rotations of 3-2b-65-5



(a) flexure cracks at Total Load of 10-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.34. Images of Progression of Static Test of 3-2b-65-5

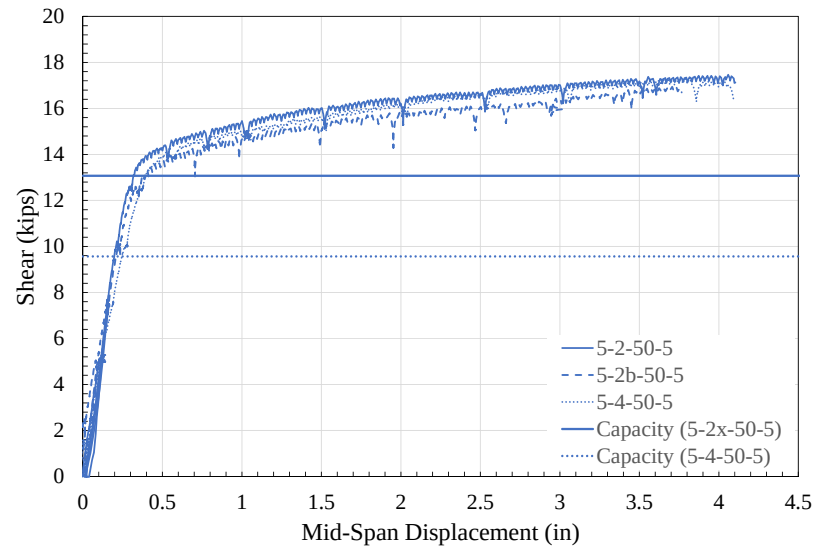


Figure 3.35. Shear-Displacement Curves and Capacity for 5-x-50-5 Beams

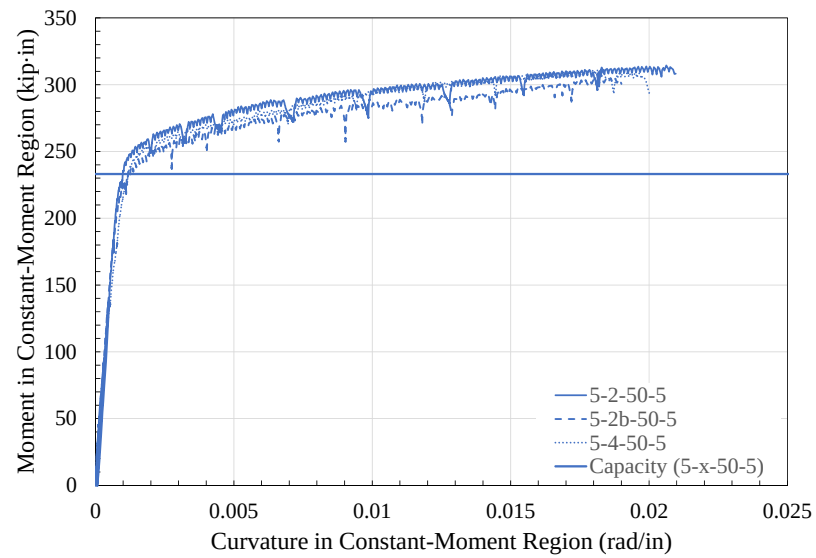


Figure 3.36. Moment-Curvature Curves and Capacity for 5-x-50-5 Beams

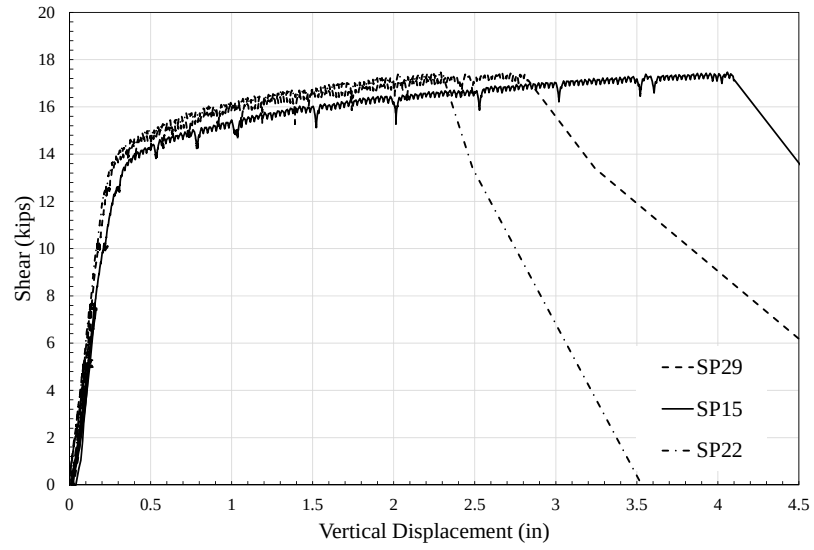


Figure 3.37. Shear Force - Vertical Deflection Response of 5-2-50-5

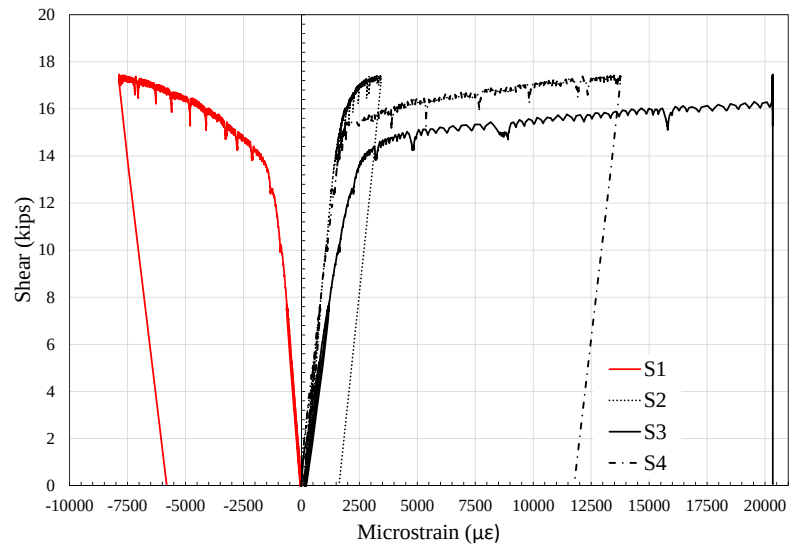


Figure 3.38. Shear vs Longitudinal Faceplate Strains of 5-2-50-5

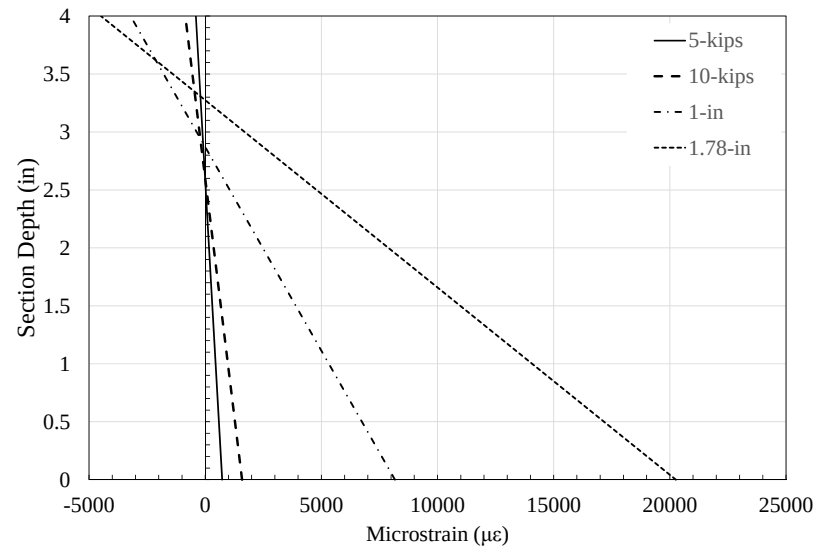


Figure 3.39. Strain Profile at Mid-Span Section of 5-2-50-5

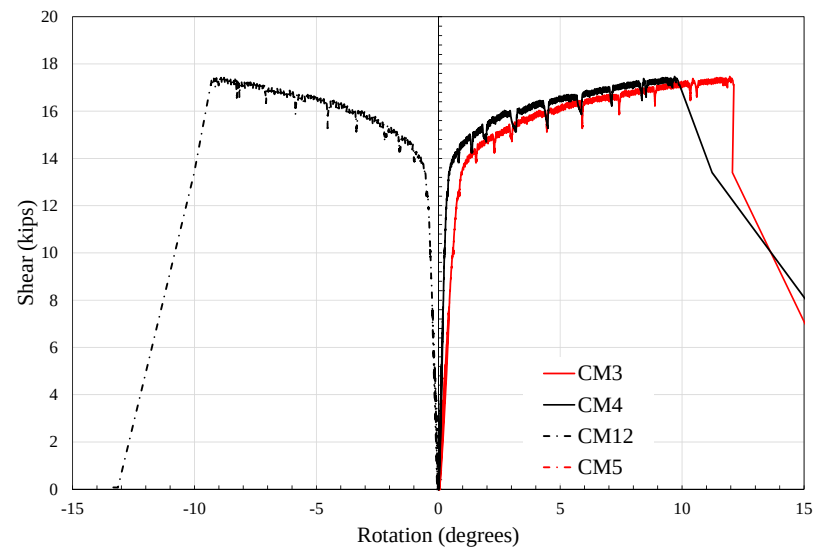


Figure 3.40. Measured Rotations of 5-2-50-5



(a) flexure cracks at Total Load of 10-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.41. Images of Progression of Static Test of 5-2-50-5

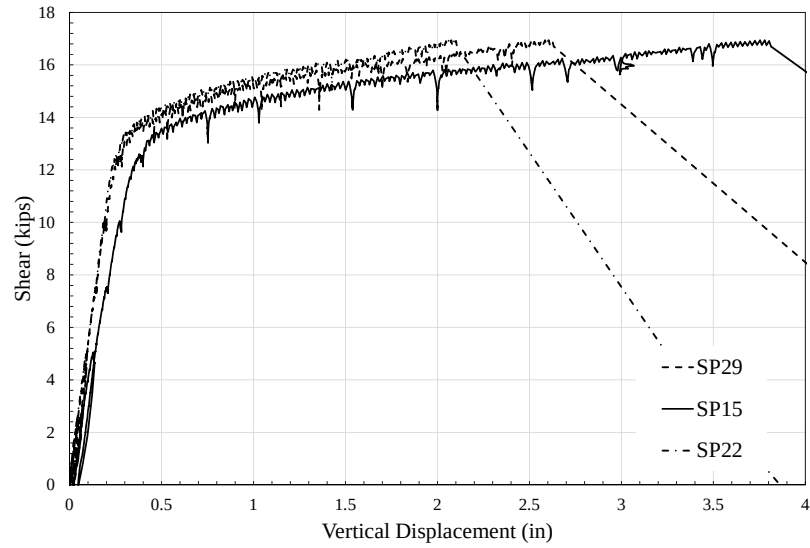


Figure 3.42. Shear Force - Vertical Deflection Response of 5-2b-50-5

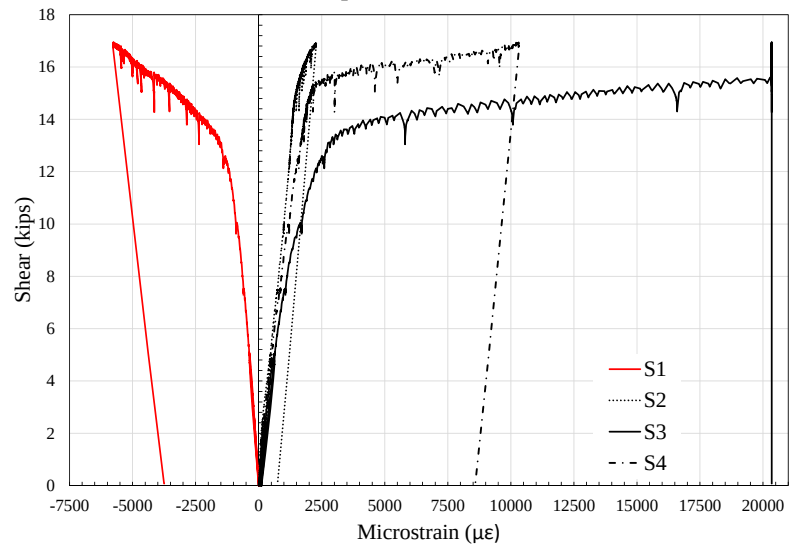


Figure 3.43. Shear vs Longitudinal Faceplate Strains of 5-2b-50-5

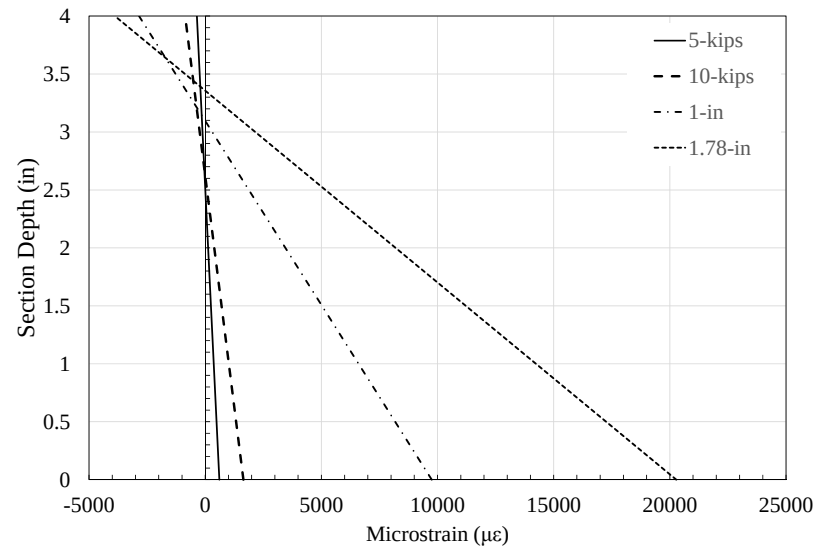


Figure 3.44. Strain Profile at Mid-Span Section of 5-2b-50-5

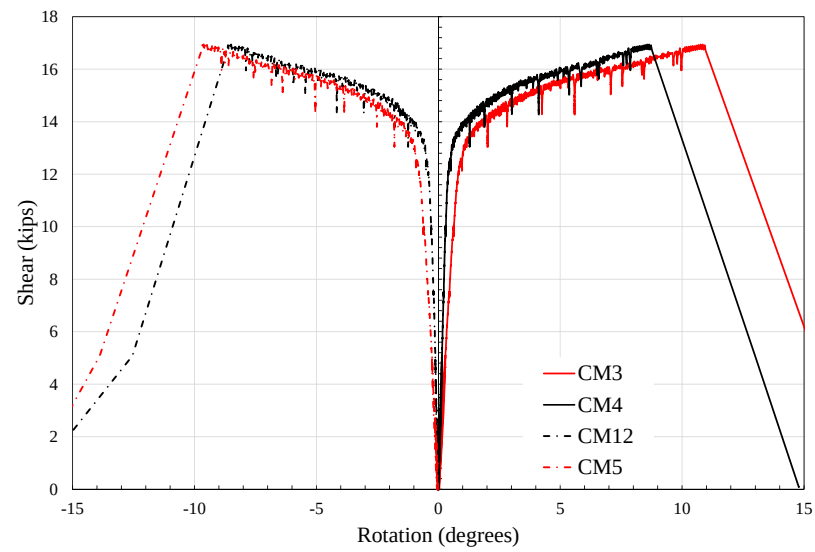


Figure 3.45. Measured Rotations of 5-2b-50-5



(a) flexure cracks at Total Load of 15-kips



(b) Shear Cracks at Total Load of 20-kips



(c) Failure

Figure 3.46. Images of Progression of Static Test of 5-2b-50-5

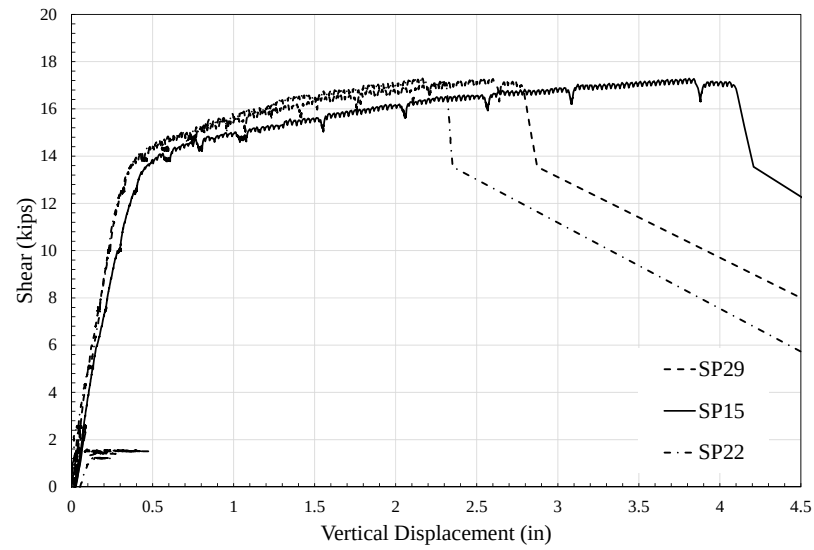


Figure 3.47. Shear Force - Vertical Deflection Response of 5-4-50-5

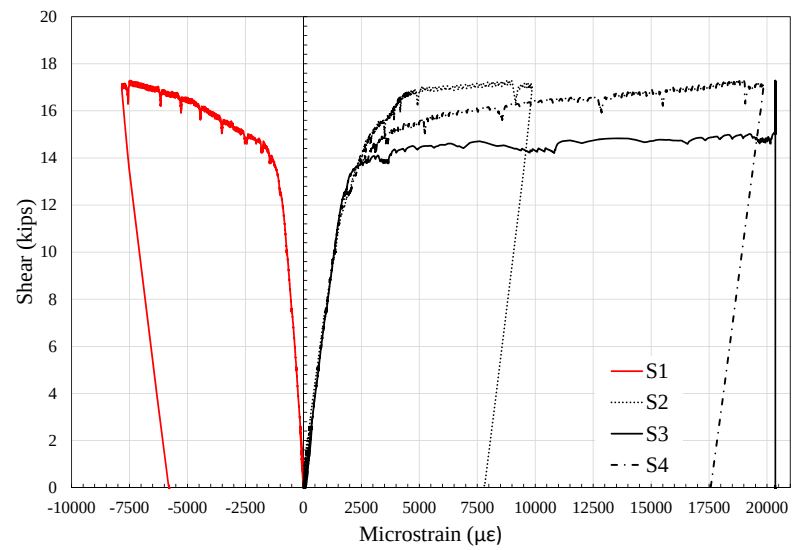


Figure 3.48. Shear vs Longitudinal Faceplate Strains of 5-4-50-5

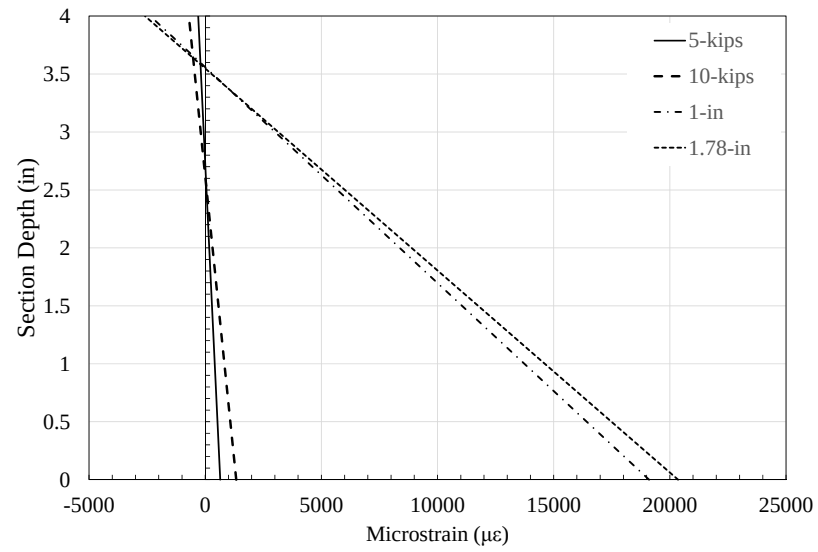


Figure 3.49. Strain Profile at Mid-Span Section of 5-4-50-5

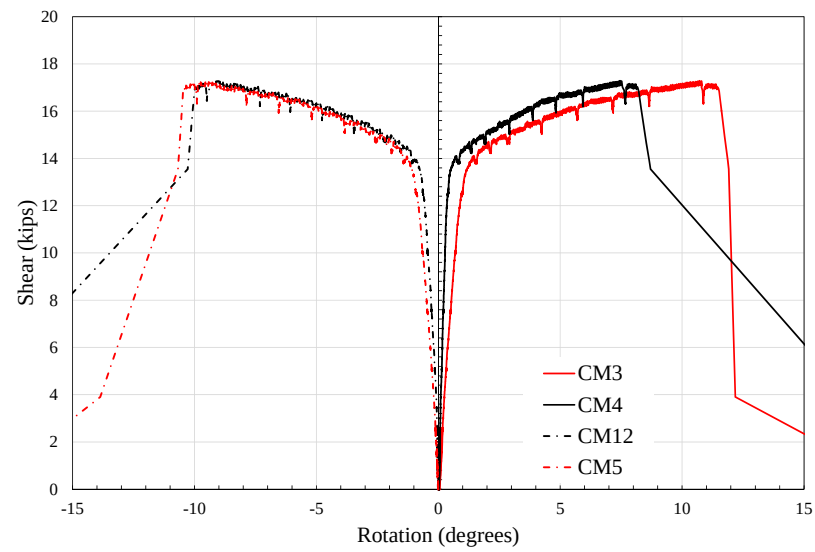


Figure 3.50. Measured Rotations of 5-4-50-5



(a) flexure cracks at Total Load of 15-kips



(b) Shear Cracks at Total Load of 25-kips



(c) Failure

Figure 3.51. Images of Progression of Static Test of 5-4-50-5

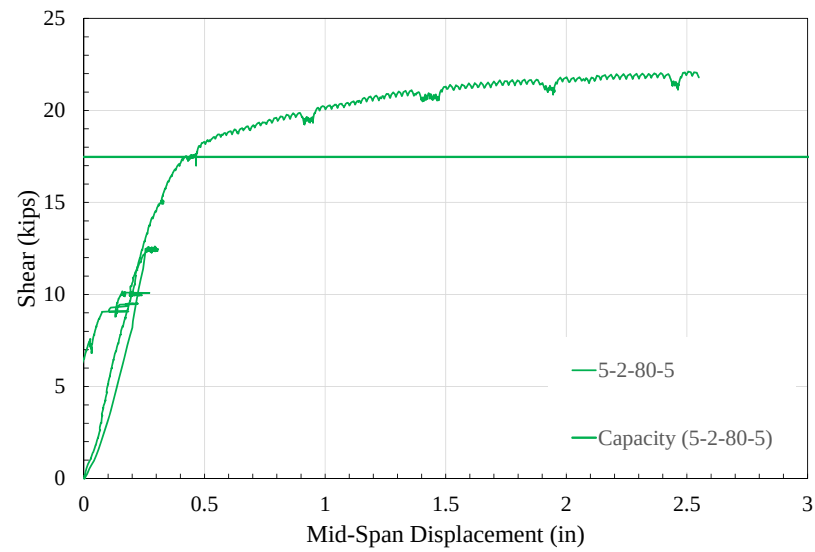


Figure 3.52. Shear-Displacement Curves and Capacity for 5-2-80-5 Beams

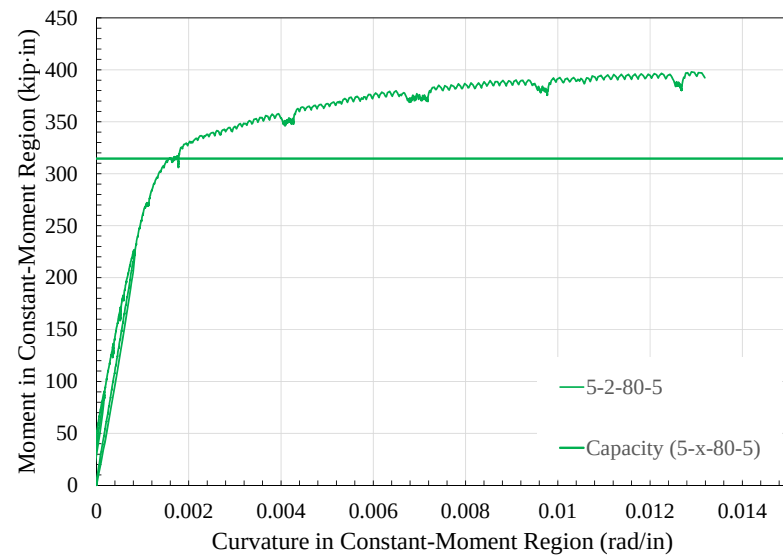


Figure 3.53. Moment-Curvature Curves and Capacity for 5-2-80-5 Beams

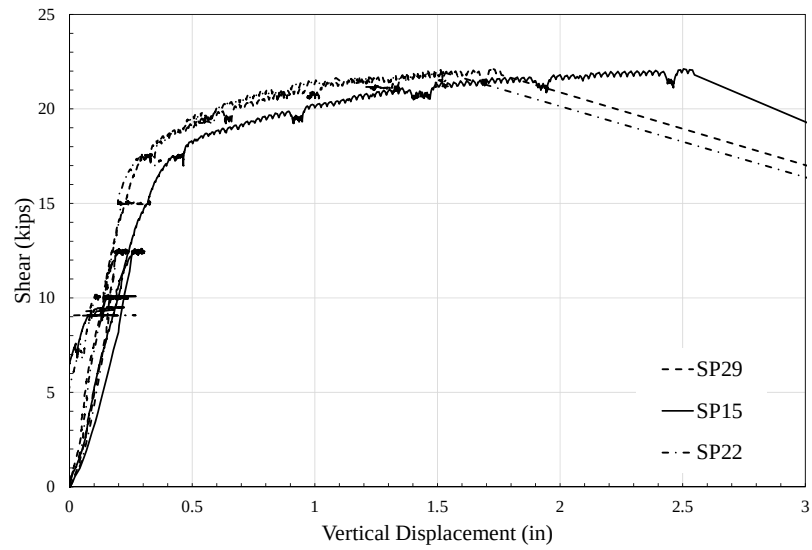


Figure 3.54. Shear Force - Vertical Deflection Response of 5-2-80-5

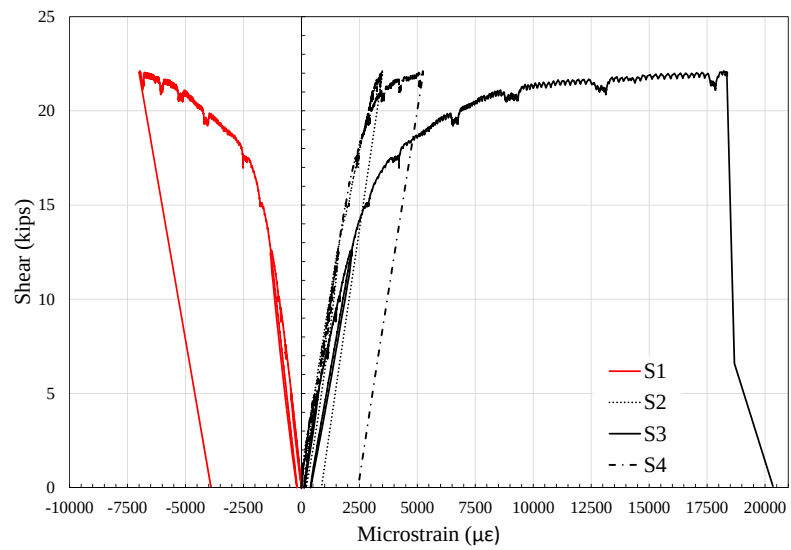


Figure 3.55. Shear vs Longitudinal Faceplate Strains of 5-2-80-5

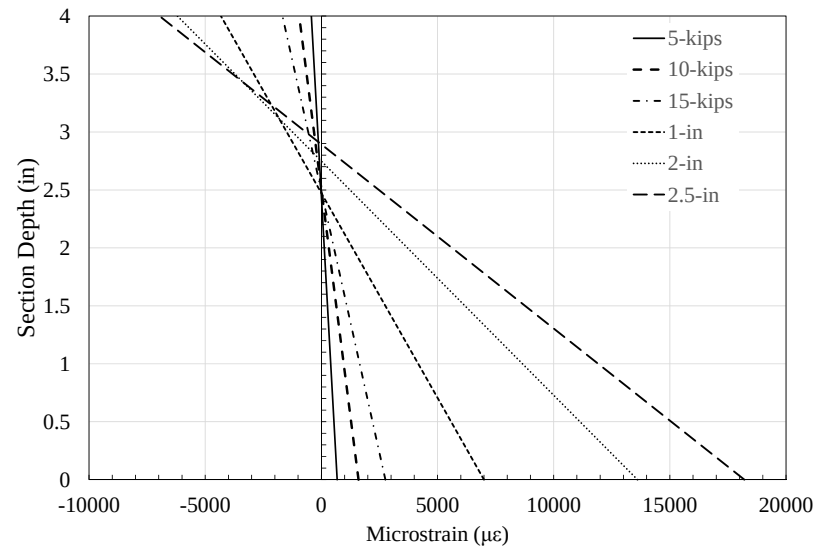


Figure 3.56. Strain Profile at Mid-Span Section of 5-2-50-5

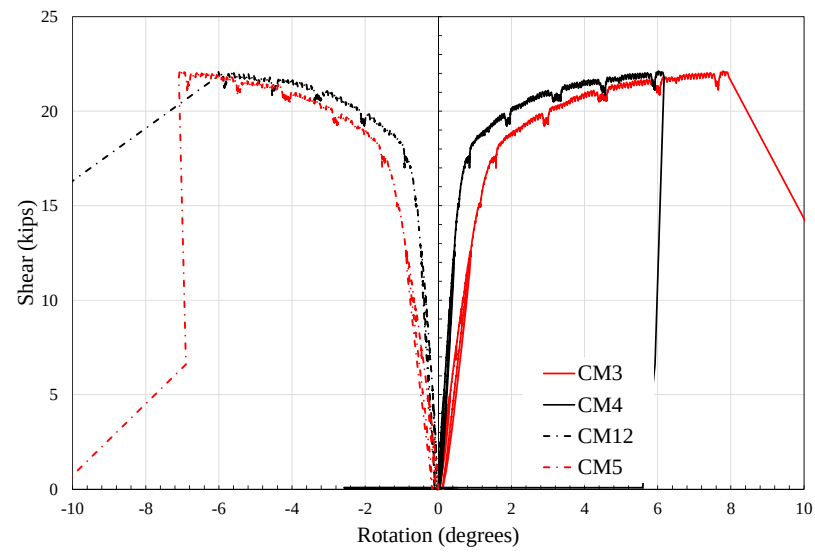
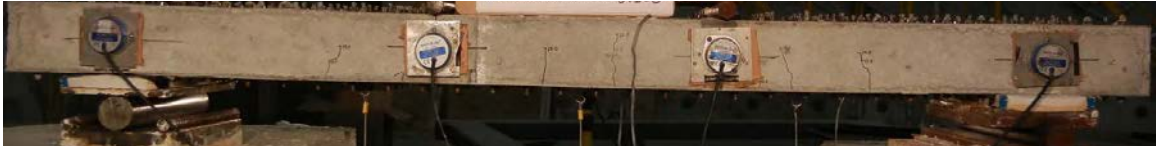


Figure 3.57. Measured Rotations of 5-2-80-5



(a) flexure cracks at Total Load of 15-kips

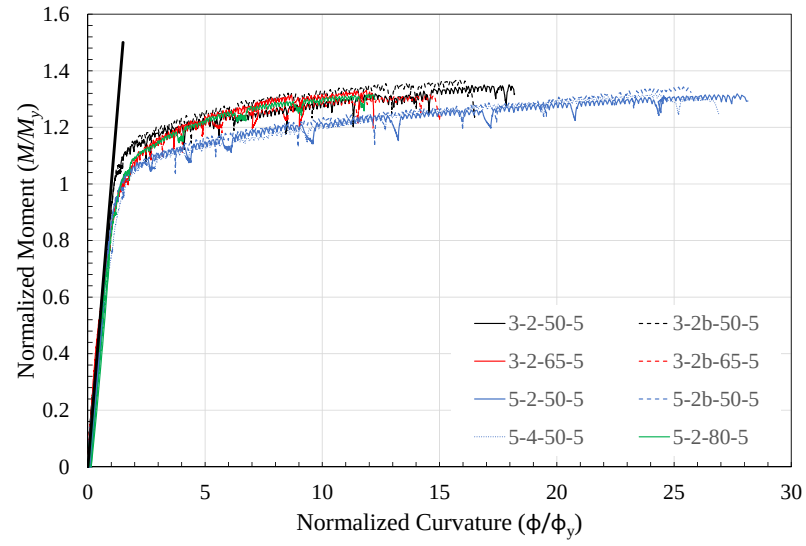


(b) Shear Cracks at Total Load of 25-kips

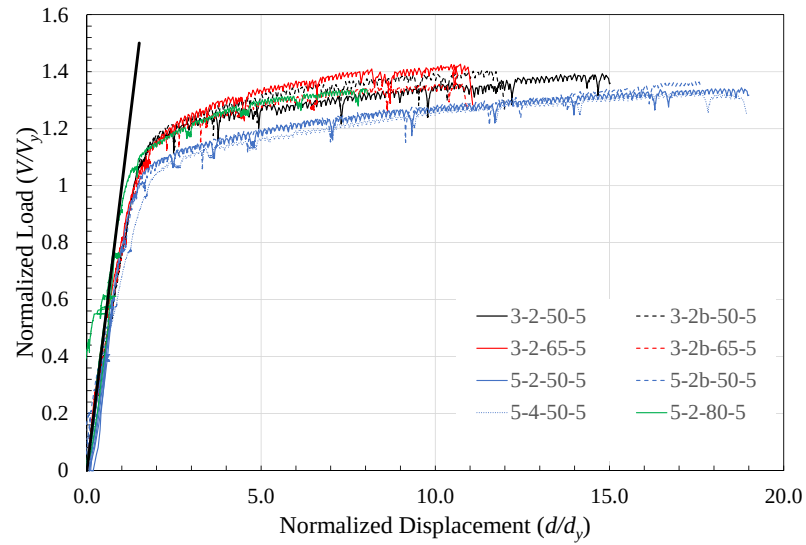


(c) Failure

Figure 3.58. Images of Progression of Static Test of 5-2-80-5



(a) Normalized Moment-Curvature



(b) Normalized Load-Displacement

Figure 3.59. Normalized Comparison of Static Experimental Results

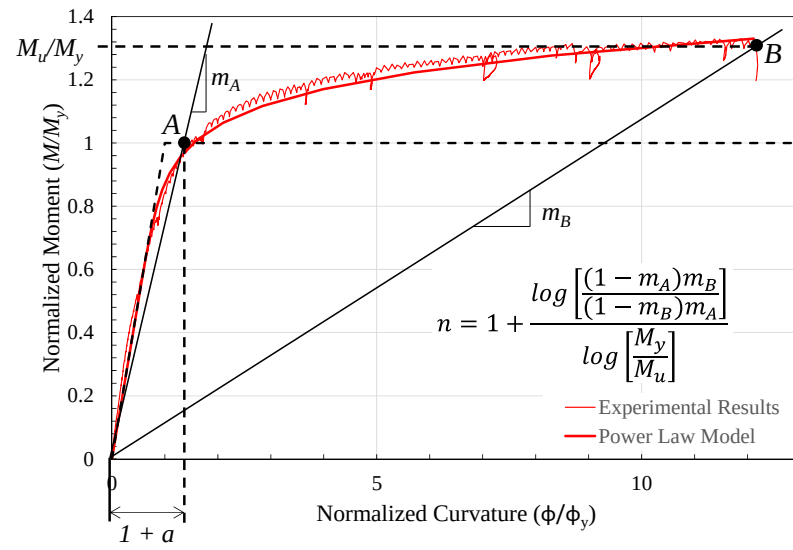


Figure 3.60. Graphical Determination of Power Law Constants

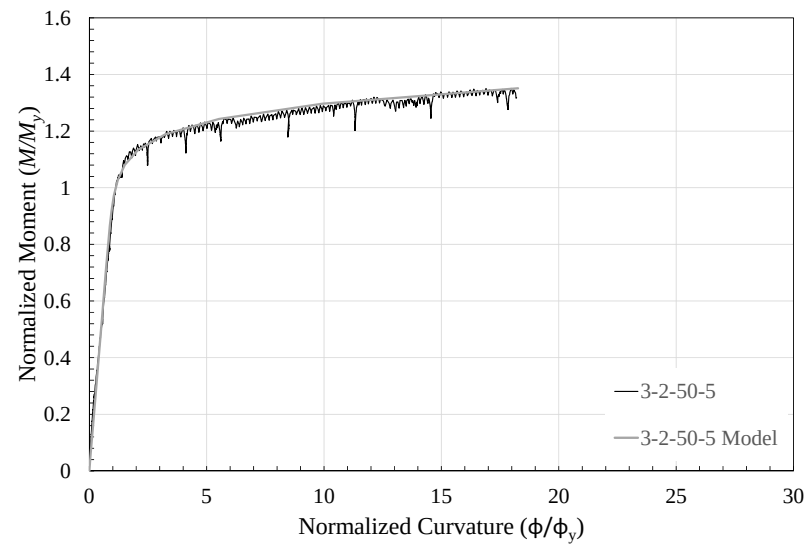


Figure 3.61. Moment-Curvature Power Law Best Fits for 3-2-50-5

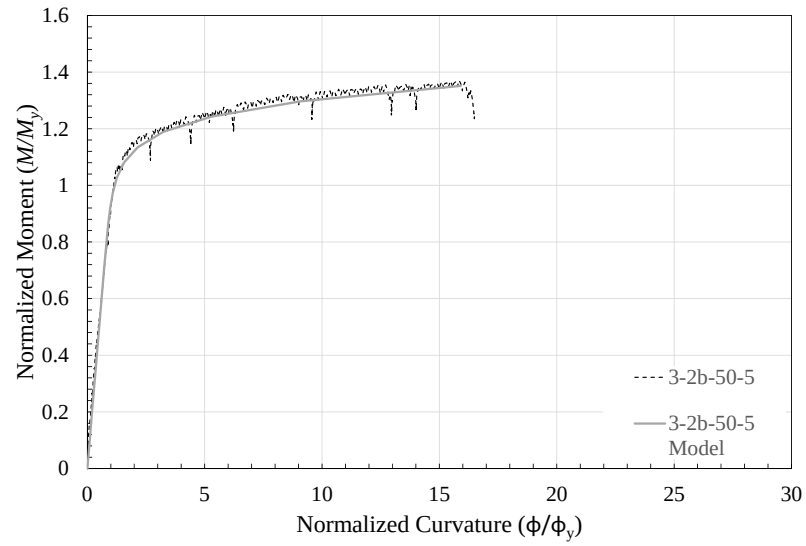


Figure 3.62. Moment-Curvature Power Law Best Fits for 3-2b-50-5

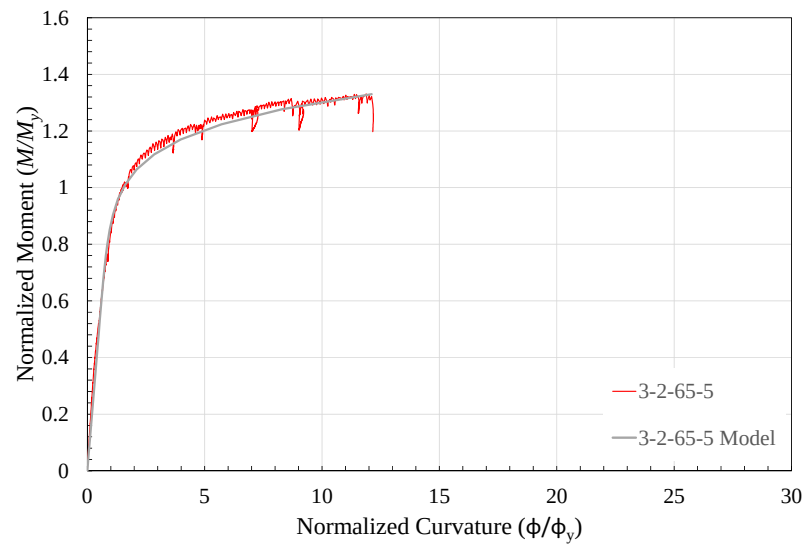


Figure 3.63. Moment-Curvature Power Law Best Fits for 3-2-65-5

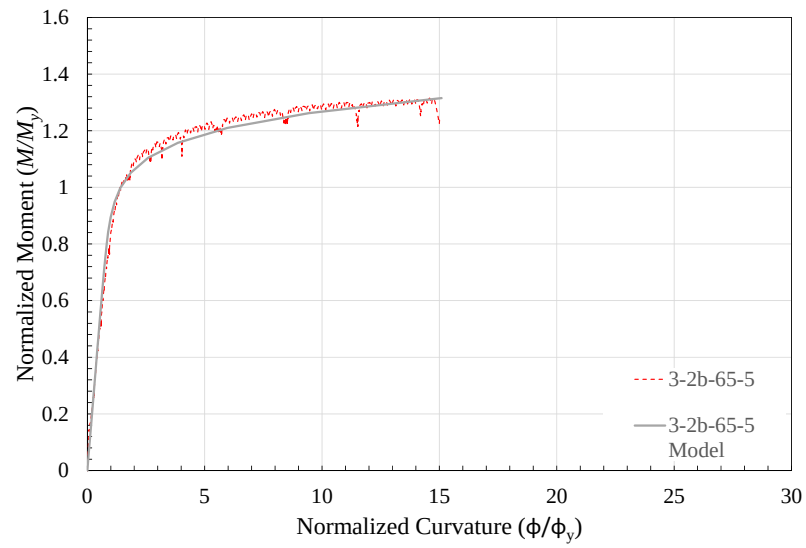


Figure 3.64. Moment-Curvature Power Law Best Fits for 3-2b-65-5

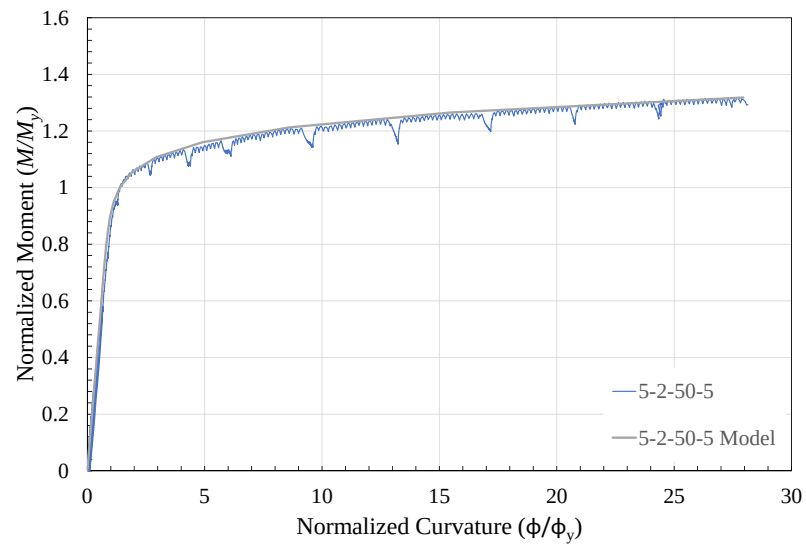


Figure 3.65. Moment-Curvature Power Law Best Fits for 5-2-50-5

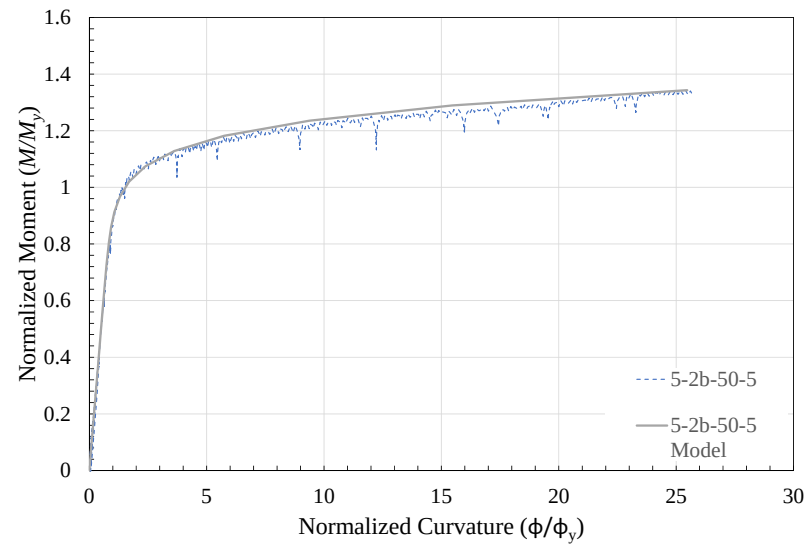


Figure 3.66. Moment-Curvature Power Law Best Fits for 5-2b-50-5

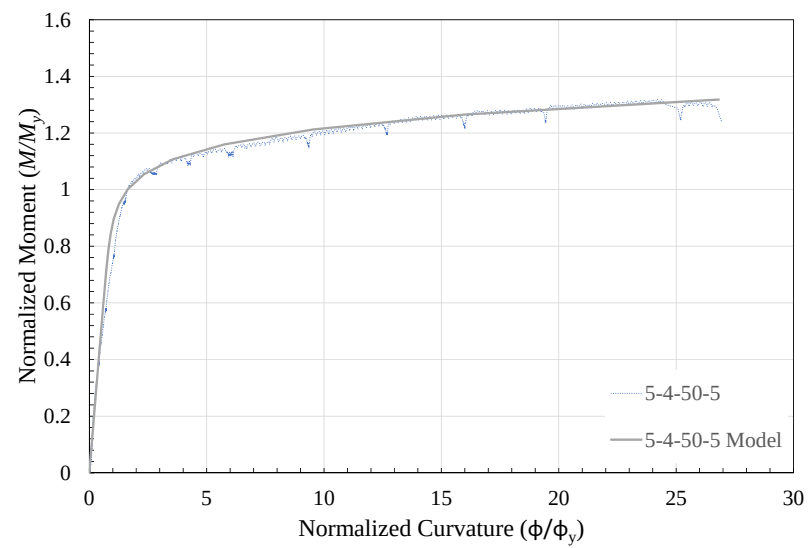


Figure 3.67. Moment-Curvature Power Law Best Fits for 5-4-50-5

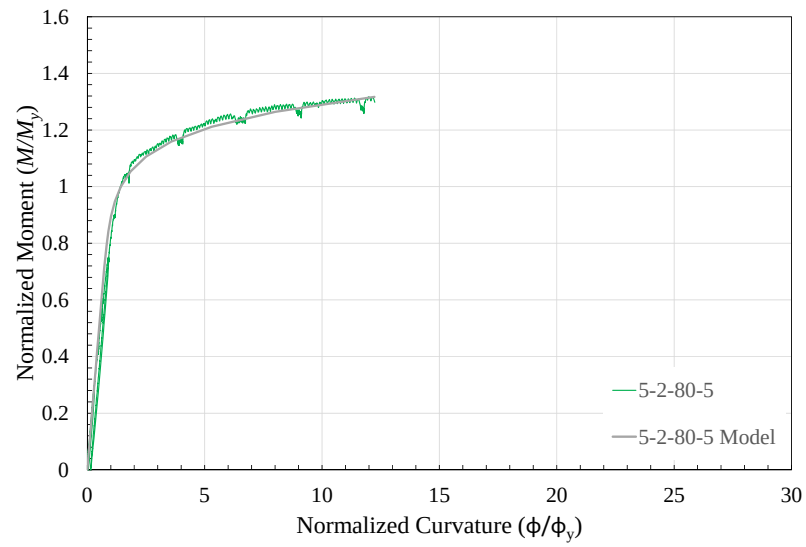


Figure 3.68. Moment-Curvature Power Law Best Fits for 5-2-80-5

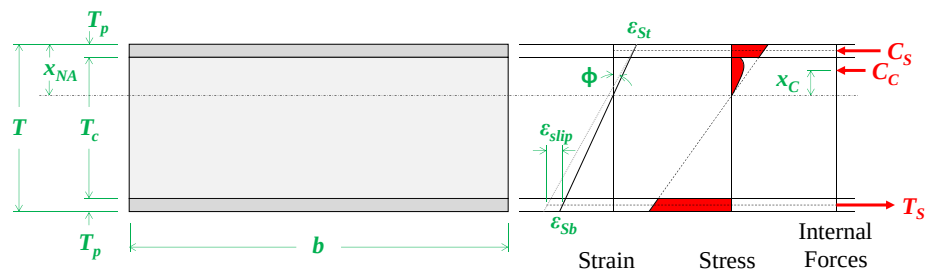


Figure 3.69. Strain Compatibility and Stress Distribution for Moment-Curvature Calculation

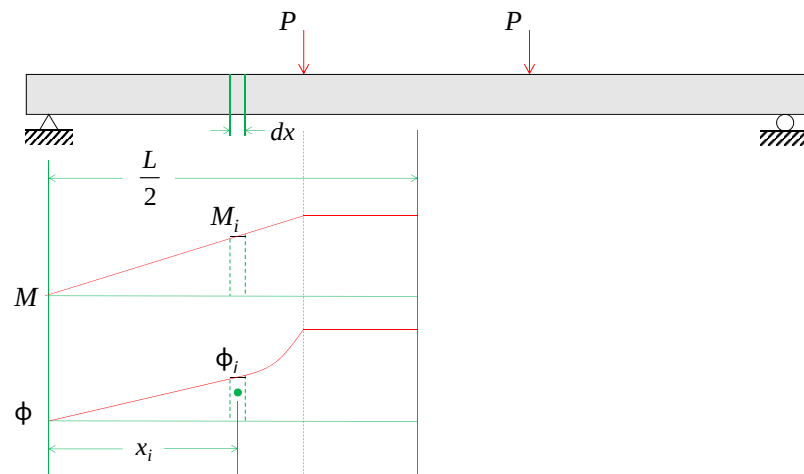
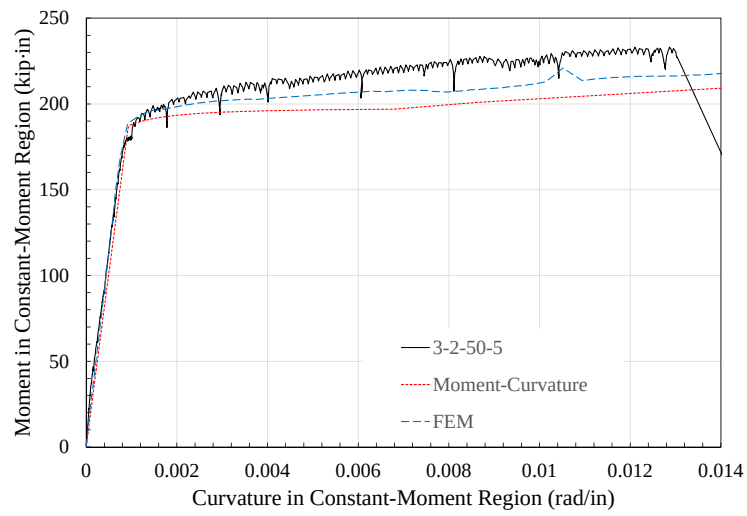
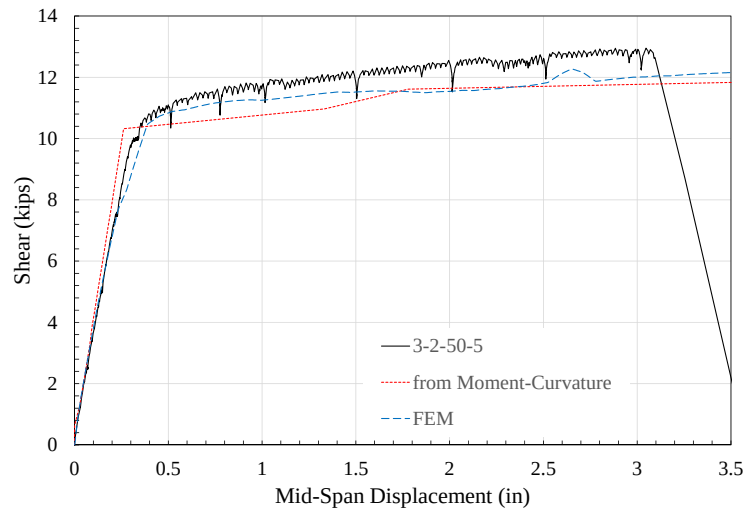


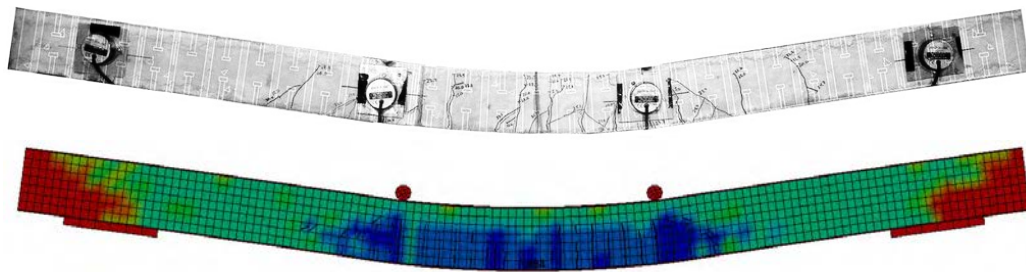
Figure 3.70. Computation of Mid-Span Displacement of Beam Loaded in Four-Point Bending by Moment-Area Method



(a) Moment-Curvature Comparisons

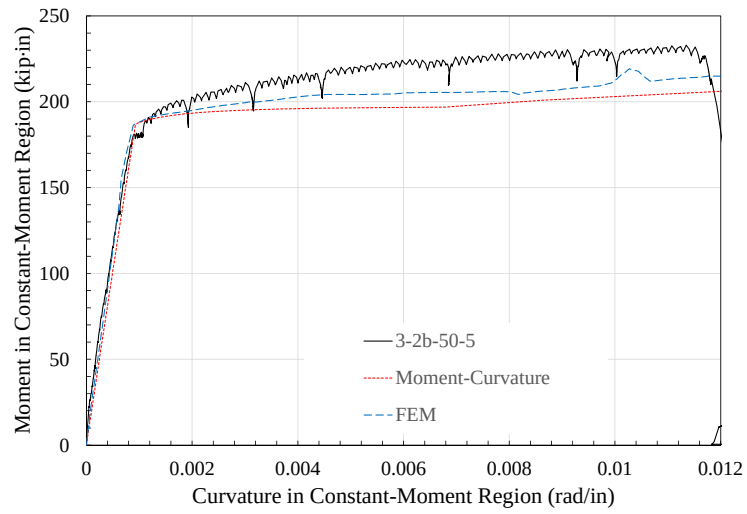


(b) Load-Displacement Comparisons

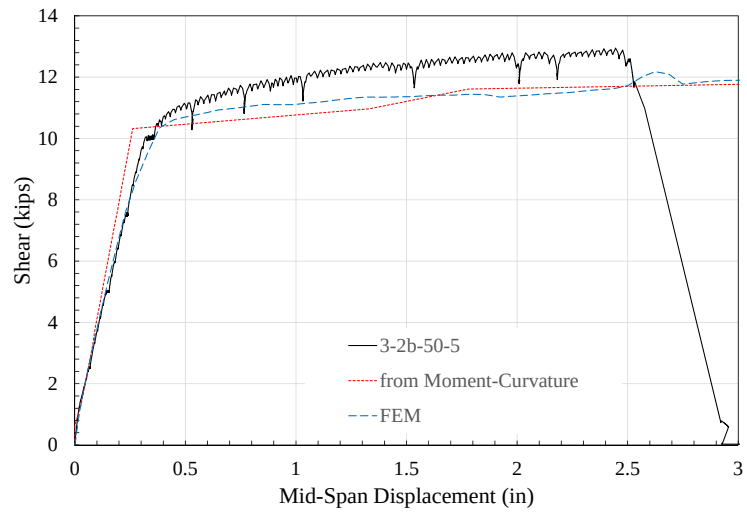


(c) Crack Pattern Comparison

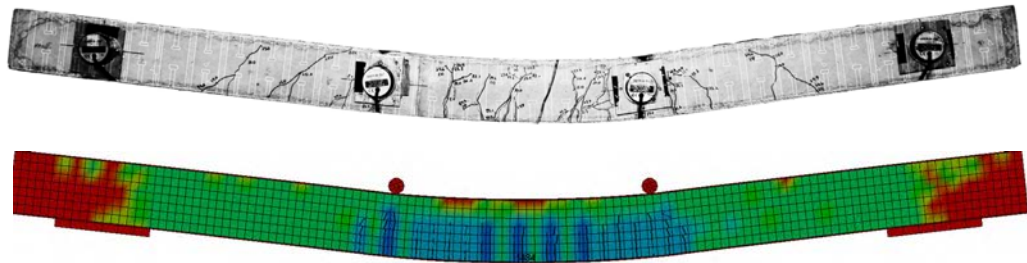
Figure 3.71. Comparison of Experimental to Model Results for 3-2-50-5



(a) Moment-Curvature Comparisons

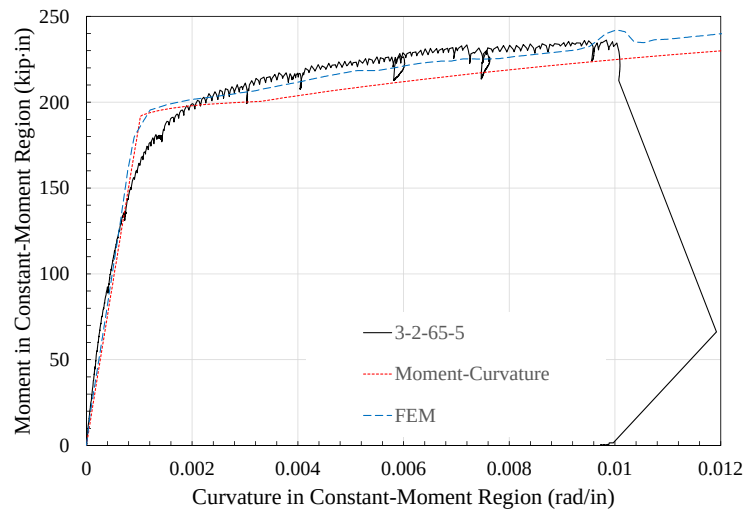


(b) Load-Displacement Comparisons

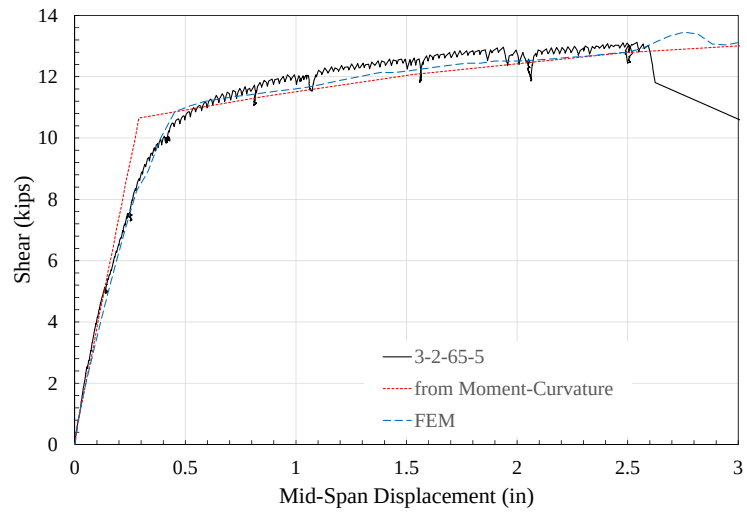


(c) Crack Pattern Comparison

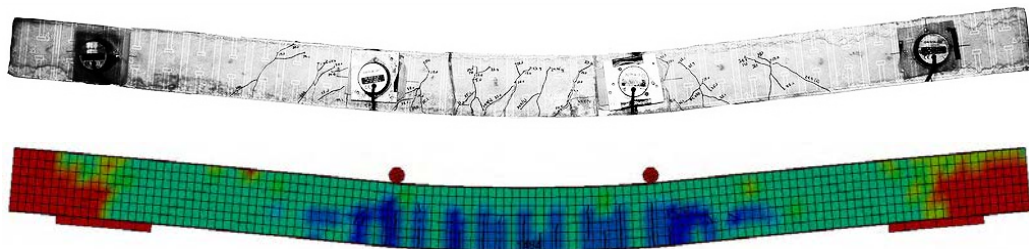
Figure 3.72. Comparison of Experimental to Model Results for 3-2b-50-5



(a) Moment-Curvature Comparisons

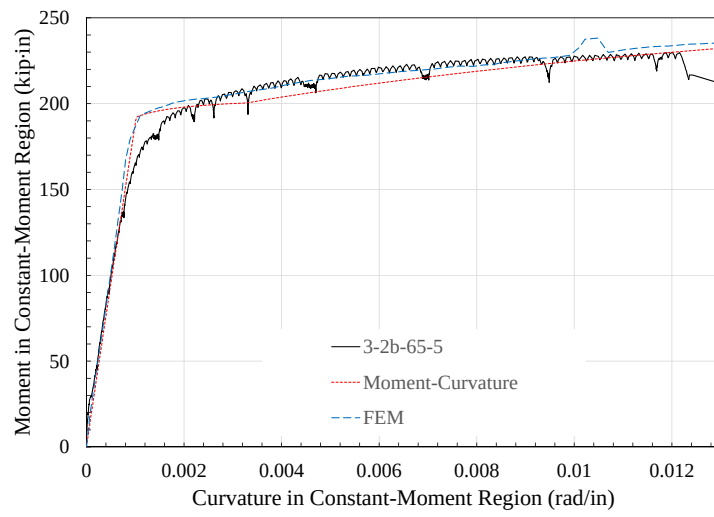


(b) Load-Displacement Comparisons

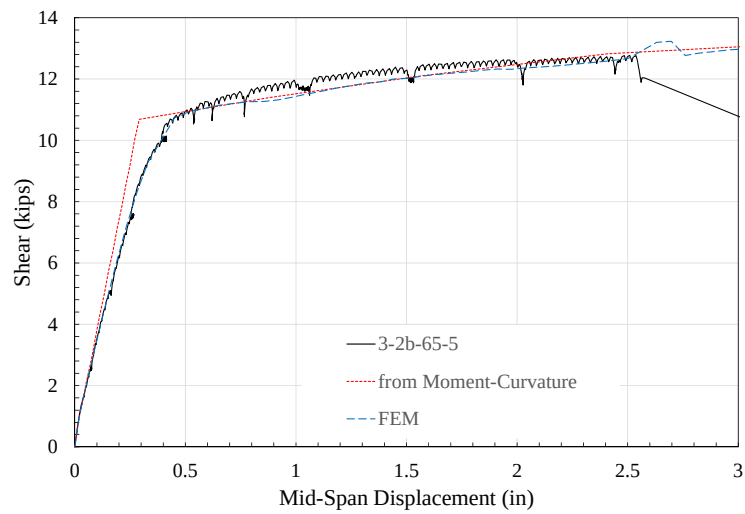


(c) Crack Pattern Comparison

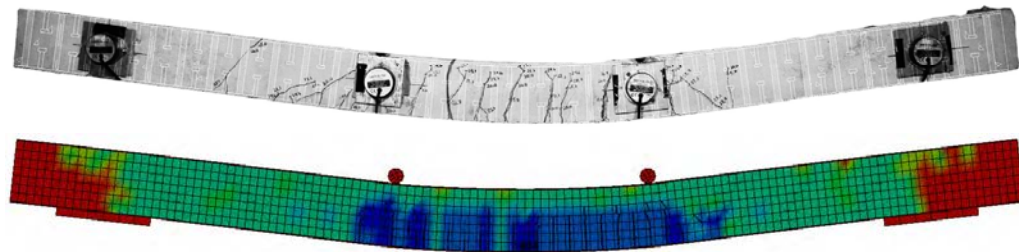
Figure 3.73. Comparison of Experimental to Model Results for 3-2-65-5



(a) Moment-Curvature Comparisons

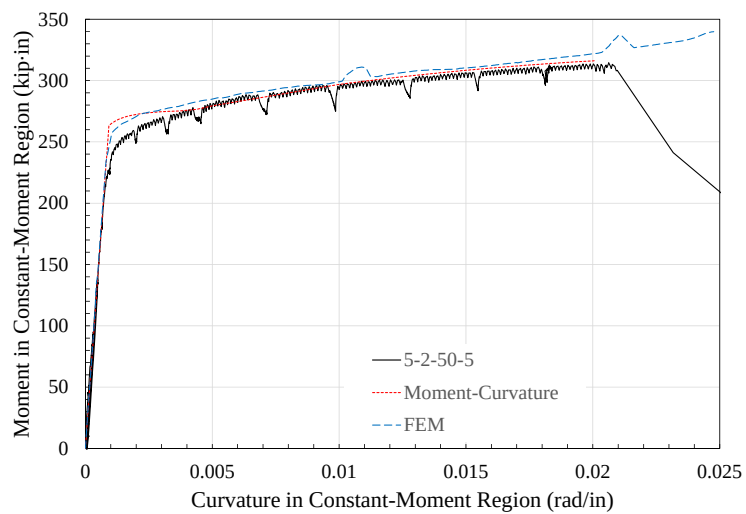


(b) Load-Displacement Comparisons

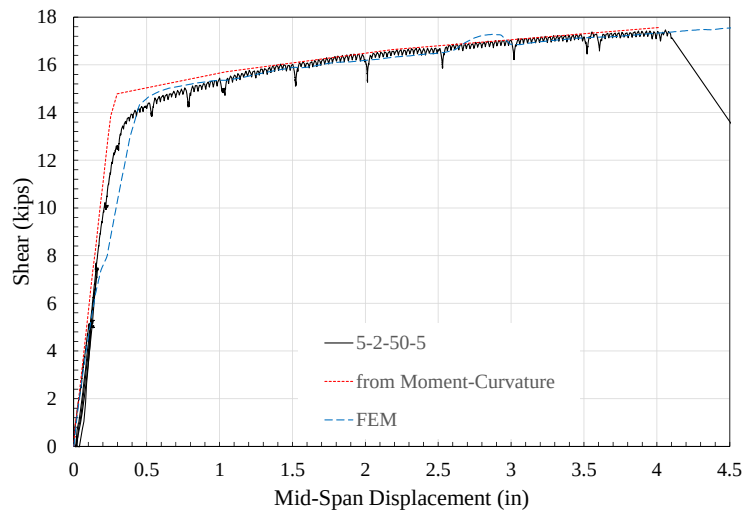


(c) Crack Pattern Comparison

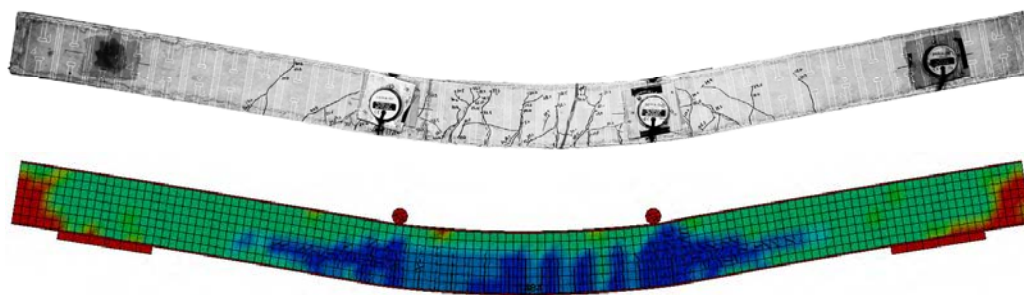
Figure 3.74. Comparison of Experimental to Model Results for 3-2b-65-5



(a) Moment-Curvature Comparisons

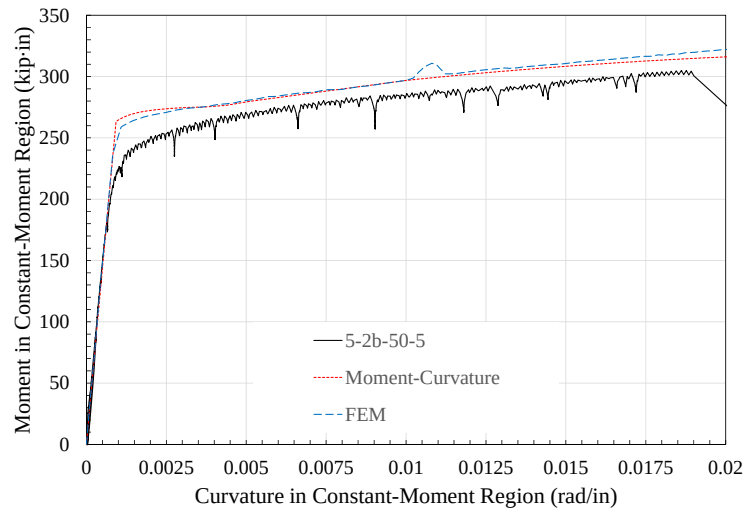


(b) Load-Displacement Comparisons

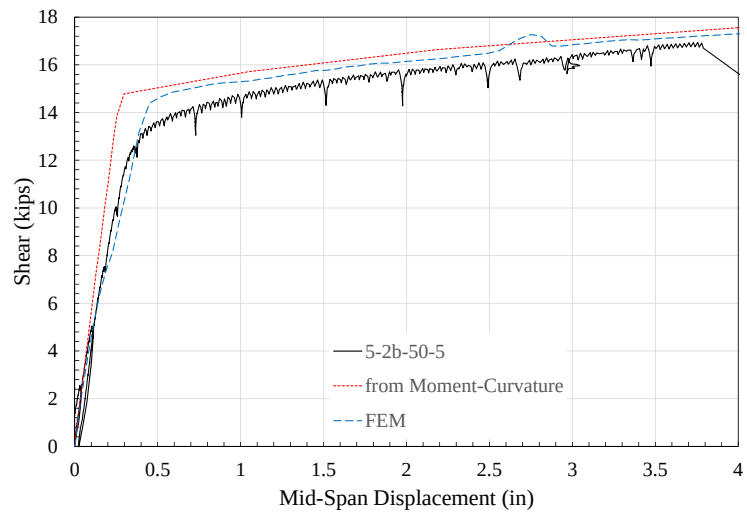


(c) Crack Pattern Comparison

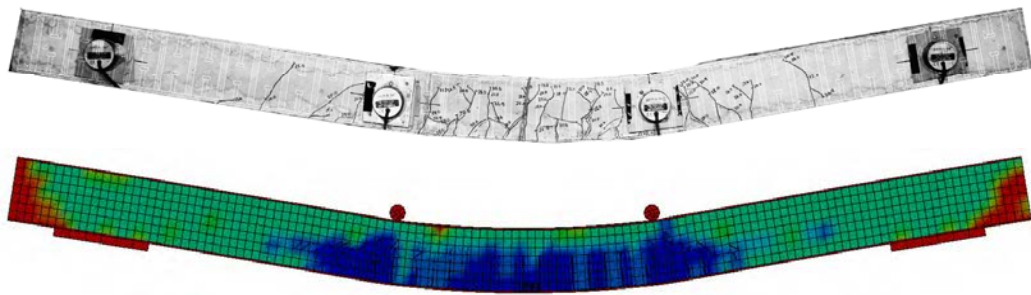
Figure 3.75. Comparison of Experimental to Model Results for 5-2-50-5



(a) Moment-Curvature Comparisons

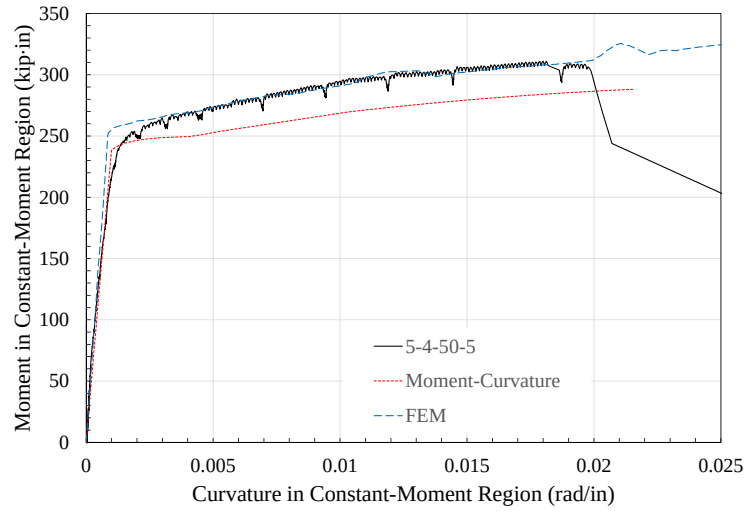


(b) Load-Displacement Comparisons

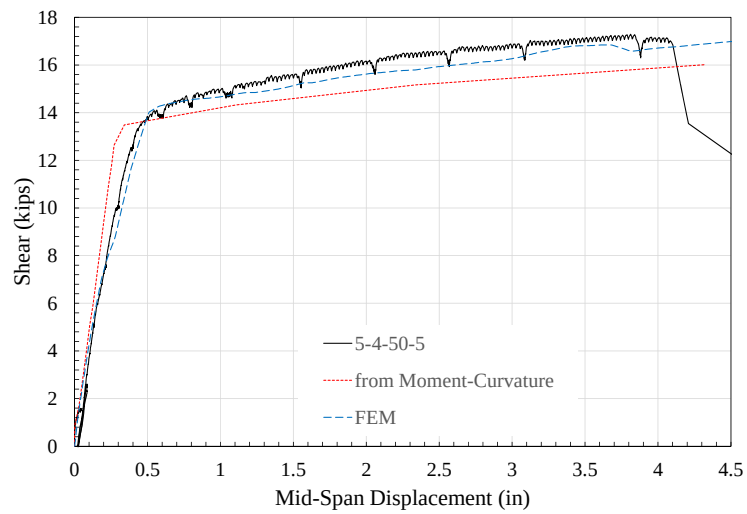


(c) Crack Pattern Comparison

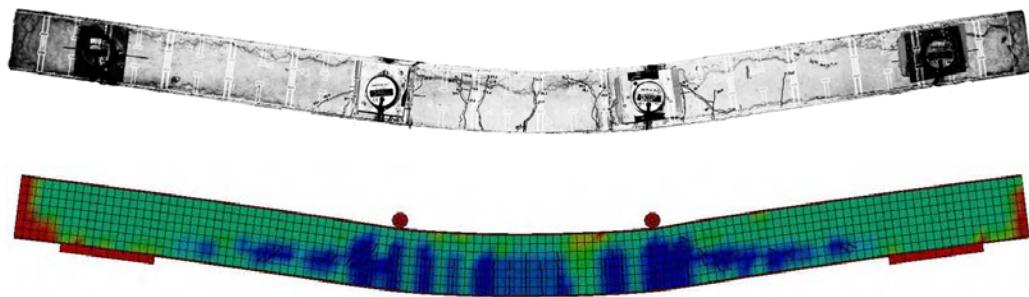
Figure 3.76. Comparison of Experimental to Model Results for 5-2b-50-5



(a) Moment-Curvature Comparisons

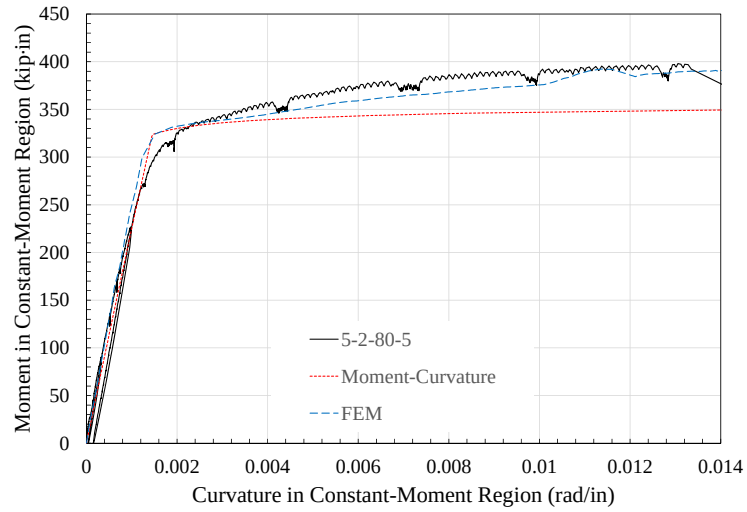


(b) Load-Displacement Comparisons

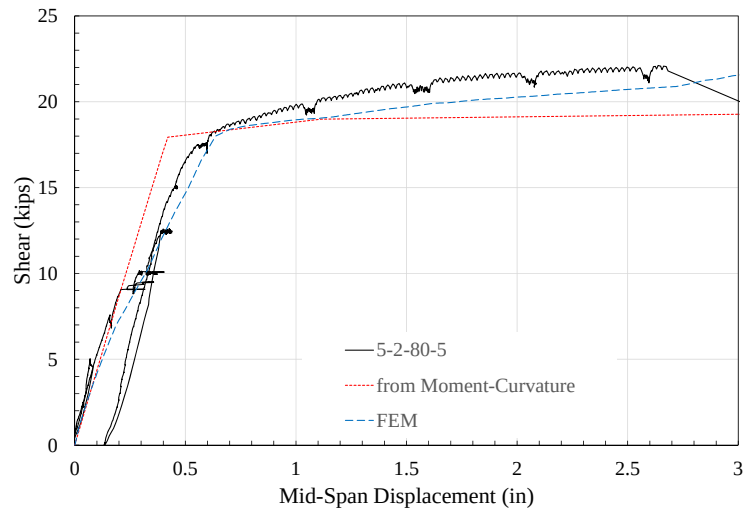


(c) Crack Pattern Comparison

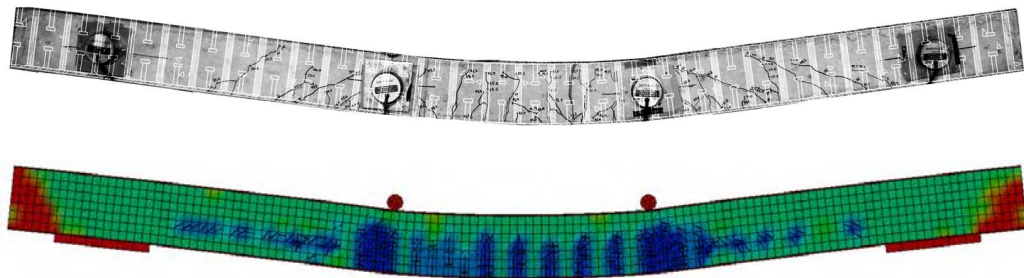
Figure 3.77. Comparison of Experimental to Model Results for 5-4-50-5



(a) Moment-Curvature Comparisons

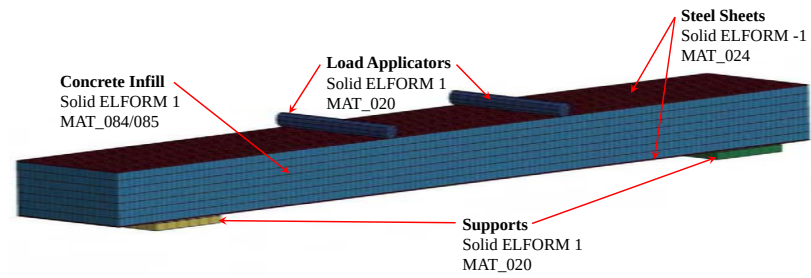


(b) Load-Displacement Comparisons



(c) Crack Pattern Comparison

Figure 3.78. Comparison of Experimental to Model Results for 5-2-80-5



(a) Complete Model



(b) Model With Concrete Removed

Figure 3.79. Static Test Specimens FE Model

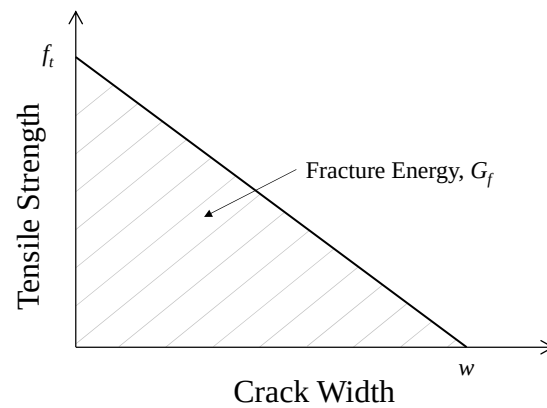
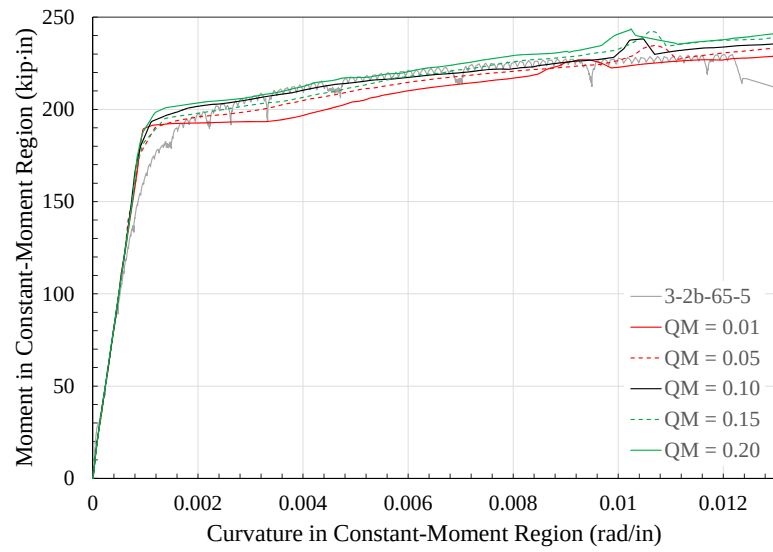
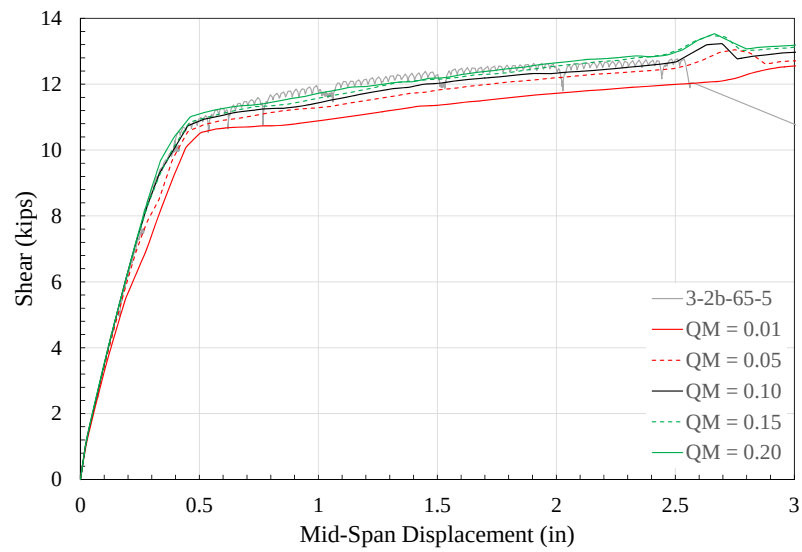


Figure 3.80. Tension Softening Model of Winfrith Concrete Model (Strain Rate Effects OFF)



(a) Moment-Curvature Comparisons



(b) Load-Displacement Comparisons

Figure 3.81. Sensitivity of Numerical Results to Hourglass Coefficient (QM)

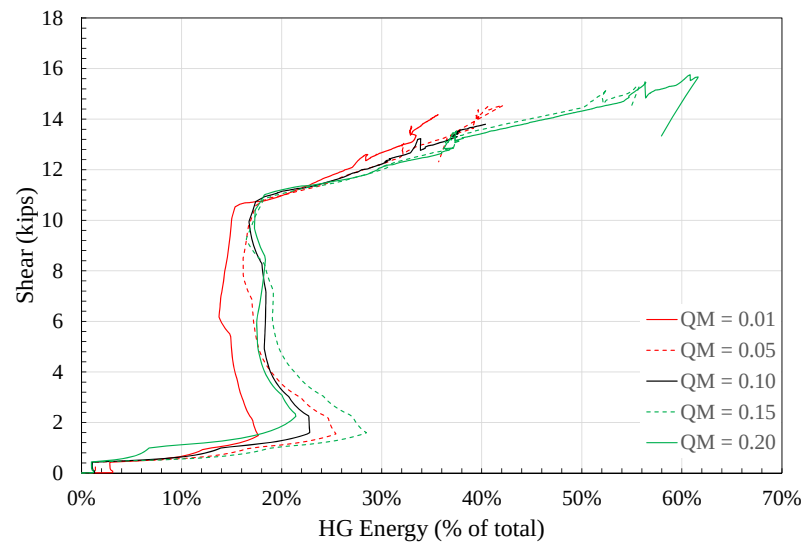
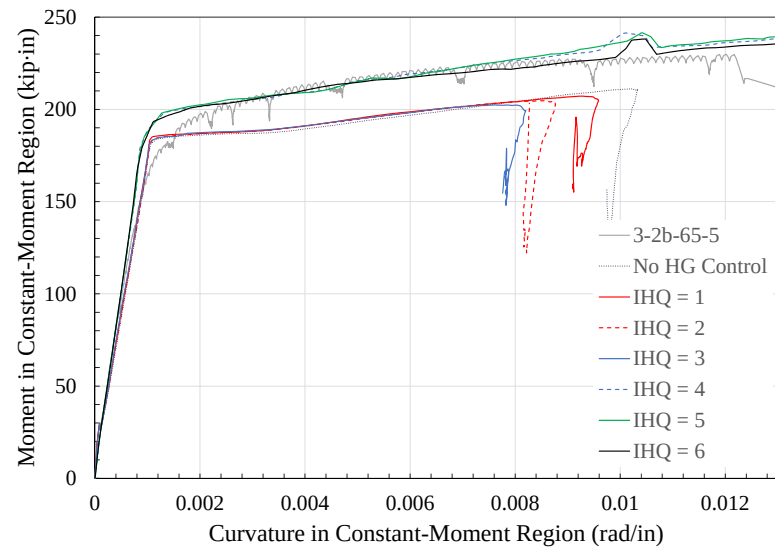
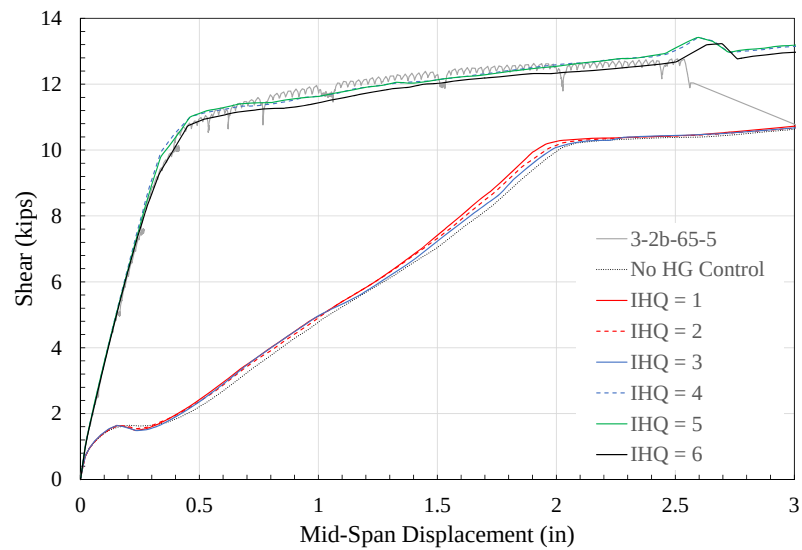


Figure 3.82. Influence of Hourglass Coefficient (QM) on Hourglass Energy



(a) Moment-Curvature Comparisons



(b) Load-Displacement Comparisons

Figure 3.83. Sensitivity of Numerical Results to Hourglass Control Type (IHQ)

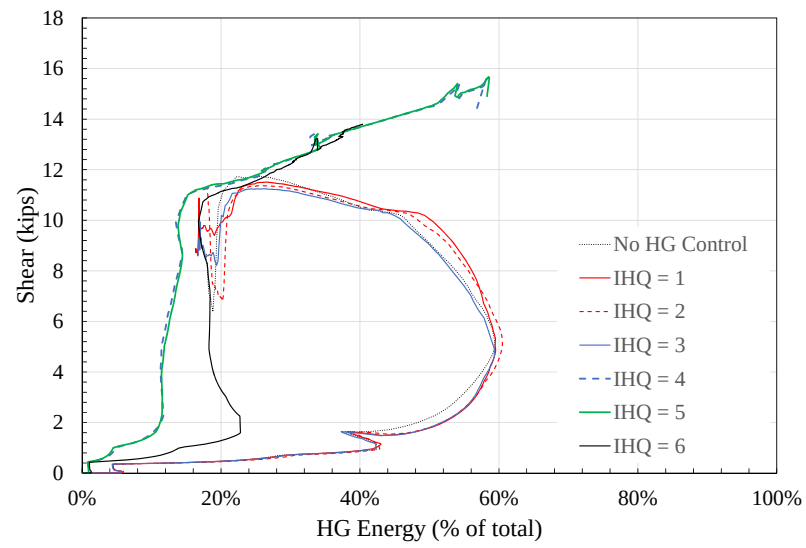
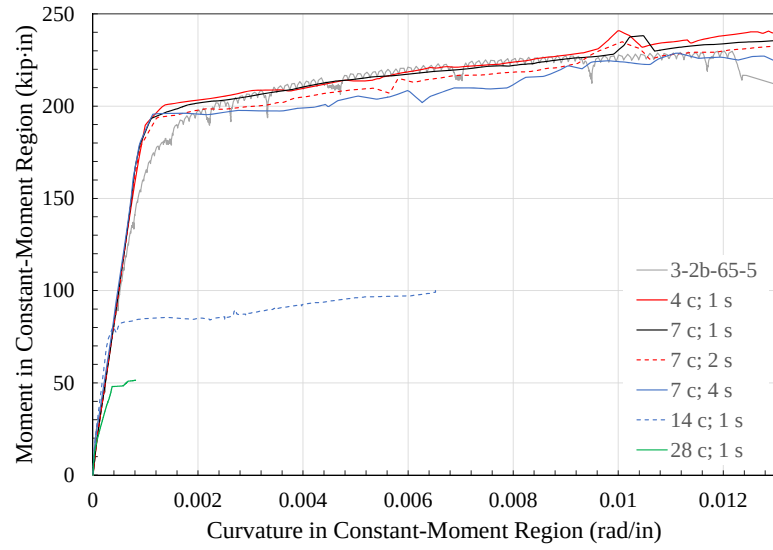
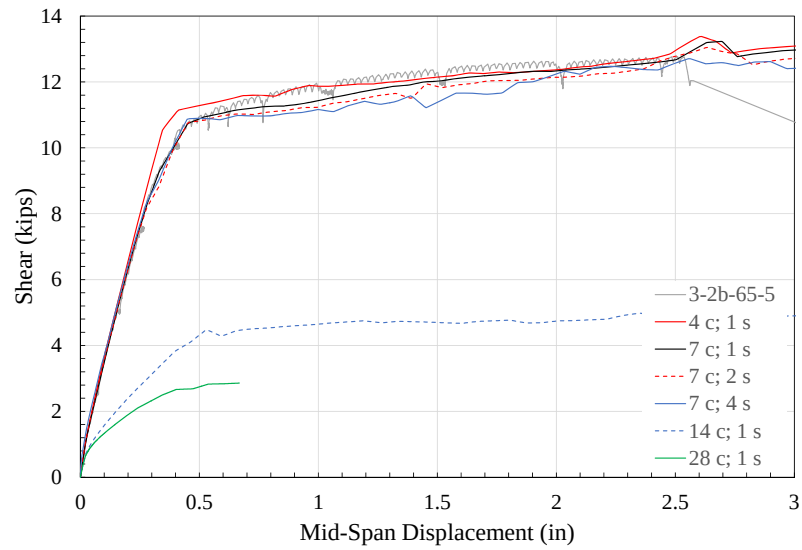


Figure 3.84. Influence of Hourglass Type (IHQ) on Hourglass Energy



(a) Moment-Curvature Comparisons



(b) Load-Displacement Comparisons

Figure 3.85. Sensitivity of Numerical Results to Mesh Size

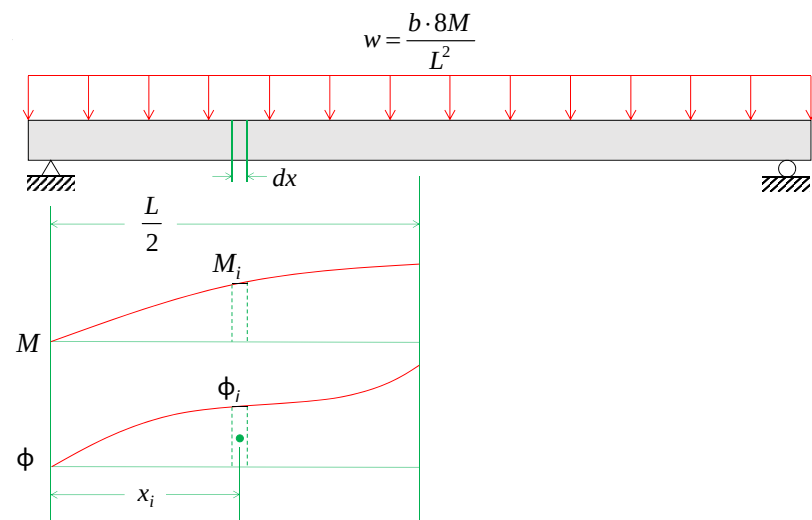
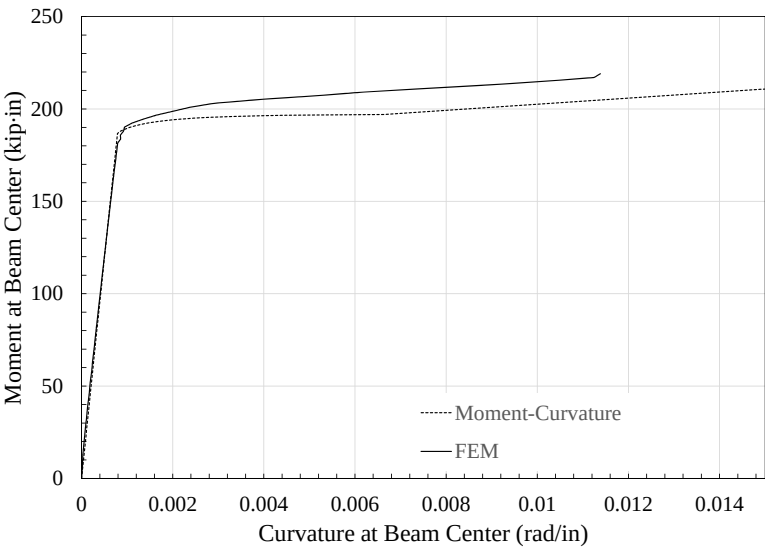
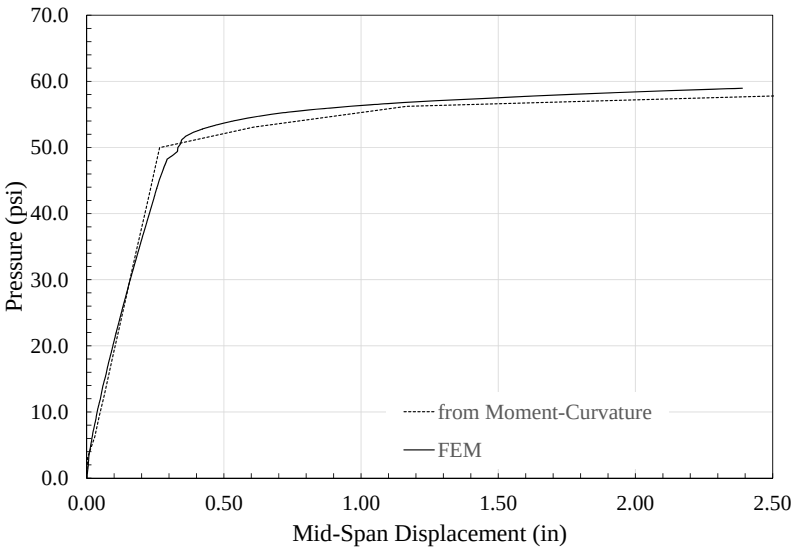


Figure 3.86. Computation of Mid-Span Displacement of Beam with Uniform Loading by Moment-Area Method

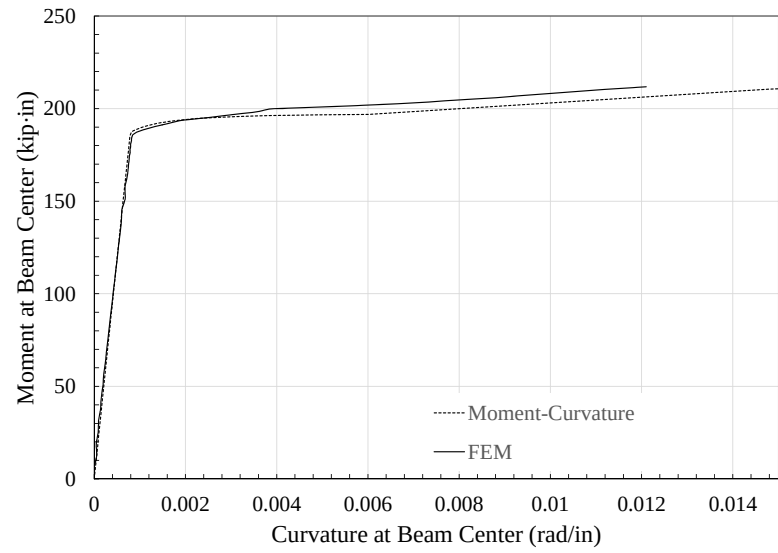


(a) Moment-Curvature Comparisons

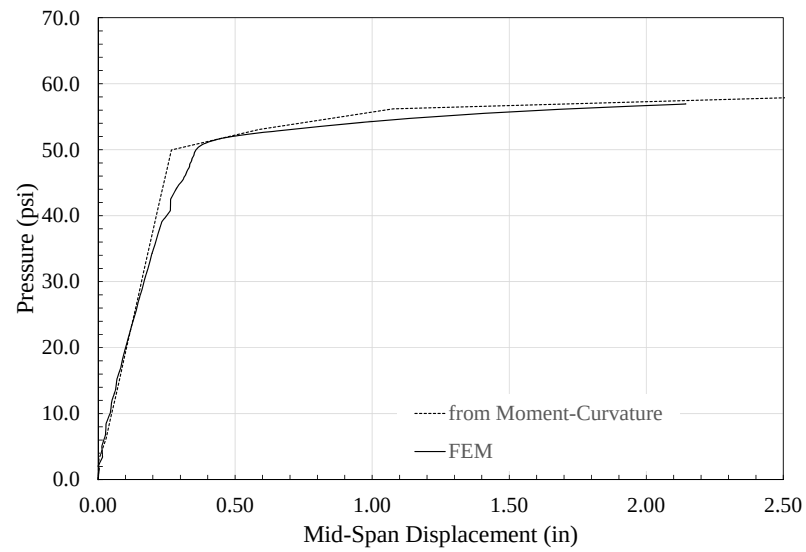


(b) Load-Displacement Comparisons

Figure 3.87. Uniform Load Case Model Results for 3-2-50-5

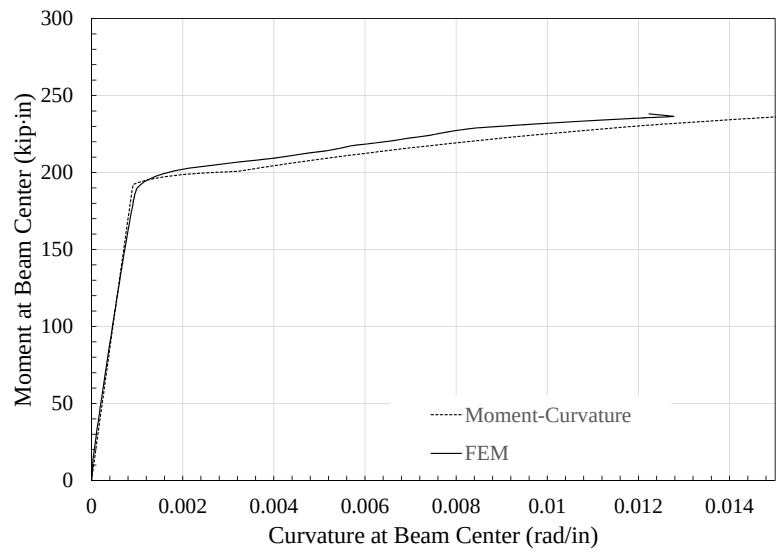


(a) Moment-Curvature Comparisons

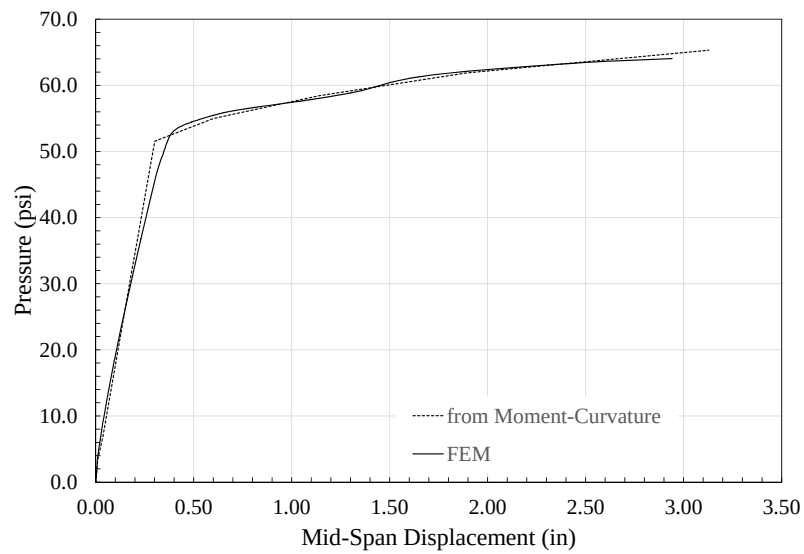


(b) Load-Displacement Comparisons

Figure 3.88. Uniform Load Case Model Results for 3-2b-50-5

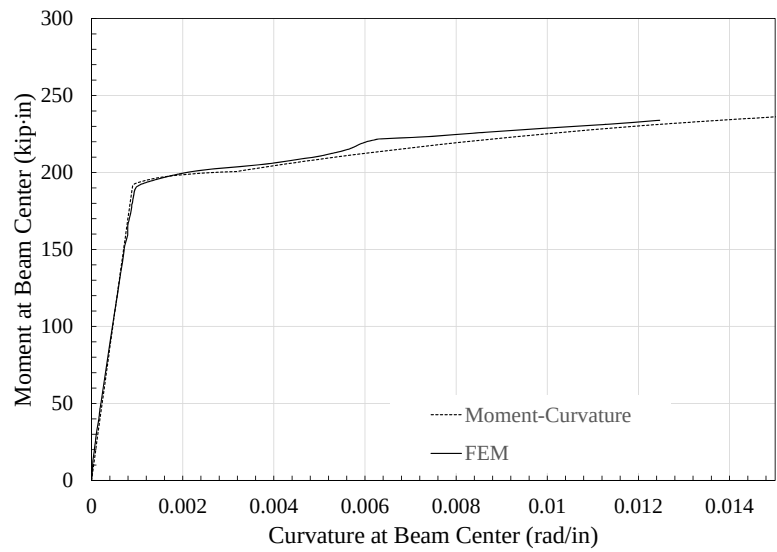


(a) Moment-Curvature Comparisons

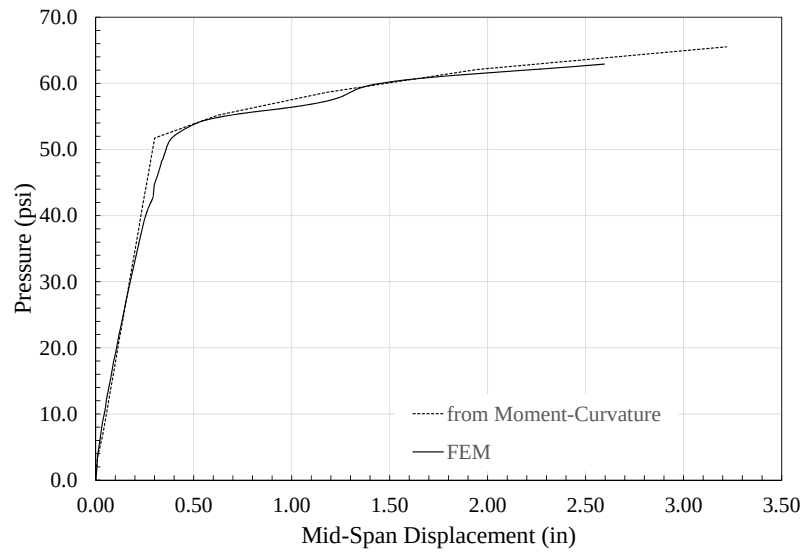


(b) Load-Displacement Comparisons

Figure 3.89. Uniform Load Case Model Results for 3-2-65-5

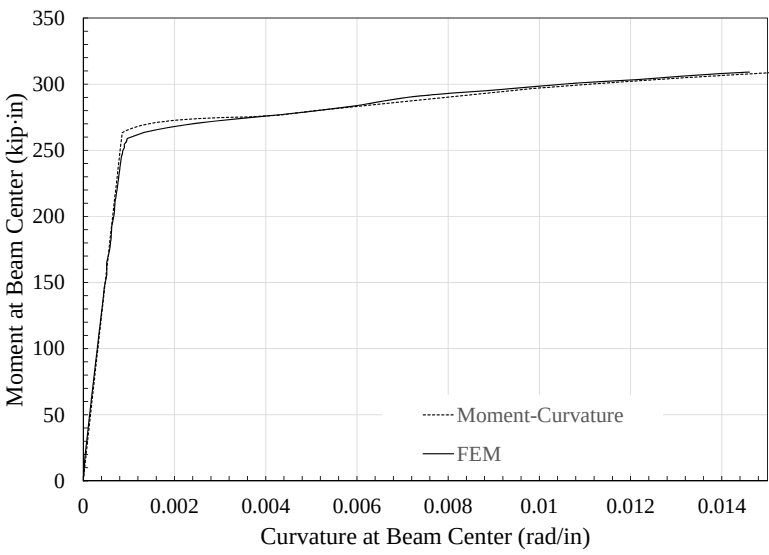


(a) Moment-Curvature Comparisons

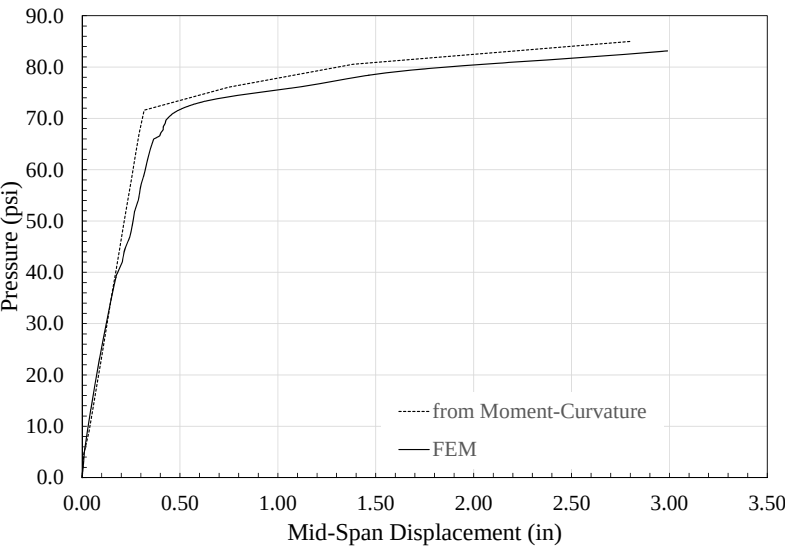


(b) Load-Displacement Comparisons

Figure 3.90. Uniform Load Case Model Results for 3-2b-65-5

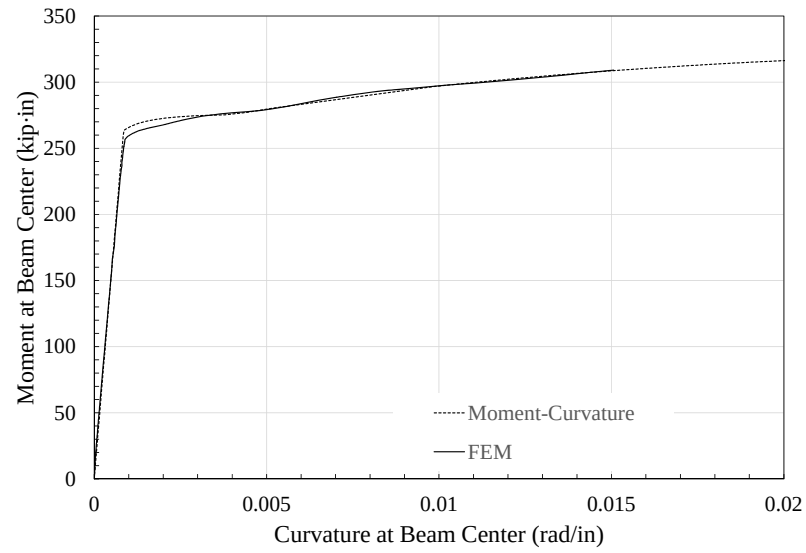


(a) Moment-Curvature Comparisons

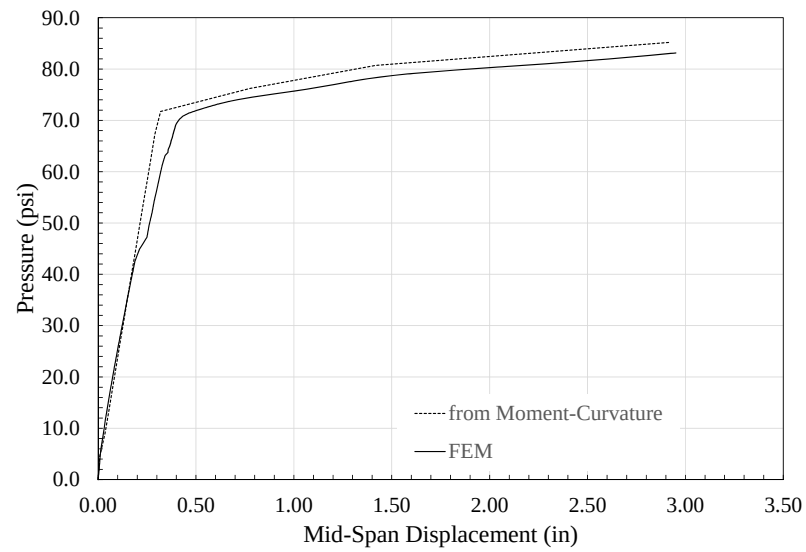


(b) Load-Displacement Comparisons

Figure 3.91. Uniform Load Case Model Results for 5-2-50-5

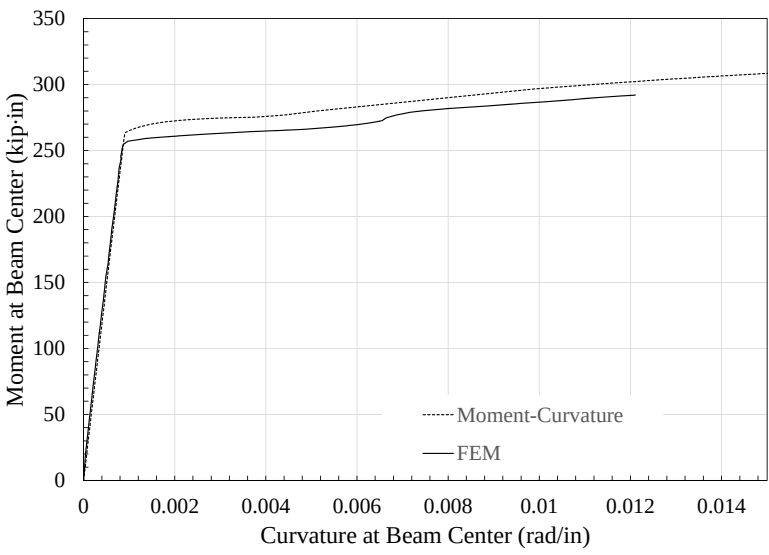


(a) Moment-Curvature Comparisons

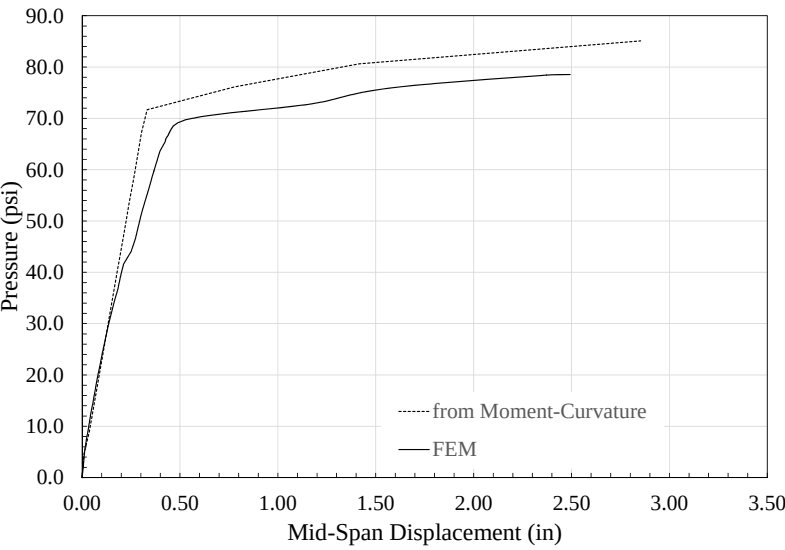


(b) Load-Displacement Comparisons

Figure 3.92. Uniform Load Case Model Results for 5-2b-50-5

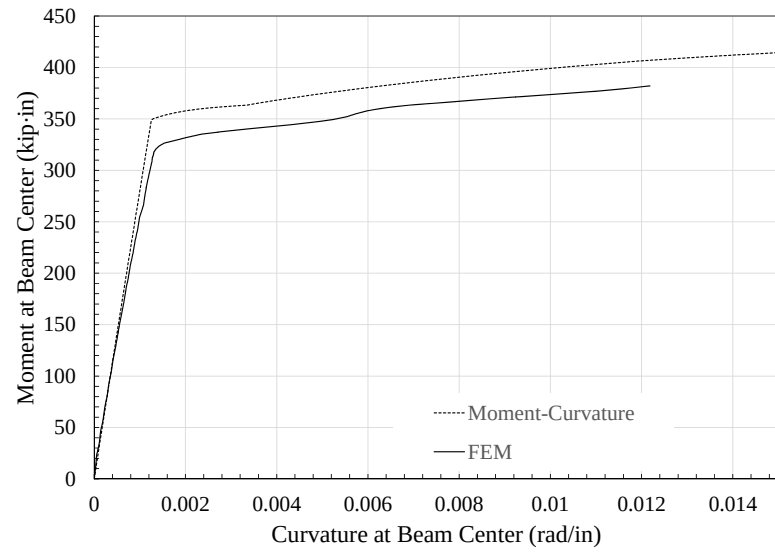


(a) Moment-Curvature Comparisons

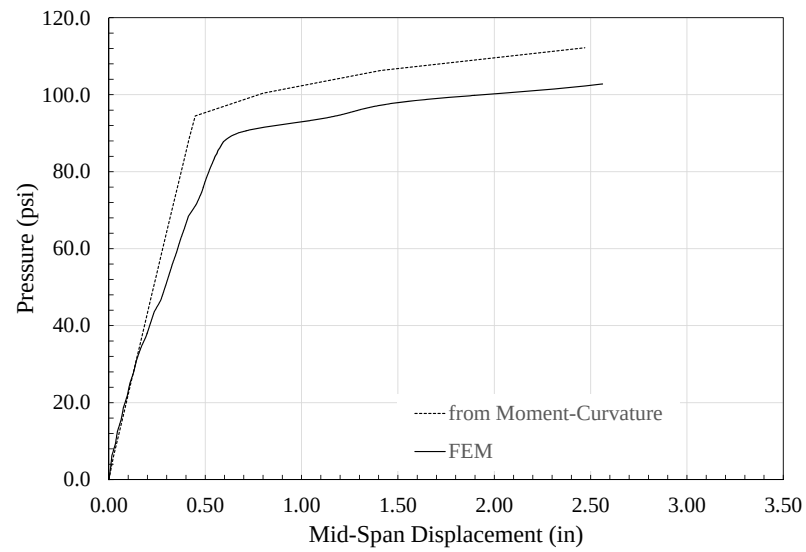


(b) Load-Displacement Comparisons

Figure 3.93. Uniform Load Case Model Results for 5-4-50-5



(a) Moment-Curvature Comparisons



(b) Load-Displacement Comparisons

Figure 3.94. Uniform Load Case Model Results for 5-2-80-5

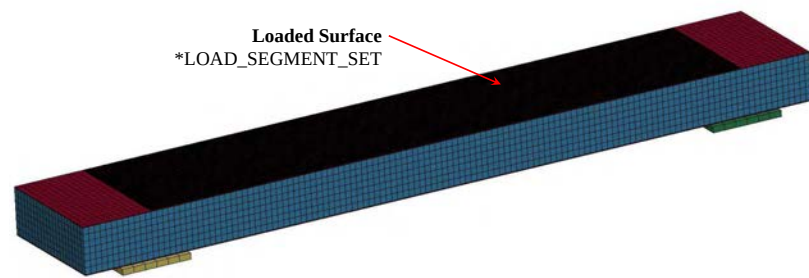


Figure 3.95. Loaded Surface for Uniform Load Case FE Models

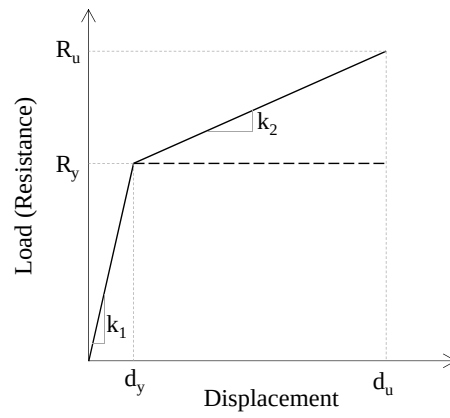


Figure 3.96. Idealized Static Resistance Function for Simply Supported One-Way SC Sections

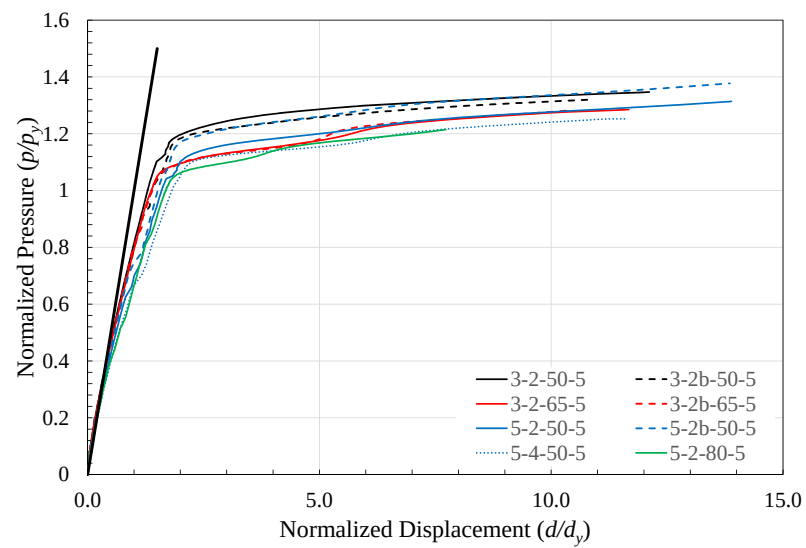


Figure 3.97. Normalized Comparison of Uniform Load Displacement Results

4. EXPERIMENTAL RESPONSE OF BLAST LOADED ONE-WAY SC WALL SECTIONS

Twelve panels were fabricated at Bowen Laboratory and tested using the Blast Load Simulator (BLS) at the USACE-ERDC in Vicksburg, MS. The panels consisted of various reinforcement ratios, steel plate strength, and tie bar spacing - identical to those tested statically as described in Chapter 3. Six panels were tested by subjecting them to two pressure pulses: the first pulse was selected to result in elastic response of the specimens and the second pulse was selected to cause plastic deformation. The other six panels were subjected to the largest pressure-impulse combination available from the BLS and were expected to deform plastically but not reach ultimate failure. The planned pressure-impulse combinations are listed in Table 4.1.

The BLS, pictured in Figure 4.1, is a compressed-gas-driven shock tube which can be tuned to simulate blast waveforms up to the equivalent of that generated by 50,000-lbs of TNT at a standoff distance of 250-ft. The BLS consists of a driver which is pressurized with a combination of helium and air, vented cone and transition components which shape the pressure wave, and a target vessel which supports the tested specimen and contains any debris [84]. When the pressurized gases reach the predetermined burst pressure, metal diaphragms placed between the driver and the vented cone are struck. The suddenly released gases expand and propagate toward the target vessel generating a pressure wave against the surface of the specimens. Vents in the vented cone allow reflected pressure waves to escape from the BLS and reduce the magnitude of any secondary pressure waves. The BLS assembly is housed in a protected structure and operated remotely from a control room.

Important benefits of the BLS over explosive field testing are: (1) the opportunity for extensive instrumentation of the specimens and (2) the absence of a fireball and

dust cloud. High-speed cameras can be protected in the BLS facility and there is nothing to obscure video recordings of the panel response to the pressure pulse. Tests conducted using the BLS tend to be less expensive than field testing. The current BLS has two important limitations: (1) tested specimens cannot exceed 71-in x 53-in and (2) only generally uniform pressure distributions can be created making it suitable only for simulating far-field explosions. USACE-ERDC is currently constructing a larger BLS which will be able to test larger, full-size, wall specimens.

4.1 Blast Tests of SC Wall Sections

4.1.1 Specimen Design and Material Properties

The same specimen designs used for the static tests were used for the blast tests. Design of these specimens was described in Chapter 3 (see Table 3.1 and Figure 3.2). Duplicates of four configurations were created for a total of twelve specimens. See Table 4.2 for a list of the twelve specimens.

Steel sheets from the same heats as for the static test specimens were used for the blast specimens (Tables 3.3-3.5 and Figures 3.3-3.6). Tie bars from the same order as the static tests were used (Table 3.4 and Figure 3.7). Studs from heat 10324140 were used for all the blast specimens (see Tables 3.6 and 3.7). The same concrete mix was ordered for the blast specimens as for the static specimens. Table 4.3 provides the delivered mix proportions for the blast specimens. The supplier delivered a nearly identical mix as for the static specimen (compare Table 4.3 to Table 3.2).

4.1.2 Test Setup

Through consultation with engineers who operated the BLS, previously calibrated pressure-impulse combinations were chosen to deliver pressure waves to meet the desired intent of each test. Prior to any tests of specimens, calibration shots were com-

pleted to confirm the desired blast waveform was produced. Twelve Kulite Model HKS-11-375SG pressure gauges located at various positions on a semi-rigid plate as shown in Figure 4.2 recorded pressure time-histories for each shot. One calibration shot at a driver-pressure vessel internal pressure of 1443-psi and two shots with internal pressure of 400-psi were completed. Measured pressure and calculated impulse values for these shots are provided in Table 4.4. These shots served two purposes: (1) confirm the desired pressure-impulse combination was achieved and (2) confirm the pressure distribution was approximately uniform across the face of the specimens. A comparison of data in Table 4.4 to the planned shots in Table 4.1 demonstrates that the first purpose was achieved. An examination of the variation in measured pressures and impulses in Table 4.4 demonstrates that the second purpose was achieved.

Three specimens were tested simultaneously during each of four tests in the BLS. Figure 4.3 depicts the setup process. The test frame consisted of internal and external portions. With the internal frame installed within the target vessel opening the three specimens were put in place with cardboard spacers at the top and bottom of the specimens to allow free end rotation during the tests. Plastic horseshoe shims were placed between the specimens to maintain $\frac{1}{4}$ -in spacing between specimens. With the specimens in place, the external frame was installed and bolted through the internal frame to the target vessel. Additional plastic horseshoe shims were used to maintain $\frac{1}{4}$ -in spacing between the specimens and the external frame. To restrict pressure flow around the frame and panels into the target vessel, X-flex® and duct-tape was used to seal the openings. The internal frame was only installed once - prior to the first test. After each test, the external frame and tested specimens were removed. The next set of specimens was then positioned and the external frame reinstalled.

The layout and type of sensors was the same for each of the four tests as shown in Figure 4.4. Eight Kulite Model HKS-11-375SG pressure gauges positioned around the perimeter of the frame recorded the pressure-time history for each shot (P1-P6, PD, and PE). Not shown in the figure is a ninth pressure gauge (IP1) located inside the target vessel to record the pressure on the non-blast side of the specimens. Lateral

mid-span deflection of each specimen was recorded using Bourns Model 3547 3-turn precision potentiometers (D1R, D2R, and D3R). As an alternate, back-up measure, mid-span lateral deflection was also calculated from acceleration measurements of each specimen recorded from Endevco Model 7270A accelerometers (A1, A2, and A3). A third deflection measurement was recorded at a point 8.75-in away from the mid-point using Celesco Model PT5A potentiometric cable-extension transducers (D1, D2, and D3). Strains were measured using Tokyo Sokki Kenkyujo YEFLA-5-5LT 120 Ω 5-mm strain gages. One strain gage (SG1) was centered at mid-span on the compression (blast) face of each specimen. Three strain gages (SG2, SG3, and SG4) were placed at the quarter-points of the span centered on the tension (non-blast) face of each specimen. The suffix L, C, and R for each strain gage shown in Figure 4.4 correspond to the left, center, and right specimens, respectively, as viewed from inside the target vessel.

Data from all sensors was transmitted over 22 AWG 6 conductor shielded mil-spec cable and recorded at 1-MS/s on a Pacific Instruments Model 5810 data acquisition system. For the first two shots (Test 1), the acquisition system was triggered with an audible trigger sending a voltage rise to the data acquisition system which recorded 250-msec pre-trigger and 800-msec post-trigger. This trigger was susceptible to early activation if the striker did not puncture the diaphragms: this occurred for shot 2 resulting in no data recorded for this test. To prevent this from happening in subsequent tests, the trigger was changed to activate the data acquisition system on a pressure increase registered by two pressure gages and recorded the same pre- and post-trigger times. This trigger method worked without error for the remaining four shots.

Two Phantom v7.3 and one Phantom v9.1 high-speed video cameras were used to document the specimen responses (left, right, and rear angles) and pre- and post-test still photos were taken. Each specimen was weighed prior to being loaded in the target vessel. Residual displacements were measured prior to removing specimens from the target vessel. After tested specimens were removed from the frame, cracks

along the sides were marked and photographed. Tests 1 and 2 consisted of two shots each. Specimens were photographed and residual displacement measurements were made with the panels in place after the first shot to avoid disturbing the specimens. Therefore, no cracks were marked after the first shot for each of the first two tests. Steel sheets were not removed from the specimens, therefore crack patterns on the tension and compression faces were not observed.

4.1.3 Blast Test Results

Measured pressures and calculated impulses for each of the six shots are provided in Table 4.5. Representative pressure data from one pressure sensor for each shot is provided in Figures 4.5-4.9 (no data was recorded for Shot 2 so there is no figure of pressure data for this shot). Maximum deflection values from each of the sensors for each of the shots are provided in Table 4.2. Residual displacement values are provided for those sensors from which meaningful data was possible. Residual deflected shapes of the specimens were measured by hand prior to specimens being removed from the frame. During the tests, some sensors malfunctioned or detached from the specimens as noted in Table 4.2.

4.1.3.1 Test 1

Three specimens were loaded in the BLS for Test 1, from left to right (as appeared inside the target vessel, see Figure 4.10): 3-2b-50-5(1), 3-2-50-5(1), and 5-4-50-4(1). Test 1 consisted of two shots. The first shot had a driver pressure of 400-psi which resulted in 20.2-psi peak pressure and 261-psi·msec impulse. During the second shot at a driver pressure of 1400-psi the striker was released. This activated the audible trigger but the diaphragms did not rupture. As pressure was being released from the pressure vessel the diaphragms ruptured suddenly at a driver pressure of 1388-psi. This occurred before the data acquisition system could be reset: no data was recorded for this test. The high-speed video cameras were reset in time, however,

and video recordings of the test were captured. From these recordings, the mid-span displacement-time history was constructed using Phantom Camera Control Application software (PCC 2.5). Because driver pressure when rupture occurred was close to the pressure from shot 5 (Test 3) it was assumed that the applied pressure and impulse for shot 2 was similar.

There was approximately 0.28-in of frame settlement during Shot 1 which is evident from still shots of the high speed video (see Figure 4.11). Frame displacement and settlement were measured from high speed videos using PCC 2.5 software. Frame displacement resulted in larger measured deflections recorded by the sensors than actually experienced by the specimens.

Raw displacement and strain gage data from Shot 1 for the three specimens are shown in Figures 4.12, 4.13, and 4.14. A 2-pass bandpass filter with a 20kHz upper limit was applied to cable extensometer and strain gage data to remove noise. As seen in the figures, there was more noise in some channels than others - the same filter was applied to all regardless of the amount of noise in the signal. No filter was applied to accelerometer data. Accelerometer data was integrated twice to obtain velocity and displacement. These calculated curves are shown with the accelerometer data in Figures 4.12(a), 4.13(a), and 4.14(a). There was no signal reference set for the potentiometers during this shot so there was no useful data from these sensors.

Maximum measured mid-span displacements from accelerometer data for shot 1 were 0.473-in, 0.421-in, and 0.559-in for 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1) respectively. The panels had negligible residual displacement although data from the cable extensometers suggested otherwise. Measured residual displacements of the specimens in the frame post-test were 0.048-in, 0.044-in, and 0.026-in and deflected shapes for the specimens are shown graphically in Figure 4.15. There was approximately 0.28-in of settlement of the internal frame during this shot. Figures 4.16, 4.17, and 4.18 show the displacements measured by both sensors. Note that cable extensometers were located 8.75-in away from mid-span and therefore were expected to register smaller displacements than those calculated at mid-span from accelerome-

ter data. Due to numerical errors associated with integrating the acceleration curve twice, only the peak displacement value was meaningful but the full transient response measured by the cable extensometers was valuable. When corrected for frame displacement, maximum mid-span displacements were 0.182, 0.130, and 0.268 for 3-2-50-5(1), 3-2b-50-5(1) , and 5-4-50-5(1).

Filtered strain gage data for all strain gages on each specimen is shown in Figures 4.19, 4.20, and 4.21. The two strain gages at the non-midspan quarter-points (SG2 and SG4) were close in value for each of the three panels which denotes symmetric bending. These figures also demonstrate that these three specimens remained elastic during shot 1 as the largest strain registered by any gage was $1000\text{-}\mu\epsilon$ in tension and $500\text{-}\mu\epsilon$ in compression. There was excessive noise in SG4 for specimen 5-4-50-5(1) which reduces the clarity of Figure 4.21. Specimen 5-4-50-5(1) was the only specimen with tie bars of adequate diameter to affix strain gages (without worry of excessive reduction of the cross-section due to preparation of the surface). As Figures 4.14(g) and 4.14(h) show, these gages recorded negligible strains.

Still shots from high speed video of Shot 2 are provided in Figure 4.22As mentioned previously, the data acquisition system did not trigger for Shot 2. Therefore, there are no figures of raw or filtered data for the specimens from this shot. Maximum measured mid-span displacements from high speed video analysis for shot 2 were 0.948-in, 0.959-in, and 0.535-in for 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1) respectively. Because these measurements were obtained from the video record, they did not require correction due to frame settlement. Residual mid-span displacements measured on the panels in the frame post-shot were 0.185-in, 0.161-in, and 0.029-in. Residual deflected shapes for the specimens are shown graphically in Figure 4.23.

Cracks were not observed after the first shot because the specimens remained in the frame. Specimens were removed after the second shot and cracks marked and photographed (see Figure 4.24). All three specimens had flexure cracks with the longest cracks located at midspan. The longest cracks extended from the tension face to within 0.25-in of the compressive face of the specimen and crack length reduced

the further away from midspan the crack appeared. No concrete crushing was evident. Figure 4.24(a) shows that the post-test crack pattern for 3-2-50-5(1) consisted exclusively of flexure cracks. Figure 4.24(b) shows that the post-test crack pattern for 3-2b-50-5(1) was similar to that for 3-2-50-5(1) with the exception of a flexural shear crack near the quarter-point. Figure 4.24(c) shows that the post-test crack pattern for 5-4-50-5(1) had two important differences: (1) long flexure cracks further away from mid-span and (2) a flexure crack originated at the blast-face and extended about $3/4$ of the way through the specimen. This flexure crack at the support suggests that the supports provided more constraint than idealized.

4.1.3.2 Test 2

Three specimens were loaded in the BLS for Test 2, from left to right (as appeared inside the target vessel, see Figure 4.25): 5-2-50-5(1), 5-2b-50-5(1), and 3-2-65-5(1). Test 2 consisted of two shots. The first shot had a driver pressure of 400-psi which resulted in 18.9-psi peak pressure and 260-psi-msec impulse. The second shot had a driver pressure of 1420-psi which resulted in 61.6-psi peak pressure and 1068-psi-msec impulse.

Frame settlement from shots 1 and 2 remained at the start of shot 3 as evident in Figure 4.26(a). An additional 0.15-in of frame displacement occurred during shot 3 and 0.23-in during Shot 4. Neither shot had any measurable frame settlement. Frame displacement and settlement were measured from high speed videos using PCC 2.5 software. Frame displacement resulted in larger measured deflections recorded by the sensors than actually experienced by the specimens.

Raw displacement and strain gage data from Shot 3 for the three specimens are shown in Figures 4.27, 4.28, and 4.29. A 2-pass bandpass filter with a 20kHz upper limit was applied to the cable extensometer and strain gage data to remove noise. As seen in the figures, there was more noise in some channels than in others - the same filter was applied to all regardless of the amount of noise in the signal. No

filter was applied to the accelerometer or potentiometer data. Accelerometer data was integrated twice to obtain velocity and displacement. These calculated curves are shown with the accelerometer data in Figures 4.27(a), 4.28(a), and 4.29(a).

Maximum measured mid-span displacements measured by the potentiometers for shot 3 were 0.284-in, 0.318-in, and 0.250-in for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1) respectively. Maximum measured mid-span displacements from accelerometer data for shot 3 were 0.298-in, 0.307-in, and 0.266-in for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1) respectively. There was negligible residual displacement of the specimens in the frame post-test (0.056-in, -0.003-in, and -0.007-in) which are similar to residual displacements measured by the Bourns potentiometers. Residual deflected shapes for the specimens are shown graphically in Figure 4.30. Figures 4.31, 4.32, and 4.33 show displacements measured by all three sensors. Note that cable extensometers were located 8.75-in away from mid-span and therefore expected to register smaller displacements than those calculated at mid-span from accelerometer data. Due to numerical errors associated with integrating the acceleration curve twice, only the peak displacement value was meaningful but the full transient response measured by the potentiometers and cable extensometers were valuable. There was good agreement between the displacement calculated from accelerometer data and that from the potentiometer for all three specimens in Shot 3. When corrected for frame displacement, maximum mid-span displacements were 0.130, 0.164, and 0.096 for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1).

Filtered strain gage data for all strain gages on each specimen is shown in Figures 4.34, 4.35, and 4.36. The two strain gages at the non-midspan quarter-points (SG2 and SG4) were close in value for each of the three panels which denotes symmetric bending. These figures also demonstrate that these three specimens remained elastic during shot 3 as the largest strain registered by any gage was $1000\text{-}\mu\epsilon$ in tension and $500\text{-}\mu\epsilon$ in compression. SG1 for specimen 5-2-50-5(1) malfunctioned and is not included Figure 4.35.

Raw displacement and strain gage data from Shot 4 for the three specimens are shown in Figures 4.38, 4.39, and 4.40. A 2-pass bandpass filter with a 20kHz upper limit was applied to the cable extensometer and strain gage data to remove noise. As seen in the figures, there was more noise in some channels than in others - the same filter was applied to all regardless of the amount of noise in the signal. No filter was applied to the accelerometer or potentiometer data. Accelerometer data was integrated twice to obtain velocity and displacement. These calculated curves are shown with the accelerometer data in Figures 4.38(a), 4.39(a), and 4.40(a).

Maximum measured mid-span displacements measured by the potentiometers for shot 4 were 0.677-in and 0.761-in for 5-2-50-5(1) and 5-2b-50-5(1) respectively. Maximum measured mid-span displacements from accelerometer data for shot 4 were 1.293-in, 0.493-in, and 0.726-in for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1) respectively. The potentiometer detached from 3-2-65-5(1) during the test. Measured residual displacements of the specimens in the frame post-test were 0.449-in, 0.097-in, and 0.049-in. Residual displacements demonstrated that the pressure-impulse of shot 4 exceeded the elastic capacity of all three specimens. Residual deflected shapes for the specimens are shown graphically in Figure 4.41. Figures 4.42, 4.43, and 4.44 show displacements measured by all three sensors. Note that cable extensometers were located 8.75-in away from mid-span and therefore expected to register smaller displacements than those calculated at mid-span from accelerometer data. Due to numerical errors associated with integrating the acceleration curve twice, only the peak displacement value was meaningful but the full transient response measured by the potentiometers and cable extensometers were valuable. The potentiometer connected to 3-2-65-5(1) detached from the specimen during the shot and is not included in Figure 4.42. There is good agreement between the displacement calculated from accelerometer data and that from the potentiometer for 5-2-50-5(1). The displacement calculated from the accelerometer for 5-2b-50-5(1) is about 75% of that measured by the potentiometer and nearly equal to that measured by the cable extensometer. Given the distance between the extensometer and the accelerometer, the data from the potentiometer

is deemed most accurate for mid-span displacement for 5-2b-50-5(1). When corrected for frame displacement, maximum mid-span displacements were 1.059, 0.443, and 0.527 for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1) respectively as tabulated in Table 4.2.

Filtered strain gage data for all strain gages on each specimen is shown in Figures 4.45, 4.46, and 4.47. The two strain gages at the non-midspan quarter-points (SG2 and SG4) were close in value for each of the three panels which denotes symmetric bending. These figures also demonstrate that the tension steel plates in these three specimens exceeded their elastic limits during Shot 4. This is corroborated by the residual tensile strain measured by SG3 for all three specimens. These sets of strain gage data, particularly that for 3-2-65-5(1) (Figure 4.45), capture the transient vibration of each specimen and show that damping stopped the vibration after five or six cycles.

Cracks were not observed after the first shot because the specimens remained in the frame. Specimens were removed after the second shot and cracks marked and photographed (see 4.48). All three specimens had flexure cracks with the longest cracks located at midspan. The longest flexure cracks extended from the tension face to within 0.10-in of the compressive face of the specimen and crack length reduced the further away from midspan the crack appeared. No concrete crushing was evident. All three specimens also exhibited flexural shear cracks. Figure 4.48(a) shows that the post-test crack pattern for 3-2-65-5(1) consisted primarily of flexure cracks with one flexural shear crack appearing near the quarter point of the specimen. Figure 4.48(b) shows that the post-test crack pattern for 5-2-50-5(1) had wider spacing than for 3-2-65-5(1) but also consisted primarily of flexure cracks and a single visible flexural shear crack near a support. Figure 4.48(c) shows that the post-test crack pattern for 5-2b-50-5(1) was similar to 5-2-50-5(1) with the exception that flexural shear cracks appeared near mid-span.

4.1.3.3 Test 3

Three specimens were loaded in the BLS for Test 3, from left to right (as appeared inside the target vessel, see Figure 4.49): 5-2-80-5(1), 3-2-50-5(2), and 3-2b-50-5(2). Test 3 consisted of a single shot (shot 5) with a driver pressure of 1420-psi which resulted in 59.9-psi peak pressure and 1032-psi·msec impulse.

Frame settlement from previous shots remained at the start of shot 5 as evident in Figure 4.50(a). The frame displaced by a maximum 0.30-in during Shot 5 but had no measurable additional settlement. Frame displacement and settlement were measured from high speed videos using PCC 2.5 software. Frame displacement resulted in larger measured deflections recorded by the sensors than actually experienced by the specimens.

Raw displacement and strain gage data from Shot 5 for the three specimens are shown in Figures 4.51, 4.52, and 4.53. A 2-pass bandpass filter with a 20kHz upper limit was applied to the cable extensometer and strain gage data to remove noise. As seen in the figures, there was more noise in some channels than in others - the same filter was applied to all regardless of the amount of noise in the signal. No filter was applied to the accelerometer or potentiometer data. Accelerometer data was integrated twice to obtain velocity and displacement. These calculated curves are shown with the accelerometer data in Figures 4.51(a) and 4.53(a). The accelerometer malfunctioned on specimen 3-2b-50-5(2), therefore no accelerometer data is shown for this specimen.

Maximum measured mid-span displacements measured by the potentiometers for shot 5 were 1.460-in, 1.450-in, and 0.720-in for 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1) respectively. Maximum measured mid-span displacements from accelerometer data for shot 5 were 0.462-in and 0.703-in for 3-2-50-5(2) and 5-2-80-5(1) respectively (the accelerometer detached from 3-2b-50-5(2)). Measured residual displacements were between 0.40-in and 0.50-in for 3-2-50-5(2) and 3-2b-50-5(2) demonstrating the pressure-impulse of shot 5 exceeded the elastic capacity of these specimens. Residual mid-span displacement of 5-2-80-5(1) was less than 0.20-in. Residual deflected shapes

for the specimens are shown graphically in Figure 4.54. Figures 4.55, 4.56, and 4.57 show the displacements measured by all three sensors. Note that cable extensometers were located 8.75-in away from mid-span and therefore expected to register smaller displacements than those calculated at mid-span from accelerometer data. Due to numerical errors associated with integrating the acceleration curve twice, only the peak displacement value was meaningful but the full transient response measured by the potentiometers and cable extensometers were valuable. There was good agreement between displacements calculated from accelerometer data and that from the potentiometer for 5-2-80-5(1) but not for 3-2-50-5(2). The potentiometer detached from specimens 3-2-50-5(2) and 3-2b-50-5(2) during the test. From analysis of high speed video they did not appear to continue to displace after the peak displacement was reached but instead began to fall. Therefore, maximum displacement measured by these two sensors was reasonable but no residual displacement value from these sensors was possible. When corrected for frame displacement, maximum mid-span displacements were 1.165, 1.160, and 0.417 for 3-2-50-5(2), 3-2b-50-5(2), and 5-2-80-5(1).

Filtered strain gage data for all strain gages on each specimen is shown in Figures 4.58, 4.59, and 4.60. The two strain gages at the non-midspan quarter-points (SG2 and SG4) were close in value for each of the three panels which denotes symmetric bending. These figures also demonstrate that the tension steel plates in these three specimens exceeded their elastic limits during Shot 5. This is corroborated by the residual tensile strain measured by SG3 for all three specimens. These sets of strain gage data captured the transient vibration of each specimen and showed that damping stopped the vibration after five or six cycles. The values recorded from SG3 of 3-2-50-5(2) are suspect - this mid-span strain should be larger than strains measured by SG2 and SG4 and it should not have negative (compressive) residual strain.

Specimens were removed from the frame after the shot and cracks marked and photographed (see Figure 4.61). Flexure cracks were most prominent in 3-2-50-5(2)

and 3-2b-50-5(2) with some flexural shear cracks in both. Figure 4.61(a) shows there was one large shear crack near one of the supports for 3-2-50-5(2) which extended from top to bottom of the specimen at approximately a 45° angle. Figure 4.61(b) shows shear cracks also formed near one of the supports for 3-2b-50-5(2) but did not extend to the faces of the specimen. 3-2b-50-5(2) also had a short flexure crack (approximately 1-in) near the support suggesting that the frame provided more constraint than was idealized. Figure 4.61(c) shows that there were only a few flexure cracks that formed near mid-span for 5-2-80-5(1). The small amount of cracking and residual displacement suggests that the response of 5-2-80-5(1) to this pressure-impulse was largely elastic. No concrete crushing was evident in any of the three specimens.

4.1.3.4 Test 4

Three specimens were loaded in the BLS for Test 4, from left to right (as appeared inside the target vessel, see Figure 4.62): 5-4-50-5(2), 5-2-50-5(2), and 3-2b-65-5(1). Test 4 consisted of a single shot (shot 6) with a driver pressure of 1420-psi which resulted in 61.6-psi peak pressure and 1110-psi·msec impulse.

Frame settlement from previous shots remained at the start of shot 6 as evident in Figure 4.50(a). The frame displaced by an additional maximum 0.15-in during Shot 6 but had no measurable additional settlement. Frame displacement and settlement were measured from high speed videos using PCC 2.5 software. Frame displacement resulted in larger measured deflections recorded by the sensors than actually experienced by the specimens.

Raw displacement and strain gage data from Shot 6 for the three specimens are shown in Figures 4.66, 4.65, and 4.64. The potentiometer detached from 3-2b-65-5(1) during the test. A 2-pass bandpass filter with a 20kHz upper limit was applied to the cable extensometer and strain gage data to remove noise. As seen in the figures, there was more noise in some channels than in others - the same filter was applied to all regardless of the amount of noise in the signal. No filter was applied

to the accelerometer or potentiometer data. Accelerometer data was integrated twice to obtain velocity and displacement. These calculated curves are shown with the accelerometer data in Figures 4.66(a), 4.65(a), and 4.64(a).

Maximum measured mid-span displacements measured by the potentiometers for shot 6 were 0.845-in and 0.852-in for 5-2-50-5(2) and 5-4-50-5(2) respectively and residual displacements were 0.457-in and 0.340-in. Maximum measured mid-span displacements from accelerometer data for shot 5 were 1.474-in, 0.694-in, and 0.455-in for 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2) respectively. Residual displacements measured by the potentiometers were larger than measured residual deflections of the specimens in the frame post-test. Residual deflected shapes for the specimens are shown graphically in Figure 4.67. Figures 4.68, 4.69, and 4.70 show the displacements measured by all sensors. Note that cable extensometers were located 8.75-in away from mid-span and therefore expected to register smaller displacements than those calculated at mid-span from accelerometer data. Due to numerical errors associated with integrating the acceleration curve twice, only the peak displacement value was meaningful but the full transient response measured by the potentiometers and cable extensometers were valuable. There was reasonable agreement between the displacement calculated from accelerometer data and that from the potentiometer for 5-2-50-5(2) but not for 5-4-50-5(2). When corrected for frame displacement, maximum mid-span displacements were 1.180, 0.551, and 0.558 for 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2).

Filtered strain gage data for all strain gages on each specimen is shown in Figures 4.71, 4.72, and 4.73. The two strain gages at the non-midspan quarter-points (SG2 and SG4) were close in value for each of the three panels which denotes symmetric bending. These figures also demonstrate that the tension steel plates in these three specimens exceeded their elastic limits during Shot 6. Of note, the strain measured by SG3 on 3-2b-65-5(1) exceeded the sensor's capacity - thus the constant value. There was more noise in the data from SG1 on 5-4-50-5(2) and the value of this data

is limited. These sets of strain gage data captured the transient vibration of each specimen and showed that damping stopped the vibration after five or six cycles.

Specimens were removed from the frame after the shot and cracks marked and photographed (see Figure 4.74). Flexure and flexural-shear cracks were prominent throughout much of the length of 3-2b-65-5(1) and the final deflected shape of the specimen, shown in Figure 4.74(a), indicates a plastic hinge formed at mid-span. Figure 4.74(b) shows that flexure cracking near mid-span and a shear crack near one of the supports formed in 5-2-50-5(2). A similar crack pattern was observed in 5-4-50-5(2) as seen in Figure 4.74(c).

4.2 Experimentally Determined Natural Period and Damping Ratio

As described above, damped vibration of each specimen was evident from measured strain time histories. Figure 4.75 depicts how the natural period could be estimated from the time between displacement peaks. This figure also shows the exponential damping curve (Equation (4.1) [85]) for various equivalent viscous damping ratios.

$$x(t) = X_m \cdot e^{\zeta\omega_n t} \quad (4.1)$$

For this example, 5-2-50-5(2) from Shot 6 (Test 4), the damping ratio that best fit the experimental decay of vibration was 0.08. The values of damping ratios which best fit the vibration decay for each specimen are listed with the experimentally measured natural period for each specimen in Table 4.6. For shots in which the response remained elastic, the damping ratio was $\approx 5\%$. For plastic response cases, the damping ratio was $\approx 10\%$. This calculated damping ratio is a “smeared” ratio which lumps together common sources of damping: material, concrete cracking, and from support conditions. These values are not recommended for use in other applications but are useful in providing some understanding of damping of one-way SC walls after a short duration load was applied.

4.3 Influence of Design Parameters on Blast Response

In order to compare the influence of design parameters, displacement results were used to compute ductility demand by dividing the maximum displacement, X_M , by the calculated elastic displacement at yield, X_E . This ductility measure provided a normalized value from which the influence of design parameters were considered. Two comparisons were made: pressure and impulse. For each, the pressure and impulse at which these deflections occurred were normalized to dimensionless force and impulse terms using Equations (4.2) and (4.3), respectively, (after [91]).

$$P = \frac{2p_r bL}{R_y} \quad (4.2)$$

$$I = \frac{i_r bL}{R_y/\omega_n} \quad (4.3)$$

The values for X_E , the yield resistance, R_y , and the natural frequency of the specimen, ω_n , were calculated using values from the idealized bi-linear static resistance function described in Section 3.6.2 and the estimated mass of the unsupported length of each specimen.

Figure 4.76 shows comparisons of experimental ductility to normalized pressure and impulse for each specimen. For each comparison an exponential curve was generated of the form in Equation (4.4) or (4.5). Values for A and B and the R^2 value for the fit are provided in Tables 4.7 and 4.8.

$$\frac{X_M}{X_E} = Ae^{B \cdot I} \quad (4.4)$$

$$\frac{X_M}{X_E} = Ae^{B \cdot P} \quad (4.5)$$

4.3.1 Performance of Panels With Varied Steel Plate Strength

Figure 4.77 provides the same data as in Figure 4.76 grouped by steel plate strength and reinforcement ratio. Also shown in this figure are the best exponential fits through the data of each subgroup with the exception of Gr80, $\rho = 3\%$,

which had a single data point. Tables 4.7 and 4.8 provide values for the best fit constants and R^2 . These trendlines assist in identifying the influence of these parameters. Figure 4.77(a) indicates that as the steel strength increased, the ductility demand decreased for a given pressure. The same conclusion was drawn from Figure 4.77(b) for lower impulses ($I \leq 6.0$) but not for higher impulses.

4.3.2 Performance of Panels With Varied Reinforcement Ratio

The data grouped by nominal reinforcement ratio is shown with the data grouped by steel plate strength in Figure 4.77. Figure 4.77(a) indicates that as the reinforcement ratio increased, the ductility demand increased for the lower range of normalized pressure ($P \leq 1.75$). This may seem counterintuitive - less steel leads to lower ductility demands. The reason for this is because the elastic displacement limit, X_E , which is dependent on reinforcement ratio, is included in the calculation of ductility demand. The maximum displacement for a given pressure was larger for the specimens with less reinforcement (see Table 4.2) but the ductility demand was less. The same conclusion was drawn, although with a smaller difference, from Figure 4.77(b) for lower impulses ($I \leq 6.0$). Conclusions about higher pressures and impulses could not be drawn because of limited data in the higher ranges of normalized pressure and impulse for specimens with 5% reinforcement ratios.

4.3.3 Performance of Panels With Varied Tie Bar Spacing

Figure 4.78 provides the same data as in Figure 4.76 grouped by tie bar diameter and spacing. The terms in the legend correspond to the second term in each specimen name. The number ‘2’ indicates tie bars designed to provide the required shear strength and spaced at 2-in on-center, ‘2b’ indicates tie bars one size larger than for ‘2’ and spaced at 2-in on-center, and ‘4’ indicates tie bars designed to provide the required shear strength spaced at 4-in on-center. Shown with the data points in this figure are the best exponential fits through the data of each subgroup. Tables 4.7

and 4.8 provide values for the best fit constants and R^2 . Figures 4.78(a) and 4.78(b) indicate that the ductility demand was larger for specimens with larger tie bar spacing for the lower ranges included in this study ($P \leq 1.5$ and $I \leq 6.0$). The influence reduced as pressure and impulse increased.

4.3.4 Performance of Panels With Varied Tie Bar Diameter

The data grouped by tie bar diameter is shown with the data grouped by tie bar spacing in Figure 4.77. Figure 4.78(a) indicates that as the tie bar diameter increased, the ductility demand decreased. The same conclusion was drawn from Figure 4.78(b) for the normalized impulse. The influence was less notable as pressure and impulse increased.

4.4 Chapter Summary

This chapter reported results from a series of dynamic experiments of twelve SC panels completed using the USACE-ERDC BLS in Vicksburg, MS. These tests included specimens with different design parameters (steel plate strength, flexural reinforcement ratio, tie bar diameter (shear reinforcement ratio), and tie bar spacing). From these test results, the influence of each design parameter was described.

The following chapter develops benchmarked SDOF and FE models using results from these dynamic experiments. These benchmarked models were used for a series of parametric studies (Chapter 6) to better understand the influence of design and load variables on structural response.

Table 4.1. Planned Blast Loads

	p_r	i_r
Series	(psi)	(psi·msec)
1 - small	20	300
1 - large	60	1100
2	60	1100

Table 4.2. Blast Specimen Test Results

Test	Shot	Panel	f'_c (psi)	Wt (lb)	p_r (psi)	i_r (psi·msec)	$X_{M,mid-span}$		$X_{r,mid-span}$		Stringpots	
							Accel (in)	Rod (in)	Rod (in)	$X_{M,qtr-pt}$ (in)	$X_{r,qtr-pt}$ (in)	
1	1	3-2-50-5(1)	6400	269.5	20.2	261	0.473	†	†	0.406	0.206	
		3-2b-50-5(1)	6400	270.0			0.421	†	†	0.308	0.167	
		5-4-50-5(1)	6400	280.0			0.559	†	†	0.510	0.247	
	2	3-2-50-5(1)	6400	269.5	59.9†	1032†	0.948†	†	†	†	†	
		3-2b-50-5(1)	6400	270.0			0.959†	†	†	†	†	
		5-4-50-5(1)	6400	280.0			0.535†	†	†	†	†	
2	3	3-2-65-5(1)	6300	265.5	18.9	260	0.298	0.284	0.035	0.238	0.023	
		5-2-50-5(1)	6300	281.5			0.307	0.318	0.059	0.234	0.005	
		5-2b-50-5(1)	6300	288.0			0.266	0.250	0.083	0.230	0.055	
	4	3-2-65-5(1)	6300	265.5	61.6	948	1.293	*	*	0.791	0.223	
		5-2-50-5(1)	6300	281.5			0.493	0.677	0.194	0.508	0.079	
		5-2b-50-5(1)	6300	288.0			0.726	0.761	0.241	0.599	0.157	
3	5	3-2-50-5(2)	6500	269.0	59.9	1032	0.462	1.460	*	0.744	0.322	
		3-2b-50-5(2)	6500	270.5			*	1.455	*	0.775	0.449	
		5-2-80-5(1)	6500	283.5			0.703	0.712	0.193	0.524	0.085	
	4	3-2b-65-5(1)	6400	265.5	61.6	1110	1.474	*	*	1.120	0.449	
		5-2-50-5(2)	6200	279.0			0.694	0.845	0.457	0.596	0.130	
		5-4-50-5(2)	6400	281.5			0.455	0.852	0.340	0.512	0.116	

† no signal reference set for this instrument; no meaningful data acquired

‡ DAQ not triggered; p_r and i_r from Test 3; $X_{M,mid-span}$ from high-speed video; no other values available

* sensor detached from specimen during test

Table 4.3. Concrete Mix Design for Blast Specimens (As Received from Supplier; 3 CY order)

Component	Type	Total Amount (lb)	Amount per CY (lb)	% Moisture	Water Adjust. (lb)
Gravel	Pea	5680	1893	1.5	83
Sand	SAND-23	4500	1500	3.9	175
Cement	Type I	1685	562		
Water		436	145		
Admixtures	GLEN 3030	34 oz	11 oz		
			Water (added on site)		0
			Total Water (lb)		695
			Water : Cement Ratio		0.41

Table 4.4. BLS Calibration Shot Data

	Cal Shot 1		Cal Shot 2		Cal Shot 3	
	$p_d = 1443$ psi		$p_d = 400$ psi		$p_d = 400$ psi	
Gage	p_r	i_r	p_r	i_r	p_r	i_r
P1	64.1	1072	17.5	251	19.5	263
P2	64.1	1056	19.4	248	22.1	249
P3	65.9	1092	22.9	253	24.0	278
P4	61.5	1072	20.5	268	20.9	260
P5	59.0	1074	18.6	257	20.6	263
P6	61.6	1066	16.8	247	18.8	254
P7	74.6	1246	21.0	284	21.7	298
P8	*	*	*	*	*	*
P9	60.7	1074	19.8	273	22.3	265
P10	63.1	1099	17.9	246	18.8	258
PD	*	*	16.6	247	*	*
PE	61.9	1075	17.7	254	19.4	260
Average	63.7	1093	19.0	257	20.8	265
Median	62.5	1074	18.6	253	20.8	262
Std Dev	4.3	55	2.0	13	1.7	14

NOTE: pressure data in psi and computed impulses in psi · msec

* pressure transducer malfunctioned

Table 4.5. Pressure Data and Calculated Impulses from Blast Tests

Gage	Test 1						Test 2						Test 3						Test 4					
	Shot 1			Shot 2			Shot 3			Shot 4			Shot 5			Shot 6			Shot 6			Shot 6		
	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r	p_d	i_r	p_r
	$400psi$			$1388psi$			$400psi$			$1420psi$			$1420psi$			$1420psi$			$1420psi$			$1420psi$		
P1	19.5	251	‡	N/A	18.7	249	60.4	1022	55	998	56.8	1073												
P2	22.5	300	‡	N/A	21.9	305	71.1	1258	68.5	1200	70.5	1279												
P3	19.3	255	‡	N/A	19	251	57	1024	58.2	1015	58.6	1069												
P4	20.7	250	‡	N/A	18.1	253	61	1042	58.7	996	61	1068												
P5	20.2	259	‡	N/A	18.8	254	63.4	1045	58.7	1018	59.4	1097												
P6	20.8	257	‡	N/A	18.7	262	59	1067	61.5	990	64.6	1094												
PD	18.8	252	‡	N/A	17.5	241	61	1002	57.4	993	56.1	1074												
PE	19.4	262	‡	N/A	18.6	266	60	1083	61.2	1045	65.9	1127												
Average	20.2	261	N/A	N/A	18.9	260	61.6	1068	59.9	1032	61.6	1110												
Median	19.9	256	N/A	N/A	18.7	254	60.7	1044	58.7	1007	60.2	1084												
Std Dev	1.2	16	N/A	N/A	1.3	20	4.2	81	4.0	70	5.0	71												
IP1	0.25	6	‡	N/A	0.17	17	0.4	23	0.4	76	0.46	31												

NOTE: pressure data in psi and computed impulses in $psi \cdot msec$

‡ data acquisition system not triggered; similar rupture pressure as in Shot 5 (Test 3)

Table 4.6. Experimentally Determined Natural Period and Damping Ratio

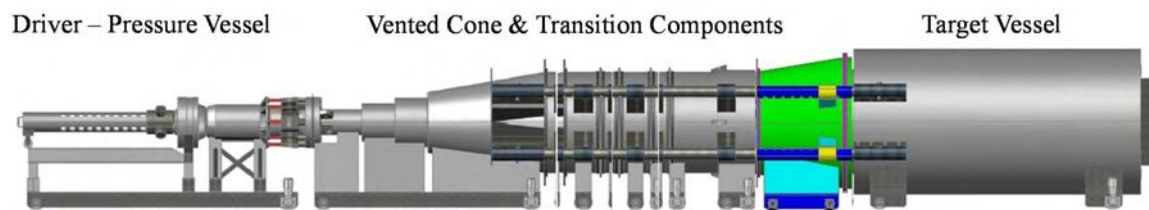
Panel	Shot	Response	T_n	ζ
			(msec)	
3-2-50-5(1)	1	E	15.6	0.07
3-2b-50-5(1)	1	E	15.2	0.07
5-4-50-5(1)	1	E	16.4	0.05
3-2-65-5(1)	3	E	14.0	0.08
5-2-50-5(1)	3	E	14.4	0.08
5-2b-50-5(1)	3	E	14.3	0.06
3-2-65-5(1)	4	P	14.1	0.08
5-2-50-5(1)	4	P	15.5	0.10
5-2b-50-5(1)	4	P	11.2	0.10
3-2-50-5(2)	5	P	14.5	0.09
3-2b-50-5(2)	5	P	13.2	0.10
5-2-80-5(1)	5	E?	13.1	0.11
3-2b-65-5(1)	6	P	13.9	0.09
5-2-50-5(2)	6	P	13.0	0.08
5-4-50-5(2)	6	P	13.0	0.12

Table 4.7. Exponential Fit of Ductility vs Normalized Pressure

	A	B	R^2
ALL	0.412	0.284	0.926
Gr50, 3%	0.385	0.290	0.978
Gr50, 5%	0.507	0.248	0.805
Gr65, 3%	0.282	0.347	0.979
Gr80, 5%		N/A	
2	0.395	0.292	0.957
2b	0.289	0.327	0.989
4	0.993	0.143	0.999

Table 4.8. Exponential Fit of Ductility vs Normalized Impulse

	A	B	R^2
ALL	0.342	1.183	0.926
Gr50, 3%	0.278	1.287	0.977
Gr50, 5%	0.441	1.030	0.806
Gr65, 3%	0.213	1.373	0.999
Gr80, 5%		N/A	
2	0.329	1.197	0.962
2b	0.218	1.386	0.996
4	0.874	0.665	0.999



(a) Schematic



(b) Photograph of Assembled BLS

Figure 4.1. USACE ERDC Blast Load Simulator (after [86])

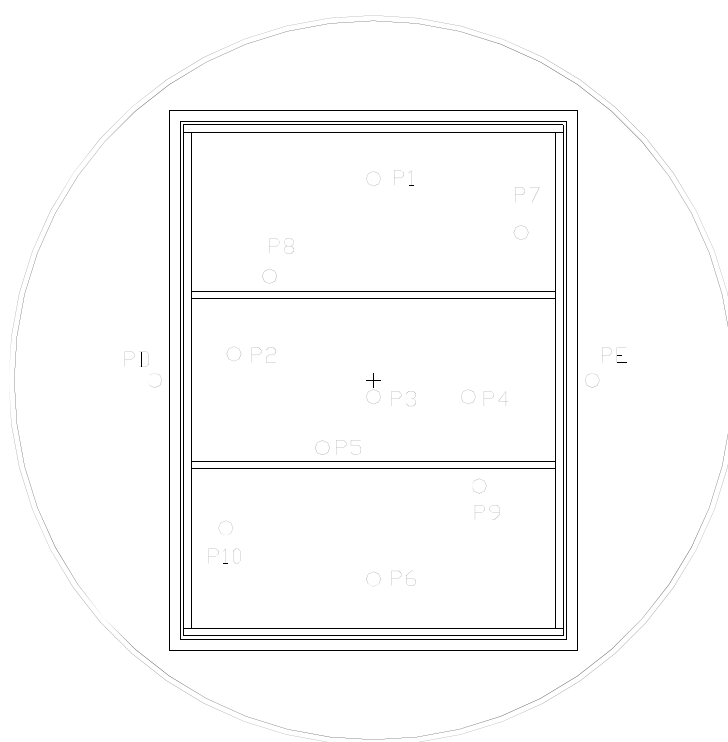


Figure 4.2. Pressure Transducer Locations for Calibration Panel



(a) Empty Target Vessel



(b) Positioning Internal Frame

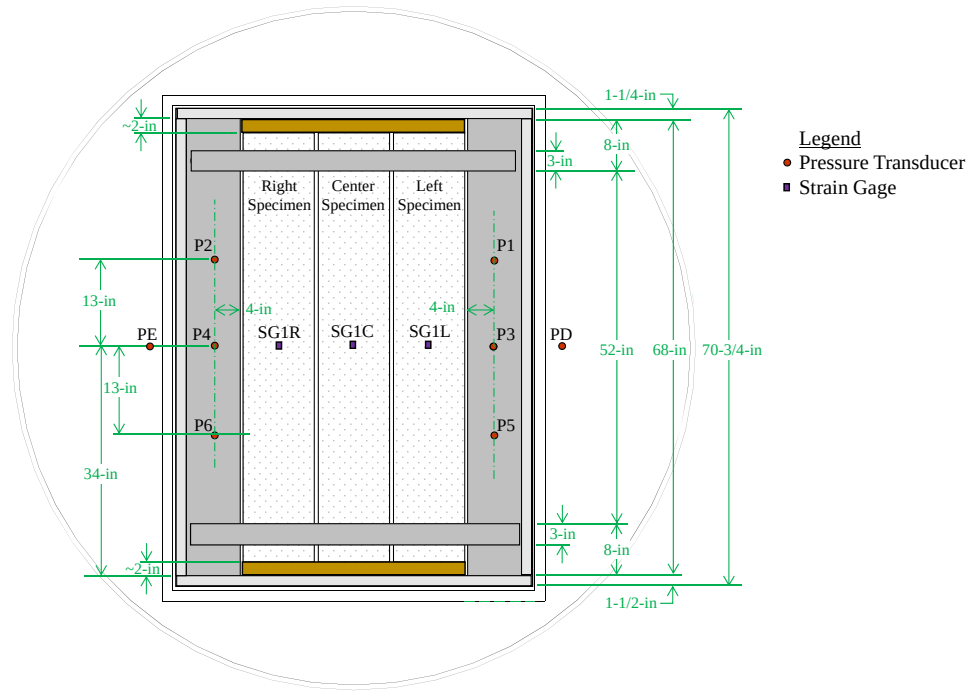


(c) Putting Specimens in Place

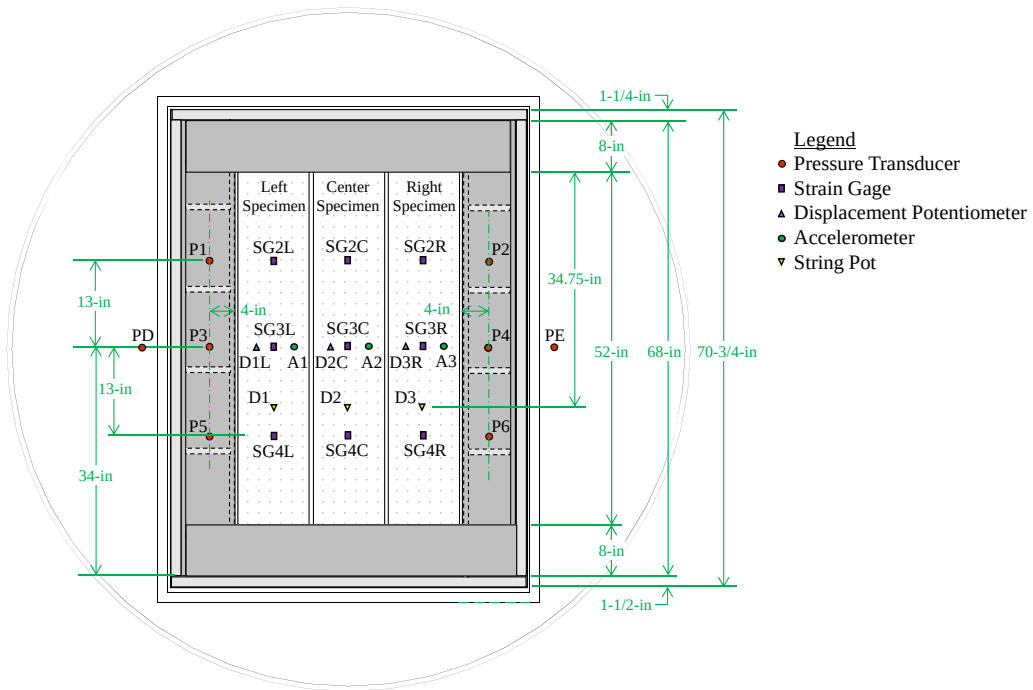


(d) Completed Setup

Figure 4.3. Blast Test Setup Procedure



(a) Blast Face



(b) Non-Blast Face

Figure 4.4. Sensor Locations for Blast Tests

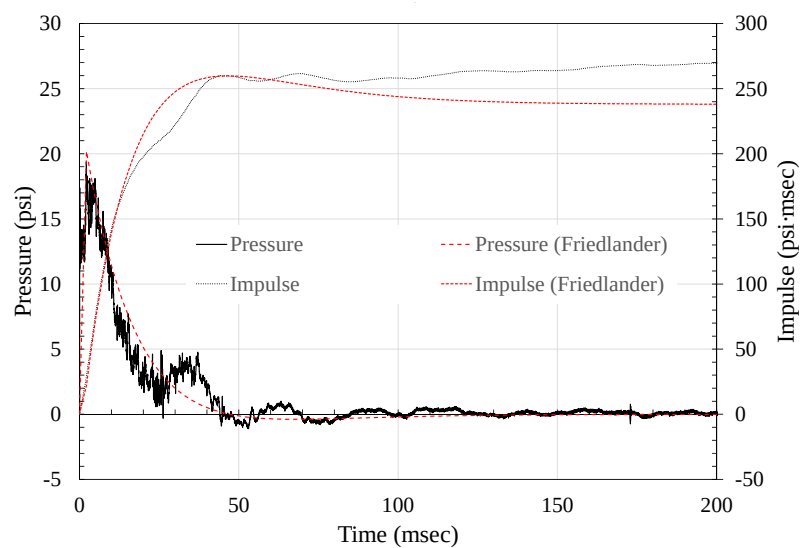


Figure 4.5. Representative Pressure-Time History, Shot 1

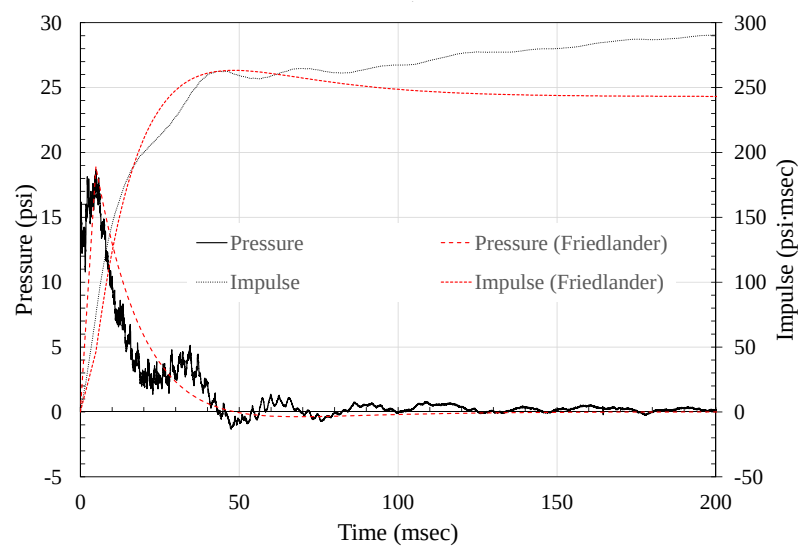


Figure 4.6. Representative Pressure-Time History, Shot 3

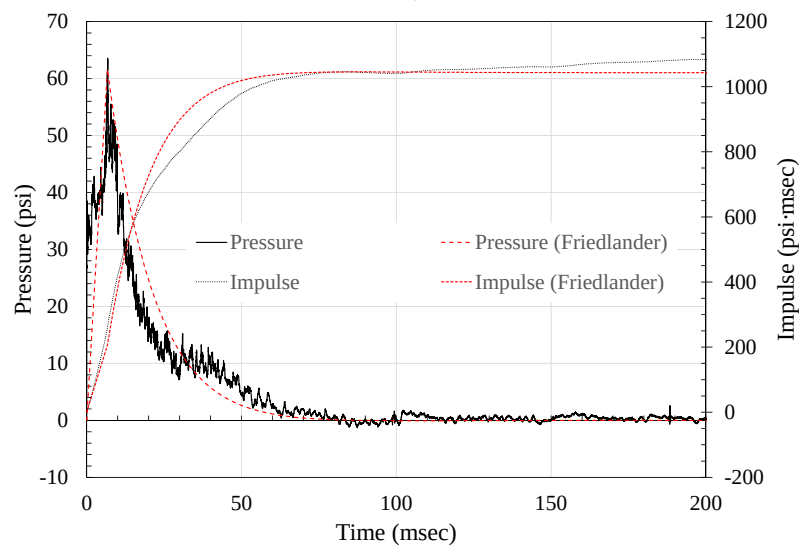


Figure 4.7. Representative Pressure-Time History, Shot 4

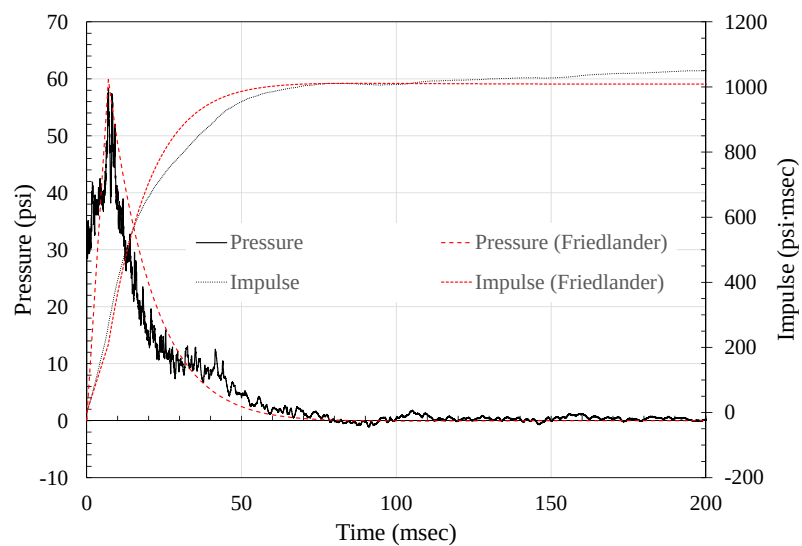


Figure 4.8. Representative Pressure-Time History, Shot 5

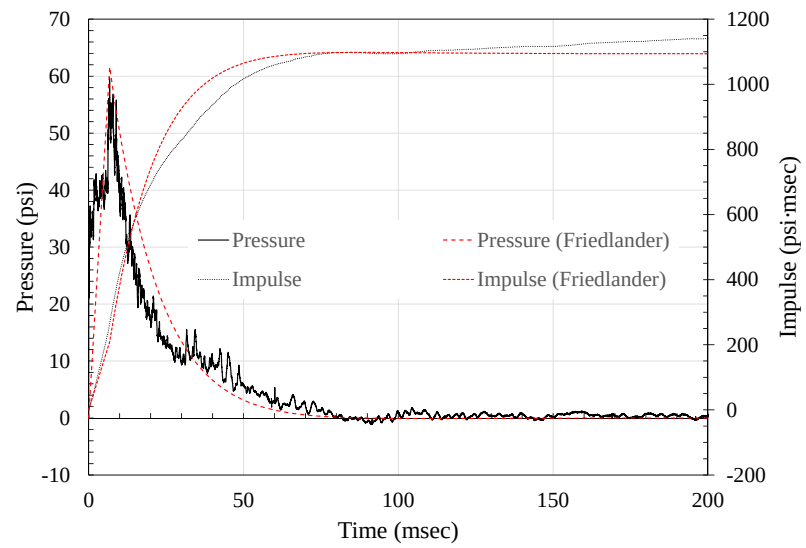
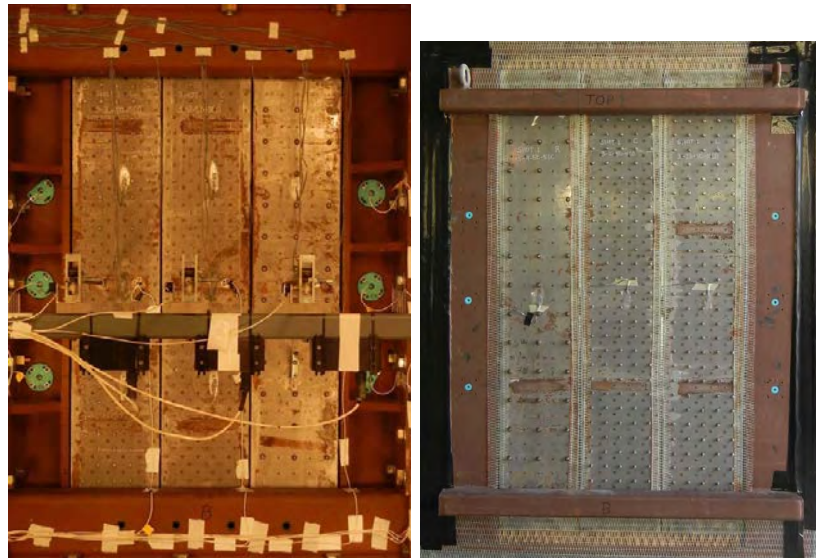
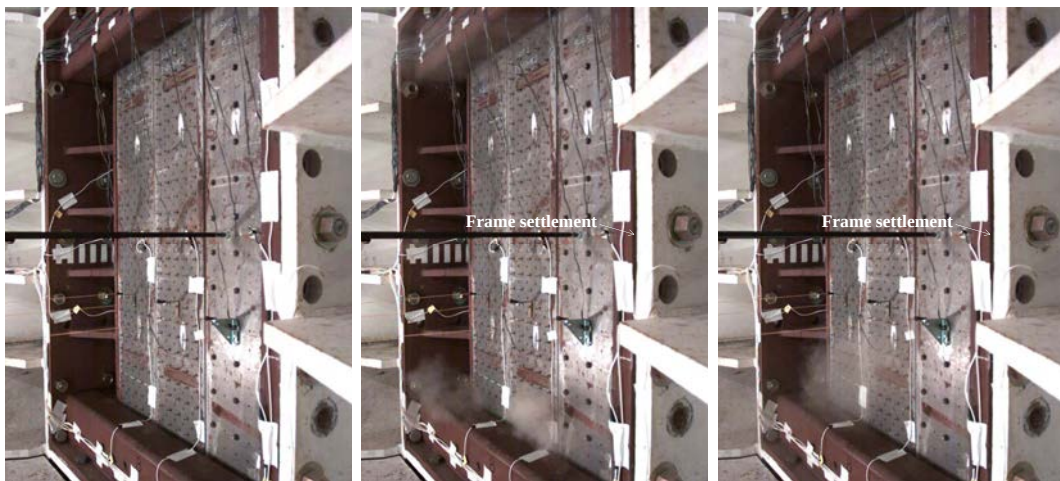


Figure 4.9. Representative Pressure-Time History, Shot 6



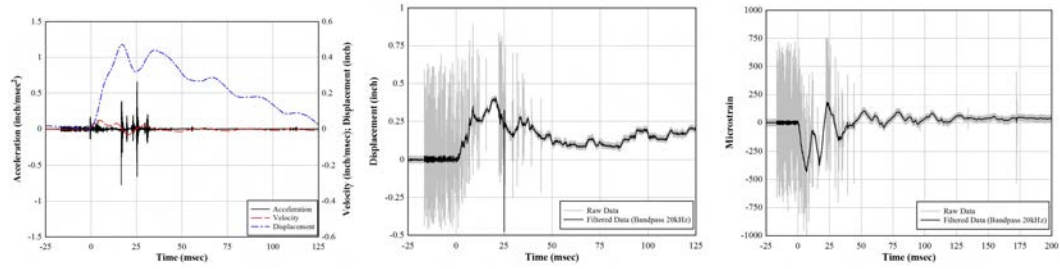
(a) View from Inside Target Vessel (b) View from Outside Target Vessel (Blast Face)

Figure 4.10. Pre-Test Photos for Test 1: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)



(a) Pressure Arrival, t_a (b) Max. Displ, $t_a + 16\text{msec}$ (c) Rest, $t_a + 200\text{msec}$

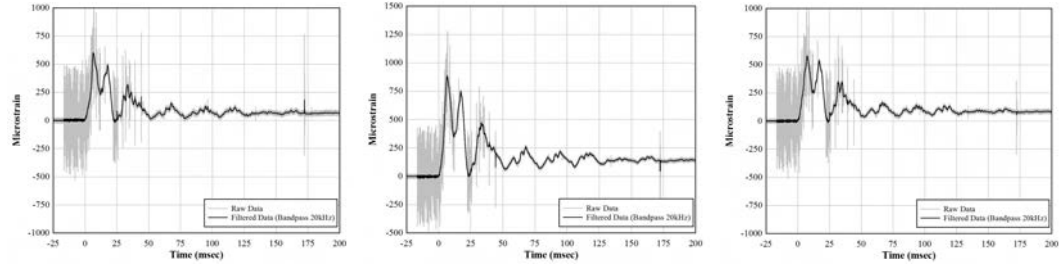
Figure 4.11. High Speed Video Frames from Test 1, Shot 1: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)



(a) Accelerometer

(b) Cable Extensometer

(c) SG1

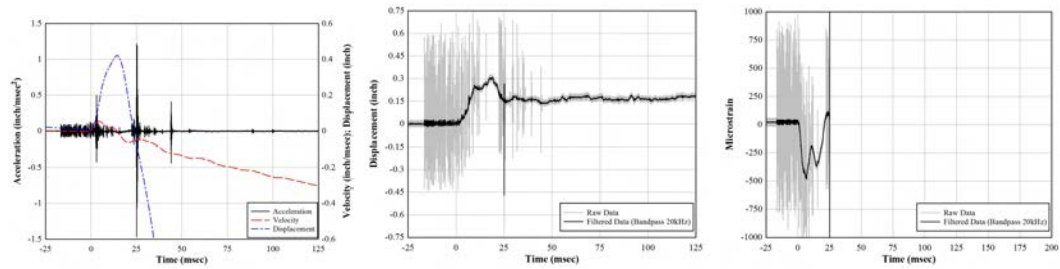


(d) SG2

(e) SG3

(f) SG4

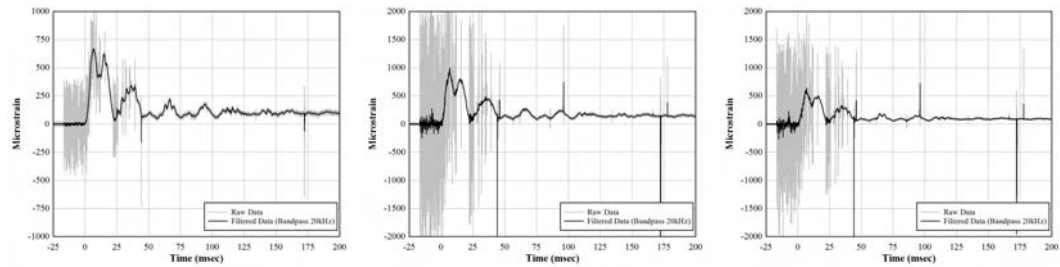
Figure 4.12. Raw and Filtered Data, 3-2-50-5(1), Shot 1



(a) Accelerometer

(b) Cable Extensometer

(c) SG1



(d) SG2

(e) SG3

(f) SG4

Figure 4.13. Raw and Filtered Data, 3-2b-50-5(1), Shot 1

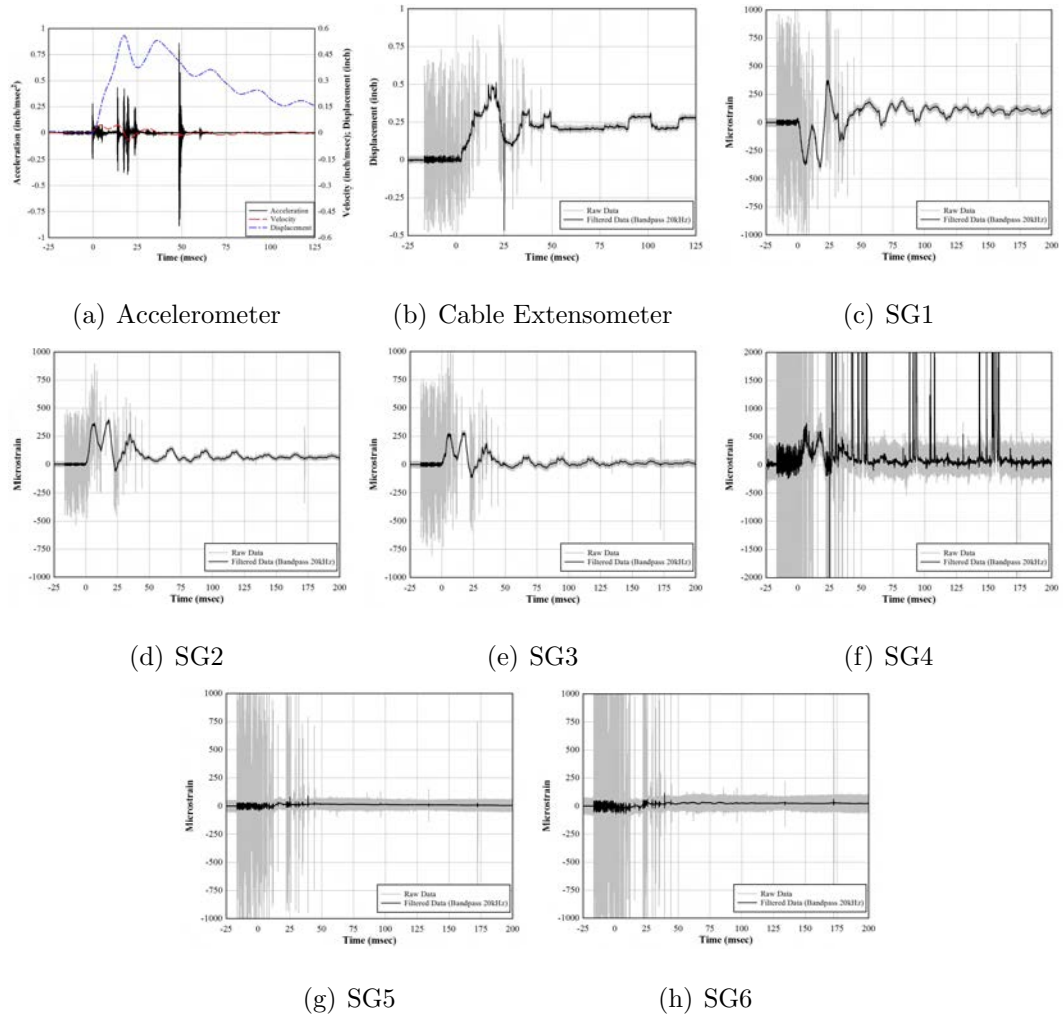


Figure 4.14. Raw and Filtered Data, 5-4-50-5(1), Shot 1

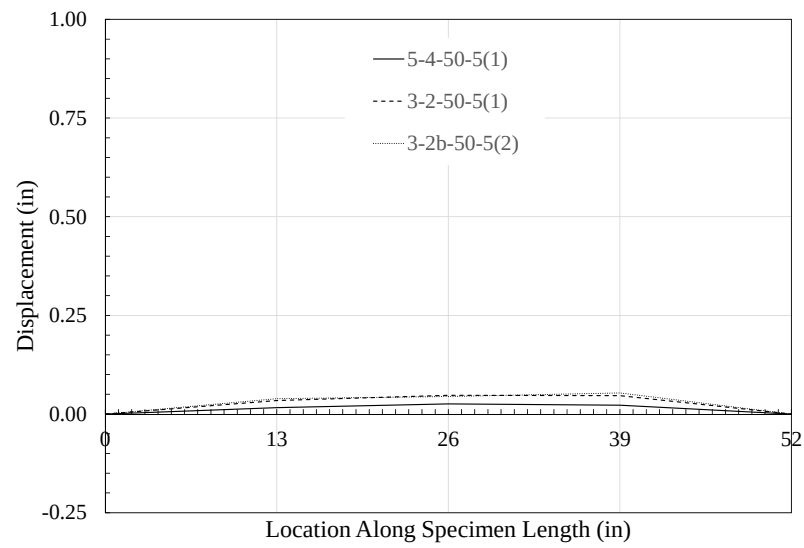


Figure 4.15. Post-Test Residual Displacement for Test 1, Shot 1: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)

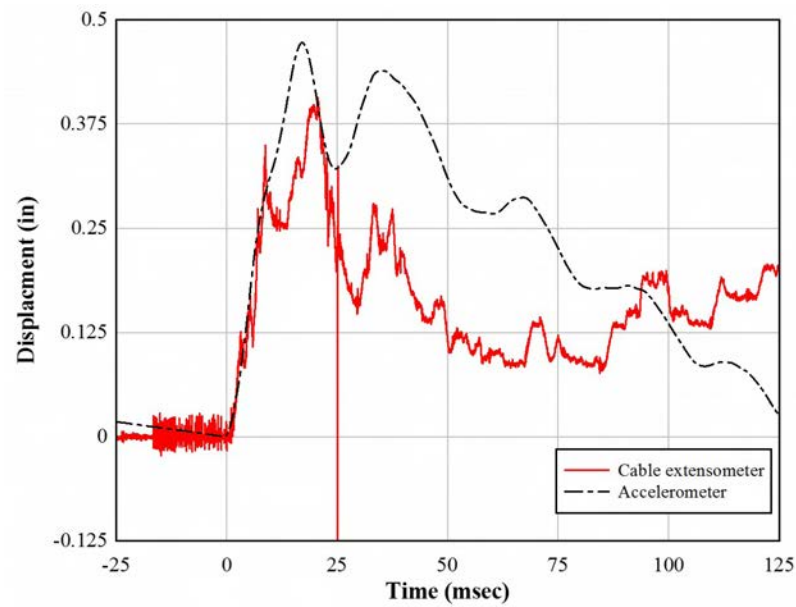


Figure 4.16. Displacement Time-History, 3-2-50-4(1), Shot 1

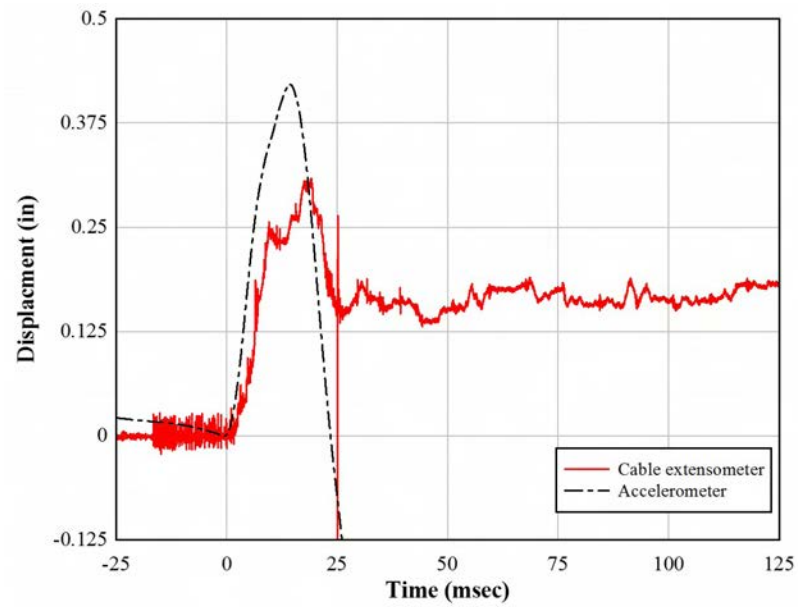


Figure 4.17. Displacement Time-History, 3-2b-50-4(1), Shot 1

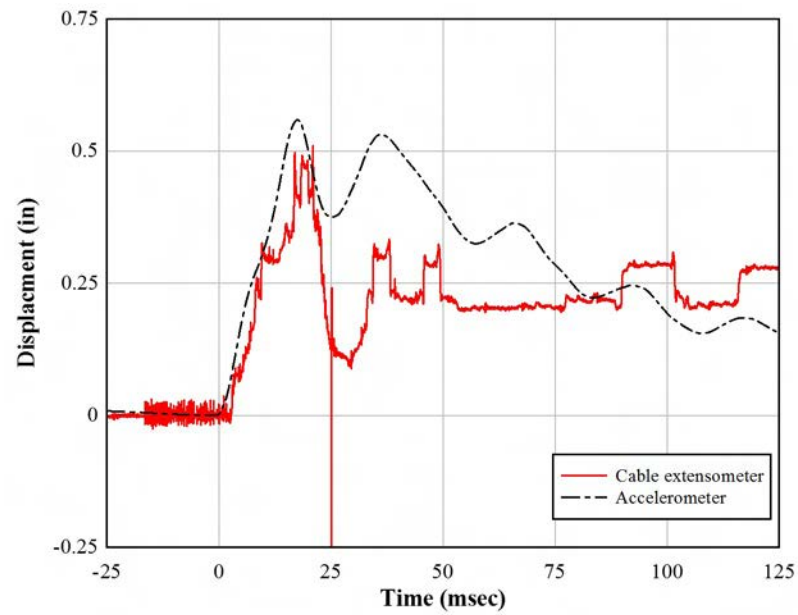


Figure 4.18. Displacement Time-History, 5-4-50-4(1), Shot 1

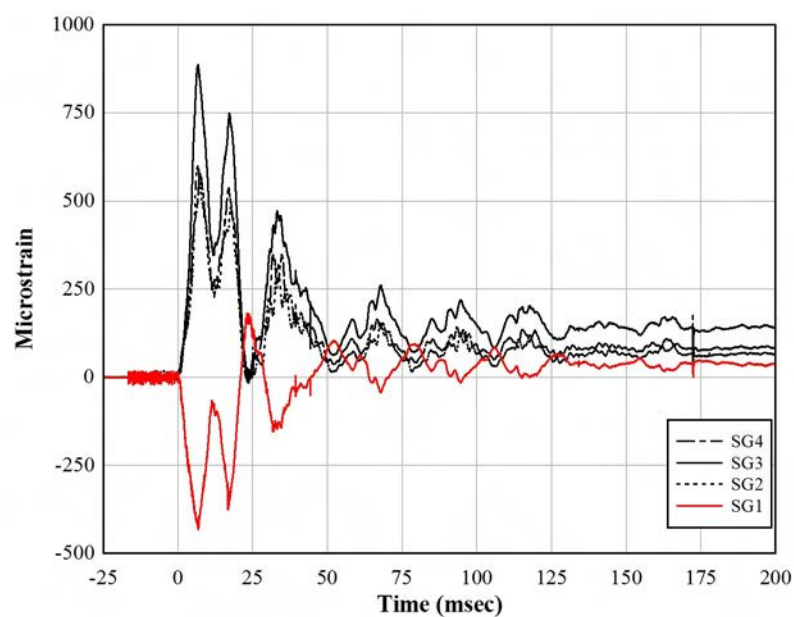


Figure 4.19. Strain Time-History, 3-2-50-4(1), Shot 1

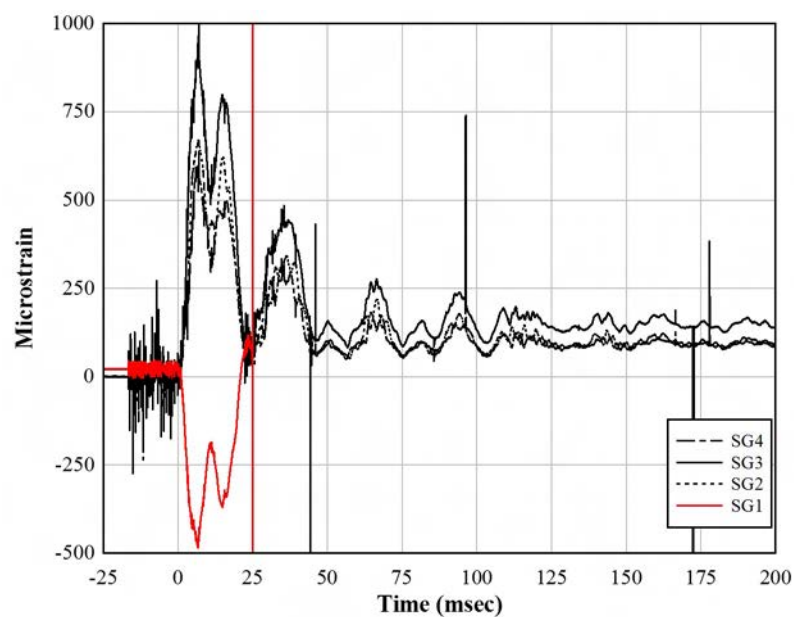


Figure 4.20. Strain Time-History, 3-2b-50-4(1), Shot 1

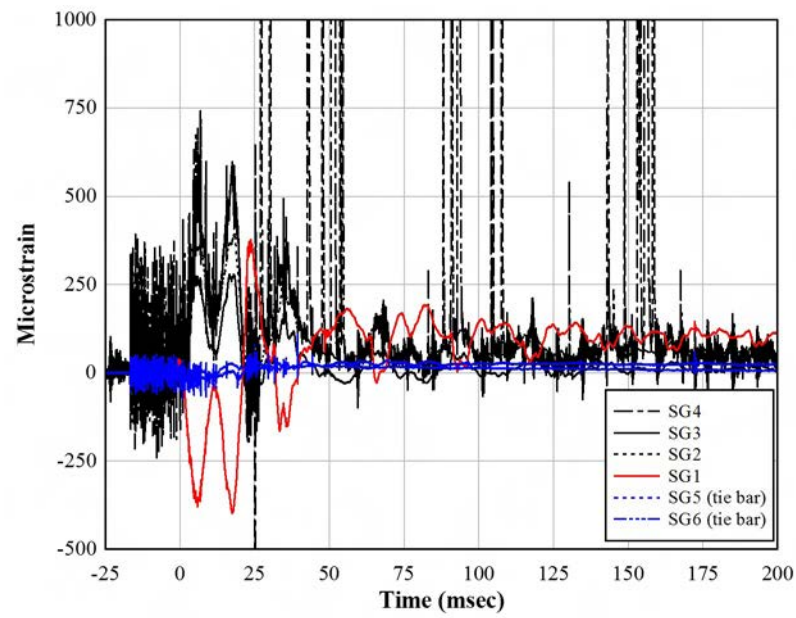
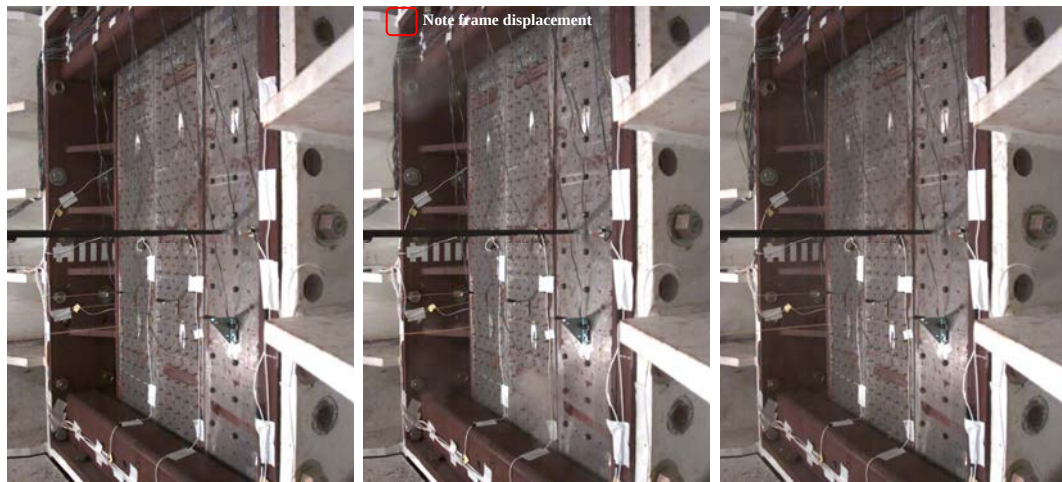


Figure 4.21. Strain Time-History, 5-4-50-4(1), Shot 1



(a) Pressure Arrival, t_a (b) Max. Displ, $t_a + 11\text{msec}$ (c) Rest, $t_a + 200\text{msec}$

Figure 4.22. High Speed Video Frames from Test 1, Shot 2: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)

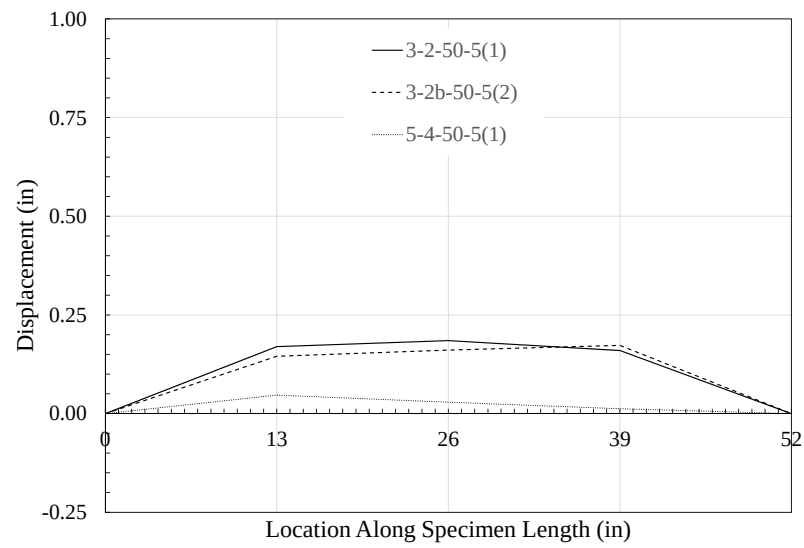


Figure 4.23. Post-Test Residual Displacement for Test 1, Shot 2: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)



(a) 3-2-50-5(1)

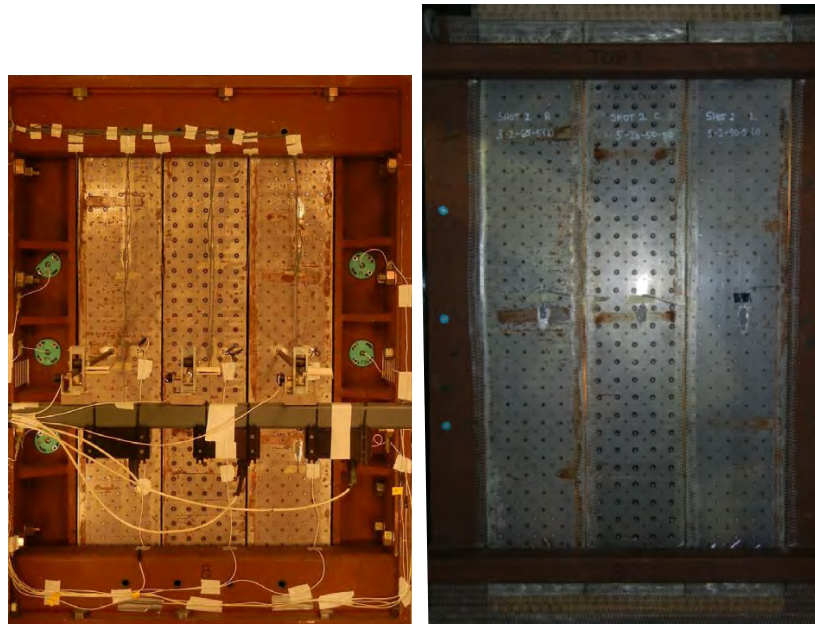


(b) 3-2b-50-5(1)



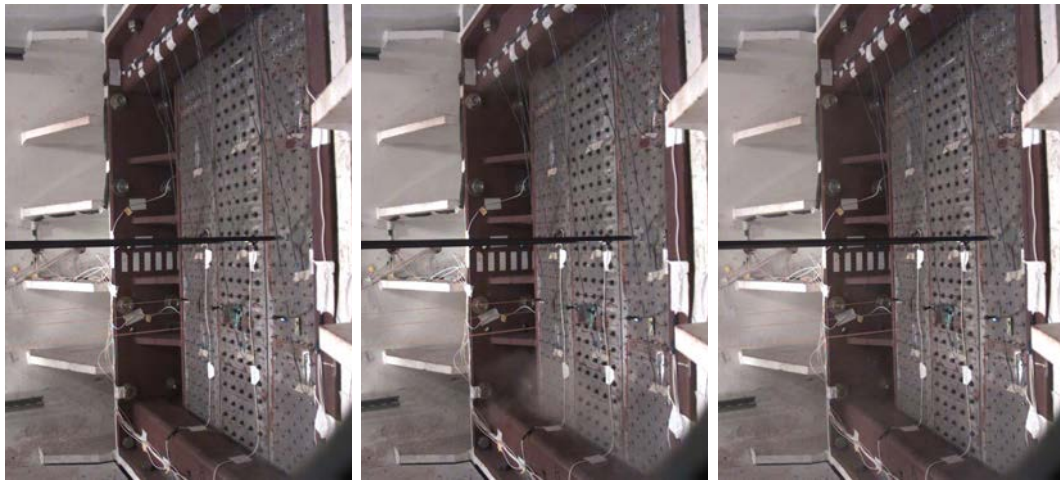
(c) 5-4-50-5(1)

Figure 4.24. Post-Test Crack Patterns from Test 1: 3-2-50-5(1), 3-2b-50-5(1), and 5-4-50-5(1)



(a) View from Inside Target Vessel (b) View from Outside Target Vessel (Blast Face)

Figure 4.25. Pre-Test Photos for Test 2: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)



(a) Pressure Arrival, t_a (b) Max. Displ., $t_a + 9$ (c) Rest, $t_a + 200$

Figure 4.26. High Speed Video Frames from Test 2, Shot 3: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)

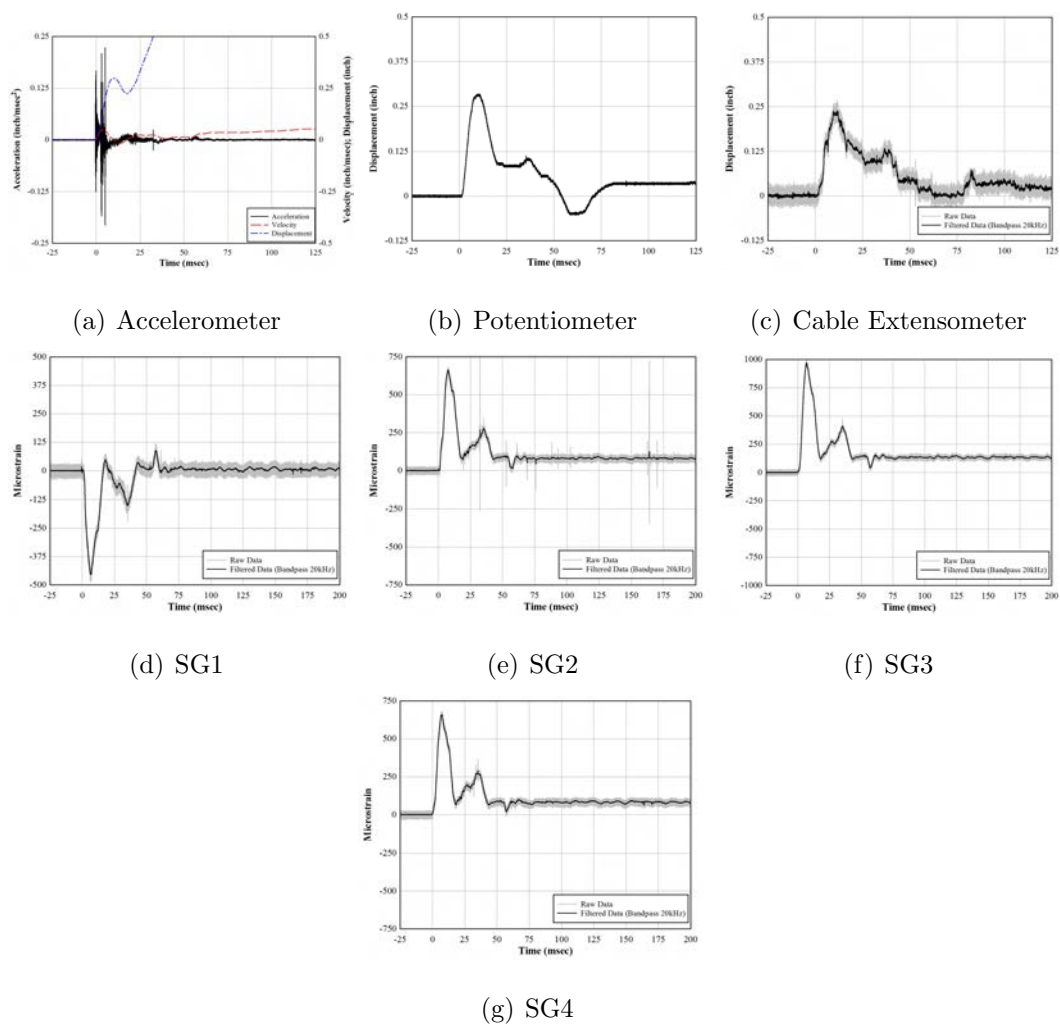


Figure 4.27. Raw and Filtered Data, 3-2-65-5(1), Shot 3

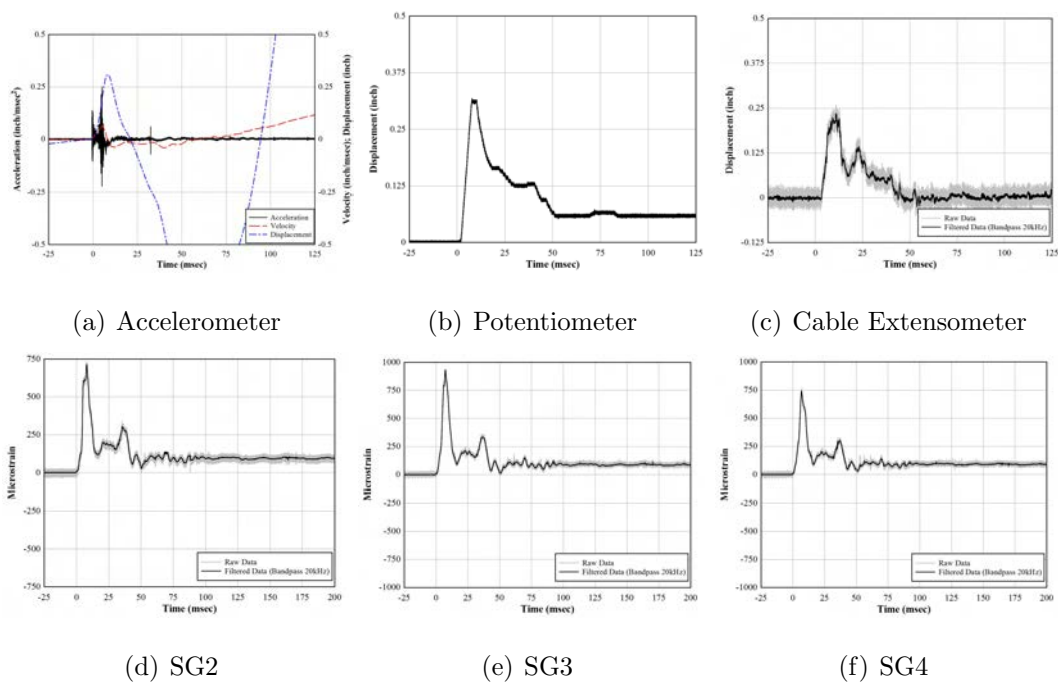


Figure 4.28. Raw and Filtered Data, 5-2-50-5(1), Shot 3

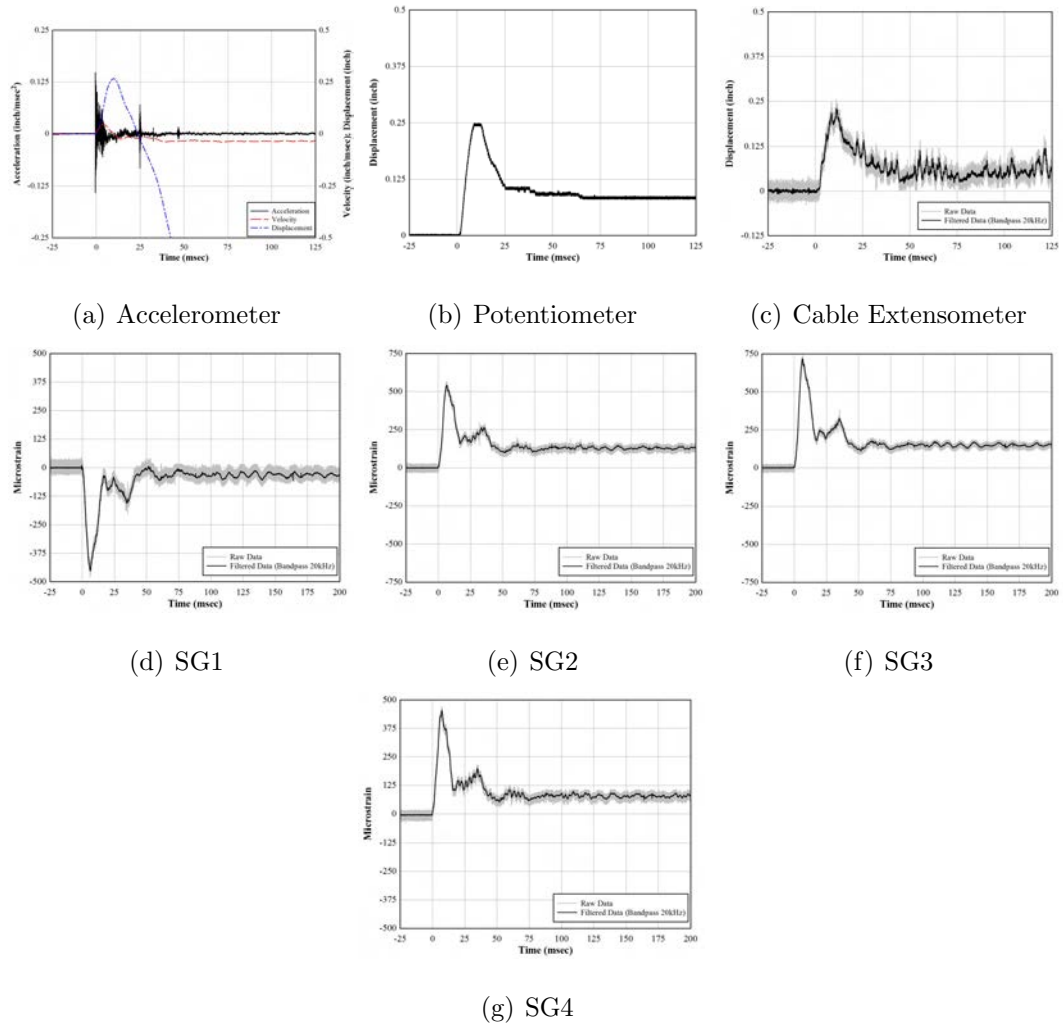


Figure 4.29. Raw and Filtered Data, 5-2b-50-5(1), Shot 3

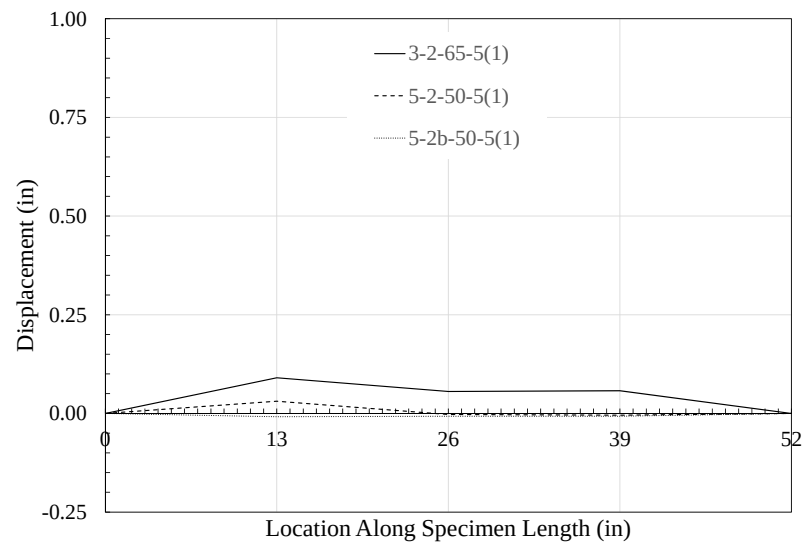


Figure 4.30. Post-Test Residual Displacement for Test 2, Shot 3: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)

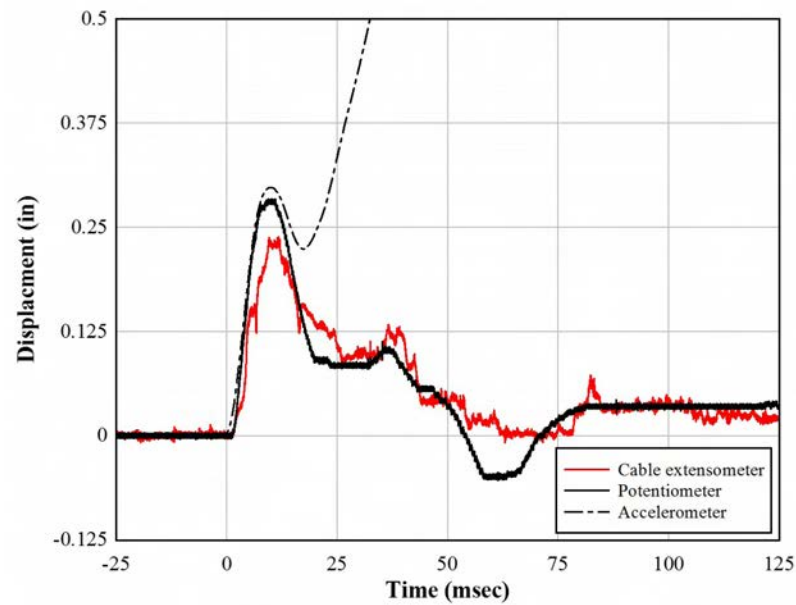


Figure 4.31. Displacement Time-History, 3-2-65-4(1), Shot 3

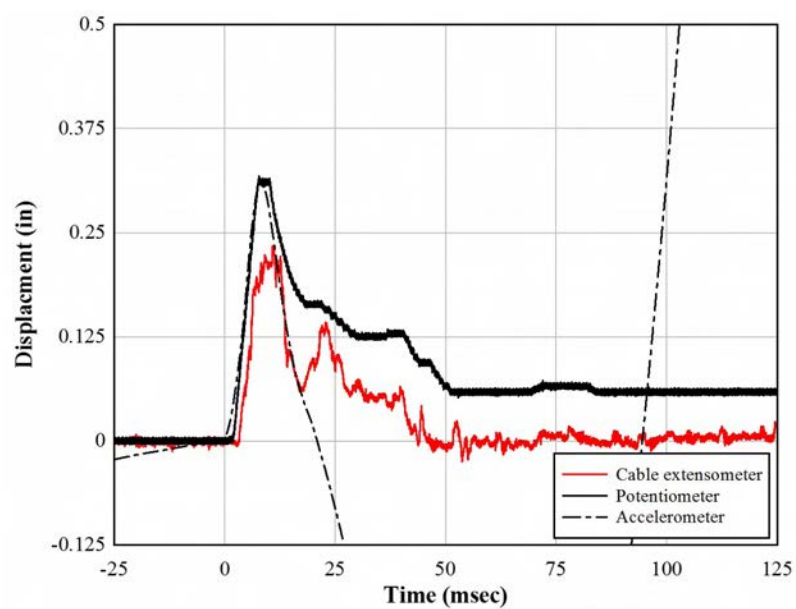


Figure 4.32. Displacement Time-History, 5-2-50-4(1), Shot 3

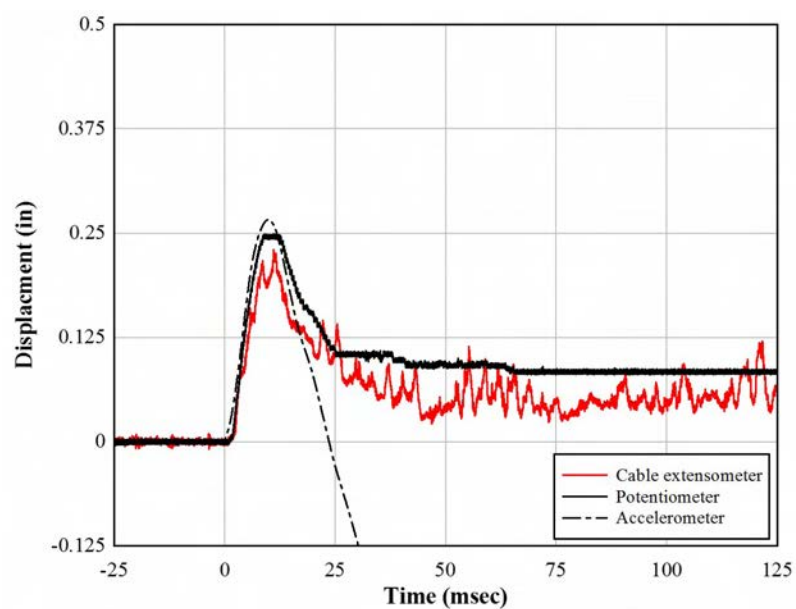


Figure 4.33. Displacement Time-History, 5-2b-50-4(1), Shot 3

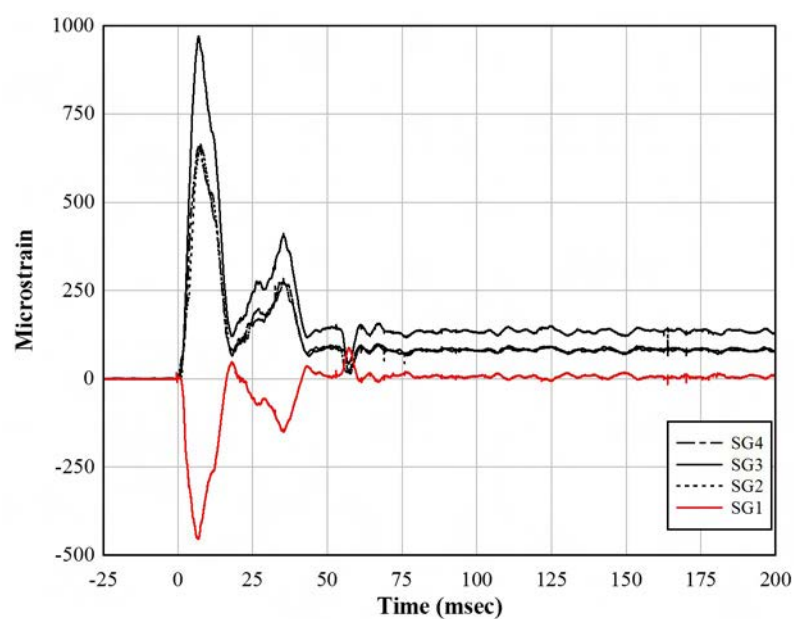


Figure 4.34. Strain Time-History, 3-2-65-4(1), Shot 3

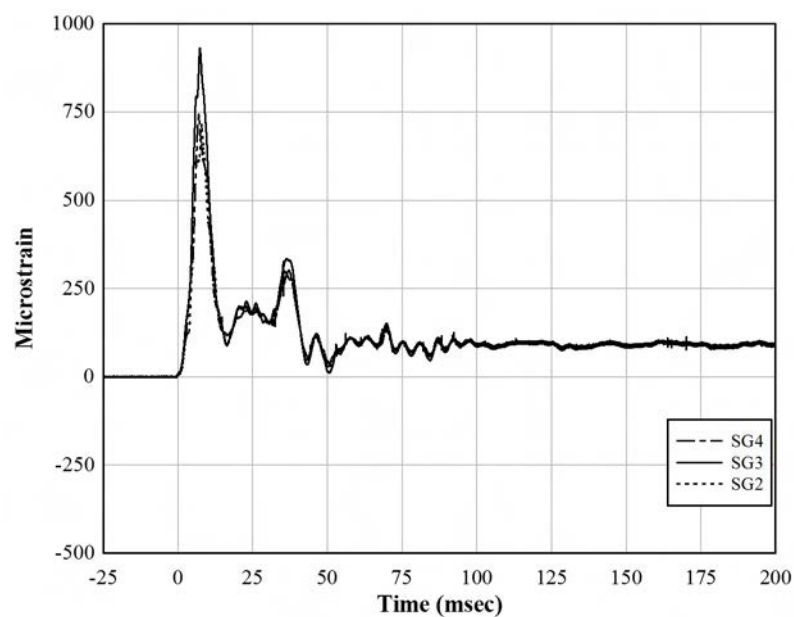


Figure 4.35. Strain Time-History, 5-2-50-4(1), Shot 3

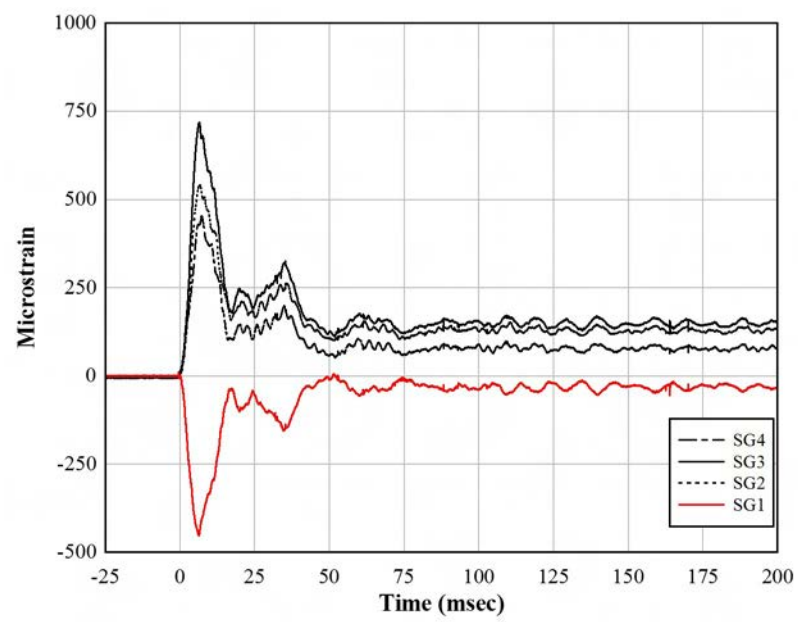
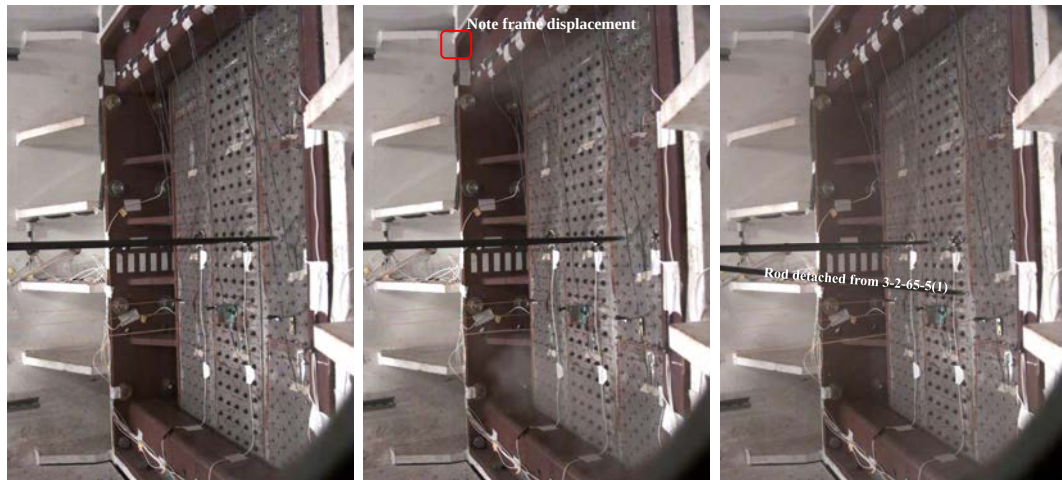


Figure 4.36. Strain Time-History, 5-2b-50-4(1), Shot 3



(a) Pressure Arrival, t_a (b) Max. Displ., $t_a + 11\text{msec}$ (c) Rest, $t_a + 200\text{msec}$

Figure 4.37. High Speed Video Frames from Test 2, Shot 4: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)

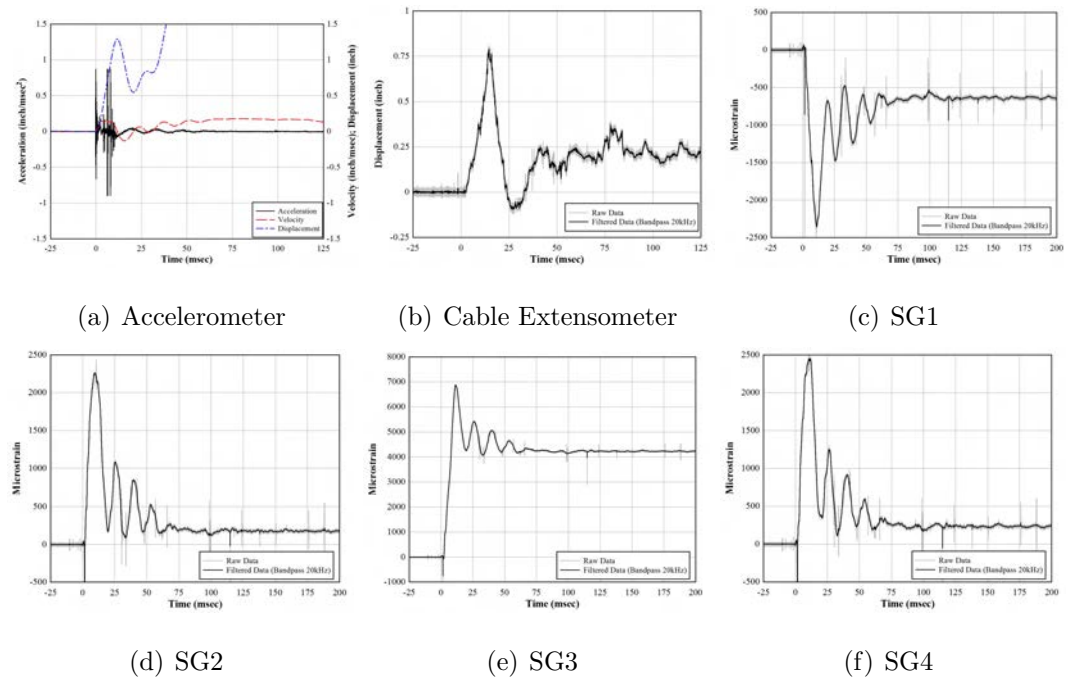


Figure 4.38. Raw and Filtered Data, 3-2-65-5(1), Shot 4

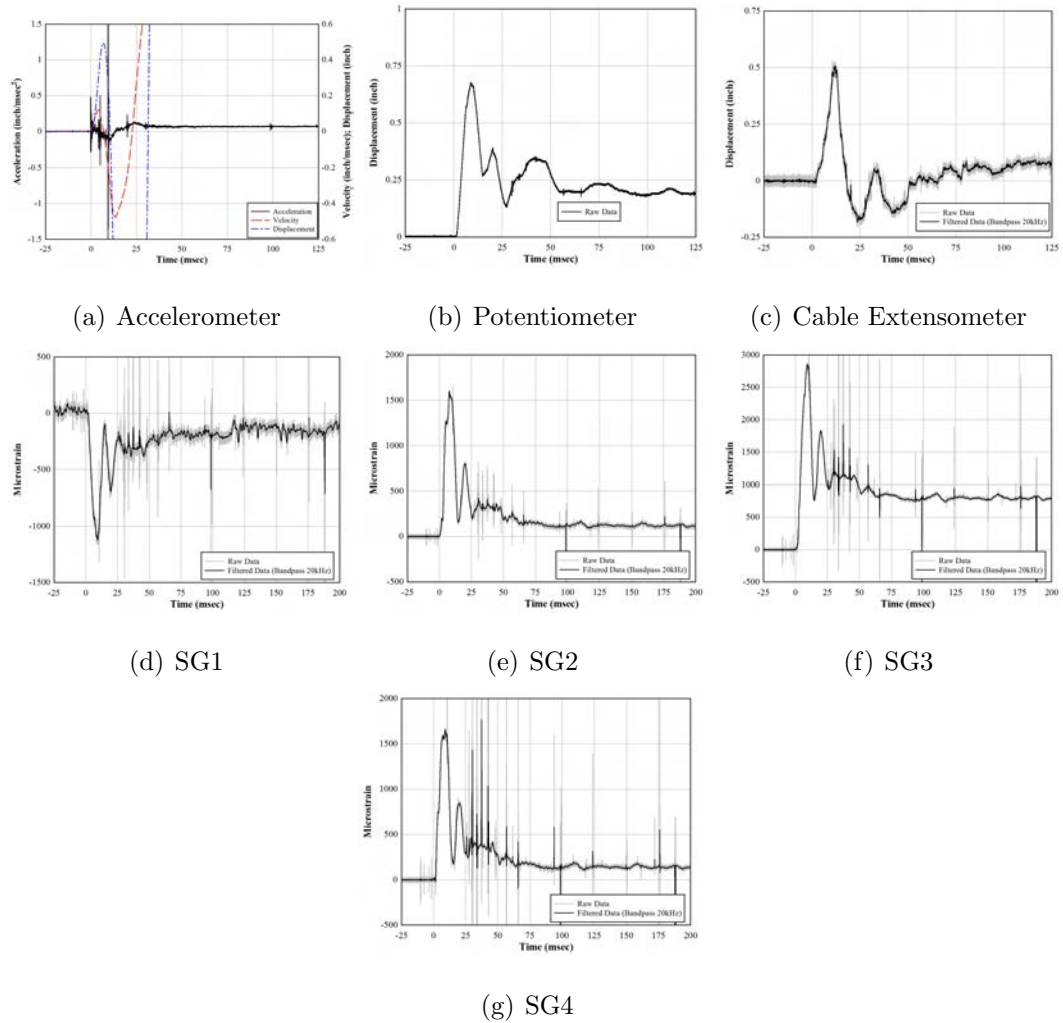
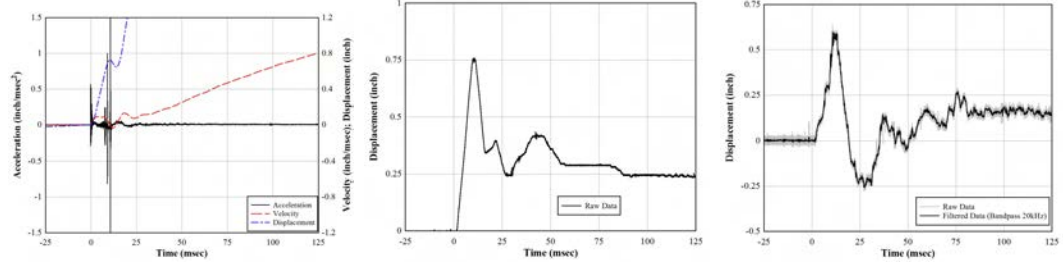


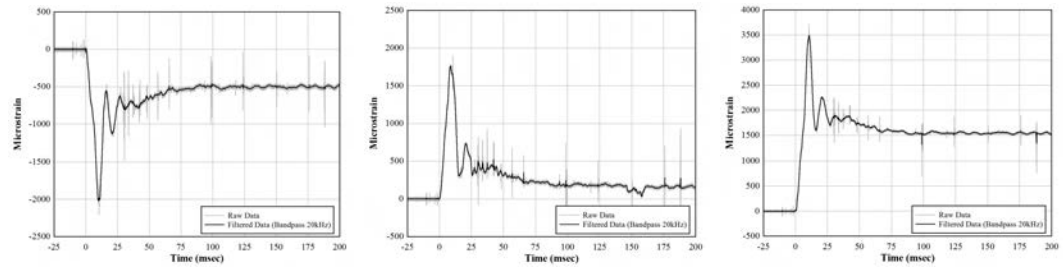
Figure 4.39. Raw and Filtered Data, 5-2-50-5(1), Shot 4



(a) Accelerometer

(b) Potentiometer

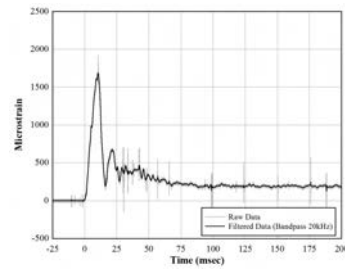
(c) Cable Extensometer



(d) SG1

(e) SG2

(f) SG3



(g) SG4

Figure 4.40. Raw and Filtered Data, 5-2b-50-5(1), Shot 4

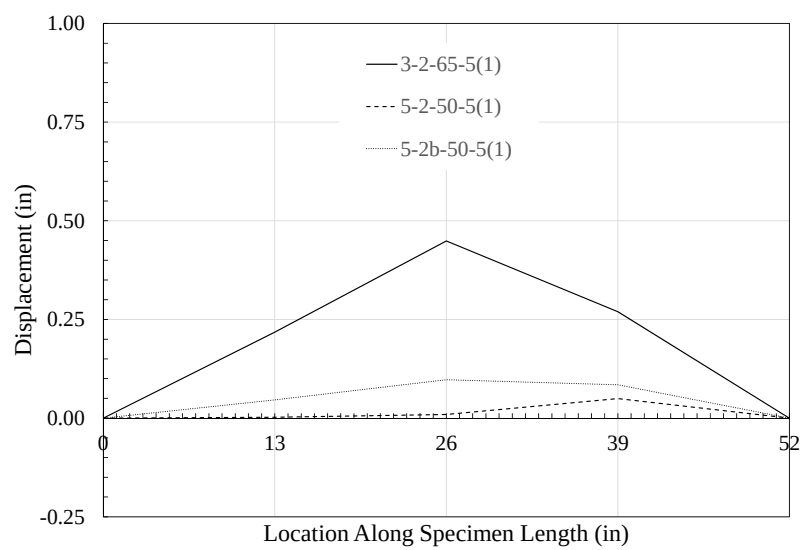


Figure 4.41. Post-Test Residual Displacement for Test 2, Shot 4: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)

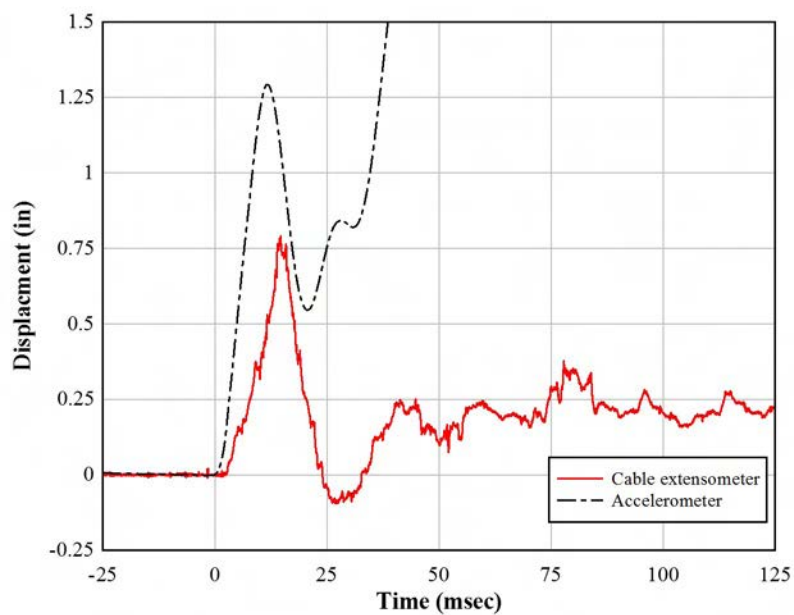


Figure 4.42. Displacement Time-History, 3-2-65-4(1), Shot 4

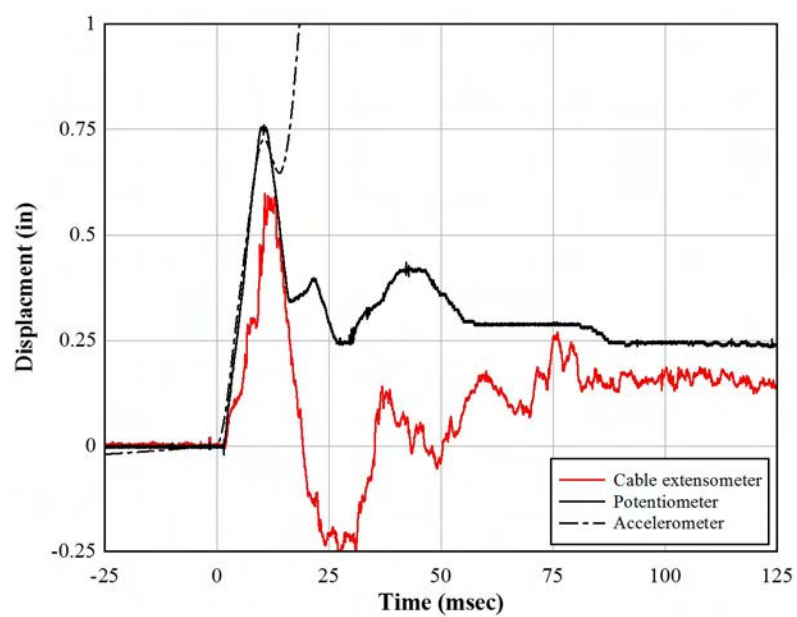


Figure 4.43. Displacement Time-History, 5-2-50-4(1), Shot 4

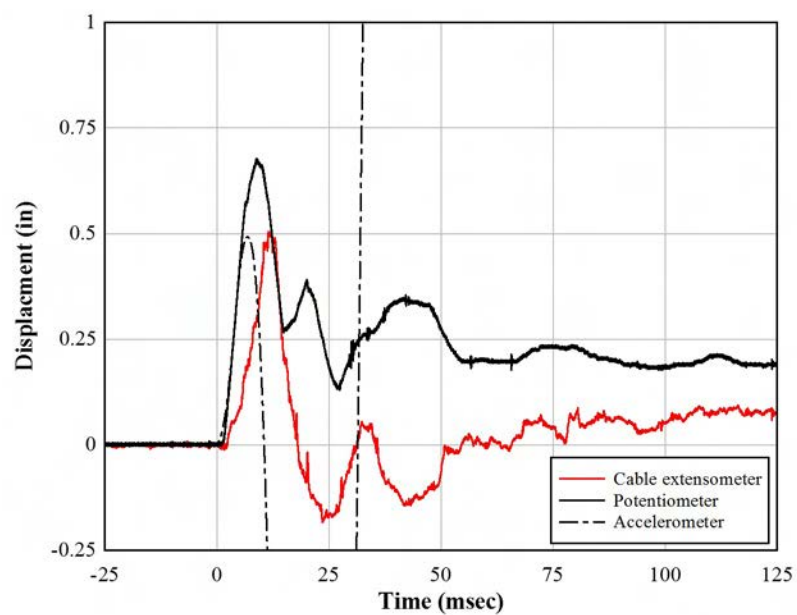


Figure 4.44. Displacement Time-History, 5-2b-50-4(1), Shot 4

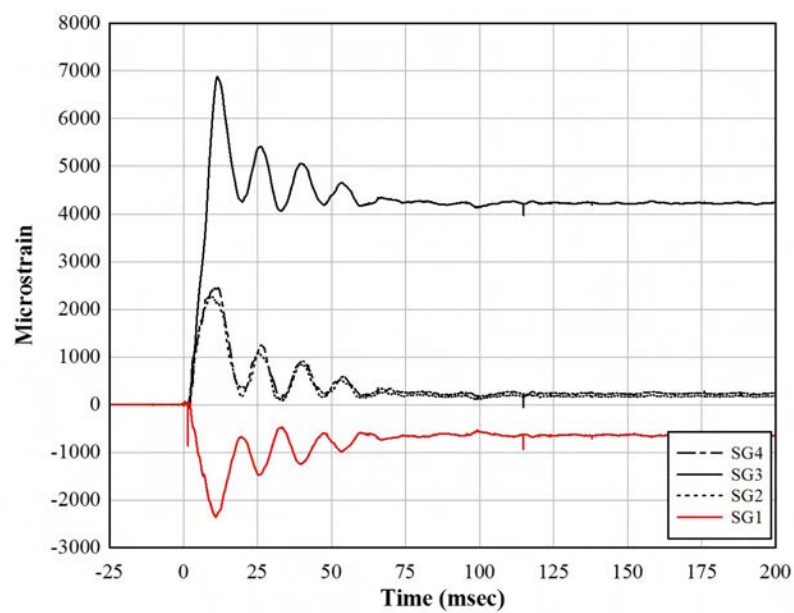


Figure 4.45. Strain Time-History, 3-2-65-4(1), Shot 4

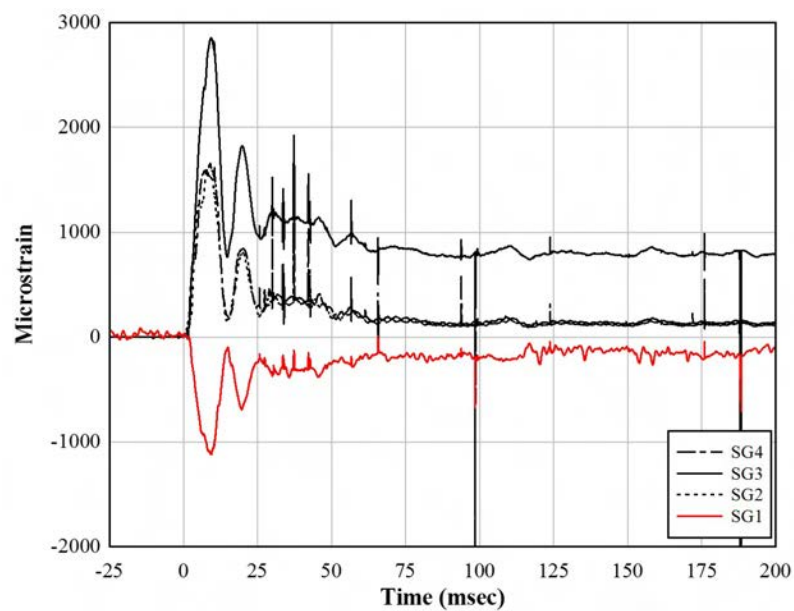


Figure 4.46. Strain Time-History, 5-2-50-4(1), Shot 4

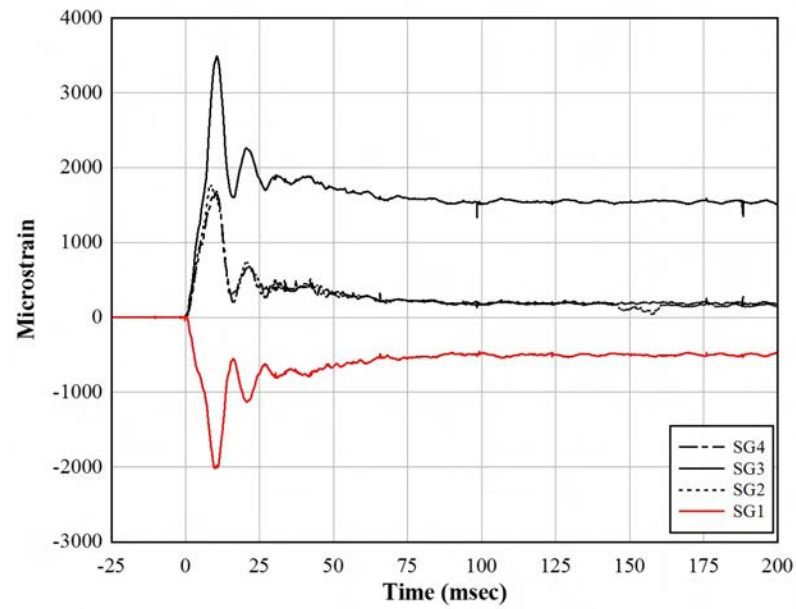


Figure 4.47. Strain Time-History, 5-2b-50-4(1), Shot 4



(a) 3-2-65-5(1)

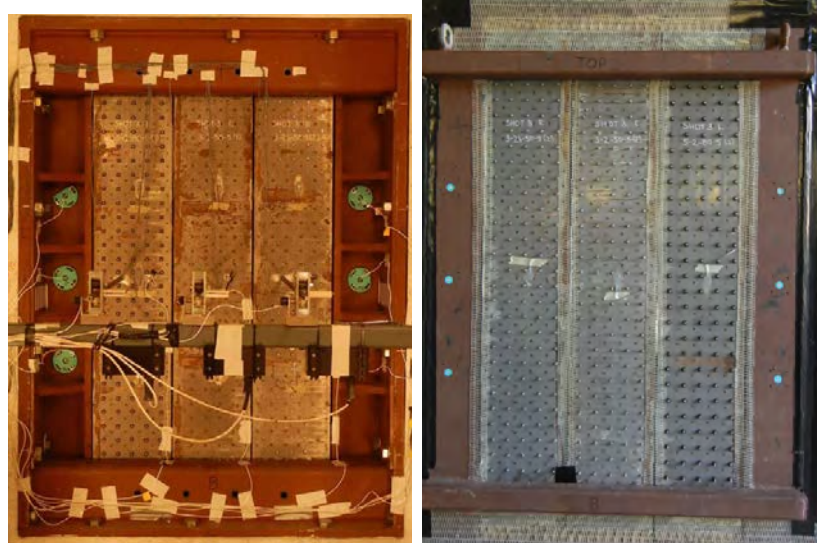


(b) 5-2-50-5(1)



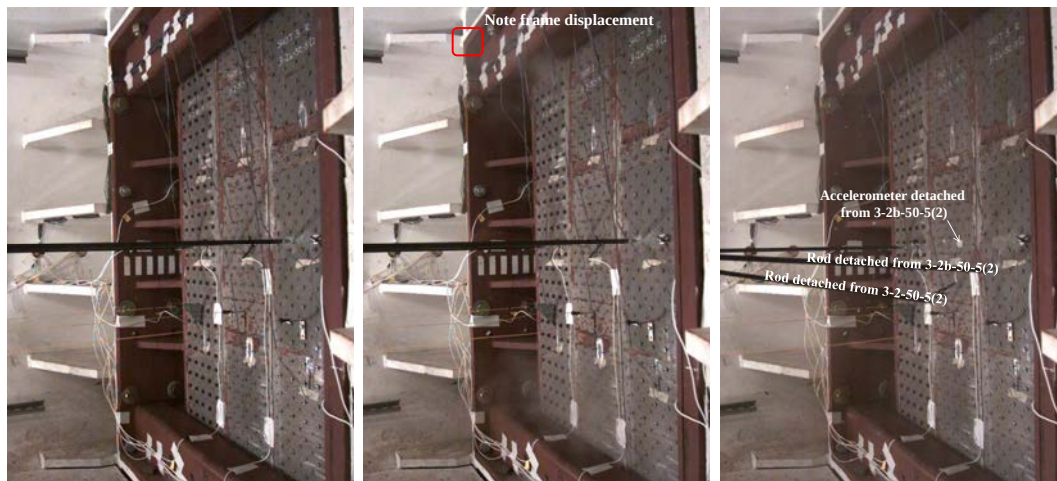
(c) 5-2b-50-5(1)

Figure 4.48. Post-Test Crack Patterns from Test 2: 3-2-65-5(1), 5-2-50-5(1), and 5-2b-50-5(1)



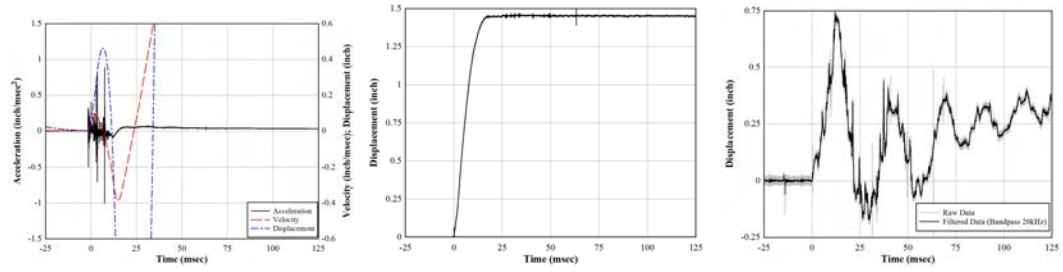
(a) View from Inside Target Vessel (b) View from Outside Target Vessel (Blast Face)

Figure 4.49. Pre-Test Photos for Test 3, Shot 5: 3-2-50-5(2), 3-2b-50-5(2), and 5-2-80-5(1)



(a) Pressure Arrival, t_a (b) Max. Displ., $t_a + 12\text{msec}$ (c) Rest, $t_a + 200\text{msec}$

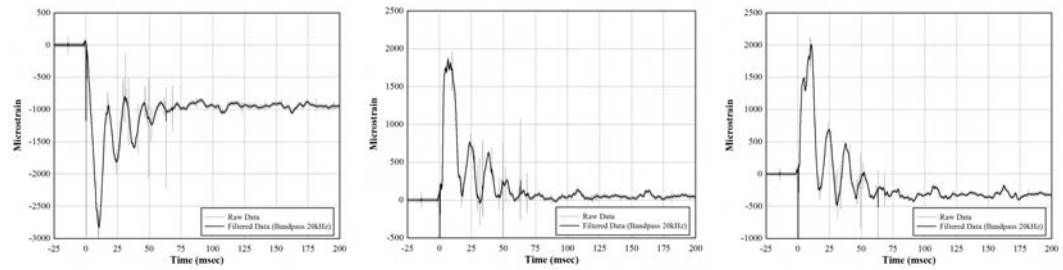
Figure 4.50. High Speed Video Frames from Test 3, Shot 5: 3-2-50-5(2), 3-2b-50-5(2), and 5-2-80-5(1)



(a) Accelerometer

(b) Potentiometer

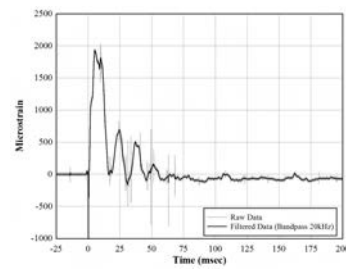
(c) Cable Extensometer



(d) SG1

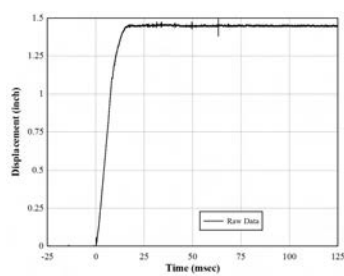
(e) SG2

(f) SG3

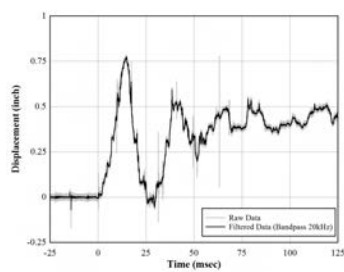


(g) SG4

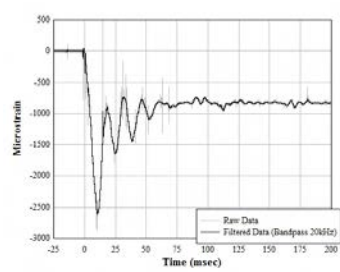
Figure 4.51. Raw and Filtered Data, 3-2-50-5(2), Shot 5



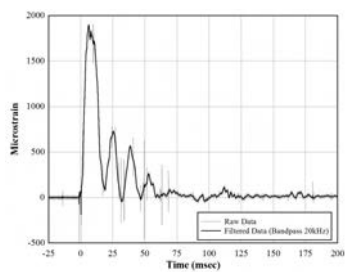
(a) Potentiometer



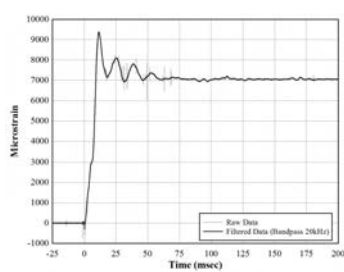
(b) Cable Extensometer



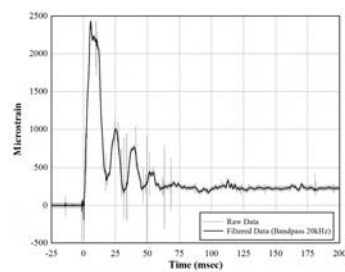
(c) SG1



(d) SG2



(e) SG3



(f) SG4

Figure 4.52. Raw and Filtered Data, 3-2b-50-5(2), Shot 5

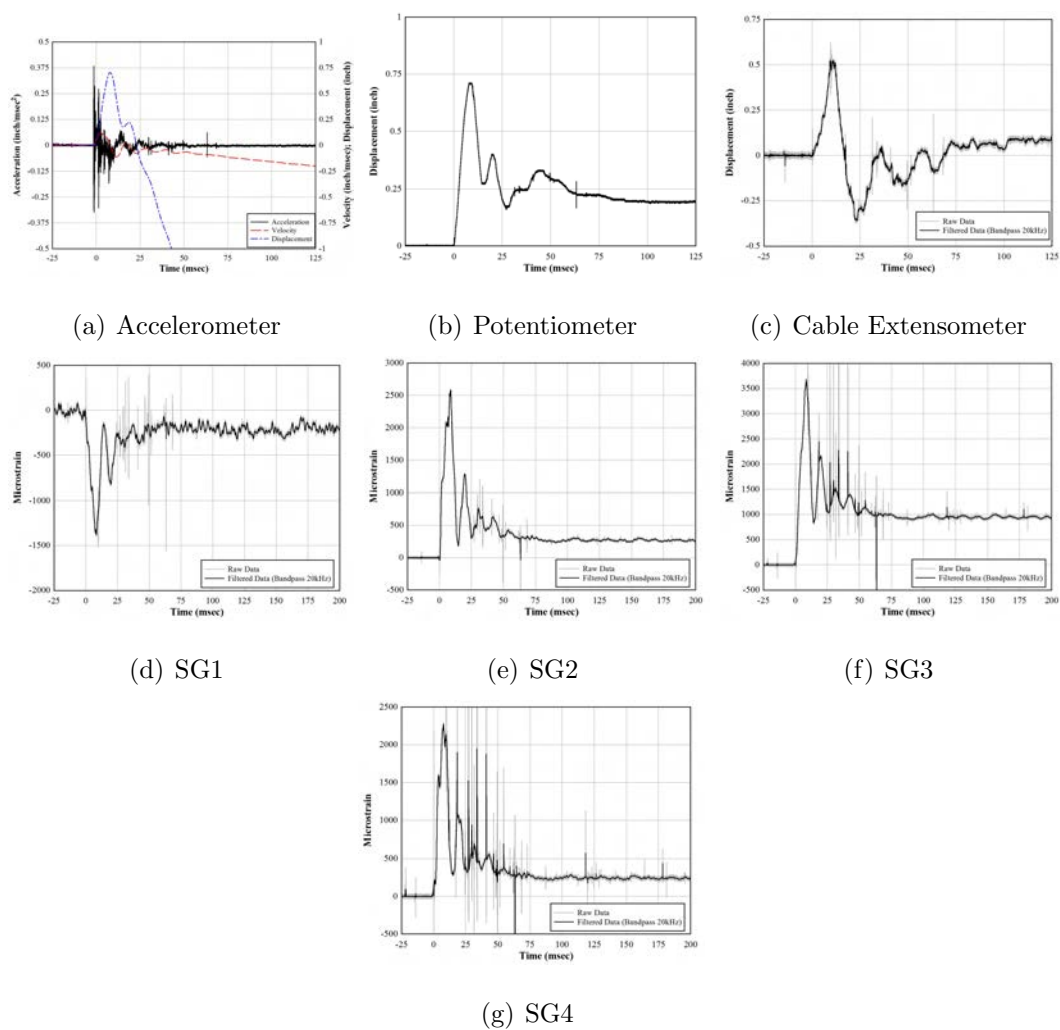


Figure 4.53. Raw and Filtered Data, 5-2-80-5(1), Shot 5

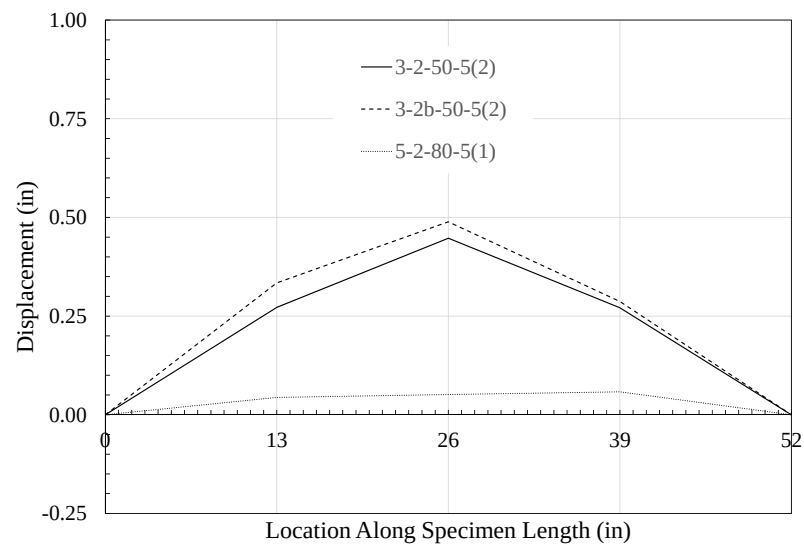


Figure 4.54. Post-Test Residual Displacement for Test 3, Shot 5: 3-2-50-5(2), 3-2b-50-5(2), and 5-2-80-5(1)

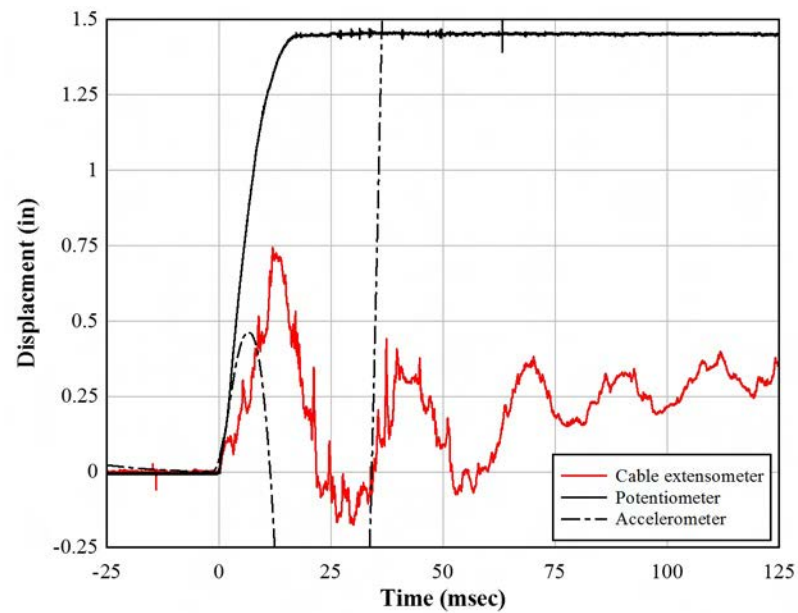


Figure 4.55. Displacement Time-History, 3-2-50-4(2), Shot 5

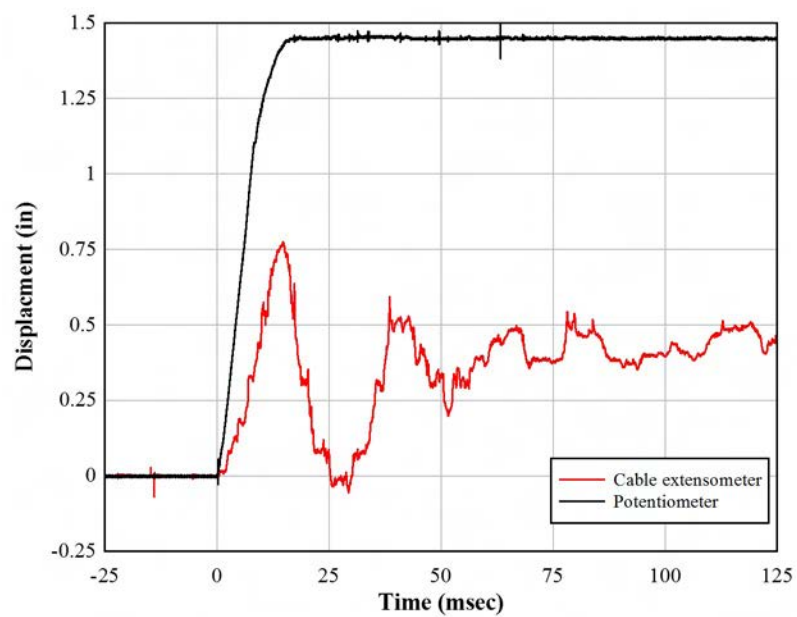


Figure 4.56. Displacement Time-History, 3-2b-50-4(2), Shot 5

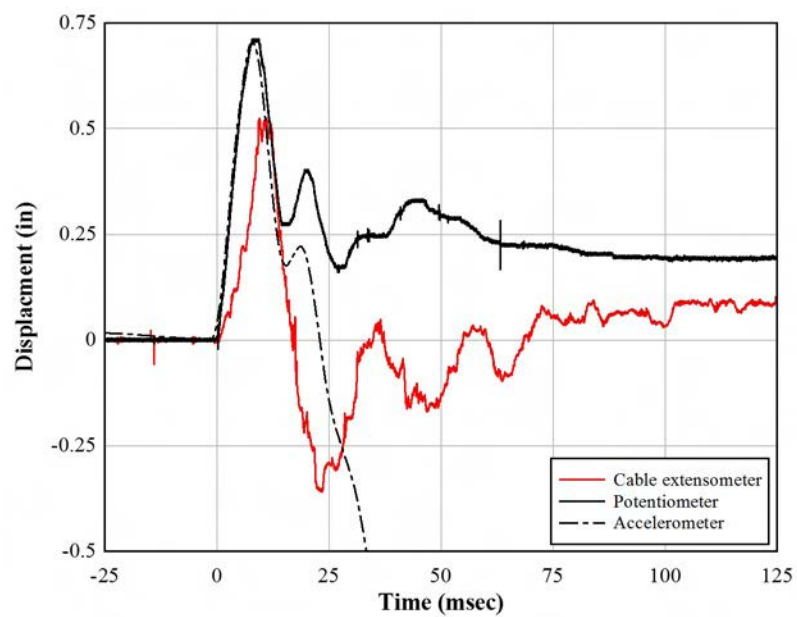


Figure 4.57. Displacement Time-History, 5-2-80-4(1), Shot 5

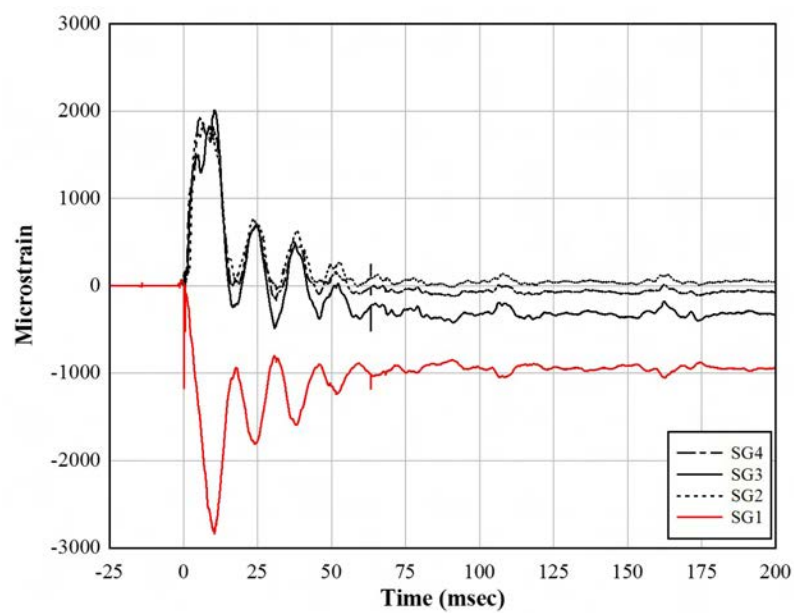


Figure 4.58. Strain Time-History, 3-2-50-4(2), Shot 5

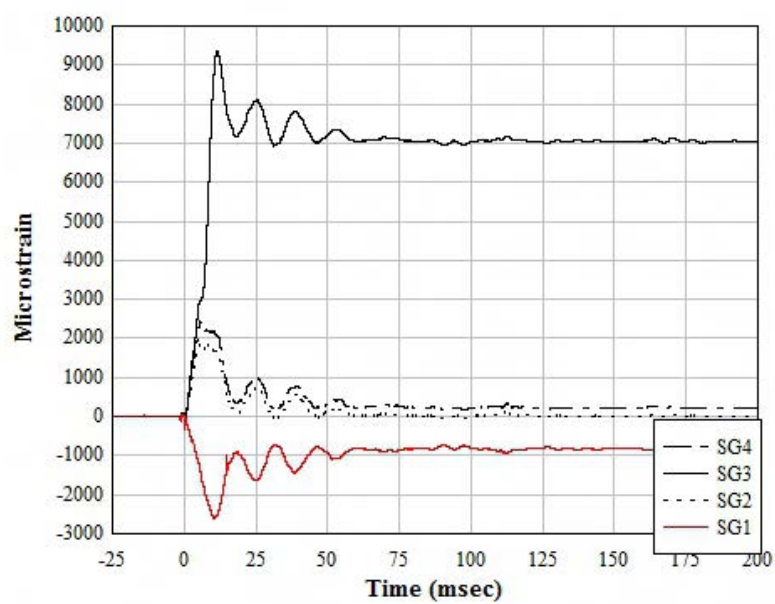


Figure 4.59. Strain Time-History, 3-2b-50-4(2), Shot 5

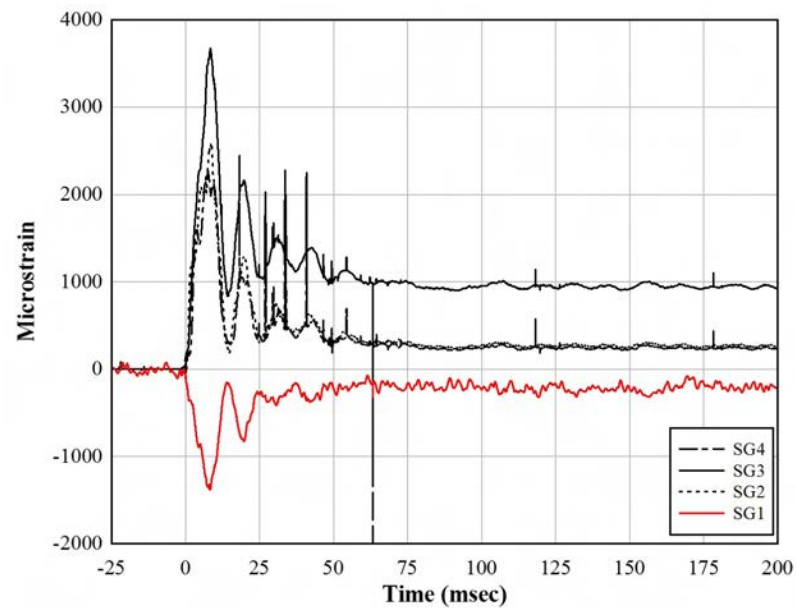


Figure 4.60. Strain Time-History, 5-2-80-4(1), Shot 5



(a) 3-2-50-5(2)



(b) 3-2b-50-5(2)



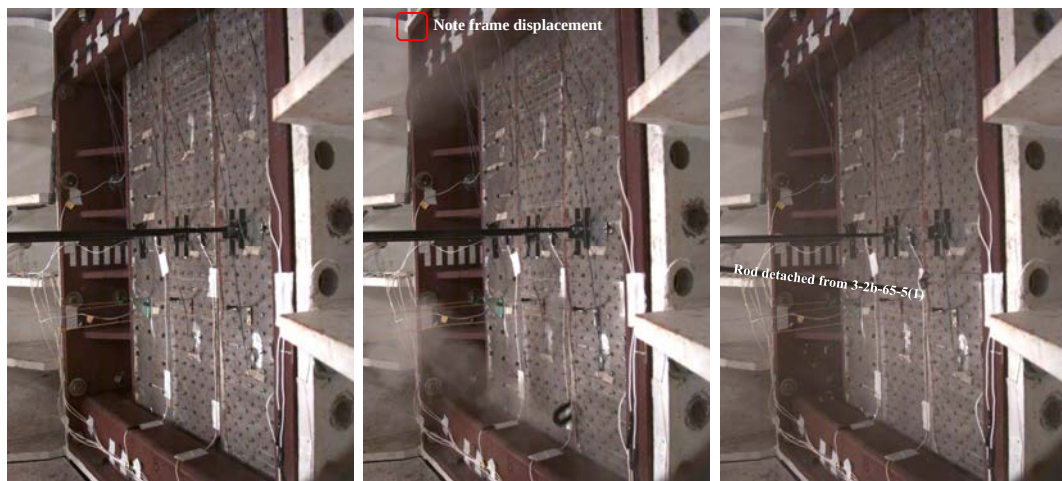
(c) 5-2-80-5(1)

Figure 4.61. Post-Test Crack Patterns from Test 3: 3-2-50-5(2), 3-2b-50-5(2), and 5-2-80-5(1)



(a) View from Inside Target Vessel (b) View from Outside Target Vessel (Blast Face)

Figure 4.62. Pre-Test Photos for Test 4, Shot 6: 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2)



(a) Pressure Arrival, t_a (b) Max. Displ., $t_a + 12\text{msec}$ (c) Rest, $t_a + 200\text{msec}$

Figure 4.63. High Speed Video Frames from Test 4, Shot 6: 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2)

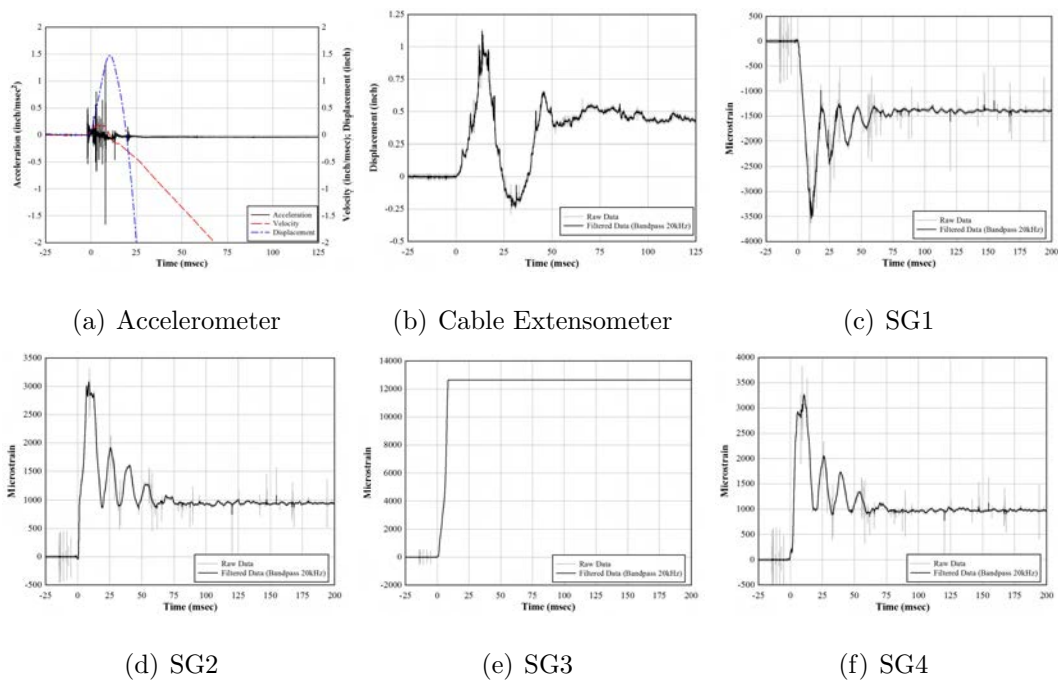
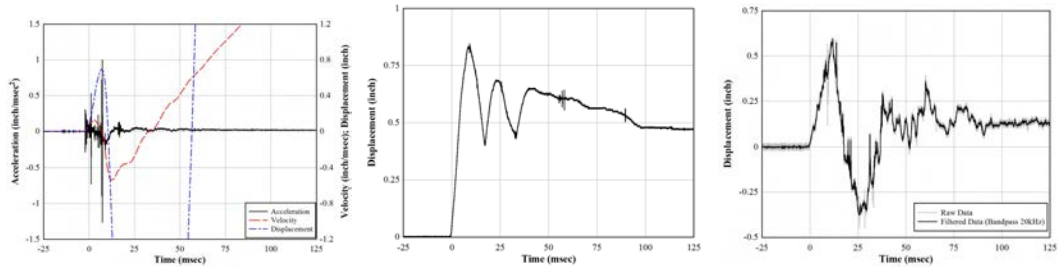


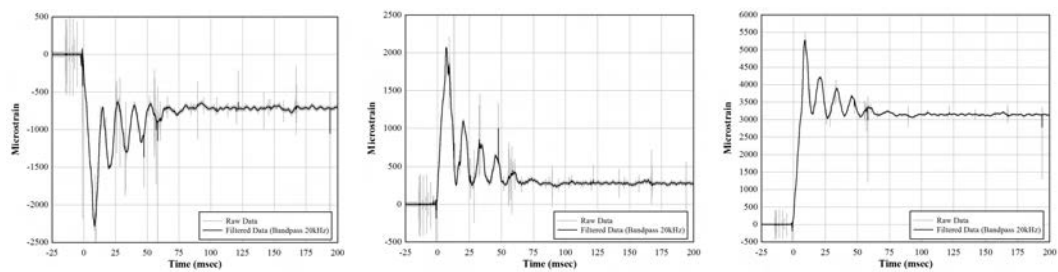
Figure 4.64. Raw and Filtered Data, 3-2b-65-5(1), Shot 6



(a) Accelerometer

(b) Potentiometer

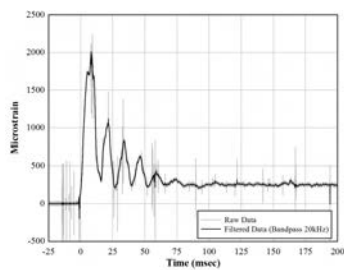
(c) Cable Extensometer



(d) SG1

(e) SG2

(f) SG3



(g) SG4

Figure 4.65. Raw and Filtered Data, 5-2-50-5(2), Shot 6

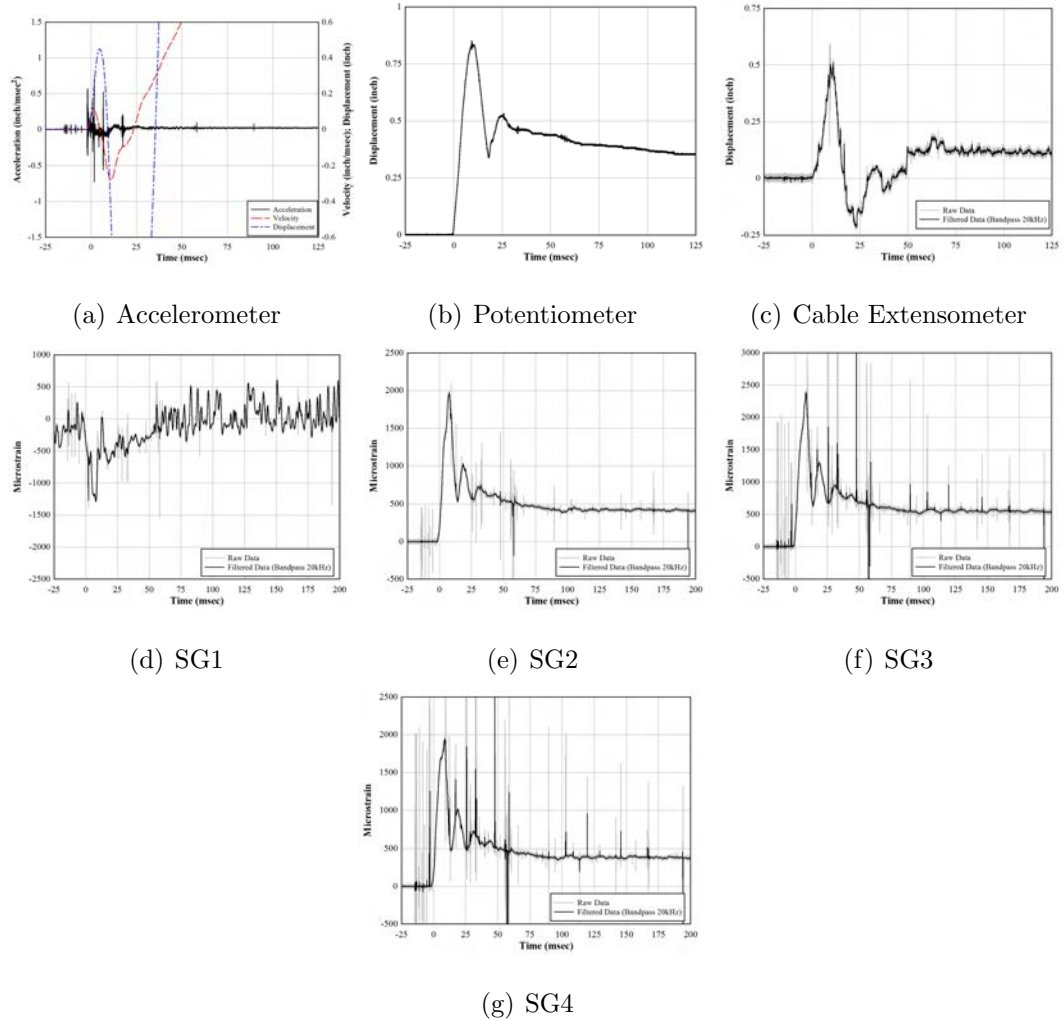


Figure 4.66. Raw and Filtered Data, 5-4-50-5(2), Shot 6

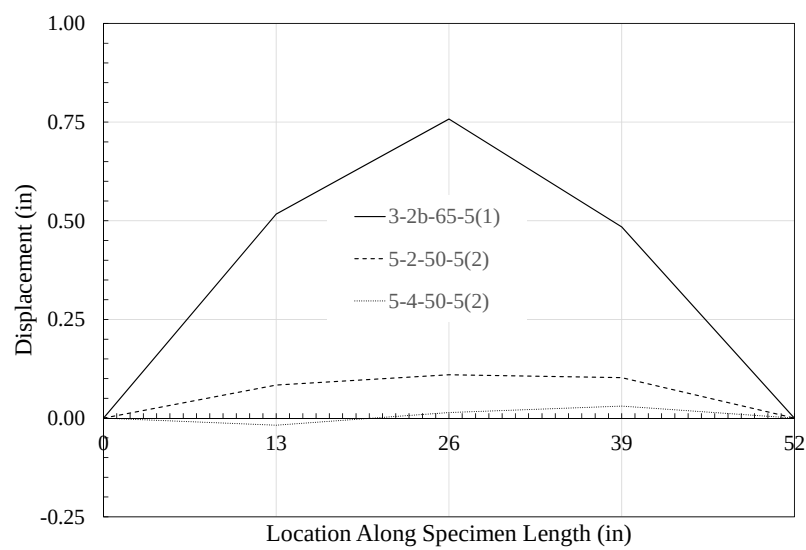


Figure 4.67. Post-Test Residual Displacement for Test 4, Shot 6: 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2)

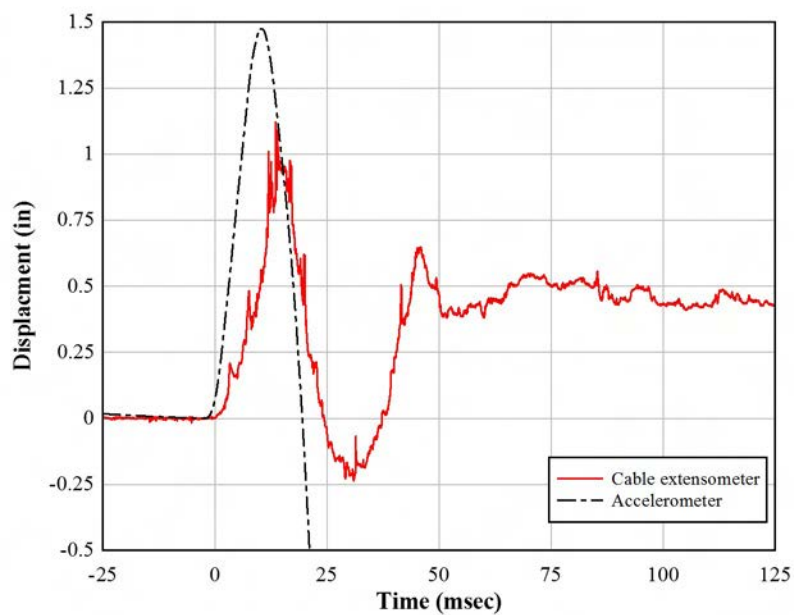


Figure 4.68. Displacement Time-History, 3-2b-65-4(1), Shot 6

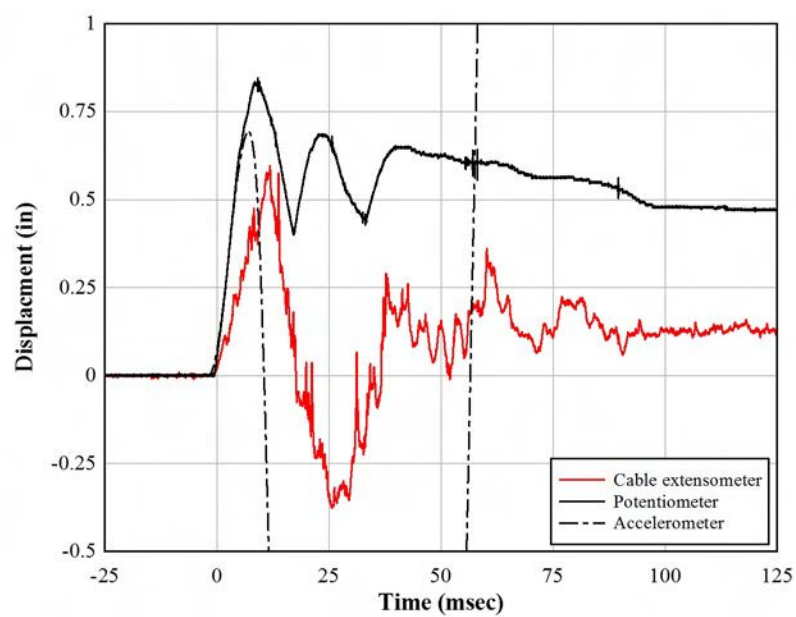


Figure 4.69. Displacement Time-History, 5-2-50-4(2), Shot 6

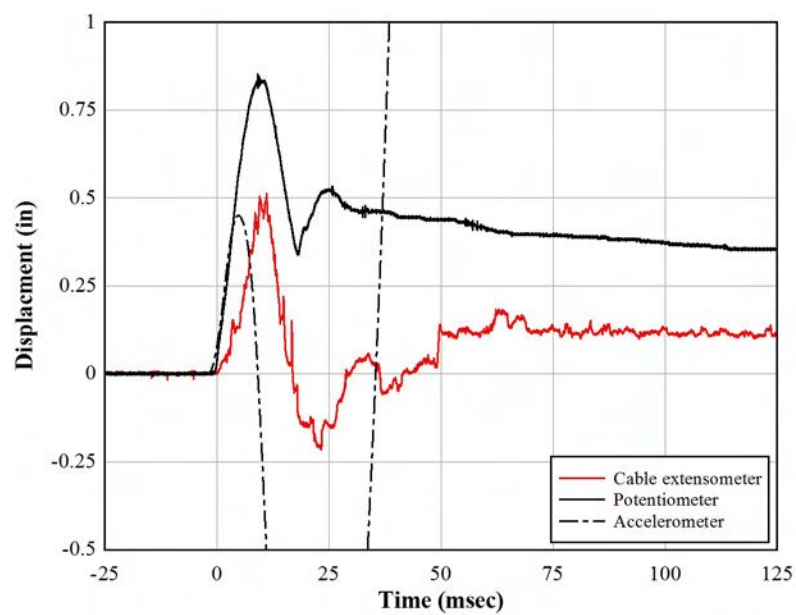


Figure 4.70. Displacement Time-History, 5-4-50-4(2), Shot 6

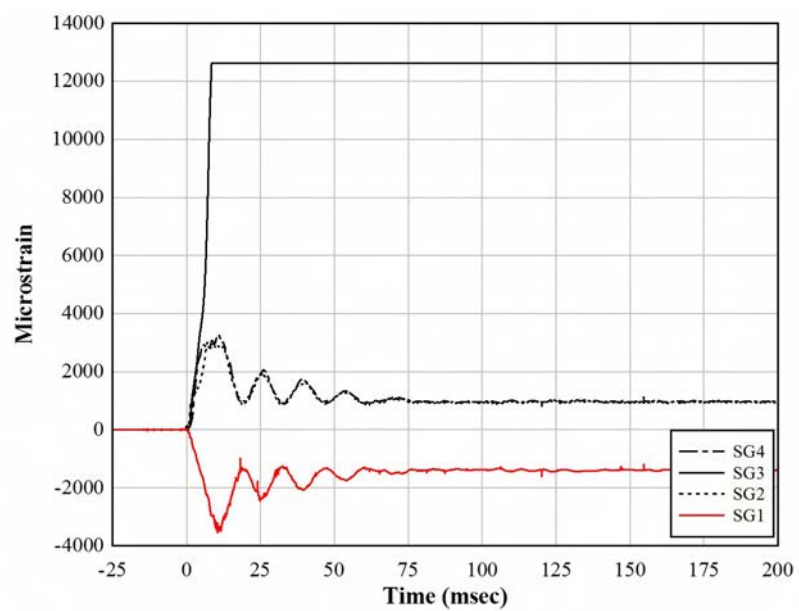


Figure 4.71. Strain Time-History, 3-2b-65-4(1), Shot 6

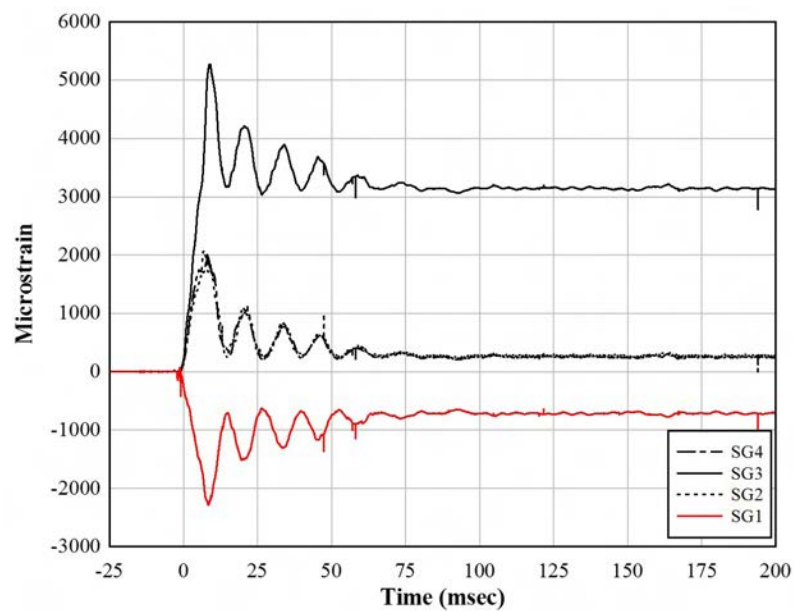


Figure 4.72. Strain Time-History, 5-2-50-4(2), Shot 6

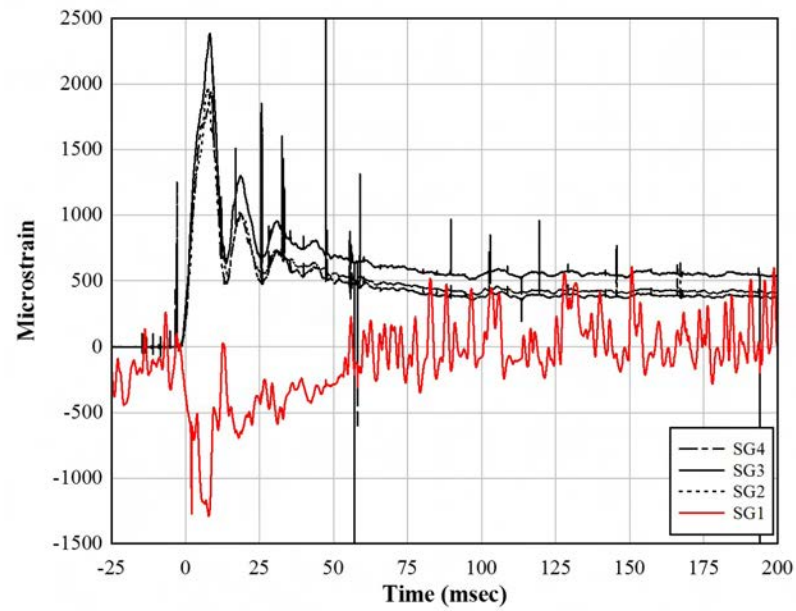


Figure 4.73. Strain Time-History, 5-4-50-4(2), Shot 6



(a) 3-2b-65-5(1)



(b) 5-2-50-5(2)



(c) 5-4-50-5(2)

Figure 4.74. Post-Test Crack Patterns from Test 4: 3-2b-65-5(1), 5-2-50-5(2), and 5-4-50-5(2)

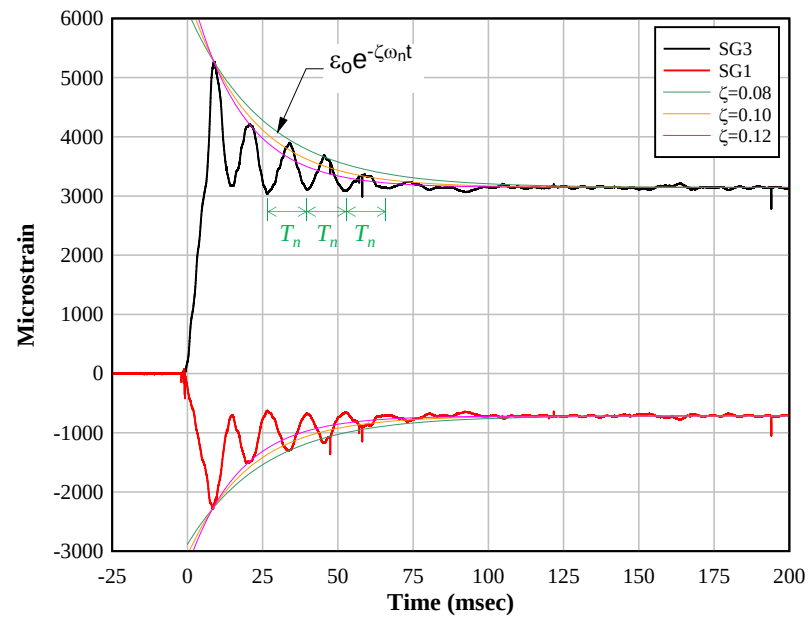
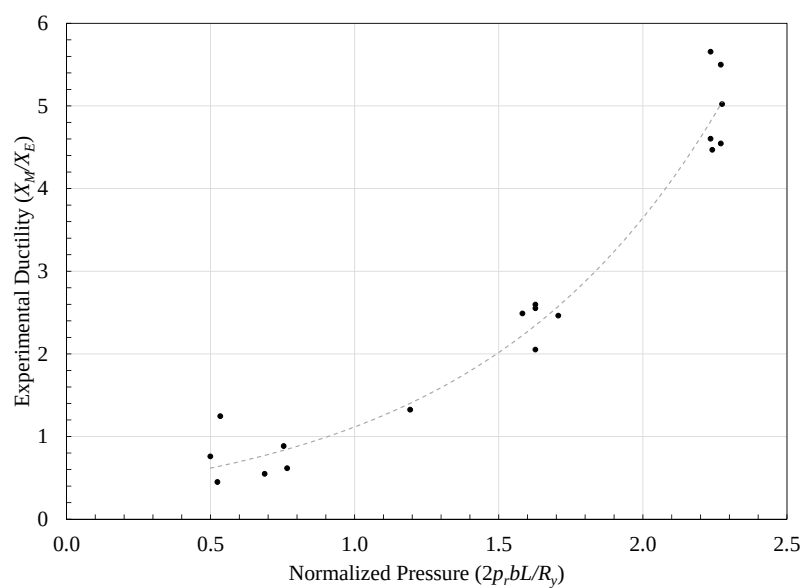
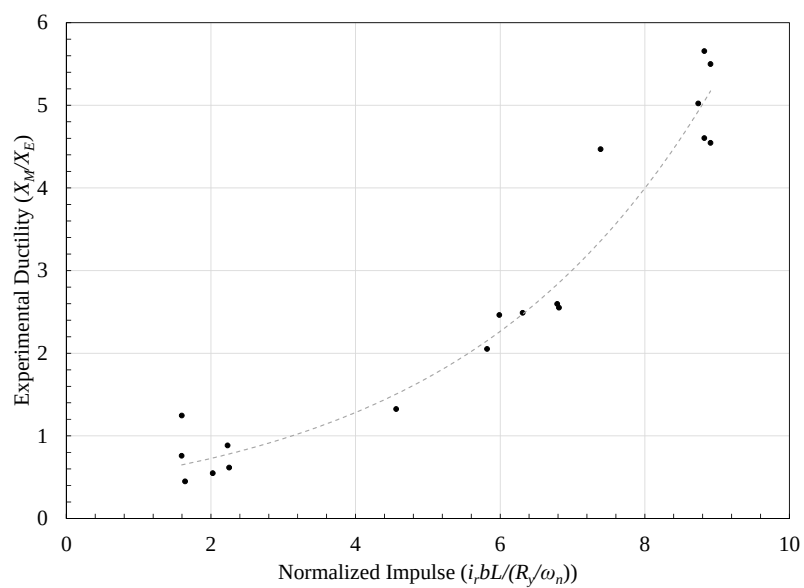


Figure 4.75. Experimental Determination of One-Way SC Wall Natural Period and Damping Ratio

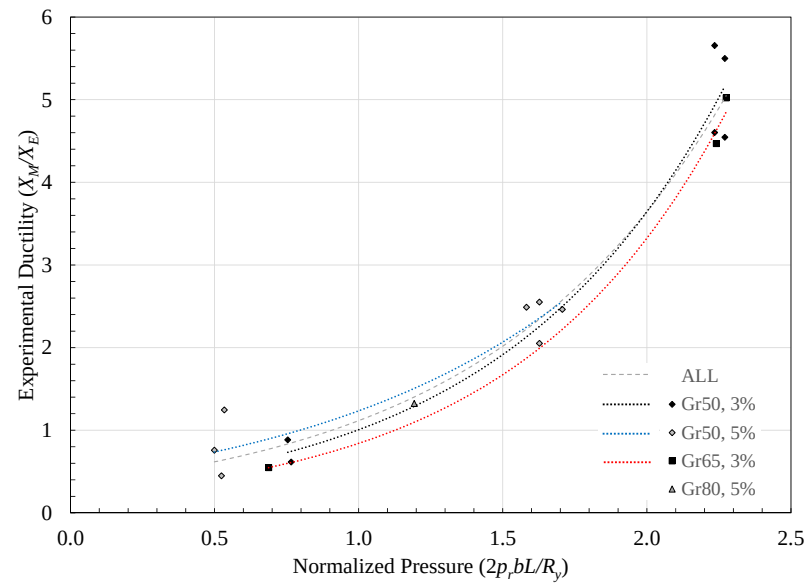


(a) Compared to Pressure

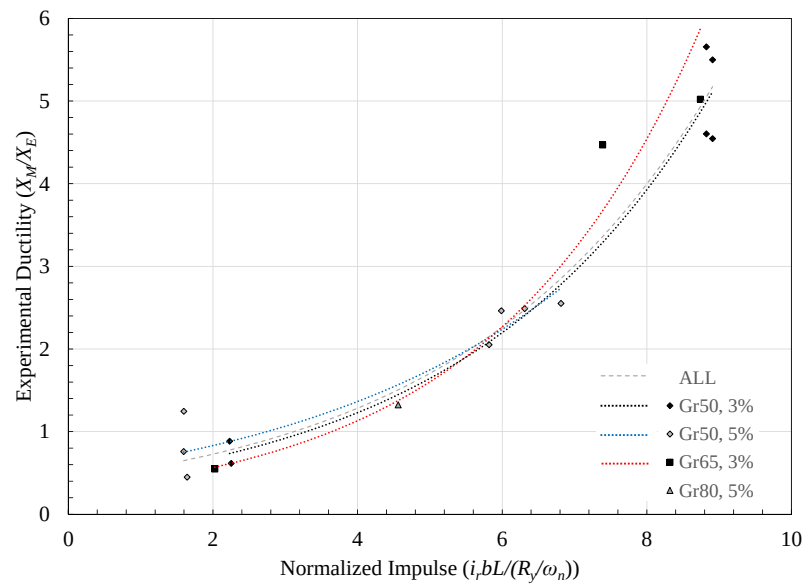


(b) Compared to Impulse

Figure 4.76. Ductility Demand Comparison for Twelve Blast Test Specimens

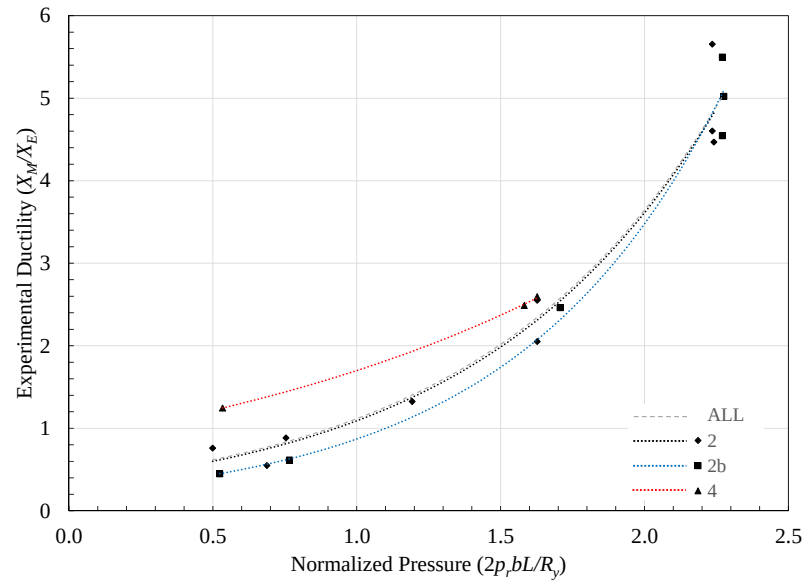


(a) Compared to Pressure

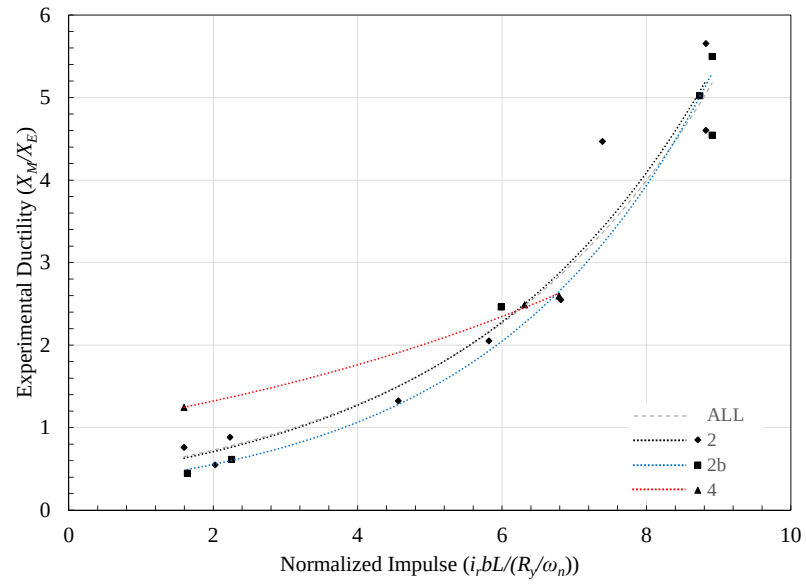


(b) Compared to Impulse

Figure 4.77. Influence of Steel Plate Strength and Reinforcement Ratio on Ductility Demand



(a) Compared to Pressure



(b) Compared to Impulse

Figure 4.78. Influence of Tie Bar Spacing and Diameter on Ductility Demand

5. BENCHMARKED MODELS OF BLAST LOADED SC WALLS

SDOF and FE numerical models were benchmarked to results from the twelve specimens subjected to blast loads. Details of both methods are described in the following sections. Results of maximum and residual displacement at mid-span, X_M and X_r respectively, from both models are provided with experimentally measured displacement (corrected for frame displacement) in Table 5.1.

5.1 FE Modeling of SC Walls Subjected to Blast Loads

The FE models created for static experiment benchmarking (see Section 3.4) were modified to benchmark the blast experiments:

- Material models were changed to account for strain rate effects on material strength,
- Supports were modified to reflect the support conditions provided by the BLS frame, and
- Loading was accomplished by pressure loading of the top surface of the top steel sheet.

Figure 5.1 provides a schematic of the FE model and Appendix B contains a list of all keywords used in the model.

5.1.1 Strain Rate Effects on Material Properties

It is well known that steel and concrete strength and stiffness are dependent on strain rate. In general, strength and stiffness increase as strain rate increases. The effect on strength is typically more dramatic than stiffness. Strain rate effects on

material strength were accounted for in these benchmarked models through use of dynamic increase factors (DIF).

The input for MAT_024 for all steel parts included “load curve, strain rate” (LCSR) input which defined a factor to increase yield stress for various strain rates (see Figure 5.2). This curve was taken from UFC 3-340-02 for A514 steel plate. It was developed based on an experimental database of tests of this steel at various strain rates. No such curve existed for A1011 steel sheets as used in this study but the chemical composition and static material behavior of A1011 is similar to A514. Having LCSR defined allowed LS-DYNA to select the DIF corresponding to the strain rate in each individual steel element during each time step of the analysis.

A similar curve could not be defined within MAT_084/085 and initial models which implemented the internal Winfrith model strain rate effects (RATE=0 (on)) produced inaccurate results. Inaccurate and unreliable results have been reported by others using the strain rate effects within the Winfrith material model [87]. For this reason, rather than using internal strain rate modifications of the Winfrith model, DIFs were applied to f'_c , f_t , and G_f and the input variable RATE was set at 1 (off). DIFs applied to each property are provided in Table 5.2 and were selected from figures available in the literature. Measured strain rates from the blast experiments were $\approx 0.5\text{-sec}^{-1}$ for the lower pressure shots and $\approx 1.0\text{-sec}^{-1}$ for the higher pressure shots. These strain rates were used to select appropriate DIFs. Values for f'_c and f_t DIFs were selected from Figures 5.3 and 5.4 both of which were produced by [88] based on 50-years of experimental data from a variety of sources. The DIF for G_f was selected from Figure 5.5 which was produced by [89] based on rather limited data. The figures show variability in the data used to develop the average DIF curves for each of these concrete material properties. The coefficient of variation (CV) of the points on each figure within the strain rate range of interest in this study was 9.8% for f'_c and 25% for both the f_t and G_f DIFs. The influence of this variability on the FE model results is described in Section 5.1.4.

5.1.2 Idealized Pressure-Time History

A representative pressure-time history from each experiment was selected as the load curve for the FE models (see Figures 4.5-4.9). The measured time-history record was idealized using the modified-Friedlander waveform (Equation (3.3)) with a linear rise from 0 to p_r . The Friedlander best fit variables for each representative pressure time-history obtained from regression analysis using Microsoft Excel® are provided in Table 5.3.

5.1.3 Benchmarked Finite Element Model Results

Mid-span displacement time history for all twelve specimens are shown in Figures 5.6-5.9. Visual comparisons can be made with the experimental results explained in Section 4.1.3 keeping in mind that the figures of experimental results also include rigid body displacement of the frame assembly. The general shape of the time history is the same between experimental and FE results: time to peak displacement, period of vibration cycles, and time before vibration damped out to a constant residual displacement.

Results of maximum and residual mid-span displacements of the benchmarked FE models are close to experimental values when corrected for frame displacement, as shown Table 5.1 and Figure 5.10. A basic statistical comparison demonstrates that the FE models, on average, underestimated the maximum response by 4% with a high of 27% (Shot 1, 3-2b-50-5(1)) and low of -51% (Shot 1, 5-4-50-5(1)). See Table 5.4 for additional statistical information. In this table, R^2 was calculated as the ratio of the explained sum of squares to the total sum of squares compared to an ideal error of 0%.

Comparison of crack patterns from experimental and FE results demonstrates that the FE model accurately captured behavior of the one-way SC wall sections under blast loading. Visual comparisons are provided in Figures 5.11-5.14. In these figures, the top two images for each specimen are the cracks marked post-test on both

sides of the specimen and the bottom image is a fringe of the crack output parameter provided by the Winfrith concrete model. In these fringes, red denotes areas that were uncracked and blue denotes areas that were fully cracked (i.e. the tensile strength reduced to zero). The black lines on the FE images are cracks that were wider than 0.010-in as output by the Winfrith concrete model. These figures also demonstrate that the residual shape of the model of each specimen was similar to the experimental result.

5.1.4 Sensitivity to Concrete Material Properties

Variability of the experimental results is due in part to variability in material properties. The variability in the data used to select DIFs for concrete material properties was described in Section 5.1.1. To assess the influence of this DIF variability on numerical results, an additional 15 models were analyzed (Table 5.5). These models were based on specimen 3-2-50-5(1) and modified only the concrete material properties in accordance with the DIFs listed. Each DIF was varied by plus and minus the CV from the data used to generate the DIF curves in Figures 5.3 - 5.5. To evaluate the influence on both the elastic and plastic response of the specimen, two load cases were run: the pressure curve from Shot 1 and the pressure curve from Shot 5.

Maximum mid-span displacement from these models are shown in Figures 5.15-5.17. As the f'_c or f_t DIF increased, the displacement decreased for both the elastic and plastic response. The change in displacement was influenced more by the f_t DIF than f'_c . As seen in Figure 5.15, the percent change in mid-span displacement was about 0.25 to 0.30 times the percent change in f'_c DIF with larger changes in the results associated with the elastic response than the plastic response. This is because of the influence of f'_c on E_c and changes in elastic stiffness have more notable influence on elastic response. The mid-span displacement varied about 10% between the model with the smallest f'_c DIF and the one with the largest.

As seen in Figure 5.16, the percent change in mid-span displacement was about 0.39 to 0.73 times the percent change in f_t DIF with larger changes in the results associated with the plastic response. The shear strength of the specimen is directly related to f_t and the most dramatic influence of this DIF was noted as the DIF decreased. This reduction led to shear cracks forming at lower stresses which increased the total deflection. The mid-span displacement varied over 35% between the model with the smallest f_t DIF and the one with the largest.

As the G_f DIF increased so did the maximum mid-span displacement by a small amount. As seen in Figure 5.17, the percent change in mid-span displacement was about 0.03 to 0.09 times the percent change in G_f DIF with larger changes in the results associated with the plastic response. The influence on the elastic response was negligible. The mid-span displacement for the plastic response varied less than 10% between the model with the smallest G_f DIF and the one with the largest. Influence was most significant for lower values of this DIF.

This sensitivity study demonstrated that between f'_c , f_t , and G_f , the influence on the result was most notable with changes in f_t . This highlights the influence of tensile strength on the performance of SC walls subjected to blast loads. The influence of strain rate on f_t and G_f are not as mature fields of research as the influence of strain rate on f'_c . Influence of strain rate on f_t is not included in the Winfrith concrete model and this is an important reason why results from models using the internal strain rate effects within the Winfrith model were inaccurate.

5.2 SDOF Modeling of SC Walls Subjected to Blast Loads

While FE models provide detailed information of how a structure or structural element responds to a blast load, an SDOF model takes less time to create and is a common tool in blast resistant design. For these benchmarked models, the resistance function was idealized as bi-linear with strain hardening as developed in Section 3.6.2. The measured weight of the specimens was used in the SDOF calculation and the

forcing function was the Friedlander waveform using the best fit variables in Table 5.3. The DIF for f'_c was the same as in the FE model and a DIF of 1.12 was used for the F_y . No damping was used in these benchmarking calculations.

The equation of motion was numerically solved using the constant velocity assumption within each time step (a method also known as the acceleration impulse extrapolation method). A MATLAB script was written to solve the equation of motion using a time step equal to $T_n/25$. Using time steps smaller than this did not change the calculated maximum displacement. The script was validated using several worked examples from dynamics textbooks.

5.2.1 Benchmarked SDOF Model Results

As the data in Table 5.1 and Figure 5.18 shows, the results of maximum mid-span and residual displacements of the benchmarked SDOF models were reasonable and generally conservative when compared to experimental values corrected for frame movement. A basic statistical comparison shows that the SDOF models, on average, overestimated the maximum response by 17% with a high of 69% (Shot 4, 5-2-50-5(1)) and low of -40% (Shot 1, 5-4-50-5(1)). See Table 5.4 for additional statistical information.

5.2.2 Sensitivity to Equivalent Viscous Damping

Analysis procedures described in UFC 3-340-02 neglect damping because peak displacement is of primary concern and transient vibration is of little interest in most blast resistant design applications. Neglecting damping provides a conservative estimate of peak displacement because all real structures have some inherent damping. As described in Section 4.2, experimental results indicated a damping ratio of 5-10% for the specimens in this study. To investigate the influence of damping on SDOF results, all twelve cases were run including 5% and 10% of critical damping. Results are provided in Table 5.6 and a basic statistical comparison is shown in Table 5.4.

Statistical analysis demonstrates that the closest comparison to experimental results was obtained by including 5% damping in the calculation. This underestimated the maximum response by 8% on average with a high of 48% (Shot 3, 5-2b-50-5(1)) and low of -39% (two cases: Shot 5, 3-2-50-5(2) and 3-2b-50-5(2)). As seen in Figure 5.19, while the statistical fit may be better, including 5% damping resulted in less conservative estimates of displacement than the model with 0% damping.

The ratios of calculated to experimental results for each tested specimen considering various levels of damping are shown in Figure 5.20. In this figure, points above the double black line (ratio of 1) indicate cases for which the model overestimated the displacement and points below the line indicate cases for which the model underestimated the displacement. This figure demonstrates that the influence of damping differs for each specimen configuration and load. A more general investigation of the influence of damping ratio on SDOF results is included in the parametric studies in Chapter 6.

5.3 Chapter Summary

This chapter presented modeling methods of blast loaded one-way SC walls benchmarked to experimental results reported in Chapter 4. These models modified the methods used to create benchmarked models of static response of one-way SC walls developed in Chapter 3. The primary purpose of these benchmarked models was to enable a series of parametric studies and better understand the influence of design and load variables on structural response as described in the following chapter.

Table 5.1. Blast Specimen Mid-Span Maximum and Residual Displacement Comparisons

Panel	Shot	Experimental★		FEM		SDOF	
		X_M	X_r	X_M	X_r	X_M	X_r
		(in)	(in)	(in)	(in)	(in)	(in)
3-2-50-5(1)	1	0.182	†	0.164	0.000	0.170	0.000
3-2b-50-5(1)	1	0.130	†	0.165	0.000	0.172	0.000
5-4-50-5(1)	1	0.268	†	0.130	0.004	0.162	0.000
3-2-50-5(1)	2	0.948‡	‡	1.013	0.595	1.367	1.125
3-2b-50-5(1)	2	0.959‡	‡	1.036	0.634	1.353	1.110
5-4-50-5(1)	2	0.535‡	‡	0.627	0.188	0.473	0.145
3-2-65-5(1)	3	0.130	0.035	0.154	0.011	0.181	0.000
5-2-50-5(1)	3	0.164	0.059	0.098	0.000	0.183	0.000
5-2b-50-5(1)	3	0.096	0.083	0.096	0.000	0.154	0.000
3-2-65-5(1)	4	1.059	*	0.988	0.531	1.204	0.933
5-2-50-5(1)	4	0.443	0.194	0.533	0.161	0.750	0.414
5-2b-50-5(1)	4	0.527	0.241	0.529	0.164	0.558	0.250
3-2-50-5(2)	5	1.165	*	0.880	0.502	1.368	1.126
3-2b-50-5(2)	5	1.160	*	0.914	0.546	1.354	1.111
5-2-80-5(1)	5	0.417	0.193	0.495	0.086	0.491	0.004
3-2b-65-5(1)	6	1.180	*	1.055	0.616	1.236	0.961
5-2-50-5(2)	6	0.551	0.457	0.468	0.114	0.459	0.148
5-4-50-5(2)	6	0.558	0.340	0.529	0.127	0.555	0.262

★ measured values corrected to account for frame movement (Shot 2 values were not corrected because the panel displacement was taken directly from high-speed video); if available, data from displacement potentiometer is listed otherwise the data was calculated from accelerometer data

†, ‡, and *: see notes on Table 4.2

Table 5.2. Concrete Material DIFs for Benchmarked FE Models

Concrete Property	DIF
f'_c	1.3
f_t	1.6
G_f	1.2

Table 5.3. Friedlander Waveform Best Fit Values

	Test 1		Test 2		Test 3	Test4
	Shot 1	Shot 2 [‡]	Shot 3	Shot 4	Shot 5	Shot 6
p_r	20.2	59.9	18.9	61.6	59.9	61.6
t_a	2.2	7.0	4.9	6.8	7.0	6.8
t_d	44.9	81.3	43.8	82.1	81.3	81.9
α	2.174	4.609	2.168	4.562	4.609	4.211
R^2	0.881	0.898	0.774	0.841	0.898	0.903

[‡] values from Shot 5; data acquisition system for Shot 2 failed to trigger; similar rupture pressures for Shot 2 and 5

Table 5.4. Maximum Displacement Statistical Comparison of Benchmarked Models to Experimental Results

		SDOF	SDOF	SDOF
	FE	($\zeta = 0\%$)	($\zeta = 5\%$)	($\zeta = 10\%$)
Mean	-4%	+17%	-8%	-17%
Standard Deviation	+21%	+28%	26%	26%
R^2	0.968	0.724	0.911	0.672

Table 5.5. Combinations of DIFs for Sensitivity Study

DIF Varied	Name	f'_c DIF	f_t DIF	G_f DIF
f'_c	S_fc_1	1.17	1.6	1.2
	S_fc_2	1.25	1.6	1.2
	S_fc_3*	1.3	1.6	1.2
	S_fc_4	1.35	1.6	1.2
	S_fc_5	1.43	1.6	1.2
f_t	S_ft_1	1.3	1.2	1.2
	S_ft_2	1.3	1.4	1.2
	S_ft_3*	1.3	1.6	1.2
	S_ft_4	1.3	1.8	1.2
	S_ft_5	1.3	2	1.2
G_f	S_Gf_1	1.3	1.6	1
	S_Gf_2	1.3	1.6	1.1
	S_Gf_3*	1.3	1.6	1.2
	S_Gf_4	1.3	1.6	1.35
	S_Gf_5	1.3	1.6	1.5
All	S_ALL_L	1.17	1.2	1
	S_ALL_M*	1.3	1.6	1.2
	S_ALL_H	1.43	2	1.5

* These models were identical but are listed for completeness

Table 5.6. Sensitivity of SDOF Calculation to Damping

Panel	Shot	Maximum Mid-Span Displacement (in)			
		Experimental	SDOF		
			0% Damping	5% Damping	10% Damping
3-2-50-5(1)	1	0.182	0.170	0.157	0.145
3-2b-50-5(1)	1	0.130	0.172	0.159	0.147
5-4-50-5(1)	1	0.268	0.162	0.149	0.137
3-2-50-5(1)	2	0.948	1.367	0.707	0.639
3-2b-50-5(1)	2	0.959	1.353	0.705	0.638
5-4-50-5(1)	2	0.535	0.473	0.416	0.389
3-2-65-5(1)	3	0.130	0.181	0.167	0.154
5-2-50-5(1)	3	0.164	0.183	0.168	0.156
5-2b-50-5(1)	3	0.096	0.154	0.142	0.131
3-2-65-5(1)	4	1.059	1.204	1.011	0.638
5-2-50-5(1)	4	0.443	0.750	0.575	0.532
5-2b-50-5(1)	4	0.527	0.558	0.460	0.414
3-2-50-5(2)	5	1.165	1.368	0.707	0.639
3-2b-50-5(2)	5	1.160	1.354	0.705	0.638
5-2-80-5(1)	5	0.417	0.491	0.455	0.425
3-2b-65-5(1)	6	1.180	1.236	1.037	0.904
5-2-50-5(2)	6	0.551	0.459	0.425	0.376
5-4-50-5(2)	6	0.558	0.555	0.449	0.401

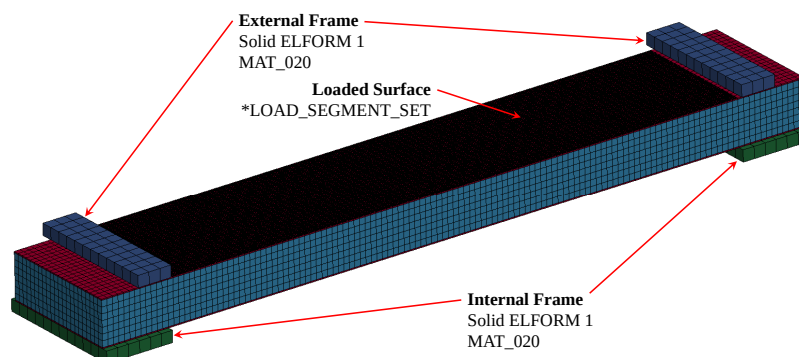


Figure 5.1. Finite Element Model of Blast Test Specimens

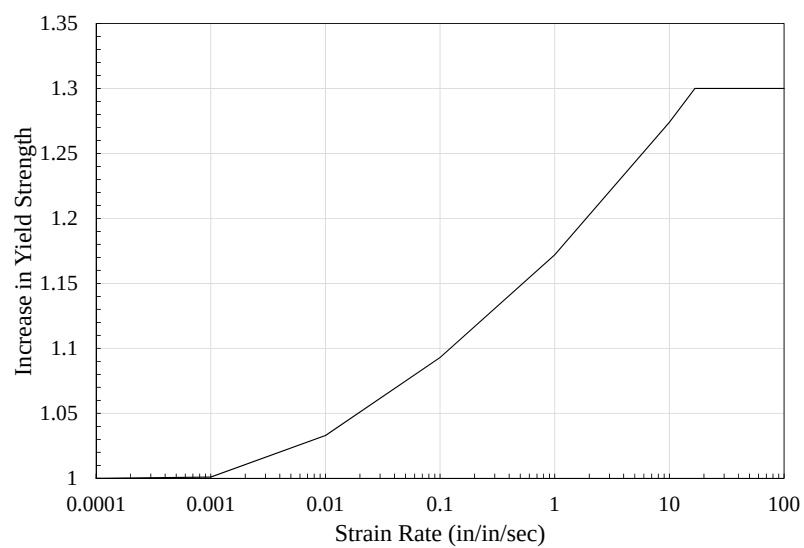


Figure 5.2. Strain Rate Effect on Steel Yield Strength (after [24])

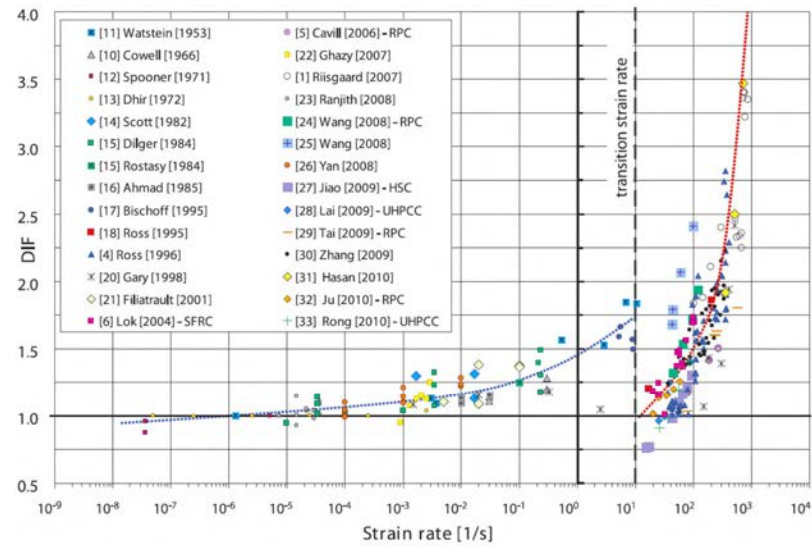


Figure 5.3. Strain Rate Effect on Concrete Compressive Strength (from [88])

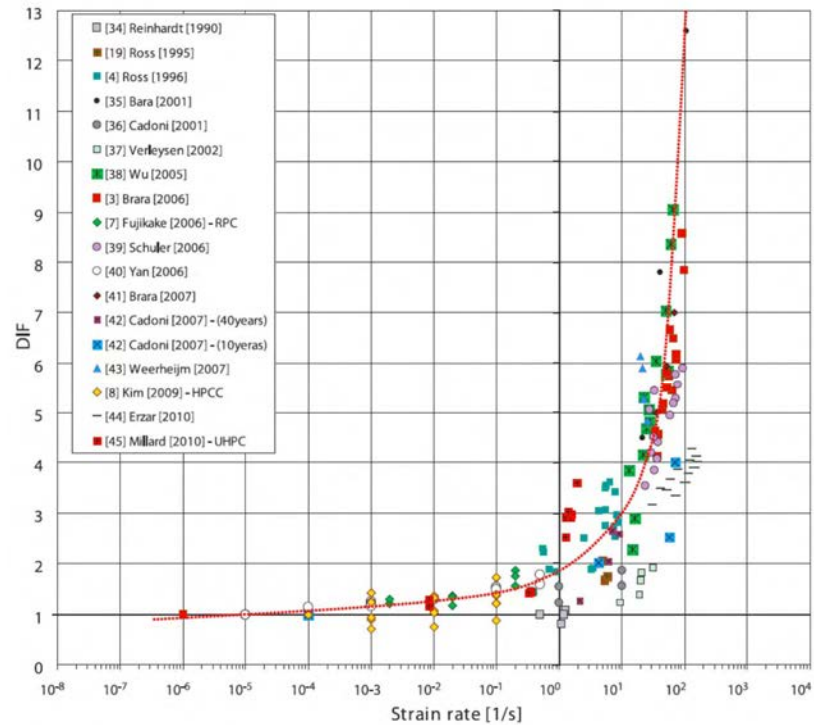


Figure 5.4. Strain Rate Effect on Concrete Tensile Strength (from [88])

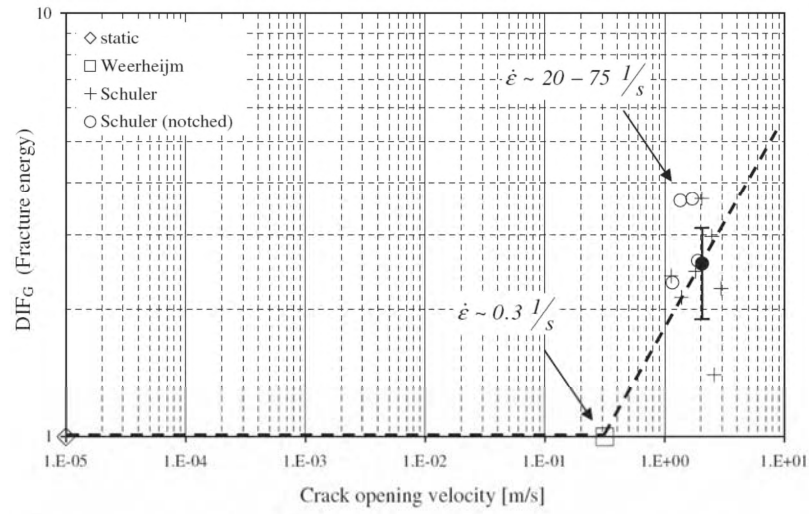


Figure 5.5. Strain Rate Effect on Concrete Fracture Energy (from [89])

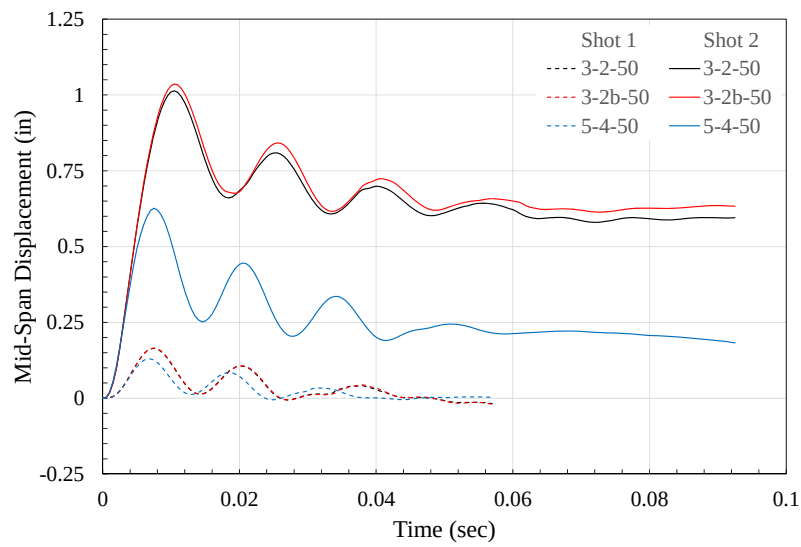


Figure 5.6. Mid-Span Displacement Time Histories from Benchmarked FE Analysis: Test 1

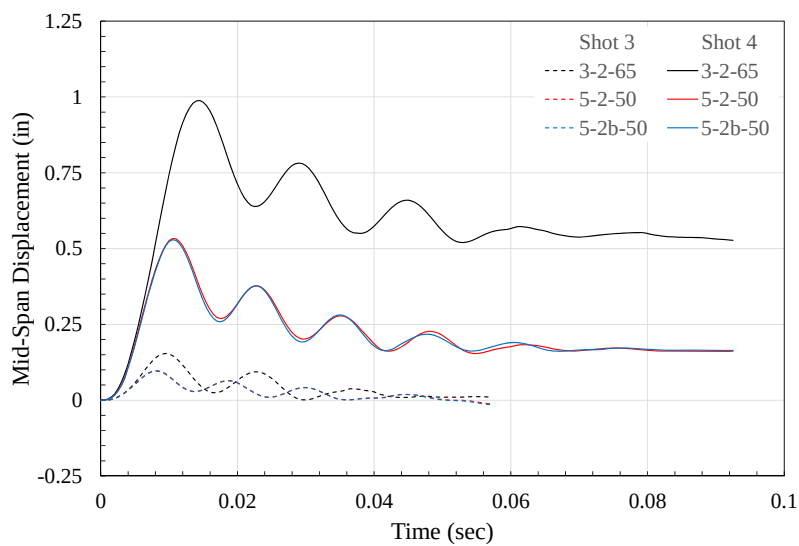


Figure 5.7. Mid-Span Displacement Time Histories from Benchmarked FE Analysis: Test 2

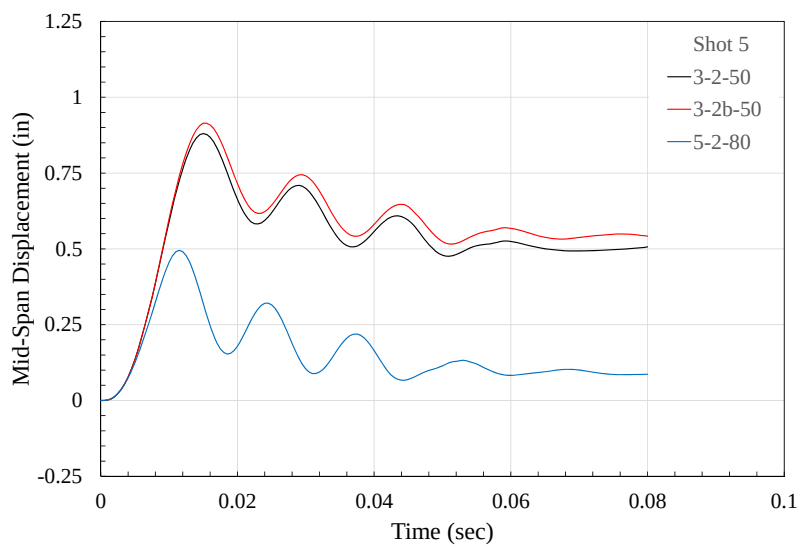


Figure 5.8. Mid-Span Displacement Time Histories from Benchmarked FE Analysis: Test 3

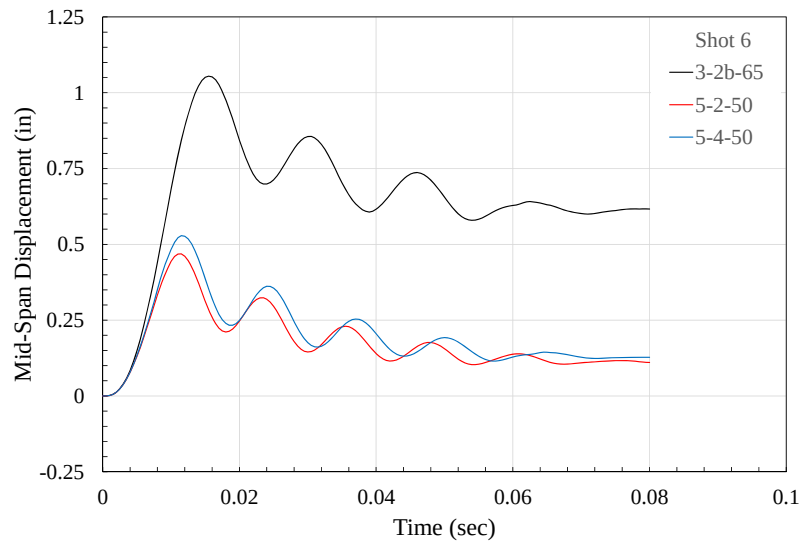


Figure 5.9. Mid-Span Displacement Time Histories from Benchmarked FE Analysis: Test 4

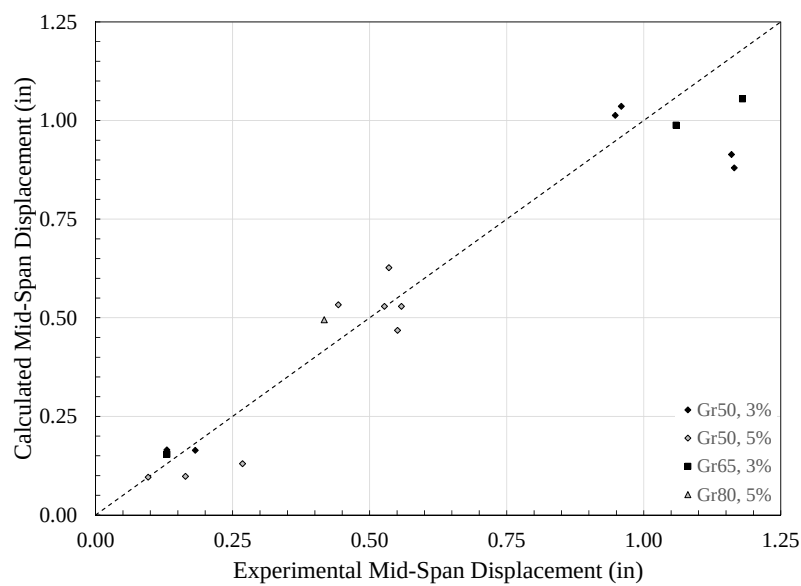
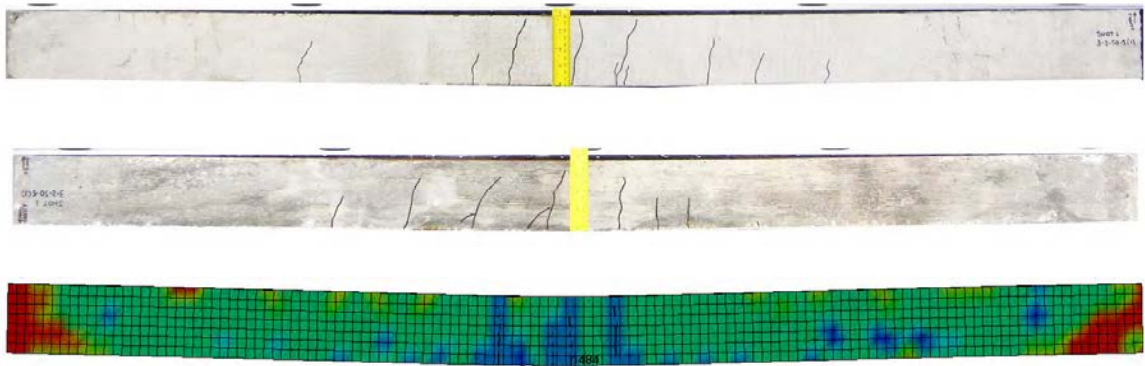
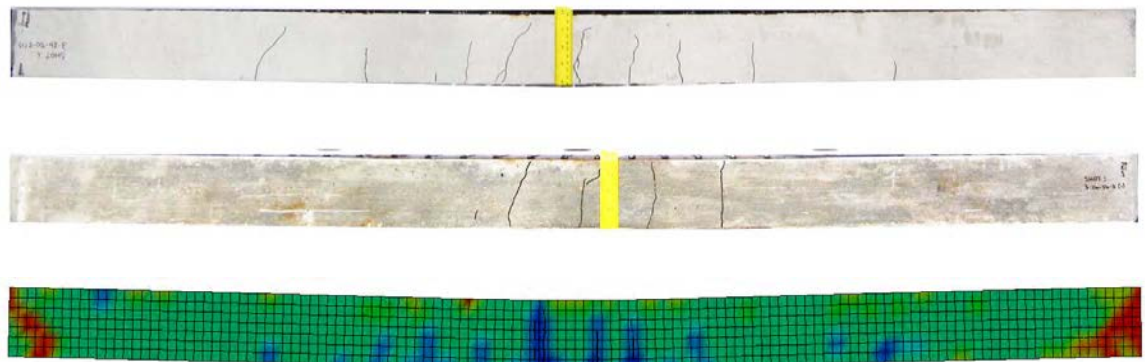


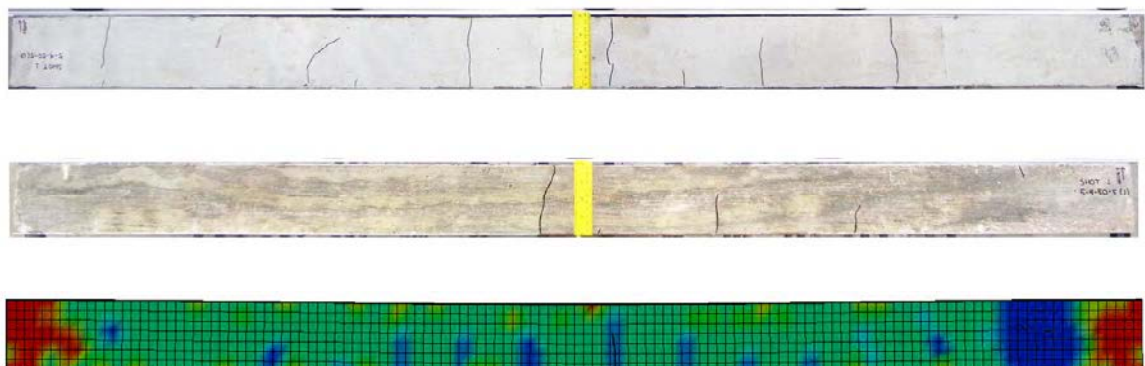
Figure 5.10. Comparison of FE Model to Experimental Blast Test Results



(a) 3-2-50-5(1)

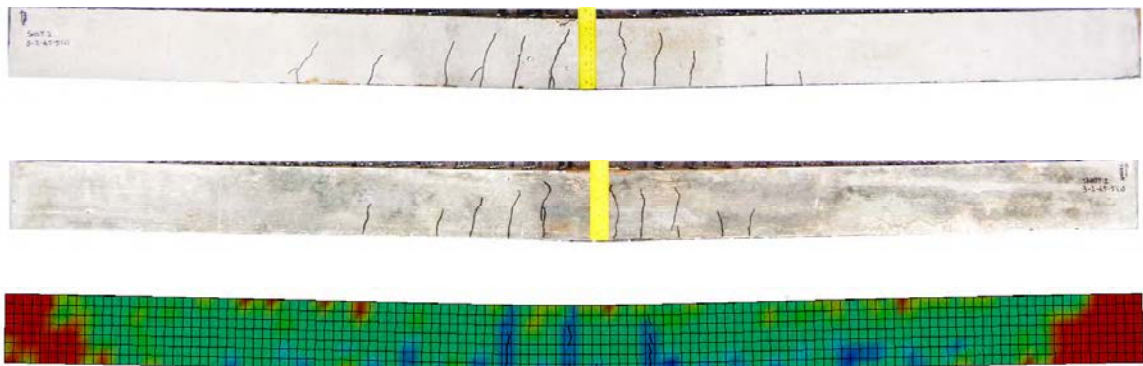


(b) 3-2b-50-5(1)

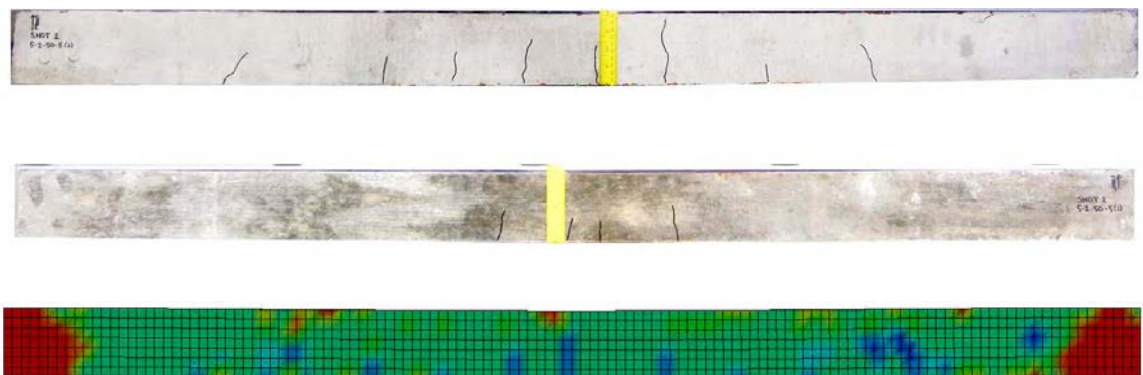


(c) 5-4-50-5(1)

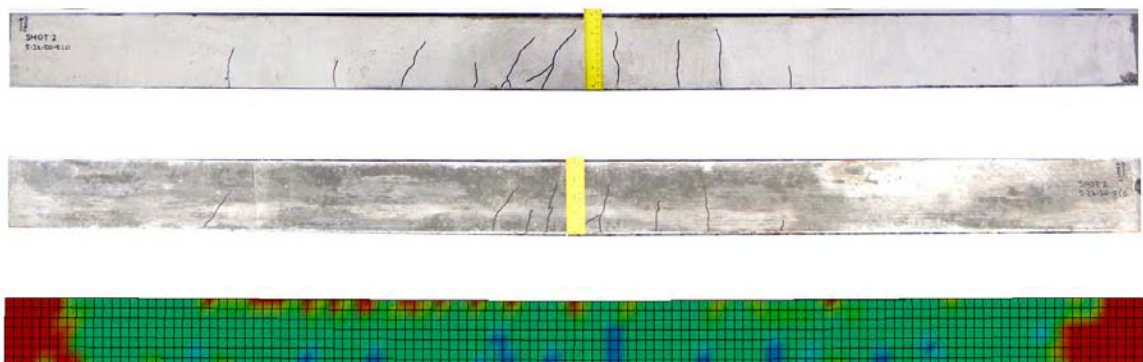
Figure 5.11. Test 1 Crack Pattern Comparison: Experimental (top two images) to FE (bottom image)



(a) 3-2-65-5(1)

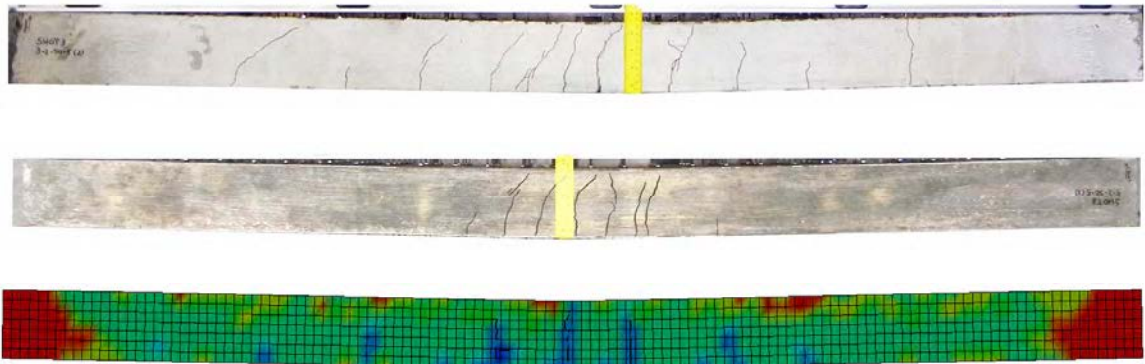


(b) 5-2-50-5(1)

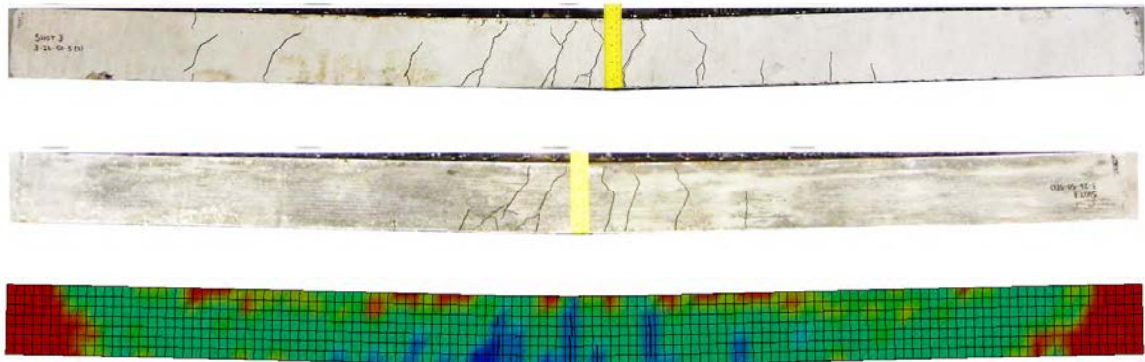


(c) 5-2b-50-5(1)

Figure 5.12. Test 2 Crack Pattern Comparison: Experimental (top two images) to FE (bottom image)



(a) 3-2-50-5(2)

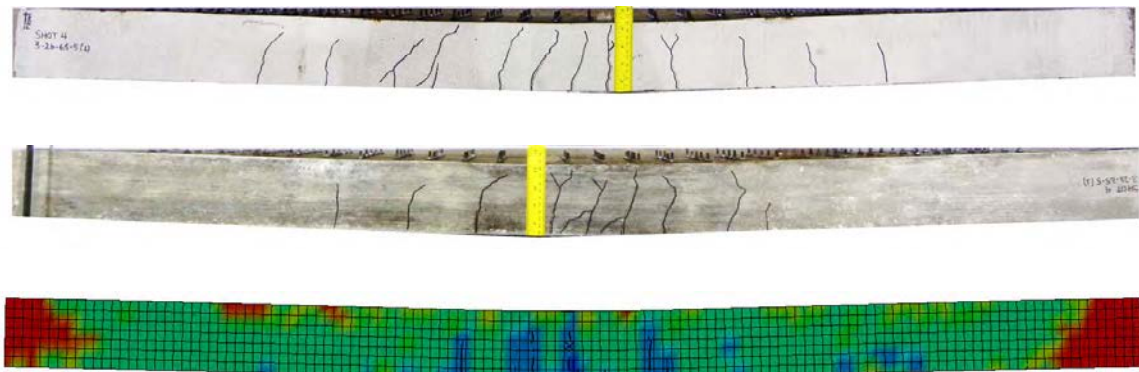


(b) 3-2b-50-5(2)

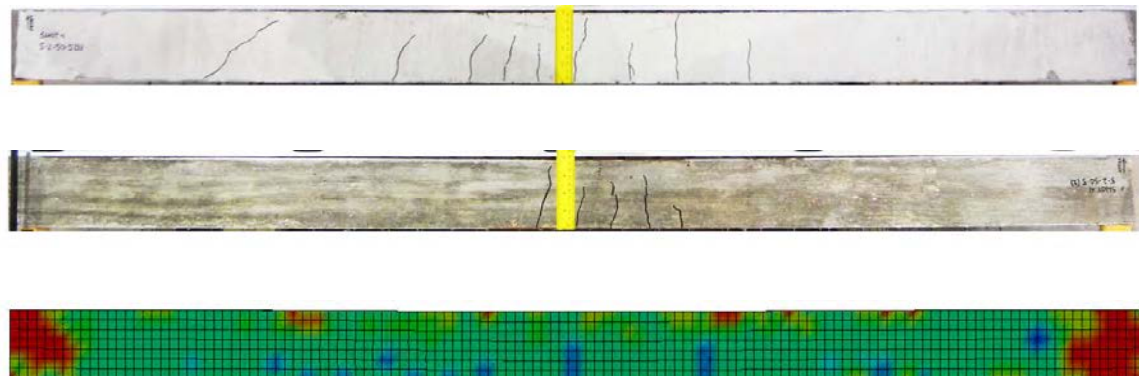


(c) 5-2-80-5(1)

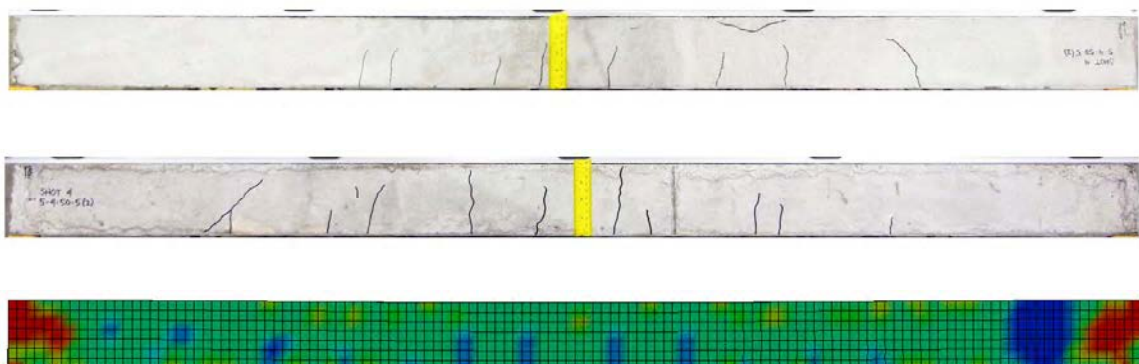
Figure 5.13. Test 3 Crack Pattern Comparison: Experimental (top two images) to FE (bottom image)



(a) 3-2b-65-5(1)



(b) 5-2-50-5(2)



(c) 5-4-50-5(2)

Figure 5.14. Test 4 Crack Pattern Comparison: Experimental (top two images) to FE (bottom image)

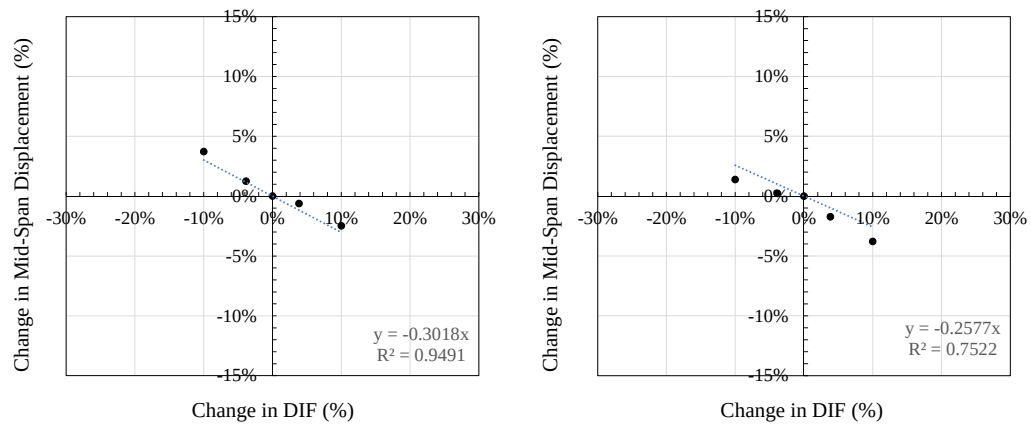


Figure 5.15. Influence of f'_c DIF on FE Model Maximum Displacement

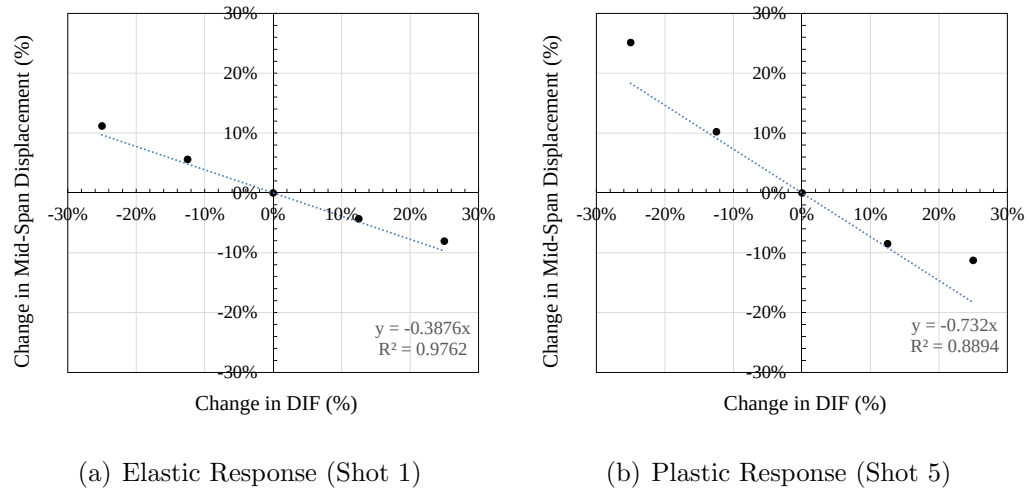


Figure 5.16. Influence of f_t DIF on FE Model Maximum Displacement

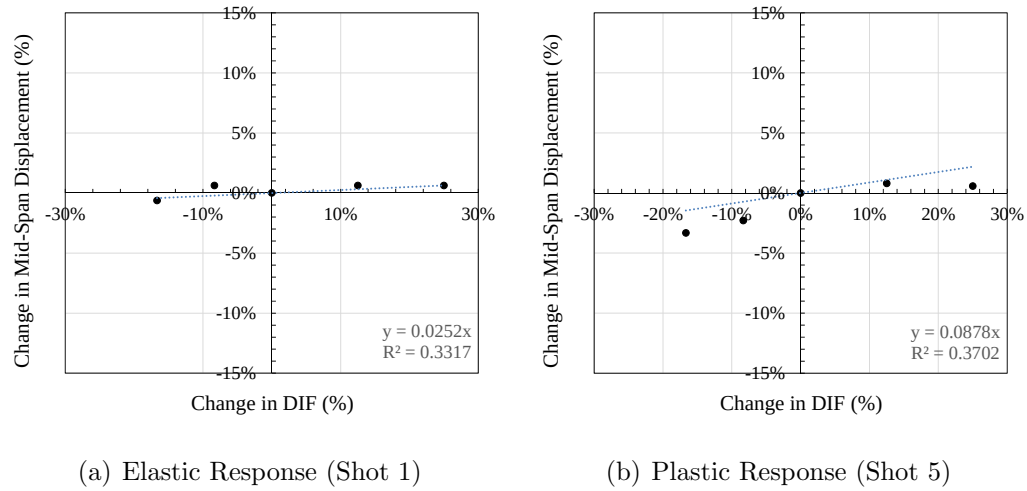


Figure 5.17. Influence of G_f DIF on FE Model Maximum Displacement

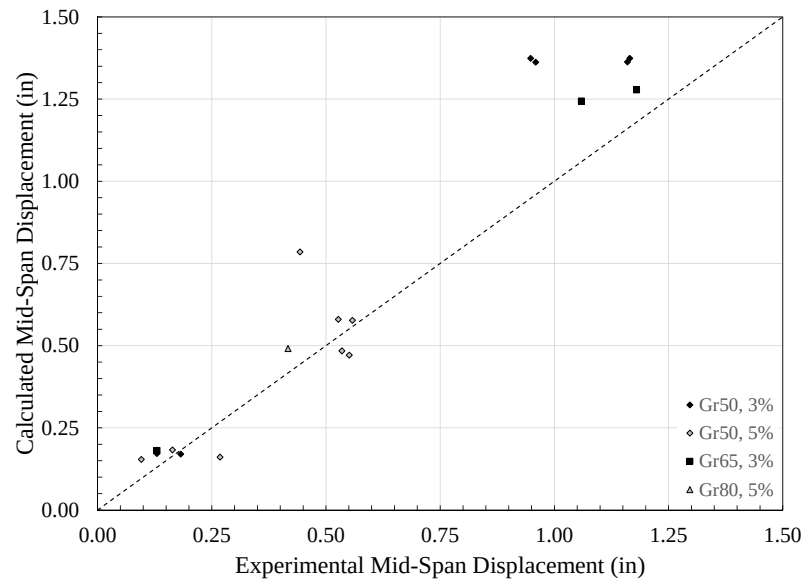


Figure 5.18. Comparison of SDOF Model with 0% Damping to Blast Test Results

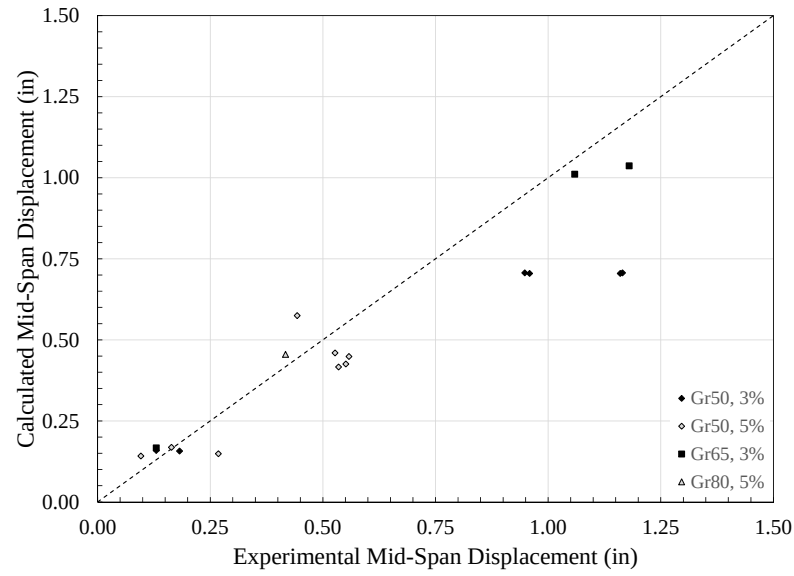


Figure 5.19. Comparison of SDOF Model with 5% Damping to Blast Test Results

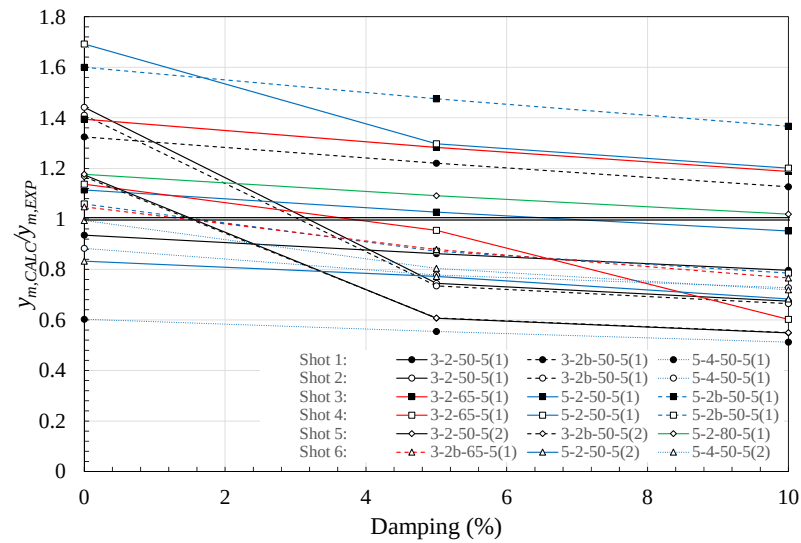


Figure 5.20. Influence of Damping Ratio on SDOF Model Maximum Displacement

6. PARAMETRIC STUDIES OF BLAST LOADED SC PANELS

Numerical modeling techniques were benchmarked to results from blast experiments of twelve SC specimens as described in Chapter 5. These benchmarked models were used to conduct further investigation of the influence of design and blast load parameters on the response of one-way SC walls.

An important phenomenon of the response of blast loaded structural members is that the same maximum mid-span displacement can result from a variety of blast load conditions. Figure 6.1 shows results from FE analysis for three different load cases on the same SC specimen (3-2-50-5) for which the maximum displacement was similar. In each figure the solid line is the triangular load pulse applied to the specimen (the right vertical axis shows the pressure associated with this pulse). The dotted line depicts the structural response (mid-span displacement) with the left vertical axis showing the magnitude of the displacement.

As seen in this figure and explained by Baker et al [90], the response of structural members to blast loads can be categorized based on the relationship between the load duration and the time of maximum response of the specimen. There are three basic response domains: impulsive, dynamic, and quasi-static. The impulsive domain includes cases in which the load is over before the maximum resistance is reached by the structure (Figure 6.1(a)). The quasi-static domain, as the name implies, includes cases in which the load is still near its maximum when the maximum resistance is reached by the structure (Figure 6.1(b)). The dynamic response domain lies between impulse and quasi-static and is characterized by cases in which the load reduces to zero at approximately the same time at which the maximum response is reached (Figure 6.1(c)). The pressure pulses in the example of Figure 6.1 range from an impulsive case of 150-psi for a total duration of about 2-msec to a quasi-static case of 25-psi for a total duration of 175-msec.

6.1 Pressure-Impulse Diagrams

Contours on pressure-impulse (p-i) diagrams of constant damage states (typically calculated as ductility) can be plotted with pressure on the vertical axis and impulse on the horizontal axis as shown in Figure 6.2. The usefulness of p-i diagrams and numerical methods to create them are described in [90] and [91]. The p-i contours for ductility of one for each of the eight SC wall configurations in this study were created by modifying the MATLAB script to solve the SDOF equation of motion as described in Section 5.2 for a triangular load pulse with zero rise time. An additional script was written to solve for a variety of pressure-impulse combinations that resulted in a desired ductility. The figure shows the pressure-impulse combinations that resulted in the specimen deflecting to its elastic limit (ductility of one). Consider specimen 3-2-50-5: a 150-psi, 200-psi-msec triangular pulse would result in yield displacement just as a 25-psi, 2500-psi-msec pulse would (or any other combination that lies on the countour line). Combinations of pressure and impulse which fall below or to the left of the damage contour would result in an elastic response while points above and to the right would exceed the elastic limit. Figure 6.2 demonstrates that there are impulsive (vertical) and quasi-static (horizontal) asymptotes which bound the contours.

6.1.1 Normalized Total Force-Total Impulse Diagrams

For more general use, p-i diagrams can be normalized as total force-total impulse (P-I) diagrams using Equations (4.2) and (4.3). Figure 6.3 shows the normalized P-I diagram for all eight specimens included in this study subjected to triangular load pulses with zero rise time. All eight p-i diagrams became the same P-I curve making this single contour useful for a wide variety of design configurations. The quasi-static asymptote corresponds to a normalized total force, P , of one and the impulsive asymptote corresponds to a normalized total impulse, I , of one. The three response domains (impulsive, dynamic, and quasi-static) are labeled on this figure. Divisions between the three domains do not correspond to exact points on the P-I

diagram. The impulse and quasi-static domains are points near the asymptotes of the P-I diagram and the dynamic domain connects the other two.

Similar diagrams can be created for any desired ductility limit as shown in Figure 6.4. For cases in which ductility limits are defined by a design specification or other source, contours of those specific limits assist in understanding what blast loads can be resisted. This understanding can inform other blast resistant design considerations such as stand-off distance and other security measures to limit the risk of exposure to blast loads beyond a design limit.

6.1.2 Experimental Validation of P-I Diagram

To confirm that the P-I diagram was in agreement with experimental results, the total force and impulse from each experimental test were normalized using Equations (4.2) and (4.3) and the maximum mid-span displacement was divided by the elastic deflection. The results are shown in Figure 6.5. Because all of the tests were clustered in the dynamic response region, a smaller portion of the P-I diagram is shown than was shown in previous figures. The black line is the damage contour associated with ductility of one and the gray lines correspond with ductility of 3 and 5 as indicated on the figure.

Each point on Figure 6.5 corresponds to the P-I coordinate for each of the twelve tested specimens and the label is the experimental ductility of that test. For example, the triangular point near the middle of the figure corresponds to the test of specimen 5-2-80-5 and the measured displacement was 0.808 times the elastic deflection. The majority of the experimental data points fall in regions of the figure as expected. Exceptions are a test of 5-4-50-5 which has a ductility of 0.780 but falls below the line for $\mu = 0.5$. Two other exceptions are: a test of 3-2-50-5 with a ductility of 4.045, and a test of 3-2b-50-5 with a ductility of 4.028, each of which fall close to the contour for $\mu = 3$. SDOF and FE analysis results described in Chapter 5 suggested that the measured response of 5-4-50-5 was an outlier. The other two high ductility

outliers are cases in which the specimen was first tested with a lower shot resulting in elastic deflection; damage from this shot contributed to larger displacements when subjected to the larger shot. This is corroborated by the two other specimens of 3-2-50-5 and 3-2b-50-5 which were tested only by the larger shot and resulted in ductility as expected (3.327 and 3.289 respectively). The favorable comparison of experimental data to the P-I curves generated from SDOF analysis demonstrate agreement between this numerical method and experimental results.

6.2 Parametric Study Design

The experimental investigation included a variety of parameters (i.e. steel plate thickness and strength, tie bar spacing and diameter) and the influence of these parameters on the blast response of one-way SC walls was discussed in Section 4.3. The two parametric studies described in this chapter focused on other parameters:

1. Changing blast loads on benchmarked models to evaluate the response to a variety of load conditions, and
2. Modifying other geometric, material, and analysis parameters: unsupported span-to-depth ratio, concrete strength, and damping ratio.

Because p-i curves for all eight configurations converged to the same normalized P-I curve, the influence of the design parameters included in the experimental study were inherently accounted for in the definitions of the idealized static resistance function for the SDOF system as explained in Section 3.6.2. This P-I diagram is useful for damage assessment of any SC wall subject to blast loads. To assist in understanding the influence of variables considered in the parametric studies described in this chapter, p-i and P-I diagrams were used.

6.3 Influence of Blast Load Parameters on SC Wall Response

A series of 80 P-I combinations were selected from Figure 6.4 to expand the experimental database and investigate the blast response of the SC specimens to blast loads throughout the impulsive, dynamic, and quasi-static domains (as shown in Figure 6.6). Points were selected to achieve larger ductility than observed experimentally to investigate what values of P-I lead to ultimate failure of the specimen (i.e. rupture of the tension plate). Triangular pressure time history curves were applied to the benchmarked FE model of specimen 3-2-50-5. This specimen was selected because it is the one-third scale specimen of the minimum practical SC wall. From the static analysis of this specimen maximum displacement was ≈ 2.40 -in corresponding to a maximum displacement ductility of 11.65. This damage contour is shown in Figure 6.6.

Pressure-impulse combinations and results from these analyses are listed in Table 6.1. The results were used to generate contour plots of the same ductility levels as shown overlaid on the SDOF ductility curves in Figure 6.7. The FE results were similar to SDOF results for ductility of one with FE estimating slightly larger loads in the dynamic response domain. Results were similar for ductility of 3 and 5 in the impulsive domain but FE estimates larger required loads in the quasi-static domain. The FE model estimates larger loads required to reach ultimate failure. These results confirm blast resistant design guidance that SDOF analysis with 0% damping provides conservative results.

The response mechanism and crack pattern differs for load cases in each of the P-I domains. Figure 6.8 shows representative crack patterns for cases in each of the three regions for which the response ductility was roughly 4. These examples are instructive concerning response mechanisms of one-way SC walls in each of these three regions. For impulsive cases, cracks were concentrated at mid-span and extended through the depth of the specimen. This leads to formation of a plastic hinge at mid-span and the deflected shape reflects this. As response becomes dynamic, flexure cracks spread from mid-span and the end regions are completely uncracked (indicated by

the red areas). The mid-span flexure cracks do not extend through the depth and the deflected shape is not as dominated by a central plastic hinge as in the impulsive load case. The flexure cracks are distributed over a larger area for the load case in the quasi-static region, do not extend through the depth, and the end regions are uncracked.

6.4 Influence of Geometric, Material, and Analysis Parameters

The experiments examined influence of several important design parameters but did not hold all other variables exactly constant nor did they include all design parameters. For example, the different grades of steel for a given gage were not exactly the same thickness leading to slightly different reinforcement ratios. All specimens had the same span to depth ratio, support conditions, and approximately the same concrete strength. Studies in this section investigate the influence of these parameters by changing input parameters to the benchmarked SDOF models.

6.4.1 Influence of Span-to-Depth Ratio

Figure 6.9 depicts influence of unsupported span-to-depth ratio on member response. These figures were generated using cross-section properties of specimen 3-2-50-5 with changing length. The value of $L : t_{sc}$ varied from 6 to 20. Figure 6.9(a) shows the p-i elastic damage contour ($\mu = 1.0$) for each $L : t_{sc}$ ratio and Figure 6.9(b) shows the damage contour for steel plate rupture ($\mu = 11.6$). The impulsive asymptote is the same for all cases indicating that the response in the impulsive domain was independent of $L : t_{sc}$. In the dynamic and quasi-static domains as $L : t_{sc}$ decreased, the pressure required to result in yield increased exponentially as shown in Figure 6.10. The influence of $L : t_{sc}$ in the quasi-static domain was significant: a 20% increase in $L : t_{sc}$ resulted in a 30% decrease in required pressure. The p-i contours in Figure 6.9 all collapsed to the same P-I contours indicating that influence of $L : t_{sc}$ was inherently included in SDOF analysis.

6.4.2 Influence of Concrete Strength

Figure 6.11 depicts influence of concrete compressive strength on member response. These figures were generated using the cross-section properties of specimen 3-2-50-5 with changing f'_c . The value of f'_c varied from 3.0-7.5-ksi, a common range of normal-weight concrete strength. Figure 6.11(a) shows the p-i elastic damage contour ($\mu = 1.0$) for each f'_c and Figure 6.11(b) shows the damage contour for steel plate rupture ($\mu = 11.6$). The quasi-static asymptote was the same for all cases indicating that the response in this domain was independent of f'_c . In the impulsive and dynamic domains as f'_c increased, the impulse required to result in yield increased slightly. This may seem counter-intuitive but the contours shown are for an achieved ductility not a displacement. As concrete strength increased the load required to achieve a specific displacement also increased as discussed in Section 5.1.4 and shown in Figure 5.15. When considering the load required to achieve a specific ductility, however, the elastic displacement is included and this value is also influenced by f'_c leading to the conclusion that lower loads resulted in the same achieved ductility as f'_c increased. The influence of f'_c in the impulsive domain was quite small: less than 1% reduction in required impulse for a 10% increase in concrete strength (see Figure 6.12). The p-i contours in Figure 6.11 all collapsed to the same P-I contours indicating that influence of f'_c was inherently included in SDOF analysis.

6.4.3 Influence of Steel Faceplate Strength

Figure 6.13 depicts influence of steel faceplate yield strength on member response. These figures were generated using modified cross-section properties of specimen 3-2-50-5 with changing F_y (the faceplate thickness was changed to 0.060-in so that the reinforcement ratio was exactly 3.0%). The value of F_y varied from 50-65-ksi, the range permitted by AISC N690s1 N9.1.1(d). Figure 6.13(a) shows the p-i elastic damage contour ($\mu = 1.0$) for each F_y and Figure 6.13(b) shows the damage contour for steel plate rupture ($\mu = 11.6$). The impulsive and quasi-static asymptotes differed

for all cases indicating that response in all three domains was dependent on F_y . As F_y increased, the pressure (in the quasi-static domain) or impulse (in the impulsive domain) required to result in yield increased linearly as shown in Figure 6.14. The influence of F_y in all domains was significant: a 10% increase in F_y resulted in a 10% decrease in required pressure or impulse. The p-i contours in Figure 6.13 all collapsed to the same P-I contours indicating that influence of F_y was inherently included in SDOF analysis.

6.4.4 Influence of Reinforcement Ratio

Figure 6.15 depicts influence of reinforcement ratio on member response. These figures were generated using modified cross-section properties of specimen 3-2-50-5 with changing ρ . The value of ρ varied from 1.5-5.0%, the range permitted by AISC N690s1 N9.1.1(c). Figure 6.15(a) shows the p-i elastic damage contour ($\mu = 1.0$) for each ρ and Figure 6.15(b) shows the damage contour for steel plate rupture ($\mu = 11.6$). The impulsive and quasi-static asymptotes differed for all cases indicating that the response in all three domains was dependent on ρ . As ρ increased, the pressure (in the quasi-static domain) or impulse (in the impulsive domain) required to result in yield increased related to the square of ρ as shown in Figure 6.16. The influence of ρ in all domains was significant: a 20% increase in ρ resulted in a 12% decrease in required pressure or impulse. The p-i contours in Figure 6.15 all collapsed to the same P-I contours indicating that influence of ρ was inherently included in SDOF analysis.

6.4.5 Influence of Damping Ratio

The effect of including damping in the benchmarked SDOF models was described in Section 5.2.2. That sensitivity study was specific to individual load cases included in the experimental investigation. Figure 6.17 shows the influence of the percent of critical damping, ζ , on member response across all three response domains. These figures were generated using the cross-section properties of specimen 3-2-50-5 with

changing ζ . The value for ζ varied from 0-10% based on results of this current study and recommendations by others [92], [93]. Figure 6.17(a) shows the p-i elastic damage contour ($\mu = 1.0$) for each ζ and Figure 6.17(b) shows the damage contour for steel plate rupture ($\mu = 11.6$). The impulsive and quasi-static asymptotes differed for all cases indicating that the response in all three domains was dependent on ζ . As damping increased, the pressure (in the quasi-static domain) or impulse (in the impulsive domain) required to result in yield increased related to the square of ζ as shown in Figure 6.18. The influence of ζ was most notable for plastic response in the impulsive response domain: including 5% damping increased the required impulse by 33% compared to the case with 0% damping. In the quasi-static domain, the same damping ratio increased the required pressure by 13%. For elastic response in impulsive and quasi-static domains, including 5% damping increased the required load by less than 10%. The p-i contours in Figure 6.17 did not collapse to the same P-I contours. Figure 6.19 shows the P-I curves for each of the values of damping considered.

6.5 Chapter Summary

This chapter presented results from parametric studies investigating influence of typical load, geometric, material, and analysis parameters. These studies were conducted by modifying parameters of benchmarked models described in Chapter 5 and expanded the experimental database reported in Chapter 4. Results from parametric studies were presented as pressure-impulse diagrams to identify the influence of each parameter in the impulsive, dynamic, and quasi-static domains. The following chapter compiles the knowledge from the experimental static and dynamic tests with findings from these parametric studies into a methodology to design one-way SC walls to resist blast loads.

Table 6.1.: Parametric Study of Blast Loads on Specimen
3-2-50-5

Number	P	I	p_r (psi)	i_r (psi·msec)	t_d (msec)	y_m (in)	Remarks
I1	3	1	80.4	117	2.91	0.256	
I2	3	2	80.4	234	5.82	0.498	
I3	3	3	80.4	351	8.73	0.714	
I4	3	4	80.4	468	11.64	0.915	
I5	3	5	80.4	585	14.55	1.12	
I6	4	1	107.2	117	2.18	0.267	
I7	4	2	107.2	234	4.37	0.565	
I8	4	3	107.2	351	6.55	0.905	
I9	4	4	107.2	468	8.73	1.27	
I10	4	5	107.2	585	10.91	1.65	
I11	5	1	134.0	117	1.75	0.247	
I12	5	2	134.0	234	3.49	0.602	
I13	5	3	134.0	351	5.24	1.14	
I14	5	4	134.0	468	6.98	1.50	
I15	5	5	134.0	585	8.73	2.01	
I16	5	6	134.0	702	10.48	2.63	plate ruptured at 2.40-in
I17	4	6	107.2	702	13.10	2.04	
I18	2	0.5	53.6	58	2.18	0.119	
I19	2.5	0.5	67.0	58	1.75	0.123	
I20	3	0.5	80.4	58	1.46	0.125	
I21	4	0.5	107.2	58	1.09	0.125	
I22	5	0.5	134.0	58	0.873	0.127	
I23	4	7	107.2	819	15.28	2.44	plate ruptured at 2.36-in
Q1	0.5	10	13.4	1170	174.6	0.097	
Q2	1	10	26.8	1170	87.3	0.243	
Q3	1.3	10	34.8	1170	67.2	0.331	
Q4	1.7	10	45.6	1170	51.4	0.488	
Q5	2	10	53.6	1170	43.7	0.688	
Q6	2.5	10	67.0	1170	34.9	1.23	
Q7	0.5	15	13.4	1755	261.9	0.098	
Q8	1	15	26.8	1755	131.0	0.246	
Q9	1.3	15	34.8	1755	100.7	0.339	
Q10	1.7	15	45.6	1755	77.0	0.515	
Q11	2	15	53.6	1755	65.5	0.754	
Q12	2.5	15	67.0	1755	52.4	1.5	
Q13	0.5	20	13.4	2339	349.2	0.099	
Q14	1	20	26.8	2339	174.6	0.249	
Q15	1.3	20	34.8	2339	134.3	0.341	
Q16	1.7	20	45.6	2339	102.7	0.535	
Q17	2	20	53.6	2339	87.3	0.796	
Q18	2.5	20	67.0	2339	69.8	1.74	

Continued on next page

Table 6.1 Continued

Number	P	I	p_r (psi)	i_r (psi·msec)	t_d (msec)	y_m (in)	Remarks
Q19	3.3	10	88.4	1170	26.5	2.3	
Q20	3	15	80.4	1755	43.7	4.17	plate ruptured at 2.36-in
Q21	2.8	20	75.0	2339	62.4	2.3	
Q22	3	20	80.4	2339	58.20	5.11	plate ruptured at 2.35-in
D1	3	6	80.4	702	17.46	1.3	
D2	3	8	80.4	936	23.28	1.66	
D3	3.5	7	93.8	819	17.46	1.98	
D4	3.5	9	93.8	1053	22.45	2.37	
D5	4	8	107.2	936	17.46	3.72	plate ruptured at 2.42-in
D6	4	10	107.2	1170	21.83	5.29	plate ruptured at 2.55-in
D7	2.5	1	67.0	117	3.49	0.248	
D8	2.5	2	67.0	234	6.98	0.439	
D9	2.5	3	67.0	351	10.48	0.584	
D10	2.5	4	67.0	468	13.97	0.707	
D11	2.5	5	67.0	585	17.46	0.825	
D12	2.5	7.5	67.0	877	26.19	1.04	
D13	2	1	53.6	117	4.37	0.233	
D14	2	2	53.6	234	8.73	0.366	
D15	2	3	53.6	351	13.10	0.451	
D16	2	4	53.6	468	17.46	0.506	
D17	2	5	53.6	585	21.83	0.553	
D18	2	7.5	53.6	877	32.74	0.633	
D19	1.7	5	45.6	585	25.68	0.432	
D20	1.7	7.5	45.6	877	38.52	0.467	
D21	1.3	1	34.8	117	6.72	0.189	
D22	1.3	3	34.8	351	20.15	0.285	
D23	1.3	5	34.8	585	33.58	0.309	
D24	1.3	7.5	34.8	877	50.37	0.325	
D25	1	1	26.8	117	8.73	0.157	
D26	1	3	26.8	351	26.19	0.214	
D27	1	5	26.8	585	43.65	0.232	
D28	1	7.5	26.8	877	65.48	0.237	
D29	0.5	3	13.4	351	52.38	0.092	
D30	0.5	5	13.4	585	87.30	0.095	
D31	0.5	7.5	13.4	877	131.0	0.096	
D32	3.8	8	101.8	936	18.38	2.82	plate ruptured at 2.42-in
D33	3.5	9.5	93.8	1111	23.70	2.44	
B1	0.75	2.2	20.2	261	25.84	0.164	
B2	2.24	8.8	59.9	1032	34.46	0.88	

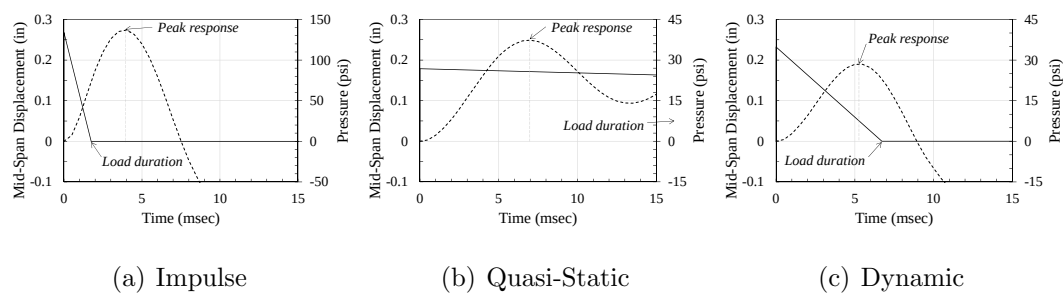


Figure 6.1. Typical Response Domains

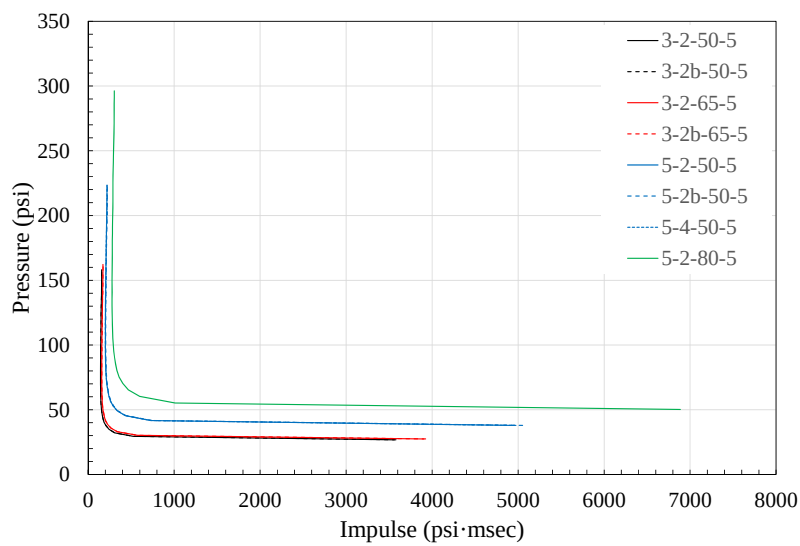


Figure 6.2. Pressure-Impulse Curves for All Eight Specimens

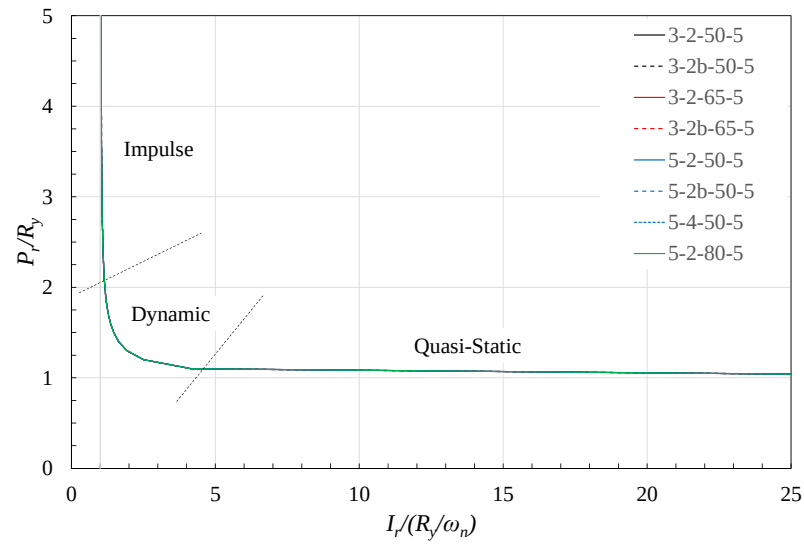


Figure 6.3. Normalized Total Force-Impulse Curves for All Eight Specimens

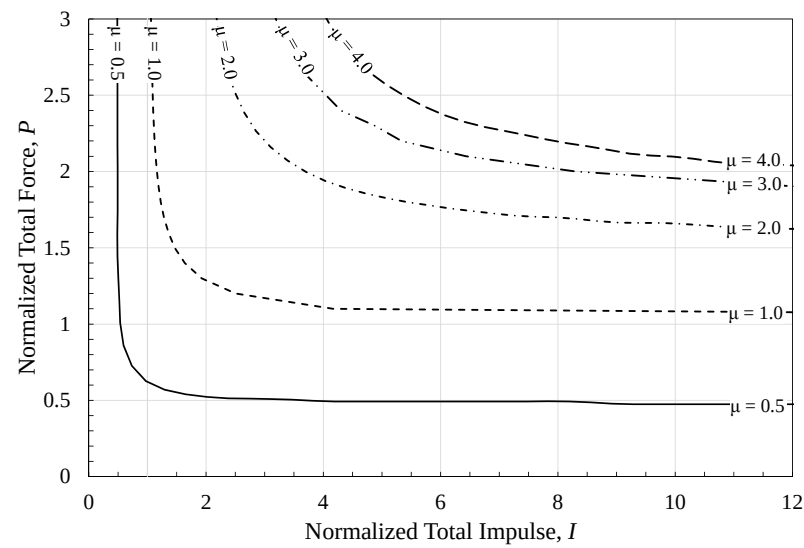


Figure 6.4. Normalized Total Force-Impulse Diagram with Multiple Ductility Limits

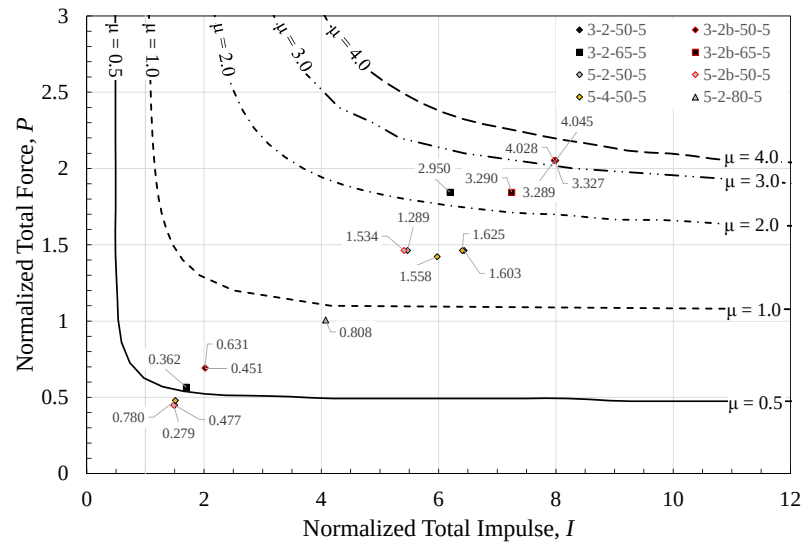


Figure 6.5. Normalized Total Force-Impulse Curve with Experimental Results

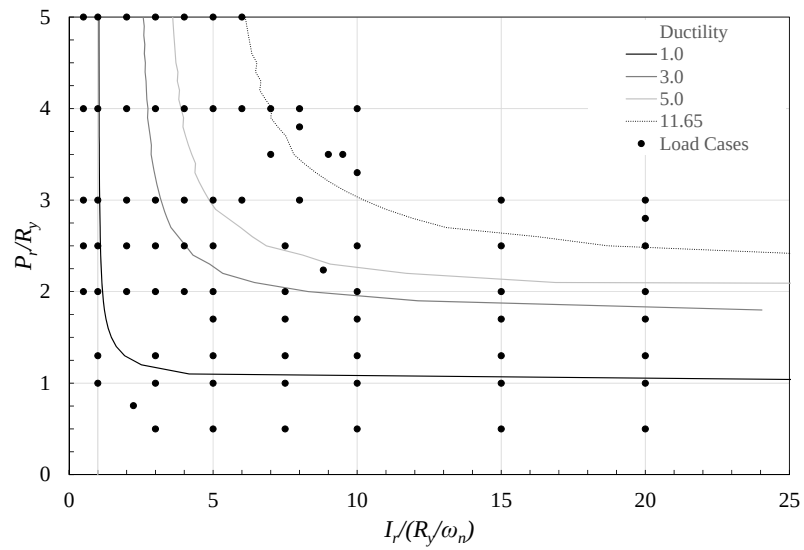


Figure 6.6. Pressure-Impulse Combinations for Parametric Study of Blast Loads on Specimen 3-2-50-5

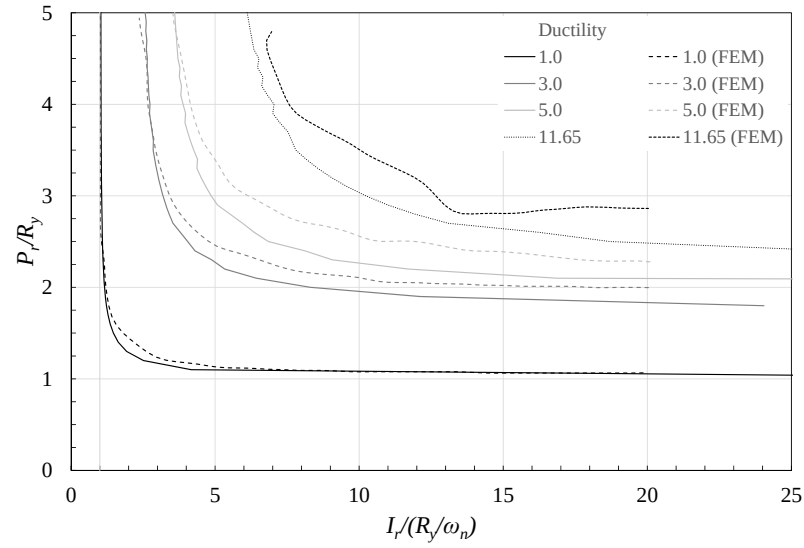
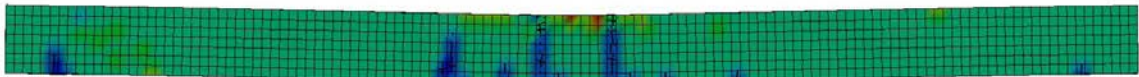
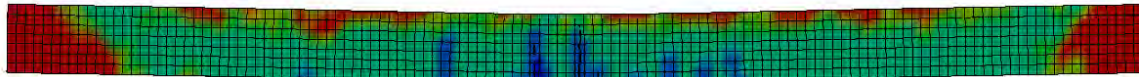


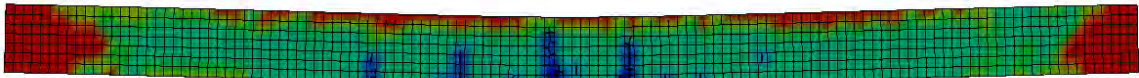
Figure 6.7. Contour Lines of Results from Parametric Study of Blast Loads on Specimen 3-2-50-5



(a) Impulsive Response; I8 (P=4; I=3)



(b) Dynamic Response; D11 (P=2.5; I=5)



(c) Quasi-Static Response; Q17 (P=2; I=20)

Figure 6.8. Crack Patterns of 3-2-50-5 Loaded with P-I Combinations ($\mu \approx 4$)

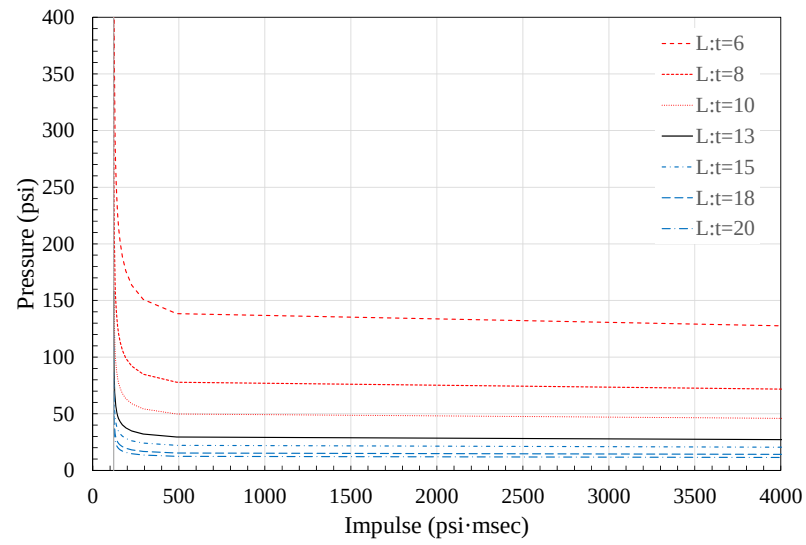
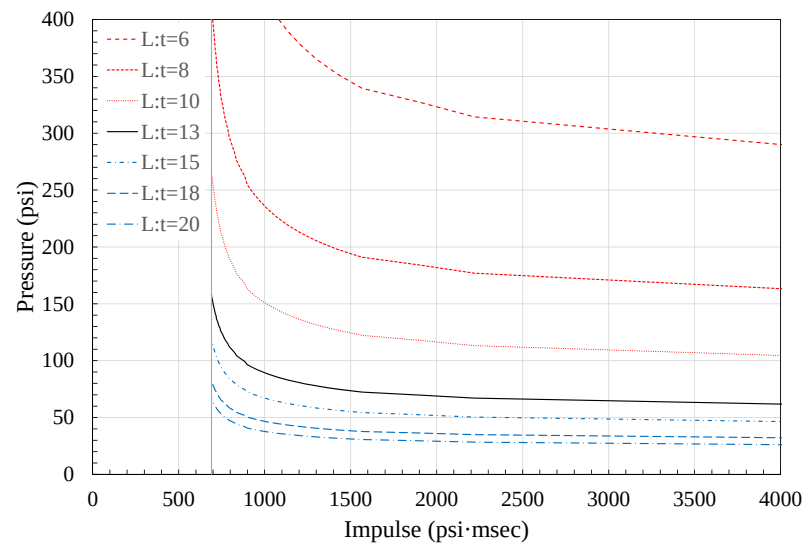
(a) $\mu = 1.0$ (b) $\mu = 11.6$

Figure 6.9. Ductility Contours of Varying Span-to-Depth Ratios (3-2-50-5 Cross-Section)

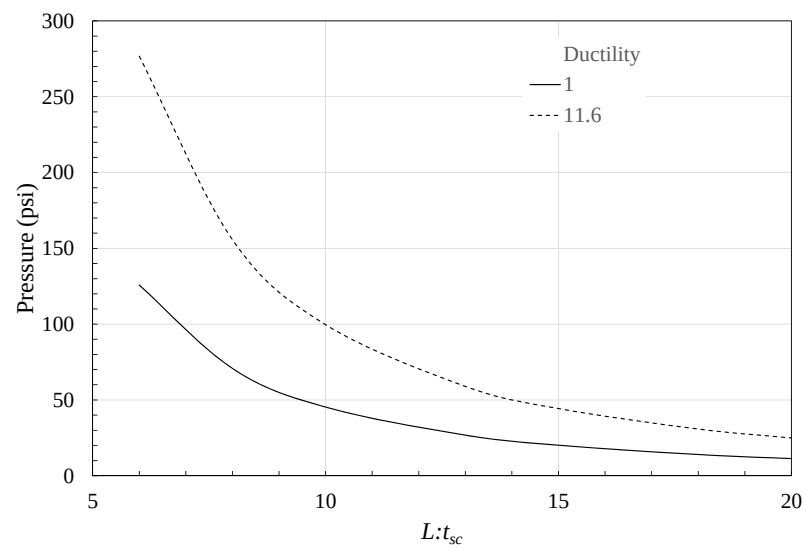


Figure 6.10. Influence of Length-to-Depth Ratio on Required Quasi-Static Pressure (3-2-50-5 Cross-Section)

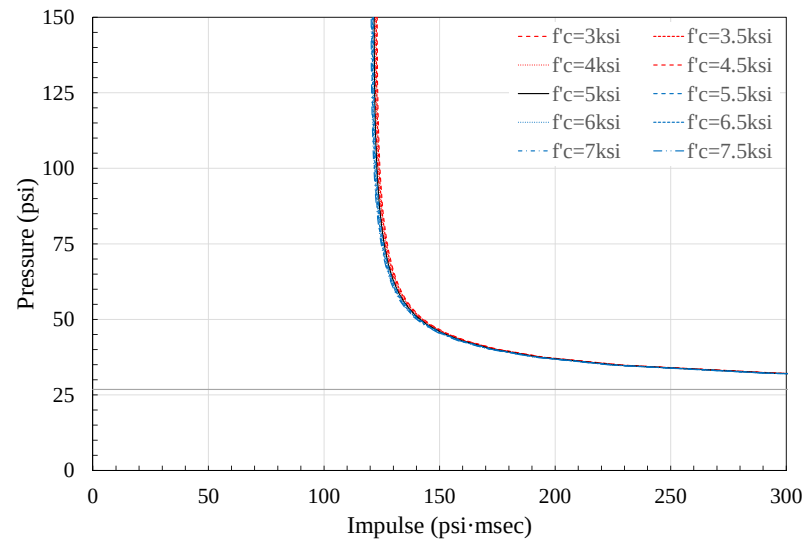
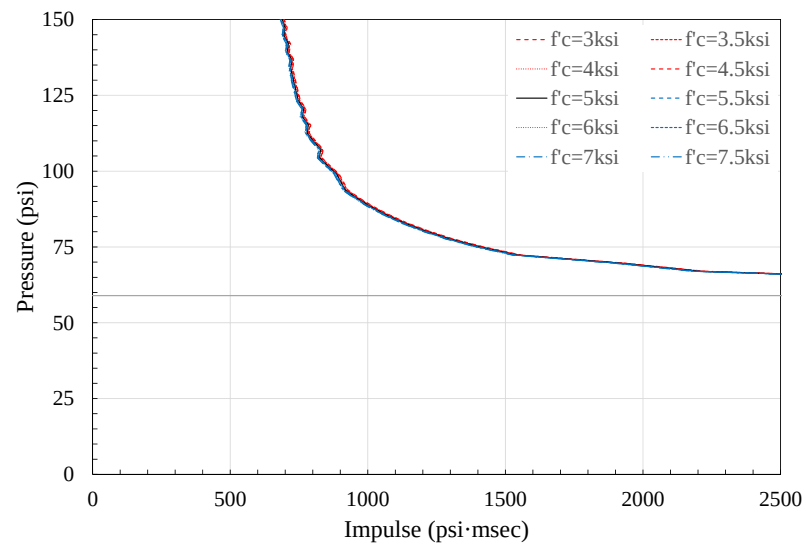
(a) $\mu = 1.0$ (b) $\mu = 11.6$

Figure 6.11. Ductility Contours of Varying Concrete Strength (3-2-50-5 Cross-Section)

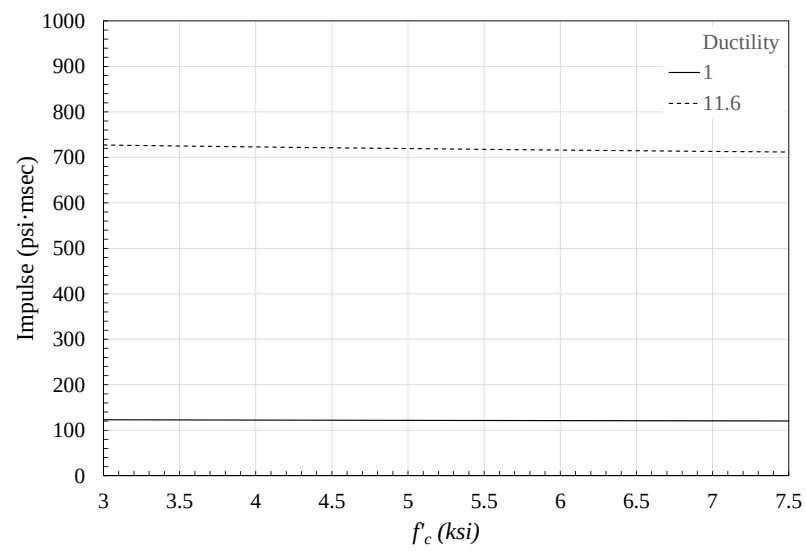


Figure 6.12. Influence of Concrete Strength on Required Impulse (3-2-50-5 Cross-Section)

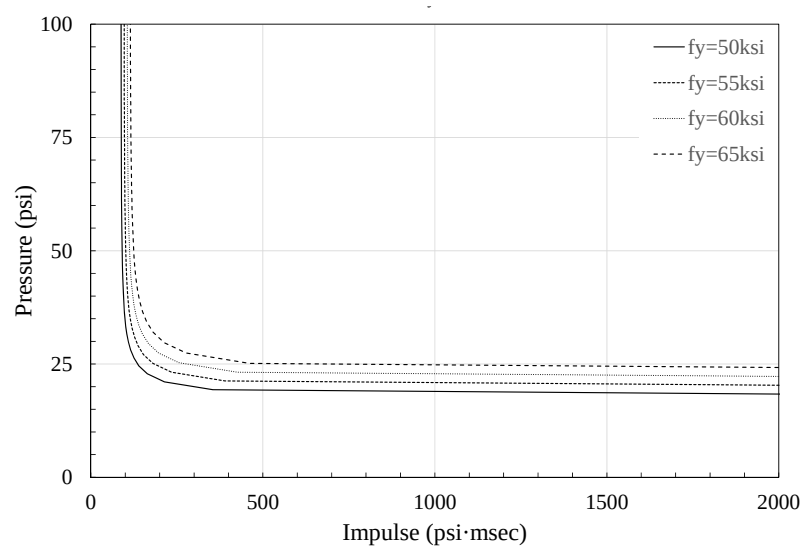
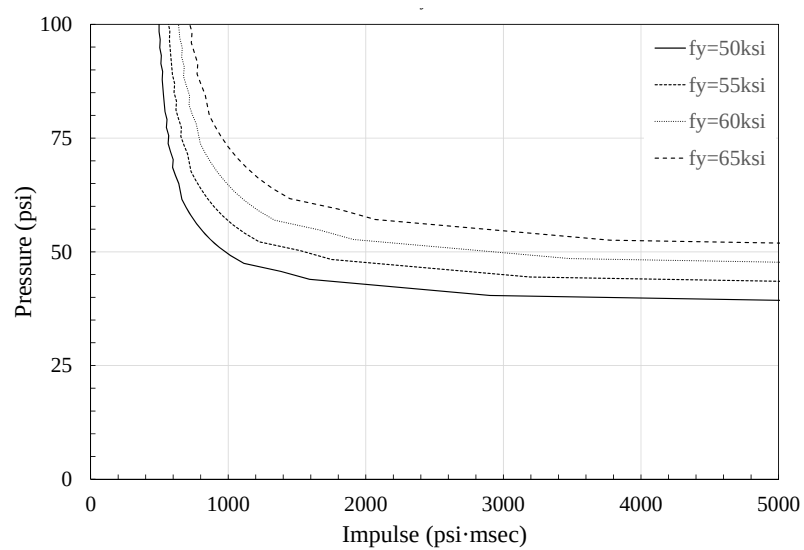
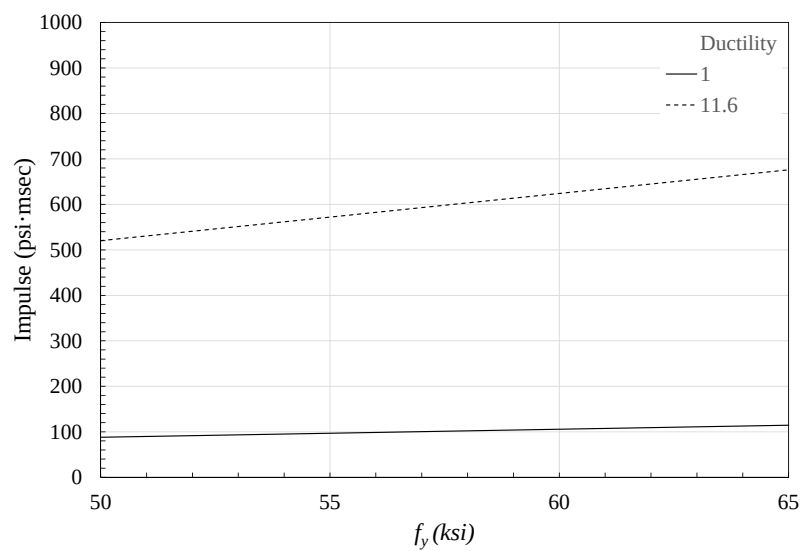
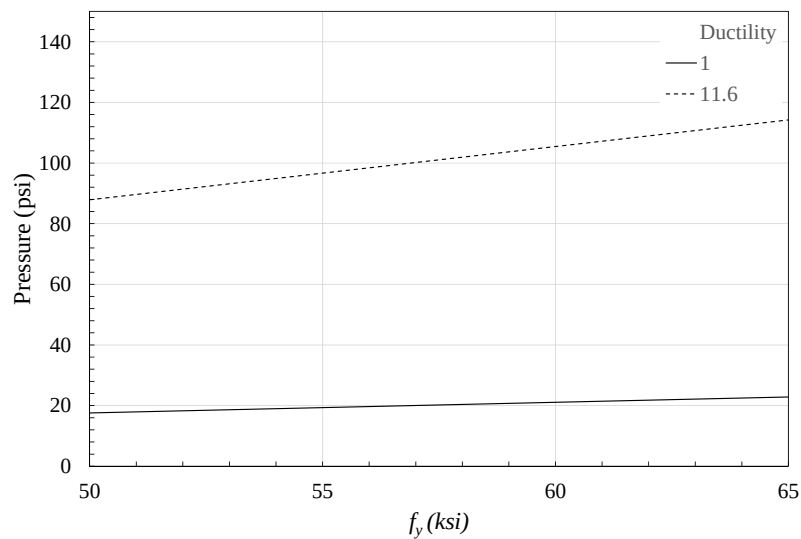
(a) $\mu = 1.0$ (b) $\mu = 11.6$

Figure 6.13. Ductility Contours of Varying Steel Faceplate Yield Strength (3-2-50-5 Cross-Section)



(a) Impulse (Impulsive Domain)



(b) Pressure (Quasi-Static Domain)

Figure 6.14. Influence of Steel Faceplate Yield Strength on Required Load (3-2-50-5 Cross-Section)

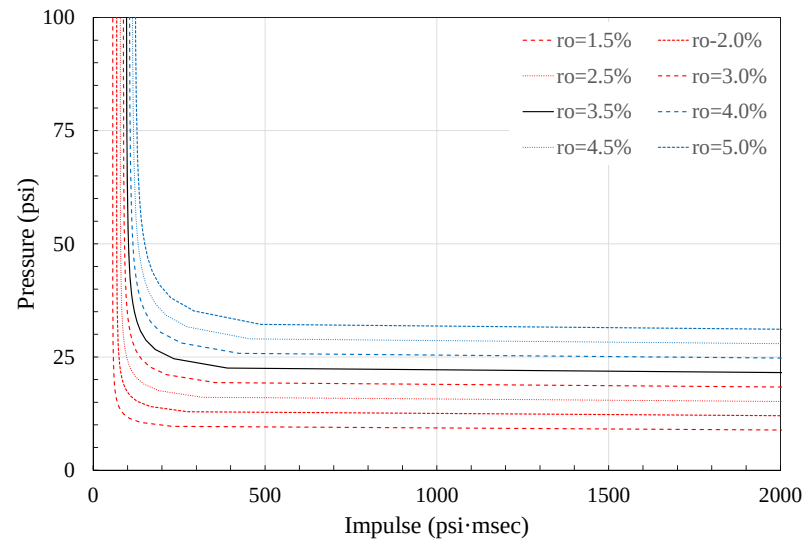
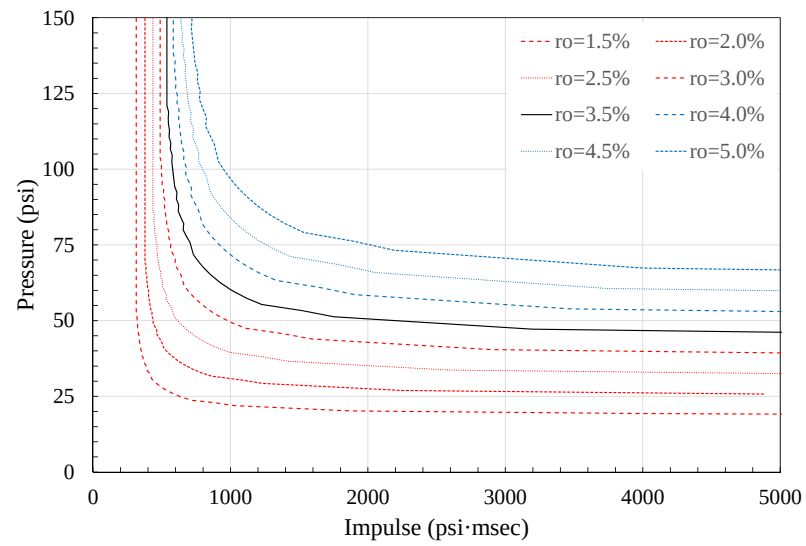
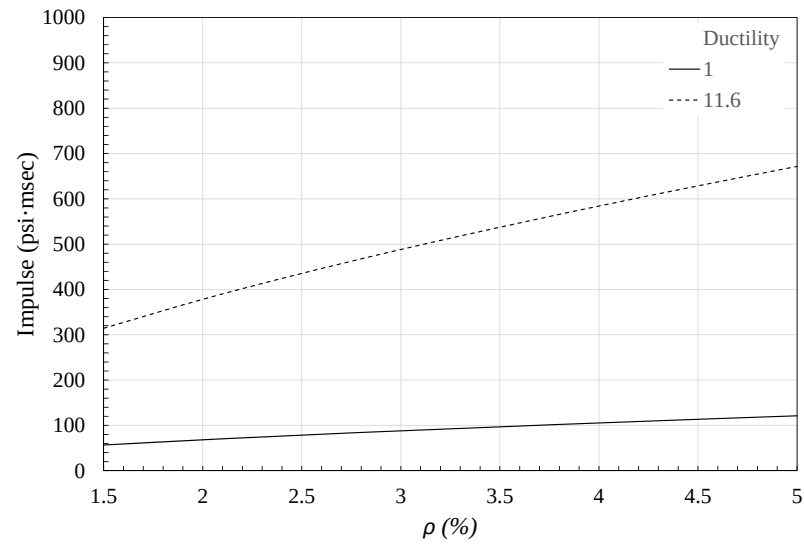
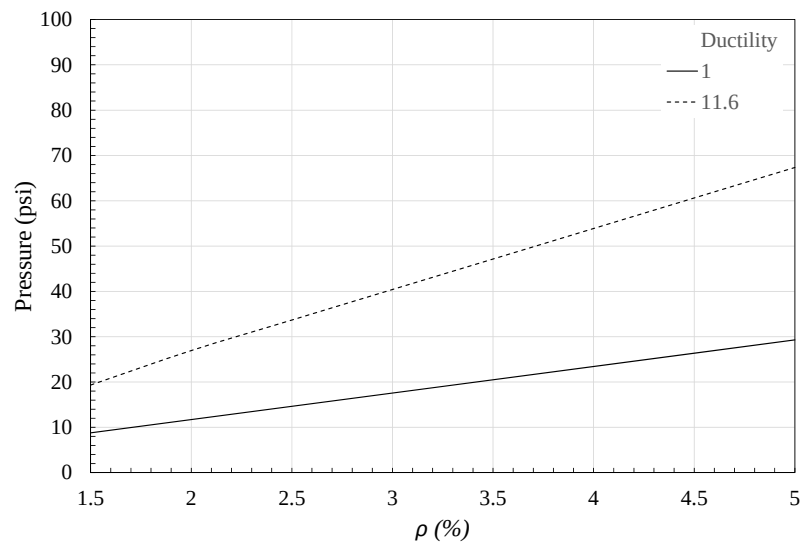
(a) $\mu = 1.0$ (b) $\mu = 11.6$

Figure 6.15. Ductility Contours of Varying Reinforcement Ratio (3-2-50-5 Cross-Section)



(a) Impulse (Impulsive Domain)



(b) Pressure (Quasi-Static Domain)

Figure 6.16. Influence of Reinforcement Ratio on Required Load (3-2-50-5 Cross-Section)

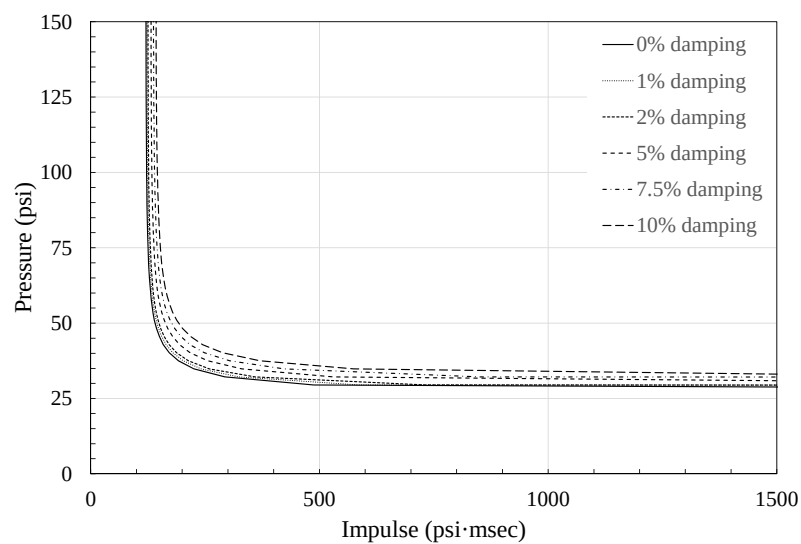
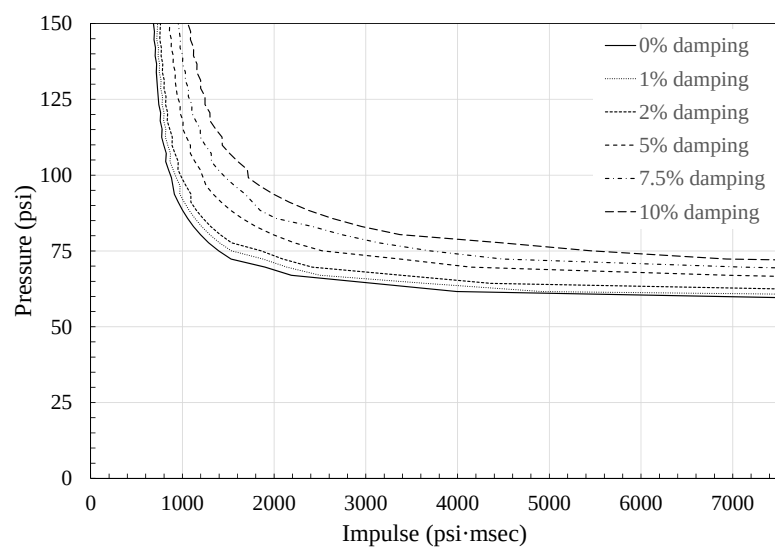
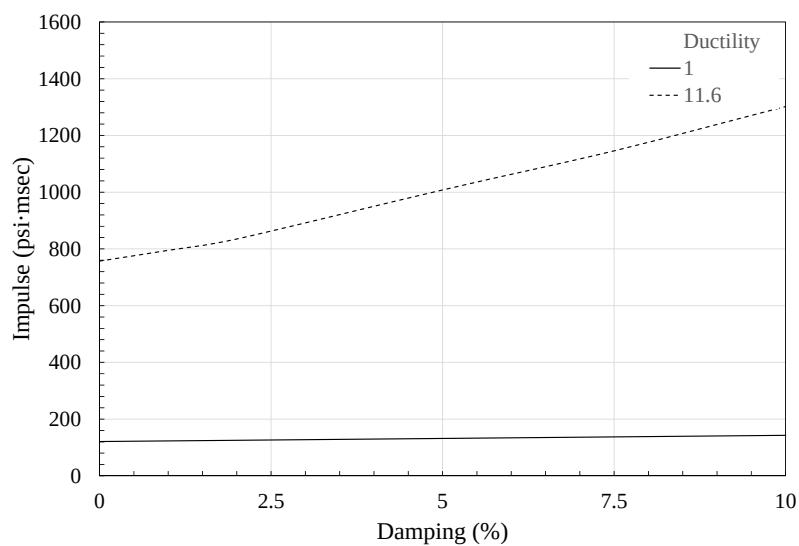
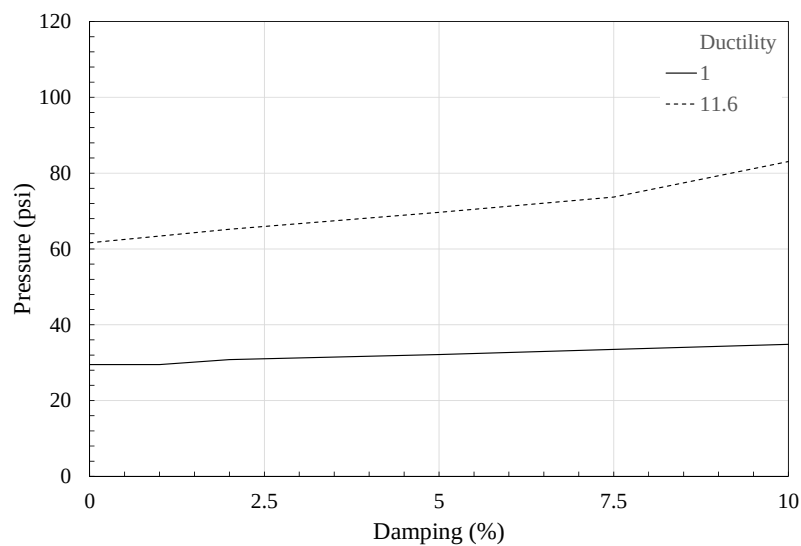
(a) $\mu = 1.0$ (b) $\mu = 11.6$

Figure 6.17. Ductility Contours of Varying Percent Damping (3-2-50-5 Cross-Section)



(a) Impulse (Impulsive Domain)



(b) Pressure (Quasi-Static Domain)

Figure 6.18. Influence of Percent Damping on Required Load (3-2-50-5 Cross-Section)

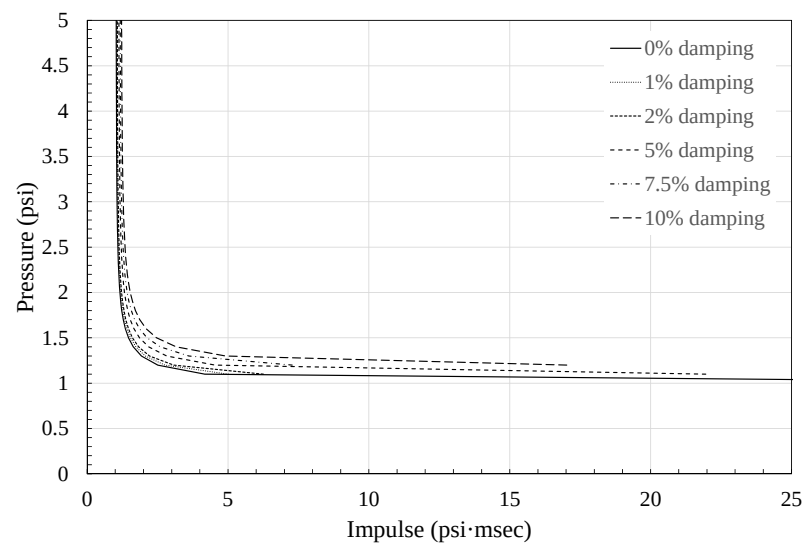


Figure 6.19. Normalized Ductility Contour of Varying Percent Damping (3-2-50-5 Cross-Section, $\mu=1.0$)

7. DESIGN RECOMMENDATIONS

This chapter describes a rational method to assess or design blast resistant SC walls and has been experimentally validated for:

- One-way flexure of simply supported SC walls without axial loads or openings,
- Far-field blast loads idealized as uniform pressure across the structure face,
- Blast pressure time-history idealized as triangular or exponential decay (e.g. Friedlander waveform), and
- Conventional construction materials ($50\text{-ksi} \leq F_Y \leq 65\text{-ksi}$; $3500\text{-psi} \leq f'_c \leq 7500\text{-psi}$).

This method merged design-basis specifications for SC structures from AISC N690s1 Appendix N9 with blast resistant design methods for RC structures from UFC 3-340-02 Chapter 4.

A structure is often designed first for design-basis loads (i.e. gravity, wind, earthquake, snow) and then checked for blast resistance. For these cases, Section 7.1 describes the process to assess the blast response of an initial design and compare to defined response limits. Methods are described to modify the design, if necessary, in order to improve performance. In cases for which a new SC wall is designed first for blast resistance, Section 7.2 describes a rational design methodology. Like most structural design processes, designing walls for blast resistance is inherently iterative and techniques for efficient design are described.

For assessment of an existing design or development of a new design, the process is similar and makes use of the P-I diagram in Figure 7.1 for assessment of design adequacy. The process is depicted in the flowchart in Figure 7.2. The primary

parameters that must be calculated or considered for each step are listed on the flowchart.

7.1 Method to Assess Blast Resistance of SC Walls

For cases in which a preliminary wall design exists and must be checked for blast resistance, a four step process can be followed:

1. Identify the blast load which the wall must resist and quantify load parameters p_r and i_r ,
2. Quantify resistance parameters R_y and ω_n of the SC wall design,
3. Confirm the SC wall design will perform within the defined limit, and
4. Refine SC wall design as needed to ensure it meets the defined limit.

Many of the equations used in this method were introduced earlier in this dissertation. For completeness, they are reproduced in this chapter with the original associated equation number and the word “revisited”. A few equations were modified from their earlier version by including a DIF; these equations are reproduced with the change and their original equation number with the phrase “incl. DIF”.

7.1.1 Identify Blast Load

The determination of what blast load to use in design is beyond the scope of this dissertation. Identifying the potential internal and external explosive threats for a given structure is a multi-disciplinary task based on risk and security assessments, planning and design assumptions, architectural and functional considerations, and required structural performance. The book *Modern Protective Structures* is a comprehensive reference design professionals can use for this important step [56].

With the threat identified, the two most important blast load parameters for the following method are the peak reflected pressure, p_r , and peak reflected impulse i_r .

These parameters can be determined in a variety of ways. ASCE 59-11 Chapter 4 describes acceptable methods for calculating these parameters [26].

7.1.2 Quantify Resistance of the SC Wall Design

In most cases, blast loads are considered beyond-design-basis. For this reason, initial design of an SC wall may have already been accomplished for the design-basis loads before assessing it for blast resistance and modifying the design as necessary. The static resistance function for the preliminary design can be developed and the natural frequency of the wall can be estimated using the preliminary design parameters and DIFs (see Table 7.1). In this table, DIFs from two different sources are provided as noted in the table footnotes. If available, measured material properties are recommended for use in these calculations. If measured values are not available, expected strength should be used.

- Calculate yield resistance using Equation (3.18) including strain rate effects on material properties

$$R_y = \frac{8M_y}{L} \quad (3.18 \text{ revisited})$$

Where M_y is calculated as in Equation (3.17) including the DIF for F_y from Table 7.1.

$$M_y = 0.9(DIF_{F_y} \cdot F_y)A_{st}t_{sc} \quad (3.17 \text{ incl. DIF})$$

- Estimate the natural frequency of the wall using Equation (7.1)

$$\omega_n = \sqrt{\frac{k_1}{M}} \quad (7.1)$$

Where k_1 is estimated from Equation (3.19), EI_{eff} is calculated from Equations (3.20)-(3.23), and the effective mass, M of the structural element is estimated from Equation (7.2). Equation (3.19) was modified to include the effect of the reduced net section as discussed in Section 3.6.2.

$$k_1 = \frac{A_n}{A_g} \frac{384EI_{eff}}{5L^3} \quad (3.19 \text{ modified})$$

$$EI_{eff} = E_s I_s + c_2 E_c I_c \quad (3.20 \text{ revisited})$$

In calculating EI_{eff} , E_s is typically assumed to be 29,000-ksi, E_c is estimated from Equation (3.8) including the dynamic increase factor on f'_c . The contributions of the moment of inertia of the steel and concrete are estimated from Equations (3.21) and (3.22) respectively.

$$I_s = \frac{bt_p(t_{sc} - t_p)^2}{2} \quad (3.21 \text{ revisited})$$

$$I_c = \frac{bt_c^3}{12} \quad (3.22 \text{ revisited})$$

$$c_2 = 0.48 \frac{2t_p}{t_{sc}} \frac{E_s}{E_c} + 0.10 \quad (3.23 \text{ revisited})$$

$$E_c = 57,000 \sqrt{DIF_{f'_c} f'_c} \quad (3.8 \text{ incl. DIF})$$

$$M = K_{LME} \frac{Lb(t_c \gamma_c + 2t_p \gamma_s) / 12^3}{g} \quad (7.2)$$

In Equation (7.2), γ_c can be assumed as 150-pcf to account for additional mass from studs and tie bars, γ_s is typically 490-pcf, and g is $386\text{-in}/\text{sec}^2$. The denominator in the fraction in the numerator of Equation (7.2) converts the γ_c and γ_s from pcf to pounds per cubic inches. The value for the elastic load-mass transformation factor, K_{LME} , is available in most structural dynamics textbooks or UFC 3-340-02. Values for uniform loads are provided for convenience in Table 7.2. Values for elastic, elasto-plastic, and plastic behavior are provided; the elastic response is the correct value for calculating k_1 as this is the elastic stiffness of the structural element.

7.1.3 Check Design Against Response Limit

Compute total force, P , and total impulse, I , from Equations (4.2) and (4.3) respectively and use Figure 7.1 to determine if the design is within the defined response limit. Four of the five ductility limits for which contour lines are provided in Figure 7.1 correspond to response limits for RC wall elements in compression as

defined in ASCE 59-11 Table 3-3 [26]. The fifth contour line corresponds to the allowable ductility defined by AISC N690s1 N9.1.6b for SC walls used in safety related nuclear facility structures. The reason for the difference in allowable ductility lies in the purpose of typical walls in nuclear facilities. Unlike a typical RC residential or commercial office building which support gravity loads from floors, shield walls in nuclear facilities often support only their self-weight and the roof. Recommending response limits for SC walls is beyond the scope of this dissertation.

$$P = \frac{2p_r bL}{R_y} \quad (4.2 \text{ revisited})$$

$$I = \frac{i_r bL}{R_y/\omega_n} \quad (4.3 \text{ revisited})$$

If the computed P-I point falls below or to the left of the applicable limit contour in Figure 7.1, the design is safe. If the point falls above or to the right of the contour the design is unsafe.

7.1.4 Refine Design

If the design is deemed unsafe, several parameters could be modified to improve the resistance. Based on findings from experimental and parametric studies presented in Chapters 4 and 6, the following methods to improve resistance are listed in order of influence (most influential listed first):

- Increase section depth, t_{sc} , or decrease unsupported length, L . The ratio $L : t_{sc}$ was shown to have the largest influence on reducing ductility demand and had the largest influence in the dynamic and quasi-static domains.
- Increase reinforcement ratio, ρ (i.e. increase faceplate thickness, t_p). This parameter was shown to have a significant influence on reducing ductility demand in all three response domains.
- Increase faceplate strength, F_y . This parameter was shown to have an influence on reducing ductility demand within all three response domains.

Increasing concrete strength, f'_c , was not recommended to improve performance because this parameter was shown to have negligible influence on ductility demand. Although increasing t_{sc} is listed first this may not be possible for cases in which other wall thickness limitations must be met. For these cases, increasing ρ and/or F_y are the best options for improving wall resistance.

7.2 Rational Design Method for Blast Resistant SC Walls

For cases in which a preliminary wall design does not exist, the following steps will lead to an efficient design:

1. Identify the blast load which the wall must resist and quantify load parameters p_r and i_r ,
2. Estimate required wall thickness,
3. Define remaining SC wall design parameters,
4. Quantify resistance parameters R_y and ω_n of the preliminary SC wall design,
5. Check preliminary SC wall design against response limit,
6. Refine preliminary design to ensure an efficient SC wall within response limit, and
7. Complete the design in accordance with AISC N690s1 Appendix N9.

7.2.1 Identify Blast Load

This step is the same for a new design as for the assessment of an existing design as explained in Section 7.1.1.

7.2.2 Estimate Required Wall Thickness

AISC N690s1 N9.1.1(a) defines minimum and maximum limits for SC walls. No SC wall can exceed 60-in in NPP applications. The minimum wall thickness is 18-in for exterior walls and 12-in for interior walls. A reasonable starting point for most far-field blast loads is the minimum thickness.

7.2.3 Complete Preliminary SC Wall Design

With the initial total wall thickness selected, the remainder of the design parameters must be defined. For conventional designs, concrete and steel faceplate strengths are often 5-ksi and 50-ksi respectively. AISC N690s1 N9.1.1 imposes limits that must be adhered to in design of SC walls for nuclear facilities and are reasonable limits for SC walls used in other applications:

- $0.25\text{-in} \leq t_p \leq 1.50\text{-in}$,
- $1.5\% \leq \rho \leq 5\%$,
- $50\text{-ksi} \leq F_y \leq 65\text{-ksi}$, and
- $f'_c \geq 4\text{-ksi}$.

Additional limitations are included in AISC N690s1 N9.1.1 that must be considered for other details of SC wall design (i.e. headed stud anchor and tie bar spacing, dimensions of holes in faceplates, and faceplate splices) but are not critical for preliminary design as described here. Design of these components is more appropriately done after the initial wall design is deemed adequate. See Section 7.2.7 for methods to design the remaining wall components.

7.2.4 Quantify Resistance of the SC Wall Design

With the basic wall design parameters defined, the process proceeds with quantifying R_y and ω_n as explained in Section 7.1.2.

7.2.5 Check Design Against Response Limit

Compute total force, P , and total impulse, I , and use Figure 7.1 to determine if the design is within the defined response limit as described in Section 7.1.3.

7.2.6 Refine Design

If the preliminary wall design is deemed unsafe, the recommendations in Section 7.1.4 can be applied to improve the resistance of the wall. If the design is safe but deemed excessively conservative, design parameters can be modified to reduce resistance and gain a more efficient design.

7.2.7 Complete Design of Remaining SC Wall Components

As described to this point, the principal parameters of an SC wall design have been selected but the design remains incomplete. Several important aspects of the design remain:

- Select diameter and spacing of tie bars,
- Select diameter and spacing of headed stud anchors,
- Confirm shear strength of the SC wall exceeds the flexural strength of the SC wall to prevent brittle shear failure, and
- Check the final design to confirm it meets all required provisions of the specification.

7.2.7.1 Design Tie Bars

Minimum tie bar area can be calculated from out-of-plane shear strength provisions of AISC N690s1 N9.3.5 accounting for DIFs on concrete and steel strength. As this section of the specification explains, the shear capacity of an SC section is the sum

of the contributions of concrete and tie bars if tie bar spacing is no larger than half the section depth ($S \leq t_{sc}/2$). Because shear demand is high in structures subjected to blast loads, tie bar spacing in blast resistant SC structures is recommended to be no larger than $t_{sc}/2$. Rearranging N690s1 Equations (A-N9-20) - (A-N9-22) to solve for the minimum area of a single tie bar results in Equation (7.3) where f'_c is in ksi.

$$A_{tie} \geq \frac{\frac{1}{\phi_{vo}} \left(\frac{L - t_{sc}/2}{L} \right) \left(\frac{R_y}{2} \right) - (0.05 \sqrt{DIF_{f'_c} f'_c}) t_c b}{\xi \frac{t_c b}{S^2} DIF_{F_y} F_{y,tie}} \quad (7.3)$$

Where ϕ_{vo} is 0.75, R_y is calculated from Equation (3.18), the DIFs are taken from Table 7.1, and ξ is either 1.0 or 0.5 for yielding or non-yielding ties, respectively. Yielding ties are defined as those for which the yield strength is no greater than 80% of the ultimate tensile strength and are recommended for blast resistant structures. The area of the ties also depends on the selected tie bar spacing, S .

With the tie dimension selected, the total shear strength provided by the ties, V_s , must be checked against a maximum limit using Equation (7.4) where f'_c is in ksi.

$$V_s = \xi \frac{t_c b}{S^2} A_{tie} DIF_{F_y} F_{y,tie} \leq \left(0.25 \sqrt{DIF_{f'_c} f'_c} \right) t_c b \quad (7.4)$$

If the condition of Equation (7.4) is met, the tie bar design and shear strength provided by the combination of concrete and tie bars are adequate. If this condition is not met, the section must be redesigned to meet the required shear strength, V_u (Equation (7.5)), at a location half the section depth away from the support. This typically requires increasing the concrete contribution to shear strength (see Equation (7.7)) to reduce the required steel contribution. In most cases, increasing the section depth will be required because increases in concrete strength have small effects on shear strength. This change in section depth increases V_u making this process iterative.

$$\phi_{vo} V_n \geq V_u \quad (7.5)$$

$$V_n = V_{conc} + V_s \quad (7.6)$$

$$V_{conc} = \left(0.05 \sqrt{DIF_{f'_c} f'_c} \right) t_c b \quad (7.7)$$

$$V_u = \left(\frac{L - t_{sc}/2}{L} \right) \frac{R_y}{2} \quad (7.8)$$

7.2.7.2 Design Headed Stud Anchors

Proper design of headed stud anchors achieves two purposes: (1) develop yield strength of the steel faceplates and (2) ensure interfacial shear strength is greater than out-of-plane shear strength of the section. The method in AISC N690s1 N9.1.4. is described in this section. No DIFs are applied to headed stud anchor strength because the literature lacks recommendations.

Before calculating the required cross-sectional area of stud anchors, the spacing, s , should be determined to meet the faceplate slenderness requirement of N690s1 N9.1.3. reproduced as Equation (7.9).

$$\frac{s}{t_p} \leq 1.0 \sqrt{\frac{E_s}{F_y}} \quad (7.9)$$

Where E_s and F_y are the elastic modulus and yield strength of the steel faceplates, respectively.

The cross-sectional area of stud anchors, A_{sa} , is governed by the larger of two limits as determined from Equations (7.10) and (7.11) (rearrangments of N690s1 Equations (A-N9-3) and (A-N9-4) with substitutions of equations for variables). Equation (7.10) ensures the yield strength of the faceplates is developed within a development length equal to three times the section depth and Equation (7.11) ensures the interfacial strength exceeds the out-of-plane shear strength as discussed in Section 7.2.7.1.

$$A_{sa} \geq \left(\frac{F_y t_p}{\phi_v F_{u,sa} (3t_{sc})} \right) \left(\frac{s}{c_1} \right)^2 \quad (7.10)$$

$$A_{sa} \geq \frac{\phi_{vo} V_n}{0.9 t_{sc}} \left(\frac{1}{\phi_v F_{u,sa} b} \right) \left(\frac{s}{c_1} \right)^2 \quad (7.11)$$

In these equations, ϕ_v is 0.65, V_n is from Equation (7.6), F_y is the yield strength of the steel faceplate, $F_{u,sa}$ is the ultimate tensile strength of the headed stud anchors, and c_1 is 1.0 or 0.7 for yielding or nonyielding stud anchors. Yielding stud anchors are required by N690s1 N9.1.4a.

7.2.7.3 Check Direct Shear Capacity

Although experimental investigations of concrete filled tubes subject to blast loads have shown that these types of structures are not as prone to direct shear failure as RC columns are [62], [63], this check is still recommended. Conservatively, the influence of the steel faceplates on direct shear resistance is neglected. Direct shear is largest at the supports and this is the location at which this check should be made as shown in Equation (7.12) (UFC 3-340-02 Equation (4-30)).

$$V_d \geq V_S \quad (7.12)$$

$$V_S = \frac{R_y}{2} \quad (7.13)$$

$$V_d = 0.16DIF_{f'_{c, shear}} f'_c b t_c \quad (7.14)$$

7.2.7.4 Check Flexure-to-Shear Capacity Ratio

At this point, the design is essentially complete. A check of the flexure-to-shear capacity with Equation (7.15) is an instructive measure to identify if the wall behavior is likely to be flexure- or shear-controlled.

$$\frac{\phi_b M_y}{\phi_{vo} V_n t_{sc}} \quad (7.15)$$

Where M_y and V_n are computed from Equations (3.17 incl. DIF) and (7.6) respectively. The strength reduction factor ϕ_b is 0.90.

This simple check results in a numerical value in terms of wall thickness (e.g. a certain wall may have a flexure-to-shear capacity of $3t_{sc}$). By normalizing this ratio to the wall thickness, some rules-of-thumb can be applied. The smaller the ratio, the more likely it is to be flexure-controlled. Specimens tested as part of this project had ratios ranging from $2.1t_{sc}$ for 5-2b-50-5 to $5.7t_{sc}$ for 5-4-50-5. All specimens were flexure-controlled in both static and dynamic tests.

AISC N690s1 N9.1.6b. allows a ductility demand limit of 10 for impulse or impact loads on flexure-controlled SC walls. The same section defines a flexure-controlled SC

wall as one in which the flexural yielding capacity is less than the out-of-plane shear capacity by at least 25%. For a uniformly loaded one-way wall with simple supports, this check can be made using Equation (7.16).

$$\frac{LV_n}{4M_y} \geq 1.25 \quad (7.16)$$

In Equation (7.16), the constant 4 in the denominator would change to 6 for a one-way fixed-fixed wall, 2 for a one-way cantilever wall, or 5 for a one-way propped cantilever wall.

7.2.7.5 Final Design Checks

AISC N690s1 Appendix N9 includes several other requirements that must be met:

- Check minimum required tie bar strength - AISC N690s1 Equation (A-N9-6), and
- Confirm general provisions of AISC N690s1 N9.1.1 are satisfied.

7.2.8 Complete Numerical Analysis if Necessary

For many cases, the design process described above will be adequate. For cases in which additional detail about the response of the designed wall to the specified blast load is desired, the benchmarked numerical methods described in Chapter 5 may be used. For example, the design parameters can be used to develop the idealized bi-linear static resistance function, the blast load can be idealized as a Freidlander waveform or triangular pulse, and the SDOF equation of motion can be solved for the maximum displacement. For more fidelity in understanding the response of the wall to the blast load, an FE model may be analyzed. The FE analysis provides maximum displacement but also provides useful insights such as areas of highest shear stress in which additional shear reinforcement may be desirable.

7.3 Assessment Example: Minimum Practical SC Wall

Consider the minimum practical interior SC wall which is 12-in thick with 0.25-in Gr50 steel faceplates with the following assumptions and requirements:

- 5-ksi concrete strength,
- Headed stud anchors and tie bars were designed to ensure this wall was flexure-controlled and meet all applicable requirements of AISC N690s1,
- This 12-ft high wall was wide enough to be modeled as one-way action and could be idealized with simple supports at top and bottom,
- The owners completed a risk assessment and concluded that the largest blast threat for this wall would be 5000-lb of TNT-equivalent at a standoff of 75-ft, and
- The damage limit from AISC N690s1 N9.1.6b. must be met.

This example follows the steps described in Section 7.1. Calculations for this example are included as Appendix C.

1. *Identify Blast Load.* Using ATBlast software [57] or Table 2-7 from UFC 4-340-02 [24], the peak pressure and impulse of this blast threat can be determined. From ATBlast: $p_r = 230\text{-psi}$, $i_r = 820\text{-psi}\cdot\text{msec}$.
2. *Quantify Resistance of the SC Wall Design.* Using equations provided in Section 7.1, resistance parameters were calculated. For this wall, R_y was 116-kips, k_1 was 164-kips/inch , and ω_n was 203-sec^{-1} . DIFs from AISC N690s1 were used for these calculations (see Table 7.1).
3. *Check Design Against Response Limit.* From the blast load and SC wall resistance parameters, the normalized total force and impulse for this case were: $P = 6.8$ and $I = 2.5$. Plotting this point on Figure 7.1 indicated that this wall subjected to this load was well within the response limit for a flexure-controlled SC wall defined by AISC N690s1 N9.1.6b.

4. *Refine Design.* Because this wall design was below the defined response when subject to this blast load, redesign is unnecessary. The assessment is concluded: this minimum interior SC wall would perform within the defined damage limit for a 5000-lb TNT equivalent explosion at 75-ft standoff.

7.4 Design Example: Resist a Specific Blast Load

For this example the required exterior wall was 20-ft high, wide enough to be modeled as one-way action, and assumed to be simply supported at top and bottom. The owner stipulated that the wall must resist a 2500-lb TNT-equivalent at a standoff of 60-ft and must meet the moderate damage limit from ASCE 59-11.

The example follows the steps described in Section 7.2. Calculations for the final design for this example are included as Appendix D.

1. *Identify Blast Load.* The blast load parameters for the defined blast threat were: $p_r = 230$ -psi and $i_r = 650$ -psi·msec.
2. *Estimate Required Wall Thickness and Complete Preliminary SC Wall Design.* Because this was an exterior wall, the starting thickness was selected as 18-in. For convenience, the starting faceplate thickness was selected as 0.25-in. Typical material properties were assumed: $f'_c = 5$ -ksi and $F_y = 50$ -ksi.
3. *Quantify Resistance of the SC Wall Design.* For this initial wall design, R_y was 120-kips, k_1 was 217-kips/inch, and ω_n was 173-sec⁻¹. DIFs from AISC N690s1 were used for these calculations (see Table 7.1).
4. *Check Design Against Response Limit.* The normalized total force and impulse for this case were: $P = 8.1$ and $I = 2.0$. Plotting this point on Figure 7.1 indicated that this design did not meet the moderate damage limit state defined by ASCE 59-11 when subjected to the required load.

5. *Refine Design.* Because the initial wall did not meet the limit state the design had to be modified. Because t_{sc} was shown to have the largest influence on resistance, the first attempt to refine the wall design was to increase t_{sc} to 24-in which reduced P to 6.1 and I to 1.8 - still not within the specified limit state. The second refinement attempt increased t_p to 0.5-in ($\rho=4.2\%$) resulting in $P = 3.1$ and $I = 1.1$. This was closer to the limit state but still inadequate. The third refinement attempt increased F_y to 60-ksi and brought the wall design to within the defined limit state ($P = 2.5$, $I = 0.94$).

6. *Complete Design of Remaining SC Wall Components*

- Minimum tie bar area for this case with tie bars spaced at $t_{sc}/2$ was 1.93-in² (#14 bars welded to both faceplates provide this). The steel contribution to shear strength for this tie bar design exceeded the limit in Equation (7.4). There were several options for redesign of shear strength: increase t_{sc} , increase F_y of the tie bars, decrease S , or increase f'_c (the influence of f'_c on shear strength is minimal). Through iteration, an adequate design was found to be: $t_{sc} = 30$ -in, $t_p = 0.375$ -in ($\rho=2.5\%$), and #7 bars at 10-in spacing (Gr60 steel) for shear reinforcement ($P = 4.2$, $I = 0.9$).
- The minimum shear stud area for this case with studs spaced at 5-in (half of the tie bar spacing) was 0.228-in². Nelson H4L 5/8" headed stud anchors met this requirement. The minimum length for this stud was 5-in (from AISC 360-10 I8.3 [34]).
- The direct shear capacity of this design was 309-kips which exceeded the maximum direct shear demand of 156-kips.
- The flexure-to-shear capacity for this design confirmed that this design was flexure-controlled. The ratio of moment to shear capacity was $1.84t_{sc}$.
- Final design checks confirmed that this wall was designed in accordance with the provisions of AISC N690s1 Appendix N9.

- SUMMARY: Components of the 20-ft SC wall to resist 2500-lb of TNT at 60-ft stand-off: $t_{sc} = 30$ -in ($f'_c=5$ -ksi), $t_p = 0.375$ -in (Gr60 steel), #7 bars at 10-in spacing (Gr60 steel), Nelson H4L 5/8" headed studs at 5-in (Gr60 steel).

7.4.1 Numerical Analysis of the Design Example

7.4.1.1 SDOF Model of Design Example

An SDOF model of the wall design described in Section 7.4 was created using the method explained in Section 5.2. The maximum mid-span displacement from this analysis was 0.635-in with 0% damping. The yield displacement for this section was 0.734-in using the idealized resistance function which leads to a ductility demand of 0.87. This demand was less than the allowable limit of 1.0 confirming the adequacy of the design for the specified load.

7.4.1.2 FE Model of Design Example

An FE model of the wall design described above was created using the method explained in Section 5.1. The maximum mid-span displacement from this analysis was 0.727-in and neither faceplate yielded indicating a ductility demand less than 1.0. Flexure cracks occurred near mid-span and some shear cracks initiated near the supports. Axial stress in the tie bars in the region in which shear cracks began to form was higher than axial stress in tie bars elsewhere, but the tie bar element with the highest stress was only at approximately 50% of yield stress under this blast load.

7.5 Chapter Summary

This chapter explained rational methods to assess or design an SC wall for blast resistance. The method was developed using provisions of AISC N690s1 Appendix N9 modified to account for strain rate effects on material properties and SDOF analysis of one-way SC walls subjected to pressure loads. A design aid was presented which provided damage contours for response limits defined in ASCE 59-11 for RC walls in compression and defined in AISC N690s1 N9.1.6b for SC walls. This design aid enables quick assessment of existing walls for specified blast loads and is useful for design. The following chapter summarizes the conclusions of this dissertation and describes recommendations for future work.

Table 7.1. Dynamic Increase Factors for Design

Material Property	For Calculating	DIF*	DIF**
f'_c	Yield Moment	1.19	1.25
f'_c	Shear Capacity	1.10	1.10
F_y	Yield Moment	1.12	1.29

* permitted by ASCE 59-11 Table 3-5 [26]

** permitted by AISC N690s1 Table A-N9.1.1 [5]

Table 7.2. Transformation Factors for Uniformly Loaded One-Way Elements (excerpt from UFC 3-340-02 Table 3-12 [24])

Edge Conditions	Range of Behavior	K_{LM}
Simply Supported	Elastic	0.78
	Plastic	0.66
Fixed-Fixed	Elastic	0.77
	Elasto-Plastic	0.78
	Plastic	0.66
Propped Cantilever	Elastic	0.78
	Elasto-Plastic	0.78
	Plastic	0.66
Cantilever	Elastic	0.65
	Plastic	0.66

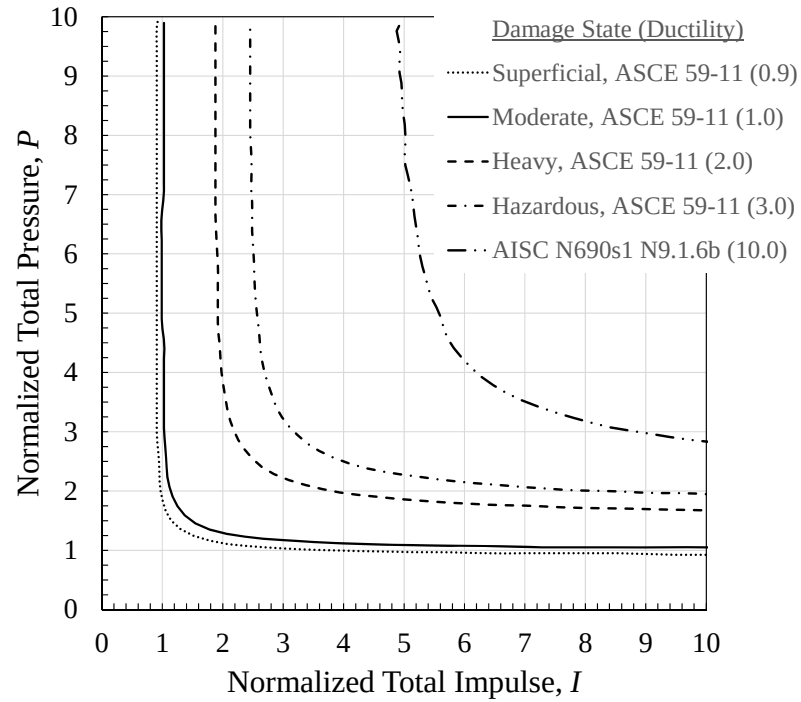


Figure 7.1. Design Aid: P-I Diagram with Wall Limit States

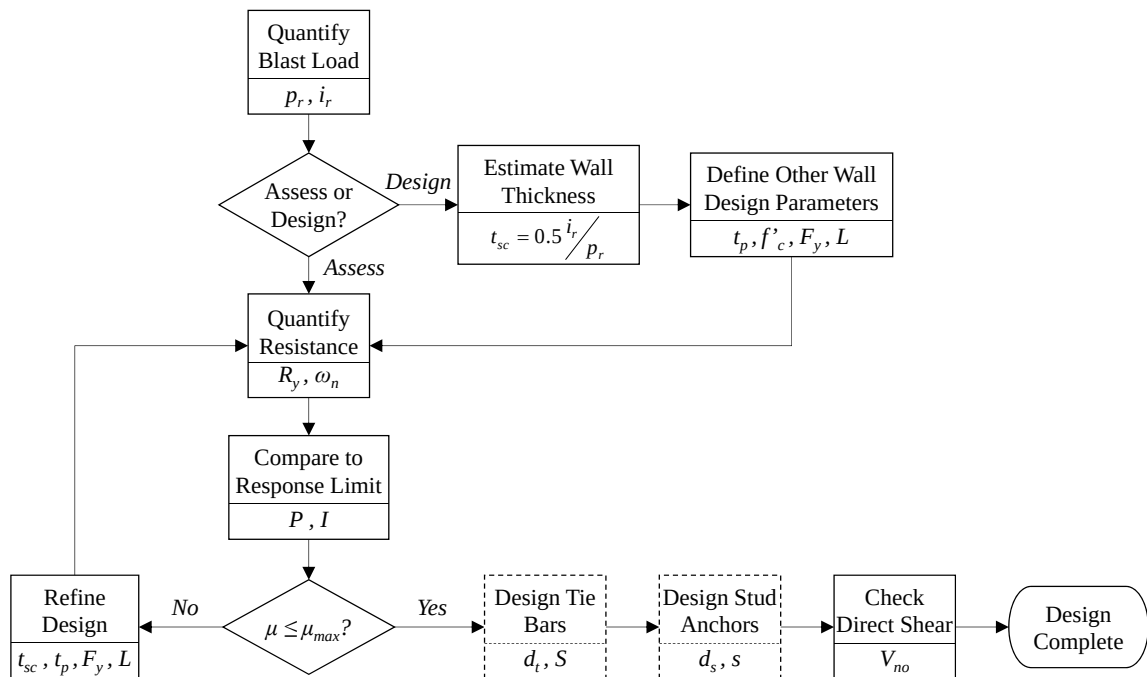


Figure 7.2. Process to Assess or Design Blast Resistant SC Wall

8. SUMMARY AND CONCLUSIONS

This chapter summarizes the findings from experimental and numerical studies of the response of simply supported one-way SC walls to blast loads. Final conclusions and observations are highlighted and recommendations for future work are presented.

8.1 Research Summary

RC has historically been the preferred choice for blast resistant structures for several reasons: the mass of the concrete, the ductility of the steel reinforcement, and the extensive experimental database of RC structures subjected to blast loads from which design methods have been validated. SC walls are a viable alternative for the same reasons as RC with the exception of an experimental database. SC offers several advantages over RC: The faceplates prevent concrete spalling, by locating the steel plates on the outside of the concrete the flexural resistance is maximized, the plates serve as formwork which reduces construction time, and SC modules can be prefabricated which improves construction efficiency. Experimental tests were conducted on SC wall sections to evaluate their performance when subjected to blast loads, begin to build a publicly accessible experimental database, and inform design recommendations.

Two series of experiments were completed to: (1) quantify the static performance and available ductility, and (2) quantify the response under different blast loads. An innovative method to quantify the static resistance function was used. Rather than using specialized equipment such as a loading tree or a vacuum or water chamber, a four-point bending test was conducted on eight different configurations of SC wall sections. FE models were benchmarked to experimental results and then the loading in the FE models was changed to increasing uniform load. The result was a static re-

sistance function for uniform pressure loads without the use of specialized equipment. This method enables researchers at most structural engineering research laboratories to develop static resistance functions at reduced expense. Dynamic tests were conducted at the USACE-ERDC BLS and used innovative instrumentation techniques to provide primary and supplementary displacement measurements and record faceplate strains.

The eight configurations of SC wall sections tested experimentally varied the flexural and shear reinforcement ratios, steel faceplate strength, and tie bar spacing. During each four-point bending test, displacement at each quarter point, rotations of the specimen at the supports and each load point, strain at mid-span of the compression faceplate and quarter points of the tension faceplate, and the total load applied were measured.

Static experiments demonstrated that each of these designs was flexure-controlled and the shear strength of the concrete was greater than $2\sqrt{f'_c}$ for these specimen dimensions because shear failure did not occur. All specimens ultimately failed due to net section rupture of the tension steel plate at deformation ductility between 7.1 and 13.1. The yield strength of all specimens was within 6% of estimated strength and stiffness was within 10% of the estimated values.

Two methods to estimate moment-curvature for an SC wall section were developed. One was based on the Ramberg-Osgood power-law model for stress-strain providing a closed-form solution for curvature along the length of a one-way bending element. The second method was a fiber model which used the Hognestad parabola [74] for concrete stress-strain and a power law developed by Varma [71] for the steel faceplate stress-strain. The fiber model directly accounted for slip between the steel plates and concrete infill for cases with less than 100% composite action while the power-law model accounted for this indirectly through the best-fit parameters. Both methods provided reasonable approximations of experimentally determined moment-curvature and could be used to estimate the static resistance function for a variety of load conditions by using the moment-area method.

FE models were benchmarked to experimental results for all eight specimens using LS-DYNA. Fundamental moment-curvature and load case specific load-displacement relationships from the FE models closely matched the experimental results. The maximum effective plastic strain for the steel faceplates was established by examining the FE results at the displacement at which tensile faceplate rupture occurred in the experiments. These maximum effective plastic strain values were used to estimate the ultimate capacity of the specimens when the load case was analytically changed from four-point bending to uniform pressure.

Results from the FE models subject to uniform load were used to develop a bi-linear approximation for the static resistance function of one-way SC wall sections. This idealization is an important input to SDOF models used to estimate the behavior of blast loaded SC walls and was developed using accepted capacity and stiffness equations for SC walls.

Twelve specimens were subjected to short duration pressure waves in the USACE-ERDC BLS. The designs for the static test specimens were used for these dynamic test specimens. Two sets of dynamic tests were conducted: (1) six specimens were first subjected to a pressure pulse for which an elastic response was expected and then the same specimens were subjected to a larger pressure pulse intended to achieve plastic deformation, and (2) six specimens were subjected to a pressure pulse intended to achieve plastic deformation. Displacement and acceleration at mid-span, displacement at a location 8.75-in from mid-span, and strains at mid-span on the compressive faceplate and at the quarter points on the tension faceplate were recorded for each specimen. Pressure time-histories at eight locations around the perimeter of the tested specimens were recorded for each shot.

The FE models initially benchmarked to the static experiments were modified and benchmarked to the dynamic experiments. An important modification was the inclusion of DIFs for concrete and steel material properties. Models using the Winfrith concrete model's internal capability to account for strain rate effects resulted in inaccurate results. Excellent results were obtained when DIFs for f'_c , f_t , and G_f , based

on extensive experimental databases available in the literature, were used. Results were sensitive to the concrete tensile strength assumed in the analysis.

SDOF models were also benchmarked to the dynamic experimental results. These models used the idealized bi-linear static resistance function described above and the pressure pulses were modeled using a modified Friedlander waveform. Results were sensitive to the amount of equivalent viscous damping included in the model and were closest to experimental results when 5% equivalent viscous damping was used.

A variety of parametric studies were completed using the benchmarked numerical models (both FE and SDOF). Results were provided as p-i and P-I diagrams from which conclusions about the influence of the parameter being investigated could be made. The baseline P-I diagram was benchmarked to results from the dynamic experiments.

8.2 Conclusions

Based on the static experimental investigation conducted on one-way SC wall sections, the following conclusions were drawn:

1. Scaled specimens of SC wall sections exhibited the same fundamental behavior as full-scale specimens governed by flexural yielding. Equations from AISC N690s1 for SC walls accurately estimated the strength and stiffness of the scaled specimens tested.
2. Deformation ductility larger than 7.0 and curvature ductility larger than 8.0 were observed for flexure-controlled SC walls with holes through the faceplates in order to fasten threaded tie bars. Ductility was higher for specimens constructed using Gr50 steel faceplates than for those using Gr65 or Gr80.
3. Specimen ductility was directly related to the steel plate material ductility and the ratio of net to gross steel plate area. Ductility was indirectly related to steel faceplate strength.

4. Ultimate failure was the result of net section fracture for flexure-controlled SC walls with holes through the faceplates in order to fasten threaded tie bars. Fracture occurred in the region of highest combined moment and shear (under the load point in the four-point bending tests).
5. No concrete compression failure occurred even at large displacements. Flexure cracks extended through nearly the entire section depth.
6. The normalized moment-curvature and load-displacement relationships were similar for all varieties of SC wall design tested. These normalized curves enabled development of idealized bi-linear static resistance functions for simply supported one-way SC walls.

Based on the dynamic experimental investigation conducted on one-way SC wall sections, the following conclusions were drawn:

1. Behavior of SC walls designed to be flexure-controlled is dominated by flexure when subjected to blast loads.
2. Ductility demand was inversely related to reinforcement ratio for a given pressure pulse. This was most notable at lower pressure and impulse combinations. As impulse increased, sensitivity to reinforcement ratio decreased.
3. Ductility demand was inversely related to faceplate yield strength for a given pressure pulse. As impulse increased, the sensitivity to faceplate strength decreased.
4. Ductility demand was inversely related to shear reinforcement ratio (tie bar diameter). This was most notable at lower pressure and impulse combinations in the elastic region of response. In the plastic region, the opposite was true for this specific wall configuration because of the decreased net section as tie bar diameter increased.

5. Maximum displacements were most reliably measured using potentiometers or extensometers rather than computing displacements from accelerometer data. Residual displacements were only able to be reliably measured using potentiometers and extensometers.

Based on the numerical investigations conducted on one-way SC wall sections, the following conclusions were drawn:

1. Behavior of SC wall sections subjected to static and blast loads can be predicted with FE models. These models provided estimates of moment-curvature and load-displacement curves for static loads that compared well to experimental results. The models also provided estimates of mid-span deflection, damping, and crack patterns for dynamic loads that compared well to experimental results.
2. The only modification between an FE model of a statically loaded SC wall and a blast loaded wall was the inclusion of DIFs to account for strain rate effects on steel faceplate and concrete infill material properties. Using the strain rate effects feature included in the Winfrith concrete model resulted in inaccurate results. More reasonable results were obtained by directly accounting for DIFs in the concrete material properties and keeping the strain rate feature off in the Winfrith concrete model.
3. Maximum mid-span deflection of SC walls subject to blast loads can be predicted with SDOF models using bi-linear static resistance functions.
4. Results from FE models of the response of SC wall sections to blast loads are sensitive to concrete tensile strength.
5. Results from SDOF models of the response of SC wall sections to blast loads are sensitive to the percent of equivalent viscous damping included.
6. Ductility demand and normalized total force and impact were the best measures to compare the response of SC wall sections of various design parameters.

7. Of the design parameters included in this study, steel faceplate strength, F_y , was observed to have the largest influence on response of an SC wall subject to blast loads in all three response domains.
8. Reinforcement ratio, ρ , was observed to have the second largest influence on the response of blast loaded SC walls in all three response domains.
9. Increases in span-to-depth ratio of an SC wall, $L : t_{sc}$, resulted in reductions in pressure required to achieve a certain ductility demand. This influence was observed for the dynamic and quasi-static response domains and was negligible in the impulsive domain.
10. Concrete compressive strength, f'_c , has minimal effect on the response of blast loaded SC walls. Response was independent of f'_c in the quasi-static domain and only a small influence was observed in the dynamic and impulsive domains.
11. Specimen vibration and damping was most clearly observed from strain data rather than displacement data.

8.3 Recommendations for Future Work

Publicly available information concerning blast response of SC walls is limited. This study has begun to fill this gap but was necessarily limited in scope. There are several areas for future research to continue to improve the understanding of the response of SC walls to a variety of blast load conditions and to further quantify the influence of design parameters on this response.

- *Blast response of two-way SC walls subjected to far-field explosions.* Because the present study demonstrated the similarity of blast response of one-way SC and RC walls, it is expected that this would extend to two-way behavior. Investigations of two-way response of SC walls to projectile impact have begun but so far this has been restricted to numerical investigation [47]–[49]. Physical

experiments to quantify static resistance of two-way SC walls will be broadly useful - improving understanding of static behavior and providing the basis for numerical analysis of projectile impact or blast loads. The USACE-ERDC BLS could be used for two-way tests but the current target vessel opening is restrictively small. Until USACE-ERDC completes their larger BLS, field tests would be necessary to physically test the blast response of two-way SC walls.

- *Blast response of SC walls subjected to near-field explosions.* Pressure from near-field explosions cannot be idealized as uniform across the surface of a structure as for far-field explosions. Experimental investigations of near-field explosive effects on concrete filled steel boxes have been completed [18] but these panels did not include headed stud anchors or tie bars in their design. Investigations to quantify the influence of all design parameters on SC walls subject to near field explosions are necessary to thoroughly understand blast response of SC structures.
- *Blast response of SC walls subjected to in-contact explosions.* Response of SC walls to in-contact explosions is expected to be similar to the response of walls to projectile impact. While there is a large database of projectile impact tests on SC walls there is no publicly available literature about the response to in-contact explosions. Results from these studies would be particularly important to understand how to design SC walls to prevent breaching.
- *Experimentally confirm blast response of SC walls to far-field explosions in the impulsive domain.* The experiments conducted for this project consisted of blast loads that were in the dynamic domain. Numerical models expanded the database into the impulsive domain. Experimental verification would be useful in expanding the database from which design methods could be further validated.
- *Accounting for strain rate effects on concrete tensile strength in numerical models.* Variability of concrete tensile strength at high strain rates was demon-

strated to account for variability in blast response of SC walls. Models in this study manually accounted for the DIF on tensile strength. Doing so requires knowledge of expected strain rates from a specific load case which is not always possible. Development of concrete material models for numerical modeling which accurately account for strain rate effects on compressive and tensile strength, elastic modulus, and fracture energy of concrete would be a valuable contribution for predicting performance of blast resistant SC structures.

- *Designing and detailing blast resistant SC structural connections.* An understanding of the structural response of wall sections to explosions enables quantifying loads that connections must resist. The capacity of SC structural wall-to-wall and wall-to-foundation connections have been experimentally investigated and design methods proposed [94]–[96]. Experimental verification of connections designed and detailed in accordance with existing specifications and subjected to explosive loads is critical to enabling complete design of protective SC structures.
- *Investigation of performance using other fill materials.* For expedient construction, consideration of materials other than concrete as infill should be investigated. The concrete provides mass which helps provide dynamic resistance but the strength of concrete was shown to have limited influence on blast resistance of SC walls. Materials such as sand or compacted earth may be suitable, particularly for expedient protection needs.

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APPENDICES

A. SPECIMEN DESIGN SAMPLE CALCULATIONS

Design of SC Wall Section for Blast Resistance (One-Way Action) - Specimen 5-2-50-5

Jakob C. Bruhl, Purdue University

Fields in yellow: user input. All others are calculated. Important results are in green.

Design is accomplished by combining guidance from AISC N690s1 Appendix N9 (Draft) with provisions of UFC 3-340-02 as described for reinforced concrete. Performance limits are recommended to be taken from ASCE 59-11. DIFs are also taken from ASCE 59-11.

Assumptions

- Simply supported beam
- Uniformly distributed load (equal pressure across the face of the panel)
- Uniformly distributed mass

Panel DimensionsLength (in): $L_w := 52$ Width (in): $l_w := 11$ Total thickness (in): $t_{sc} := 4$ Plate thickness (in): $t_p := 0.1046$ Total area of steel (in²): $A_s := t_p \cdot l = 1.151$ Concrete thickness (in): $t_c := t_{sc} - t_p = 3.895$ Reinforcement ratio: $\rho := \frac{2t_p}{t_{sc}} = 0.052$ Tie bar diameter, spacing (in): $d_t := 0.164$ $s_t := 2$ Stud diameter, spacing (in): $d_s := 0.25$ $s_s := 2$ **Design Blast Load (triangular)**Peak reflected pressure (psi): $p_r := 60$ Reflected impulse (psi*sec): $i_r := 1.100$ Compute duration (sec): $t_d := 2 \cdot \frac{i_r}{p_r} = 0.037$
 $s_{tt} := s_t$ $s_{tl} := s_t$ $s_{st} := s_s$ $s_{sl} := s_s$ Assume studs and ties spacing is equal in orthogonal directions.

 $n_t := \text{ceil}\left(\frac{L}{s_t}\right) \cdot \text{ceil}\left(\frac{1}{s_t}\right) = 156$ Total number of ties along span length

 $n_s := \text{ceil}\left(\frac{L}{s_s}\right) \cdot \text{ceil}\left(\frac{1}{s_s}\right) - n_t = 0$ Total number of studs along span length

 $n_{ties} := \text{ceil}\left(\frac{1}{s_t}\right) = 6$ Number of ties in a row

 $n_{studs1} := \text{ceil}\left(\frac{1}{s_s}\right) = 6$ Number of studs in a row without ties

 $n_{studs2} := \text{ceil}\left(\frac{1}{s_s}\right) - n_{ties} = 0$ Number of studs in a row with ties

Material Properties

Concrete compressive strength (psi):	$f_c := 5000$		
Concrete modulus of elasticity (psi):	$E_c := 57000 \cdot \sqrt{f_c} = 4.031 \times 10^6$		
Concrete mass density (lb*sec ² /in ⁴):	$\gamma_c := 2.36 \cdot 10^{-4}$		
Steel yield, ultimate strength (psi):	$f_y := 58000$ $f_u := 67000$	Ties $f_{ny} := 51000$ $f_{nt} := 65000$	
Young's modulus of steel (psi):	$E_s := 29000000$	Studs $f_{ys} := 61600$ $F_u := 76100$	
Steel mass density (lb*sec ² /in ⁴):	$\gamma_s := 7.33 \cdot 10^{-4}$		
Modular ratio:	$n := \frac{E_s}{E_c} = 7.195$		

Strength Increase Factors

	Concrete	Steel	
Dynamic increase factor (DIF) in flexure:	$DIF_{cf} := 1.25$	$DIF_{sf} := 1.29$	Ref: N690 Table A-N9.1.1
DIF in shear:	$DIF_{cs} := 1.10$	$DIF_{ss} := 1.10$	
Static increase factor (SIF):	$SIF_c := 1.1$	$SIF_s := 1.0$	Ref: ASCE 59-11 Table 3-4 (1.0 if input measured properties)

Check inputs are within N690 limits

Reinforcement ratio: (N9.1.1(c))	$reinf := \begin{cases} \text{"OK"} & \text{if } 0.015 \leq \rho \leq 0.05 \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	$\rho = 0.052$
Plate thickness: (N9.1.1(b))	$thick := \begin{cases} \text{"OK"} & \text{if } 0.25 \leq t_p \leq 1.5 \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	Scaled specimens
Steel plate strength: (N9.1.1(d))	$plate := \begin{cases} \text{"OK"} & \text{if } 50000 \leq f_y \leq 65000 \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	
Concrete strength: (N9.1.1(e))	$concrete := \begin{cases} \text{"OK"} & \text{if } f_c \geq 4000 \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	
Rupture strength: (N9.1.1(i))	$A_{sn} := A_s - \text{ceil}\left(\frac{1}{s_{tt}}\right) \cdot \left(d_t + \frac{1}{16}\right) \cdot t_p = 1.008$ $net := \begin{cases} \text{"OK"} & \text{if } f_u \cdot A_{sn} > f_y \cdot A_s \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	
Stud spacing: (N9.1.3, Eq'n (A-N9-2))	$spacing_{stud} := \begin{cases} \text{"OK"} & \text{if } \frac{s_{st}}{t_p} \leq 1.0 \cdot \sqrt{\frac{E_s}{f_y}} \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	
Tie bar spacing: (N9.1.5)	$spacing_{tie} := \begin{cases} \text{"OK"} & \text{if } s_{tt} \leq t_{sc} \\ \text{"NoGo"} & \text{otherwise} \end{cases}$	

Tie bar material strength:
(Eq'n (A-N9-5))

$$F_{req} := \begin{cases} \text{"OK"} & \text{if } f_{nt} \geq \left(\frac{t_p \cdot f_y \cdot t_{sc}}{4} \right) \cdot \left(\frac{s_{tt}}{s_{tl}} \right) \cdot \left[\frac{6}{18 \cdot \left(\frac{t_{sc}}{s_{tl}} \right)^2 + 1} \right] \\ \text{"NoGo"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Compute Static Flexural Capacity:

Nominal moment capacity (lb*in): $j := 0.9$ $M_n := f_y \cdot A_s \cdot j \cdot t_{sc} = 2.402 \times 10^5$ N9.3.3, Eq'n (A-N9-18)

Compute Static Shear Capacity:

Concrete contribution to shear strength (lb): $V_{conc} := 0.05 \cdot \sqrt{\frac{f_c}{1000}} \cdot t_c \cdot l \cdot 1000 = 4.791 \times 10^3$

Steel (tie bars) contribution to shear strength (lb): N9.3.4, Eq'n (A-N9-21)

$$\xi := \begin{cases} 1.0 & \text{if } f_{ny} \leq 0.8 \cdot f_{nt} \\ 0.5 & \text{otherwise} \end{cases} = 1$$

$$p_s := \frac{t_c}{s_{tl}} = 1.948$$

$$F_t := \frac{\pi}{4} \cdot d_t^2 \cdot f_{ny} = 1.077 \times 10^3$$

$$V_s := \min \left(\xi \cdot p_s \cdot F_t \cdot \frac{l}{s_{tt}}, 0.25 \cdot \sqrt{\frac{f_c}{1000}} \cdot t_c \cdot l \cdot 1000 \right) = 1.154 \times 10^4 \text{ Eq'n (A-N9-22)}$$

Total nominal shear strength (lb):

$$V_n := \begin{cases} (V_{conc} + V_s) & \text{if } s_{tt} \leq \frac{t_{sc}}{2} \\ \max(V_{conc}, V_s) & \text{otherwise} \end{cases} = 1.633 \times 10^4 \text{ N9.3.4(a), Eq'n (A-N9-20)}$$

Flexural to Shear Capacity Ratio

$$\text{Ratio} := \frac{M_n}{V_n \cdot t_{sc}} = 3.678 \text{ Ratio in terms of total section depth}$$

This ends the static strength assessment. Contained in the following pages is a modified version of the dynamic strength design methods for reinforced concrete subject to blast loads (UFC 3-340-02, Chapter 4) modified to account for the mechanics of steel-plate reinforced concrete walls.

Strength and stiffness calculations are completed IAW AISC N690s1 Appendix N9 as noted. These equations are modified to account for dynamic strength increases.
SDOF calculations are completed IAW UFC 3-340-2.

Dynamic Strength:

Dynamic compressive strength of concrete (psi): $f_{cddyn} := f_c \cdot DIF_{cf}$

$$f_{cddyn} = 6.25 \times 10^3$$

Dynamic yield strength of steel (psi): $f_{yddyn} := f_y \cdot SIF_s \cdot DIF_{sf}$

$$f_{yddyn} = 7.482 \times 10^4$$

Compute depth to neutral axis at yield moment:

$$c_c := 2 \cdot \left(\frac{A_s \cdot f_{yddyn}}{12 \cdot f_c} - n \cdot t_p \right) = 1.364 \quad \text{in} \quad \text{Eq'n (C-A-N9-11)}$$

Compute moment capacity and ultimate resistance (N9.3.3):

$$j_w := 0.9$$

$$M_{nALL} := f_{yddyn} \cdot A_s \cdot j \cdot t_{sc} = 3.099 \times 10^5 \quad \text{lb-in} \quad \text{Eq'n (A-N9-18) modified to include DIF}$$

$$M_{nALT} := A_s \cdot f_{yddyn} \cdot (t_{sc} - t_p) - 6 \cdot f_c \cdot c_c \cdot \left(\frac{c_c}{3} + \frac{t_p}{2} \right) = 3.146 \times 10^5 \quad \text{lb-in} \quad \text{Eq'n (C-A-N9-10)}$$

$$R_m := 8 \cdot \frac{M_n}{L} = 4.768 \times 10^4 \quad \text{lb} \quad \text{Table 3-1, UFC 3-340-02 (one-way, simple support, uniform load)}$$

Compute effective flexural stiffness (N9.2.2):

$$I_s := \frac{1 \cdot t_p \cdot (t_{sc} - t_p)^2}{2} = 8.73 \quad I_c := \frac{1 \cdot t_c^3}{12} = 54.184$$

$$\rho_n := \rho \cdot n = 0.376 \quad c_2 := 0.48 \cdot \rho_n + 0.10 = 0.281$$

$$EI_{eff} := E_s \cdot I_s + c_2 \cdot E_c \cdot I_c = 3.144 \times 10^8 \quad \text{Eq'n (A-N9-8)}$$

Compute stiffness and elastic deflection:

$$\text{Stiffness (lb/in): } k_e := \frac{384 \cdot EI_{eff}}{5 \cdot L^3} = 1.718 \times 10^5$$

$$\text{Elastic deflection (in): } X_e := \frac{R_m}{k_e} = 0.278$$

Compute natural period of the panel:

$$K_{LMe} := 0.78 \quad K_{LMp} := 0.66$$

Transformation factors from
Table 3-12 (UFC 3-340-02)
(one-way, simple supports,
uniform load)

$$\text{Load mass factor: } K_{LM} := \frac{K_{LMe} + K_{LMp}}{2} = 0.72$$

$$\text{Mass (lb}\cdot\text{sec}^2/\text{in): } M := [(t_p \cdot l) \cdot \gamma_s + [(t_{sc} - t_p) \cdot l] \cdot \gamma_c] \cdot L = 0.57$$

$$\text{Natural period (s): } T_n := 2\pi \cdot \sqrt{\frac{K_{LM} \cdot M}{k_e}} = 9.71 \times 10^{-3}$$

Compute maximum mid-span deflection of panel from blast load

Use MATLAB script "SDOF_SC_Uniform_Pressure" to compute time history of panel response.
Enter computed values below. Alternatively, the maximum mid-span displacement can be computed directly using a series of calculations and charts from UFC 3-340-02.

NOTE: Using the MATLAB script is preferred - avoid chart interpolation, solve the equation of motion, account for material strain hardening, etc.

Response limits from ASCE 59-11 (see Tables 3-2, 3-3)

$$\theta_{\text{limit}} := 4\text{deg}$$

$$\mu_{\text{limit}} := 2 \quad \text{NOTE: N690 allows ductility of 10 for impulsive loads (N9.1.6b)}$$

$$X_m := 0.31$$

$$\text{SDOF}_{\text{ok}} := \begin{cases} \text{"OK"} & \text{if } \frac{t_d}{T_n} \geq 0.3 \\ \text{"Use impulse method"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$\theta := \text{atan}\left[\frac{X_m}{\left(\frac{L}{2}\right)}\right] = 0.683 \cdot \text{deg} \quad \mu := \frac{X_m}{X_e} = 1.117$$

$$\text{Rotation} := \begin{cases} \text{"Meets rotation criteria"} & \text{if } \theta < \theta_{\text{limit}} \\ \text{"Does not meet rotation criteria"} & \text{otherwise} \end{cases} = \text{"Meets rotation criteria"}$$

$$\text{Ductility} := \begin{cases} \text{"Meets ductility criteria"} & \text{if } \mu < \mu_{\text{limit}} \\ \text{"Does not meet ductility criteria"} & \text{otherwise} \end{cases} = \text{"Meets ductility criteria"}$$

Check shear capacity of the section

Direct shear check

$$\text{Shear at support (lb): } V_s := \frac{R_m}{2} = \blacksquare$$

$$\text{Ultimate direct shear strength (lb): } V_d := 0.16 \cdot (f_c \cdot \text{DIF}_{\text{cs}}) \cdot l \cdot t_c = 3.771 \times 10^4$$

(UFC 3-340-02 Equation (4-30))

$$\text{Shear}_{\text{direct}} := \begin{cases} \text{"OK"} & \text{if } V_d > V_s \\ \text{"NoGo"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Diagonal tension check

$$\text{Ultimate shear (lb): } V_u := \left(\frac{L}{2} - \frac{t_{sc}}{2}\right) \cdot \frac{R_m}{L} = 2.201 \times 10^4$$

NOTE: Modified equation for V_u to find shear at distance $T/2$

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Ultimate shear stress (psi):

$$v_u := \frac{V_u}{l \cdot t_c} = 513.564$$

from support rather
than T

$$\text{Shear}_{\text{ult}} := \begin{cases} \text{"OK"} & \text{if } v_u \leq 10 \cdot \sqrt{f_c \cdot \text{DIF}_{\text{cs}}} = \text{"OK"} \\ \text{"NoGo"} & \text{otherwise} \end{cases}$$

Concrete shear strength (psi): $v_{c1} := (1.9 \cdot \sqrt{f_c \cdot \text{DIF}_{\text{cs}}}) + 2500 \cdot \frac{\rho}{2}$ or $v_{c2} := 3.5 \cdot \sqrt{f_c \cdot \text{DIF}_{\text{cs}}}$
(UFC 3-340-02 Eq'ns (4-23a) and (4-23b))

$$v_c := \begin{cases} v_{c1} & \text{if } v_{c1} \leq v_{c2} = 206.283 \\ v_{c2} & \text{otherwise} \end{cases}$$

Required shear reinforcement (in²): $A_v := \frac{(v_u - v_c) \cdot l \cdot s_t}{0.85 \cdot (f_{nt} \cdot \text{DIF}_{\text{sf}})} = 0.095$
(UFC-3-340-02 Equation (4-26))

Total Tie bar area in one row (in²): $A_t := n_{\text{ties}} \cdot \left(\frac{\pi}{4}\right) \cdot d_t^2 = 0.127$

$$\text{Tie_bar} := \begin{cases} \text{"OK"} & \text{if } A_v \leq A_t = \text{"OK"} \\ \text{"NoGo"} & \text{otherwise} \end{cases}$$

Check shear capacity IAW N690s1 N9

Same as shear check completed for static properties above but includes dynamic strengths to augment the shear checks IAW UFC 3-340-02 above.

$$V_{\text{conc}} := 4 \cdot \sqrt{f_c \cdot \text{DIF}_{\text{cs}}} \cdot t_c \cdot l = 1.271 \times 10^4$$

$$\xi := \begin{cases} 1 & \text{if } f_{ny} \leq 0.8 f_{nt} = 1 \\ 0.5 & \text{otherwise} \end{cases} \quad \rho_s := \frac{t_c}{s_{tl}} = 1.948 \quad F_s := A_t \cdot f_{ny} = 6.464 \times 10^3$$

$$V_{\text{ties}} := \min \left(21.2 \cdot \sqrt{f_{\text{cddyn}}} \cdot t_c \cdot l, \xi \cdot \rho_s \cdot F_s \cdot \frac{1}{s_{tt}} \right) = 6.924 \times 10^4$$

$$V_{\text{no}} := \begin{cases} (V_{\text{conc}} + V_{\text{ties}}) & \text{if } s_{tt} \leq \frac{t_{\text{sc}}}{2} = 8.196 \times 10^4 \\ \max(V_{\text{conc}}, V_{\text{ties}}) & \text{otherwise} \end{cases}$$

$$\text{Shear}_{\text{N690}} := \begin{cases} \text{"OK"} & \text{if } 0.75 \cdot V_{\text{no}} \geq V_u = \text{"OK"} \\ \text{"NoGo"} & \text{otherwise} \end{cases}$$

Check interfacial shear

$$R_g := 1 \quad R_p := 0.75 \quad A_{\text{sa}} := \left(\frac{\pi}{4}\right) \cdot d_s^2 = 0.049$$

$$Q_n := \min \left(0.5 \cdot A_{\text{sa}} \cdot \sqrt{f_c \cdot \text{DIF}_{\text{cs}}} E_c, R_g \cdot R_p \cdot A_{\text{sa}} \cdot \text{DIF}_{\text{ss}} \cdot F_u \right) = 3.082 \times 10^3$$

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$$v_i := \frac{Q_n}{s_s} = 770.457$$

$$\text{Shear}_{\text{interfacial}} := \begin{cases} \text{"OK"} & \text{if } v_i \geq v_u \\ \text{"NoGo"} & \text{otherwise} \end{cases} = \text{"OK"}$$

This completes the dynamic assessment.

B. LS-DYNA KEYWORDS USED IN BENCHMARKED FE MODELS

Table B.1.: LS-DYNA Keywords Used in Benchmarked
FE Models

Keyword	Static	Blast	Remarks
*CONTROL_CONTACT	x	x	
*CONTROL_ENERGY	x	x	
*CONTROL_HOURLASS	x	x	IHQ 6; QH 0.01 for all models
*CONTROL_TERMINATION	x	x	Termination time varied based on load conditions
*CONTROL_TIMESTEP	x	x	Applied mass scaling for static models (negative input value for DT2MS); TSSFAC 0.70 for blast models (no mass scaling)
*DATABASE_BNDOUT	x	x	
*DATABASE_GLSTAT	x	x	
*DATABASE_MATSUM	x	x	
*DATABASE_RCFORC	x	x	
*DATABASE_SCFORC	x	x	
*DATABASE_BINARY_D3PLOT	x	x	
*DATABASE_EXTENT_BINARY	x	x	NEIHP 56 to write Winfrith his- tory variables; STRFLG 1 to write strain data; BEAMIP 3 to write data from beam elements
*BOUNDARY_PRESCRIBED_MOTION_RIGID	x		Applied constant velocity to load applicators in static model
*BOUNDARY_SPC_SET	x	x	Restrained two nodes to prevent lateral slip
*SET_NODE	x	x	Identified the two nodes restrained against lateral slip

Continued on next page

Keyword	Static	Blast	Remarks
*CONTACT_AUTOMATIC_SURFACE_TO_SURFACE	x	x	Defined between concrete in fill and steel plates; between plates and supports; between plates and load applicator
*ELEMENT_SOLID	x	x	
*ELEMENT_BEAM	x	x	
*CONSTRAINED_LAGRANGE_IN_SOLID	x	x	Embedded tie bars and stud anchors in concrete; assumes perfect bond
*HOURGLASS	x	x	IHQ 6; QH 0.01 for all models
*MAT_RIGID	x	x	Used for supports and load applicators (various restraints)
*MAT_PIECEWISE_LINEAR_PLASTICITY	x	x	Input measured stress-strain curves for steel materials
*MAT_WINFRITH_CONCRETE	x	x	Non-strain rate model used for all cases
*MAT_ELASTIC_SPRING_DISCRETE_BEAM	x	x	Used for zero-length connector elements
*NODE	x	x	
*PART	x	x	
*SET_SEGMENT		x	Identify surface loaded by blast pressure
*SECTION_SOLID	x	x	ELFORM 1 for concrete and rigid parts; ELFORM -1 for steel plates
*SECTION_BEAM	x	x	ELFORM 1 for studs and tie bars; ELFORM 6 for connector elements

Continued on next page

Keyword	Static	Blast	Remarks
*DEFINE_CURVE	x	x	Constant velocity for static tests; steel stress-strain curves for all steel parts; load-slip for connectors; pressure curves for blast models; steel DIF for blast models
*LOAD_SEGMENT_SET		x	Applied blast pressure curve to loaded surface for blast models
*END	x	x	

C. BLAST RESISTANCE ASSESSMENT OF SC WALL: EXAMPLE

Blast Resistance Assessment of SC Wall

Last updated: 18 FEB 14

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Procedures Explained in Chapter 7 of Dissertation

Enter values for parameters highlighted in yellow and the remaining calculations assess the wall for resistance to the defined blast load. The figure at the end of the calculation shows the P-I coordinate for the case with contour lines of response limits defined for RC walls subject to compression by ASCE 59-11 and for SC walls by AISC N690s1 N9.1.6b.

Define Blast Load Parameters (7.1.1)

Peak Pressure: $p_r := 230$ psi NOTES: 5000-lb TNT at 75-ft

Peak Impulse: $i_r := 820$ psi*msec

Quantify SC Wall Resistance Parameters (7.1.2)

Input existing wall design parameters:

Section Width: $b := 12$ inches (unit width)

Unsupported Length: $L_w := 144$ inches

Total section depth: $t_{sc} := 12$ inches

Faceplate Thickness: $t_p := 0.25$ inches

Concrete Compressive Strength: $f_c := 5$ ksi

Faceplate Yield Strength: $F_y := 50$ ksi

Calculated values:

Concrete Thickness: $t_c := t_{sc} - 2t_p = 11.5$ inches

Reinforcement Ratio: $\rho := \frac{2 \cdot t_p}{t_{sc}} = 0.042$

Concrete Elastic Modulus: $E_c := 57 \sqrt{1000 \cdot f_c \cdot \text{DIF}_{fcM}} = 4.506 \times 10^3$ ksi

Steel Moment of Inertia: $I_s := \frac{b \cdot t_p \cdot (t_{sc} - t_p)^2}{2} = 207.094$ inches⁴

Concrete Moment of Inertia: $I_c := \frac{b \cdot t_c^3}{12} = 1.521 \times 10^4$ inches⁴

Calibration Constant for Eleff: $c_2 := 0.48 \cdot \frac{2t_p}{t_{sc}} \cdot \frac{E_s}{E_c} + 0.1 = 0.229$

Effective Flexural Stiffness: $El_{eff} := E_s \cdot I_s + c_2 \cdot E_c \cdot I_c = 7.573 \times 10^6$ kips-inch²

Effective Mass: $M := K_{LME} \cdot \frac{[L \cdot b \cdot (t_c \cdot \gamma_c + 2 \cdot t_p \cdot \gamma_s)]}{12^3 \cdot g} = 3.981$

$$\text{Stiffness Reduction Constant: } r_1 := 1.05 - 0.05 \cdot (\rho \cdot 100) = 0.842$$

$$\text{Yield Moment: } M_y := 0.9 \cdot F_y \cdot DIF_{F_y} \cdot t_p \cdot b \cdot t_{sc} = 2.09 \times 10^6 \text{ kip} \cdot \text{inch}$$

Resistance Parameters:

$$\text{Yield Resistance: } R_y := \frac{8 \cdot M_y}{L} = 116.1 \text{ kips}$$

$$\text{Resistance Function Initial Stiffness: } k_1 := r_1 \cdot \frac{384 \cdot EI_{eff}}{5 \cdot L^3} = 163.943 \frac{\text{kips}}{\text{inch}}$$

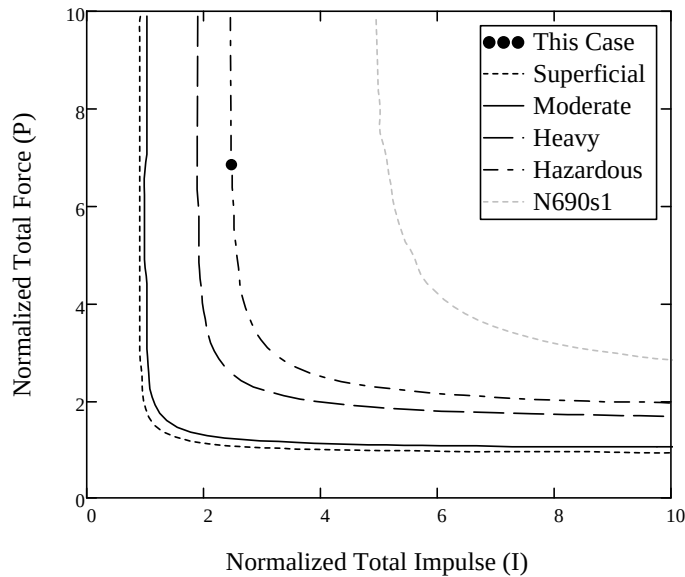
$$\text{Natural Frequency: } \omega_n := \sqrt{\frac{k_1 \cdot 1000}{M}} = 202.936 \text{ sec}^{-1}$$

Check Against Response Limit (7.1.3)

$$\text{Normalized Total Force: } P := \frac{2 \cdot p_r \cdot b \cdot L}{R_y \cdot 1000} = 6.847$$

$$\text{Normalized Total Impulse: } I := \frac{\frac{i_r}{1000} \cdot b \cdot L}{\left(\frac{R_y \cdot 1000}{\omega_n} \right)} = 2.477$$

The figure below shows the P-I coordinate for the current case with damage contours for defined response limits by ASCE 59-11 for RC walls subject to compression (Superficial ($\mu=0.9$), Moderate ($\mu=1$), Heavy ($\mu=2$), and Hazardous ($\mu=3$)) and the response limit defined by AISC N690s1 N9.1.6b for flexure-controlled SC walls in safety related nuclear facilities.



D. BLAST RESISTANT DESIGN OF SC WALL: EXAMPLE

Blast Resistant Design of SC Wall

Last updated: 18 FEB 14

Jakob C. Bruhl

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Procedures Explained in Chapter 7 of Dissertation

Enter values for parameters highlighted in yellow and the remaining calculations assess the wall for resistance to the defined blast load. The figure at the end of the calculation shows the P-I coordinate for the current design parameters with contour lines of response limits defined for RC walls subject to compression by ASCE 59-11 and for SC walls by AISC N690s1 N9.1.6b.

Define Blast Load Parameters (7.2.1)

Peak Pressure: $p_r := 228$ psi NOTES: 2500-lb TNT at 60-ft

Peak Impulse: $i_r := 650$ psi*msec

Estimate Required Wall Thickness and Complete Preliminary SC Wall Design (7.1.2, 7.1.3)

Input existing wall design parameters:

Section Width: $b := 12$ inches (unit width)

Unsupported Length: $L_w := 240$ inches

Total section depth: $t_{sc} := 30$ inches

Faceplate Thickness: $t_p := 0.375$ inches

Concrete Compressive Strength: $f_c := 5$ ksi

Faceplate Yield Strength: $F_y := 60$ ksi

Calculated values:

Concrete Thickness: $t_c := t_{sc} - 2t_p = 29.25$ inches

Reinforcement Ratio: $\rho := \frac{2 \cdot t_p}{t_{sc}} = 0.025$

Concrete Elastic Modulus: $E_c := 57 \sqrt{1000 \cdot f_c \cdot \text{DIF}_{fcM}} = 4506$ ksi

Steel Moment of Inertia: $I_s := \frac{b \cdot t_p \cdot (t_{sc} - t_p)^2}{2} = 1975$ inches⁴

Concrete Moment of Inertia: $I_c := \frac{b \cdot t_c^3}{12} = 25025$ inches⁴

Calibration Constant for Eleff: $c_2 := 0.48 \cdot \frac{2t_p}{t_{sc}} \cdot \frac{E_s}{E_c} + 0.1 = 0.177$

Effective Flexural Stiffness: $El_{eff} := E_s \cdot I_s + c_2 \cdot E_c \cdot I_c = 77251793$ kips-inch²

Effective Mass: $M := K_{LME} \cdot \frac{[L \cdot b \cdot (t_c \cdot \gamma_c + 2 \cdot t_p \cdot \gamma_s)]}{12^3 \cdot g} = 16.01$

$$\text{Stiffness Reduction Constant: } r_1 := 1.05 - 0.05 \cdot (\rho \cdot 100) = 0.925$$

$$\text{Yield Moment: } M_y := 0.9 \cdot F_y \cdot DIF_{F_y} \cdot t_p \cdot b \cdot t_{sc} = 9404 \quad \text{kip} \cdot \text{inch}$$

Quantify Resistance of the SC Wall Design (7.2.4)

Resistance Parameters:

$$\text{Yield Resistance: } R_y := \frac{8 \cdot M_y}{L} = 313 \quad \text{kips}$$

$$\text{Resistance Function Initial Stiffness: } k_1 := r_1 \cdot \frac{384 \cdot EI_{eff}}{5 \cdot L^3} = 397 \quad \frac{\text{kips}}{\text{inch}}$$

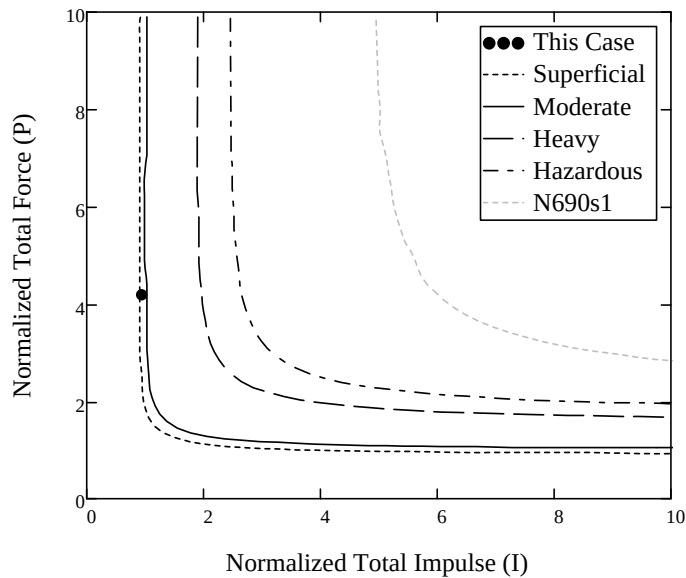
$$\text{Natural Frequency: } \omega_n := \sqrt{\frac{k_1 \cdot 1000}{M}} = 157 \quad \text{sec}^{-1}$$

Check Against Response Limit (7.2.5)

$$\text{Normalized Total Force: } P := \frac{2 \cdot p_r \cdot b \cdot L}{R_y \cdot 1000} = 4.19$$

$$\text{Normalized Total Impulse: } I := \frac{\frac{i_r}{1000} \cdot b \cdot L}{\left(\frac{R_y \cdot 1000}{\omega_n} \right)} = 0.94$$

The figure below shows the P-I coordinate for the current case with damage contours for defined response limits by ASCE 59-11 for RC walls subject to compression (Superficial ($\mu=0.9$), Moderate ($\mu=1$), Heavy ($\mu=2$), and Hazardous Damage ($\mu=3$)) and the response limit defined by AISI N690s1 N9.1.6b for flexure-controlled SC walls in safety related nuclear facilities.



Refine Design (7.2.6)

Make changes in section (7.1.2, 7.1.3) above in the following priority order: t_{sc} , ρ (t_p), F_y
 Changes to f'_c have negligible influence on blast resistance

Complete Design of Remaining SC Wall Components (7.2.7)Design Tie Bars (7.2.7.1)

Tie Bar Yield Strength: $F_{yTIE} := 60$ ksi

Tie Bar Ultimate Strength: $F_{uTIE} := 90$ ksi

Yielding or Non-Yielding Ties: $\xi := \begin{cases} 1.0 & \text{if } F_{yTIE} \leq 0.8F_{uTIE} \\ 0.5 & \text{otherwise} \end{cases} = 1$

Maximum Tie Bar Spacing: $S_{MAX} := \frac{t_{sc}}{2} = 15$ inches

Tie Bar Spacing: $S := 10$ inches

Minimum Tie Bar Area: $A_{tieMIN} := \frac{\frac{1}{\phi_{vo}} \cdot \left(\frac{L - \frac{t_{sc}}{2}}{L} \right) \cdot \left(\frac{R_y}{2} \right) - (0.05 \sqrt{DIF_{fcV} \cdot f'_c}) \cdot t_c \cdot b}{\xi \cdot \left(\frac{b \cdot t_c}{S^2} \right) \cdot DIF_{Fy} \cdot F_{yTIE}} = 0.57$

Minimum Tie Bar Diameter: $d_{tieMIN} := \sqrt{\frac{4 \cdot A_{tieMIN}}{\pi}} = 0.85$ inches

Tie Bar Diameter: $d_{tie} := 0.875$ inches (#7 bars) Specify to meet d_{tieMIN}

Steel Shear Strength Contribution: $V_s := \xi \cdot \frac{t_c \cdot b}{S^2} \cdot \frac{\pi \cdot d_{tie}^2}{4} \cdot DIF_{Fy} \cdot F_{yTIE} = 163$ kips

$V_{sLIMIT} := 0.25 \cdot \sqrt{DIF_{fcV} \cdot f'_c} \cdot t_c \cdot b = 206$ kips

LIMIT := $\begin{cases} \text{"OK"} & \text{if } V_s \leq V_{sLIMIT} \\ \text{"Reduce Vs"} & \text{otherwise} \end{cases} = \text{"OK"}$

Confirm Shear Capacity Exceeds Maximum Shear Demand

Maximum Shear Demand: $V_n := \left(\frac{L - \frac{t_{sc}}{2}}{L} \right) \cdot \left(\frac{R_y}{2} \right) = 146.939$ kips

Concrete Shear Strength Contribution: $V_{conc} := 0.05 \cdot \sqrt{DIF_{fcV} \cdot f'_c} \cdot t_c \cdot b = 41.158$ kips

Shear Check: $\text{Shear} := \begin{cases} \text{"OK"} & \text{if } \phi_{vo} \cdot (V_{conc} + V_s) \geq V_n \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Design Headed Stud Anchors (7.2.7.2)

Stud Yield Strength: $F_{ySTUD} := 60 \text{ ksi}$

Stud Ultimate Strength: $F_{uSTUD} := 80 \text{ ksi}$

Yielding or Non-Yielding Studs: $c_1 := \begin{cases} 1 & \text{if } F_{ySTUD} \leq 0.80F_{uSTUD} \\ 0.7 & \text{otherwise} \end{cases} = 1$

Maximum Stud Spacing: $s_{MAX} := 1.0 \sqrt{\frac{E_s}{DIF_{F_y} \cdot F_y}} \cdot t_p = 7.26 \text{ inches}$

Stud Spacing: $s_w := 5 \text{ inches}$ Specify to meet s_{MAX} ; consider S as multiples of s

Minimum Stud Area: $A_{sa1} := \frac{F_{ySTUD} \cdot t_p}{\phi_v \cdot F_{uSTUD} \cdot 3 \cdot t_{sc}} \cdot \left(\frac{s}{c_1}\right)^2 = 0.12$

$$A_{sa2} := \frac{\phi_{vo} \cdot (V_{conc} + V_s)}{0.9 \cdot t_{sc}} \cdot \frac{1}{\phi_v \cdot F_{uSTUD} \cdot b} \cdot \left(\frac{s}{c_1}\right)^2 = 0.228$$

$$A_{saMIN} := \begin{cases} A_{sa1} & \text{if } A_{sa1} \geq A_{sa2} \\ A_{sa2} & \text{otherwise} \end{cases} = 0.228$$

Minimum Stud Diameter: $d_{saMIN} := \sqrt{\frac{4 \cdot A_{saMIN}}{\pi}} = 0.538 \text{ inches}$

Stud Diameter: $d_{sa} := 0.625 \text{ inches}$ (H4L 5/8") Specify to meet d_{scMIN}

Check Direct Shear Capacity (7.2.7.3)

Direct Shear Capacity: $V_d := 0.16 \cdot DIF_{fcV} \cdot f_c \cdot t_c \cdot b = 309 \text{ kips}$

Direct Shear Check: $\text{DirectShear} := \begin{cases} \text{"OK"} & \text{if } V_d \geq \frac{R_y}{2} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Flexure-to-Shear (7.2.7.4)

Ratio in terms of t_{sc} :

$$\text{Ratio} := \frac{\phi_b \cdot M_y}{\phi_{vo} \cdot (V_{conc} + V_s) \cdot t_{sc}} = 1.84 \text{ Smaller value suggests flexure-controlled}$$

Check Flexure-Controlled: $\text{Ratio2} := \frac{L \cdot (V_{conc} + V_s)}{4 \cdot M_y} = 1.3$

$$\text{Flexure} := \begin{cases} \text{"Flexure Controlled"} & \text{if } \text{Ratio2} \geq 1.25 \\ \text{"Shear Controlled"} & \text{otherwise} \end{cases}$$

$$\text{Flexure} = \text{"Flexure Controlled"}$$

Final Design Checks (7.2.7.5)

Minimum Tie Bar Strength:

$$F_{\text{req}} := \left(\frac{t_p \cdot F_y \cdot t_{\text{sc}}}{4} \right) \cdot \left[\frac{6}{18 \cdot \left(\frac{t_{\text{sc}}}{S} \right)^2 + 1} \right] = 6.2 \quad \text{kips}$$

$$F_{\text{tie}} := F_{y\text{TIE}} \cdot \left(\frac{\pi \cdot d_{\text{tie}}^2}{4} \right) = 36.1 \quad \text{kips}$$

$$\text{TieStrength} := \begin{cases} \text{"OK"} & \text{if } F_{\text{tie}} \geq F_{\text{req}} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Confirm N9.1.1 General Provisions

(a) Section Thickness:

$$\text{Thickness} := \begin{cases} \text{"OK"} & \text{if } 12 \leq t_{\text{sc}} \leq 60 \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(b) Faceplate Thickness:

$$\text{Faceplate} := \begin{cases} \text{"OK"} & \text{if } 0.25 \leq t_p \leq 1.50 \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(c) Reinforcement Ratio:

$$\text{Reinforcement} := \begin{cases} \text{"OK"} & \text{if } 0.015 \leq \rho \leq 0.050 \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(d) Faceplate Strength:

$$\text{PlateStrength} := \begin{cases} \text{"OK"} & \text{if } 50 \leq F_y \leq 65 \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(e) Minimum Concrete Strength:

$$\text{ConcreteStrength} := \begin{cases} \text{"OK"} & \text{if } 4 \leq f_c \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

(f) Steel Faceplates Nonslender:

$$\text{Slenderness} := \begin{cases} \text{"OK"} & \text{if } \left(\frac{s}{t_p} \right) \leq 1.0 \sqrt{\frac{E_s}{F_y}} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

VITA

VITA

Lieutenant Colonel (LTC) Jakob C. Bruhl, U.S. Army, was born in Cape Girardeau, Missouri, on August 9, 1974. In 1996 he earned a Bachelor of Science in Civil Engineering from Rose-Hulman Institute of Technology and was commissioned a Second Lieutenant of Engineers in the United States Army through the ROTC program. His first assignment was to Fort Leonard Wood, Missouri where he served in staff and leadership positions in the 2nd Battalion of the 47th Infantry Regiment and 5th Engineer Battalion. In 2001, LTC Bruhl earned a Master of Science in Engineering Management from the University of Missouri - Rolla. While serving as a company commander in the 46th Engineer Battalion at Fort Polk, Louisiana, in 2003, LTC Bruhl deployed to Kuwait and led his company into Iraq as part of Operation Iraqi Freedom. In 2006, he earned a Master of Science in Civil Engineering from the University of Illinois - Urbana/Champaign and served as an instructor and assistant professor in the Department of Civil and Mechanical Engineering at the U.S. Military Academy from 2006 to 2008. In 2009, LTC Bruhl was assigned to the 130th Engineer Brigade at Schofield Barracks, Hawaii, where he deployed to Mosul, Iraq as the Brigade Design Engineer. Upon redeployment, LTC Bruhl served as the Operations Officer for the 84th Engineer Battalion and then as the Officer in Charge of the 14th Engineer Detachment. In 2013, LTC Bruhl was selected to return to the U.S. Military Academy as an assistant professor and began pursuit of a Doctor of Philosophy in civil engineering at Purdue University. LTC Bruhl is a graduate of the Engineer Officer Basic Course, the Infantry Captains Career Course, and the Air Command and Staff College. He holds a Professional Engineer license in the state of Missouri. He is married to the former Lesley Dawn Hamilton of Spencer, Indiana, and they have three children: Elizabeth, Madalyn, and Andrew.

Summary of Blast Tests on Steel-Plate Reinforced Concrete Walls

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Abstract

Conventional reinforced concrete (RC) structures have historically been the preferred choice for blast and impact resistant structures. Recent research demonstrates advantages of steel-plate composite (SC) structures that make them an excellent candidate for efficient construction of protective structures. Understanding of the behavior, analysis, and design of SC walls for safety-related nuclear structures in the US and abroad has advanced in the past decade resulting in the publication of a new specification for their design. A body of knowledge also exists to design SC structures to resist the effects of projectile impacts. Research on the behavior of SC walls subjected to blast does not exist in the public domain – a loading which has been identified as a continuing concern for protective structure design. This paper summarizes results from blast tests of SC walls.

INTRODUCTION

Steel-plate composite (SC) walls consist of steel faceplates with a poured concrete core (see Figure 1). Steel headed stud anchors provide composite action between the concrete infill and steel faceplates. Tie bars provide stability during construction and in-service shear strength. SC walls have recently been used in nuclear power plant (NPP) projects and are being considered for the next generation of small modular reactors (SMR). A specification to design SC structures for NPP applications, AISC N690s1 Appendix N9 (AISC, 2014), contains guidance to design each component within an SC wall. For safety-related NPP facilities these walls may be required to resist the effects of impact or impulsive loads. While there are methods to design SC walls to resist the local effects of projectile impact (i.e. Bruhl et al. (2015)) there are no accepted methods to design blast resistant SC walls.

There is limited experimental data on the response of SC walls to blast loads that can be used to validate a design method. Heng et al. (1995) and Lan et al. (2005) described the results of blast tests on various configurations of steel-concrete composite panels. None of these configurations included headed stud anchors or tie bars, which are both important components of the SC structures discussed in this paper. They tested fully enclosed steel-concrete-steel (SCS) panels and thin profiled steel sheeting cast compositely with concrete, similar to that in a composite deck.

Liew & Wang (2011) reported results from a series of blast tests on two different types of steel-concrete composite panels: cellular stiffened panels and a unique configuration which used J-hooks in place of headed stud anchors and tie bars. Hulton & Gough (1999) and Hulton (1995) provided quantitative descriptions of blast tests of what eventually became the Bi-Steel® system. Of the different configurations previously tested, Bi-Steel® is most similar to the general SC system described in this paper but is proprietary and has some key differences from SC structures designed in accordance with AISC N690s1 Appendix N9.

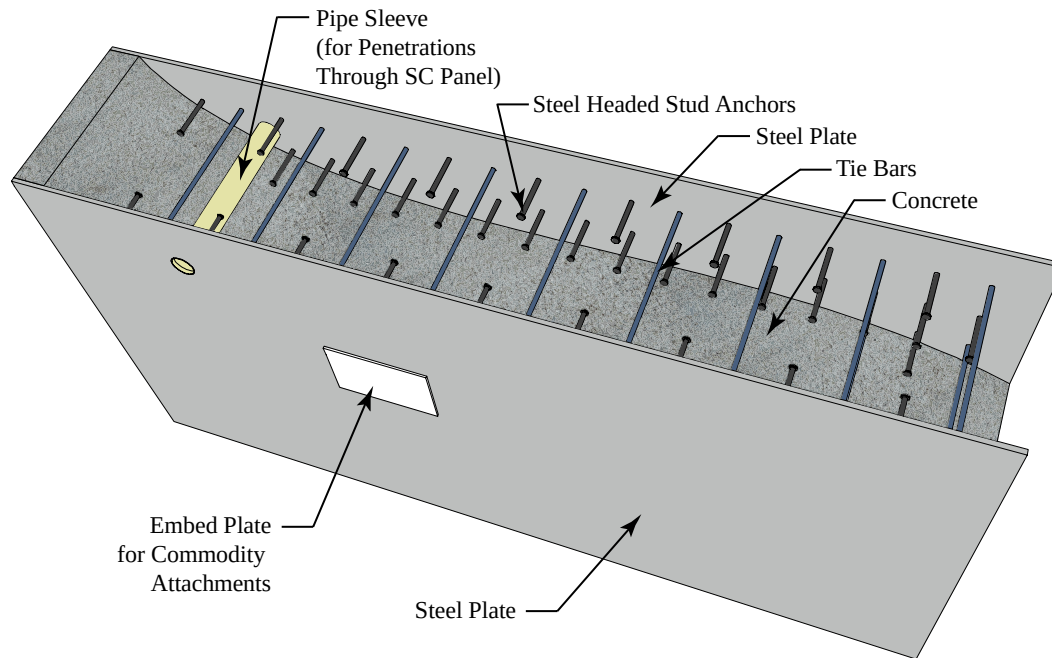


Figure 1. Typical SC Wall Section (from Johnson et al. (2014))

The purpose of this paper is to summarize: (1) the blast test program conducted on a variety of SC walls designed in accordance with AISC N690s1 Appendix N9, (2) results from a representative test specimen, and (3) comparisons of the blast response of SC walls and RC walls.

EXPERIMENTAL PROGRAM

Twelve one-way SC wall specimens were subjected to blast loads using the U.S. Army Corps of Engineers – Engineering Research and Development Center (USACE-ERDC) Geotechnical and Structures Laboratory (GSL) Blast Load Simulator (BLS) located in Vicksburg, MS. This facility can be tuned to create pressure waves, which replicate those generated by high explosives at various far-field stand-off distances. The components, operation, and limitations of the BLS are described by Johnson (2013).

Eight different SC wall designs were tested which varied steel plate thickness, steel plate strength, tie bar spacing, and tie bar diameter (see Table 1). Figure 2 provides design details and Figure 3 depicts a specimen prior to being filled with

concrete. The specimens were one-third scale with the following dimensions: 4-inches total thickness, t_{sc} ; 11-inches wide; and 64-in long. The support frame within the BLS was designed to provide simple supports with 6-inches of overhang; the unsupported length was 52-inches. Various grades and thickness of A1011 HSLA hot-rolled steel sheet were used. Concrete had day-of-test compressive strengths, f'_c , between 6200-psi and 6600-psi. Various diameters of mild steel threaded rod were used as tie bars, the size of which was determined using AISC N690 Appendix N9 and accounting for dynamic shear effects. To accommodate tie bars, holes were drilled in each steel sheet and hex nuts placed inside and outside of the steel sheet to secure tie bars prior to concrete placement. Composite action for all specimens was achieved using 0.25-inch diameter, 1-1/8-inches long, Nelson headed stud anchors spaced at 2-inches on center.

Table 1 provides details about the design of each specimen: plate (sheet) thickness, t_p ; flexural reinforcement ratio, ρ ; ratio of stud spacing, s , to plate thickness (the slenderness ratio); ratio of stud diameter, d_s , to plate thickness; grade of A1011 HSLA steel sheet; tie bar diameter, d_t , and corresponding shear reinforcement ratio, ρ_s ; and the ratio of tie bar spacing, S , to total specimen thickness, t_{sc} .

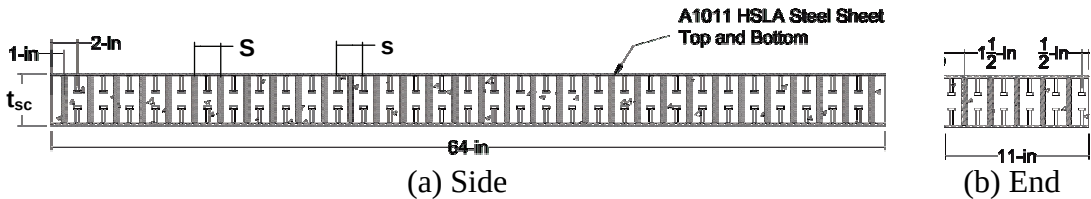


Figure 2. Blast Test Specimen Design

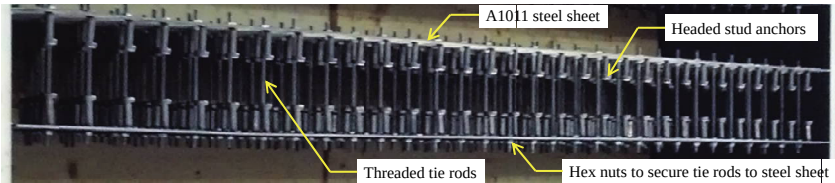


Figure 3. Photo of SC Specimen Prior to Infilling with Concrete

Table 1. Specimen Dimension and Material Details

Specimen	t_p (in)	ρ (%)	s/t_p	d_s/t_p	Sheet Grade	d_t (in)	ρ_s (%)	S/t_{sc}
3-2-50-5	0.079	4.0%	25.3	3.16	Gr50	0.1380	0.37%	0.50
3-2b-50-5	0.079	4.0%	25.3	3.16	Gr50	0.1640	0.53%	0.50
3-2-65-5	0.069	3.5%	29.0	3.62	Gr65	0.1380	0.37%	0.50
3-2b-65-5	0.069	3.5%	29.0	3.62	Gr65	0.1640	0.53%	0.50
5-4-50-5	0.112	5.6%	35.7	2.23	Gr50	0.3125	0.48%	1.50
5-2-50-5	0.112	5.6%	17.9	2.23	Gr50	0.1640	0.53%	0.50
5-2b-50-5	0.112	5.6%	17.9	2.23	Gr50	0.2500	1.23%	0.50
5-2-80-5	0.099	5.0%	20.2	2.53	Gr80	0.2500	1.23%	0.50

Prior to conducting dynamic tests in the BLS, static four-point bending tests of each specimen were completed. Figure 4 depicts the static test setup. A shear arm of 18-inches ($a/t_{sc} = 4.5$) was used for each test. Load was applied by a single actuator and spreader beam. Displacements were measured at the three quarter-points of the specimen using linear extensometers. Rotations were measured at the ends and directly under the load points using inclinometers. Strains on the tension face were measured at the quarter points and strain on the compression face was measured at mid-span.

Static results were used to benchmark numerical models which were then used to analyze the static behavior of each specimen loaded with increasing uniform pressure. Two numerical methods were used: (1) a fiber model to calculate moment-curvature relationships and moment-area method to generate load-displacement relationships, and (2) non-linear explicit finite element analysis using LS-DYNA. The result of these experiments and analyses was the static resistance function of each specimen for use in single-degree-of-freedom (SDOF) dynamic analysis to compare to experimental results.

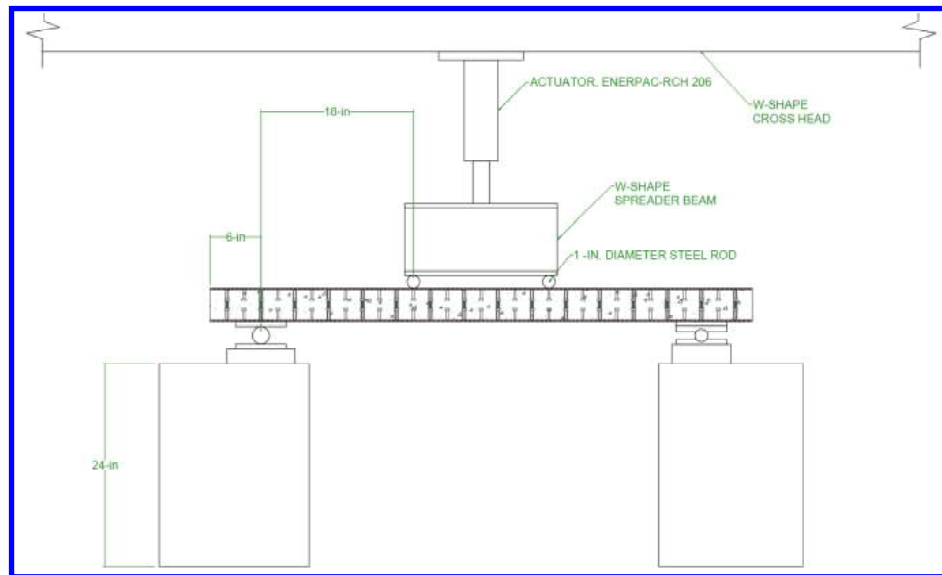
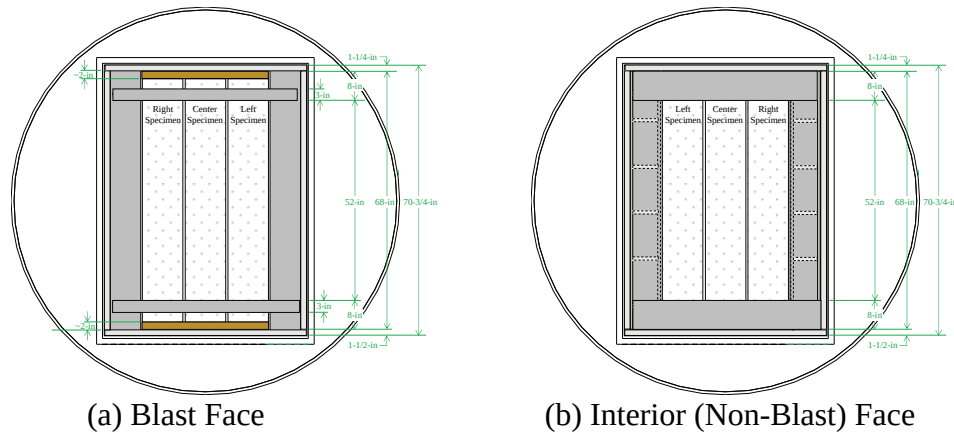


Figure 4. Static Four-Point Bending Test Setup

Three specimens were tested simultaneously in the BLS by loading them adjacent to one another with 0.25-inches between each specimen and 0.25-inches between the end specimens and the BLS frame, as shown in Figure 5. Displacements at mid-span were measured using linear potentiometers and were also calculated from accelerometer data as a check. Displacements 8.75-inches from mid-span were measured using linear extensometers. Strains on the tension (non-blast) face were measured at the quarter points and strain on the compression (blast) face was measured at mid-span. Pressure was measured by eight pressure transducers placed around the perimeter of the frame. Three high-speed video cameras captured the specimen response from different angles.



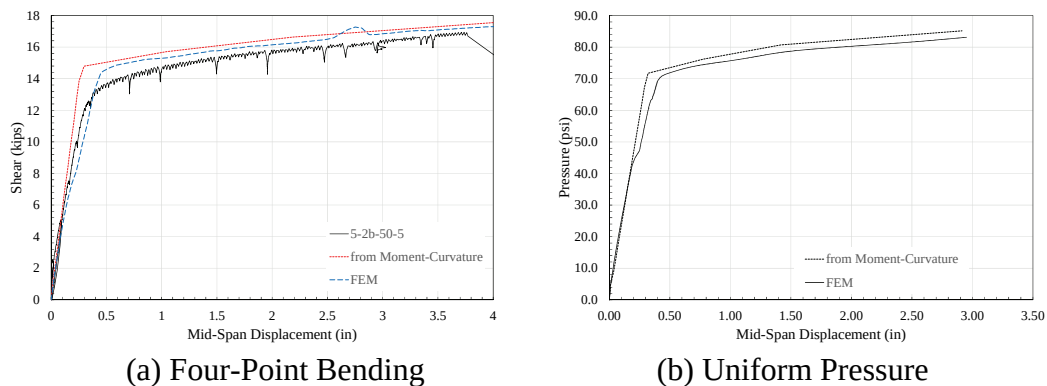
(a) Blast Face (b) Interior (Non-Blast) Face

Figure 5. Test Specimens in BLS Frame and Instrumentation Plan

Four tests were conducted with a total of six BLS shots. The first two tests consisted of two shots: a lower pressure-impulse intended to stay within the elastic range of response and a higher pressure-impulse combination intended to exceed the elastic capacity of the specimens. The second two tests consisted of only the higher pressure-impulse combination.

REPRESENTATIVE RESULTS

The test results of only one specimen are summarized for brevity. This specimen (5-2b-50-5) represents a typical SC wall design and was a one-third scale specimen of the minimum practical SC wall (12-in total thickness with 0.25-in steel plates). It was comprised of A1011 HSLA Grade 50 gage 12 hot-rolled steel sheets and 1/4-inch diameter tie bars at 2-inch spacing. The measured material properties are provided in Bruhl (2015). The flexural reinforcement ratio using measured steel sheet thickness was 5.6%. The holes drilled in the steel sheet to accommodate tie bars resulted in 12.8% reduction in section. Calculated moment capacity of the net section was 223-kip-inches. Static test of the specimen confirmed this moment capacity and yielded the static resistance functions shown in Figure 6.



(a) Four-Point Bending (b) Uniform Pressure

Figure 6. Static Resistance Functions (5-2b-50-5)

This specimen was tested dynamically twice: first with a lower pressure and then with a higher pressure. The average measured pressures and impulses were 18.9-psi and 260-psi·msec for the first shot and 61.6-psi and 1068-psi·msec for the second shot. Representative pressure-time history and calculated impulse for each shot are shown in Figure 7.

Transient displacement and strain measurements for both shots are shown in Figure 8 and Figure 9. When corrected for frame movement, the maximum displacement of the specimen from the first shot was 0.11-inches with 0.08-inches of residual displacement. Displacement measured by the potentiometer was close to that calculated from accelerometer data. The displacement measured by the linear extensometer was smaller than that from the other two sensors because it was not located at mid-span as the others were. While yield strain was not reached in the steel sheets, the strain gages measured small residual strains.

Maximum displacement from the second shot was 0.49-inches with 0.24-inches residual displacement (corrected to account for frame movement). The displacement measured by the potentiometer was larger than that calculated from accelerometer measurements. The tension steel sheet yielded during the second shot as evidenced by the change in slope of the strain history at approximately 2000- $\mu\epsilon$, the residual strain of approximately 1500- $\mu\epsilon$, and the residual displacement of the specimen.

After the second shot, the specimens were removed from the frame, cracks marked and photographs taken. This was not done after the first shot so as not to disturb the specimen before the second shot. As seen in Figure 10, flexural cracks were most prevalent near mid-span and there was limited shear cracking. Residual displacement is evident in this figure.

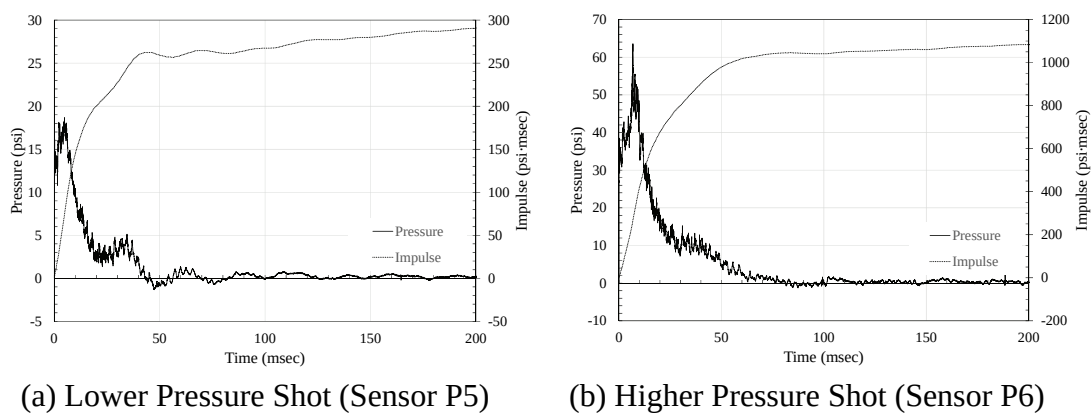


Figure 7. Measured Pressure-Time History and Calculated Impulse

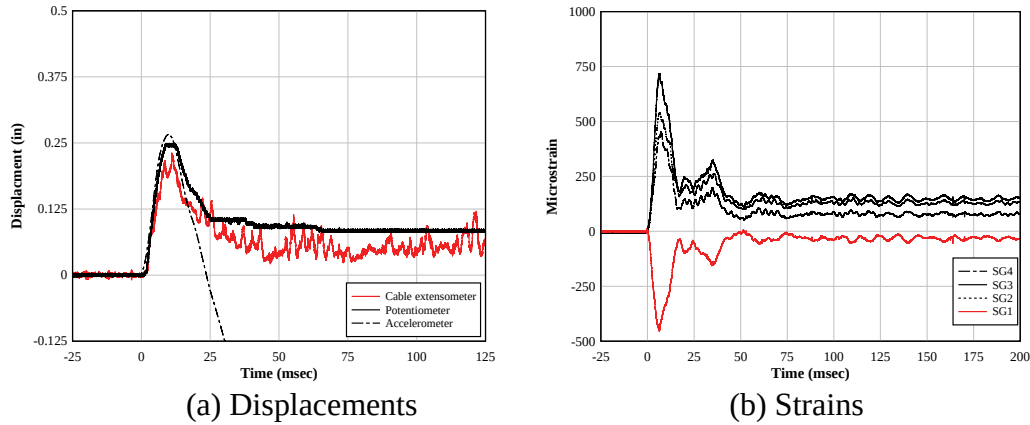


Figure 8. Dynamic Response (5-2b-50-5; $p_r = 18.9\text{-psi}$; $i_r = 260\text{-psi}\cdot\text{msec}$)

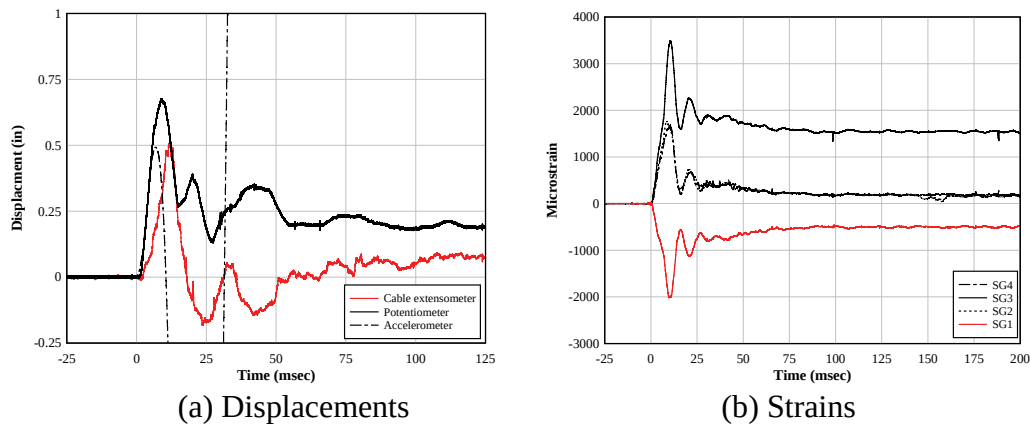


Figure 9. Dynamic Response (5-2b-50-5; $p_r = 61.6\text{-psi}$; $i_r = 1068\text{-psi}\cdot\text{msec}$)



Figure 10. Post-test Crack Map (5-2b-50-5)

COMPARISON OF SC AND RC PERFORMANCE

While flexural reinforcement ratios of the SC specimens included in this study were larger than typical RC reinforcement ratios, comparison of their performances is instructive. Thiagarajan, et al., (2014) reported experimental results from a program which investigated the influence of different reinforcement and concrete strengths. Their tests of 4-in thick RC slabs were conducted in the BLS and applied pressure waves nearly identical to those of the high pressure shot shown in Figure 7(b). Each panel in their study was doubly reinforced with 1.5% total flexural reinforcement. The panels included shrinkage reinforcement but did not have shear reinforcing steel. Two varieties of concrete were investigated: normal strength (NSC, 4000-psi) and high-strength (HSC, 15,500-psi). Two steel reinforcement strengths were investigated: conventional (NR, 50-ksi yield) and HSLA-V (VR, 83-ksi yield).

Maximum mid-span deflections were largest for NSC-NR panels (8.7-inches) and smallest for HSC-VR panels (4.8-inches) indicating that the use of higher strength materials improved the blast resistance of RC structures. Comparing the response of the NSC-NR panels to specimen 5-2b-50-5 suggests that typical SC walls deflect much less than typical RC walls subjected to the same blast pressure.

Coyle & Cormie (2009) suggested that the same level of protection could be provided by a Bi-Steel wall which was less than half the thickness of an RC wall. Their comparison was between a Bi-Steel wall with 8% and a RC wall with 1% total flexural reinforcement. A numerical investigation of SC and RC wall sections with the same reinforcement ratio was reported by Bruhl & Varma (2014). Their study included nine SC models and nine RC models consisting of 12-inch thick wall sections comprised of conventional construction materials (5000-psi concrete and Grade 50 steel). The flexural reinforcement ratio was approximately 5% for both the SC and RC section. Various lengths and pressure-impulse combinations were included in the study. Results indicated smaller deflections for SC structures compared to RC structures. The authors concluded that this was due, in part, to the fact that the steel reinforcement in an SC wall is at the extreme fiber while it cannot be for an RC wall due to clear cover requirements. Thus, for the same section depth, the moment capacity and flexural rigidity is inherently larger for an SC wall than an RC wall of the same thickness. It is important to note that RC walls have significant rebar congestion and concrete placement issues at reinforcement ratios greater than 1.5%. The advantages of SC walls become evident for high reinforcement ratios.

CONCLUSIONS

This paper provided an overview of a test program investigating the performance of one-way SC structures subjected to blast pressures and presented results from one specimen. Results from this study confirmed the influence of reinforcement ratio and steel strength on blast resistance. Results corroborated conclusions by others that SC structures provide improved blast resistance when compared to RC structures. Continued investigation of the blast response of SC structures to other than far-field explosions is necessary. Improved understanding of connection details and the influence of shear reinforcement are important.

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Title: Protection Using SC Composite Structures: Overview of Blast and Impact Resistance

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ABSTRACT

Steel-plate composite concrete (SC) structures consist of external steel-plates acting compositely with a concrete core. These steel plates provide reinforcement and serve as formwork decreasing construction time when compared to conventional reinforced concrete (RC). SC structures have recently been used for nuclear power plant (NPP) construction in the United States, Japan, and Korea. The structural behavior of SC structures has been extensively studied and design standards have recently been published as AISC N690 Appendix N9. This standard provides specific guidance for design basis loads and general guidance for impulsive and impact loads.

While the use of SC structures in the United States has been almost exclusively in NPPs, there are important benefits for engineers of other protection needs to be aware of. For example, it has been demonstrated that the same level of projectile protection can be achieved from an SC wall which is 30% thinner than an RC wall. Similar protective benefits are provided by SC walls against explosive loads. This paper includes an explanation of design philosophy and recommended design methods for impulsive and impact loads.

INTRODUCTION

Steel-plate composite concrete (SC) structures (see Figure 1) consist of external steel-plates which serve as flexural reinforcement and formwork. These steel plates act compositely with a poured concrete core through headed stud anchors welded to the steel plates. Tie bars provide shear reinforcement in service and stability of the steel plates during construction. Because the steel plates serve as formwork construction time is decreased when compared to that of conventional reinforced concrete (RC). SC structures have recently been used for nuclear power plant (NPP) construction in the United States, Japan, and Korea [1,2] and are being considered for the next generation of small modular reactors (SMR).

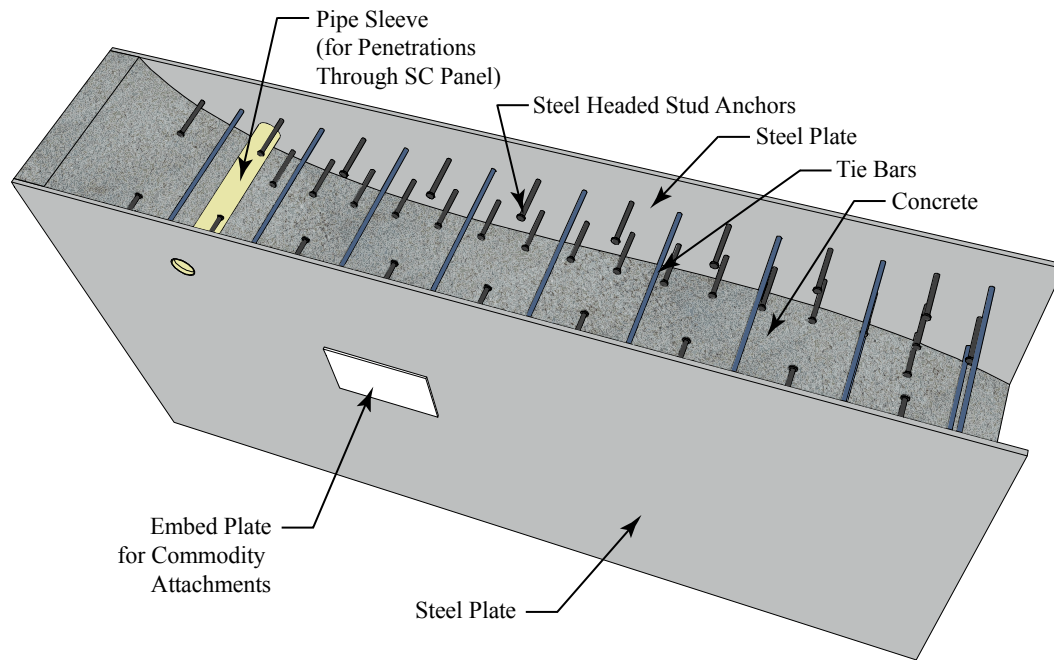


Figure 1. Typical SC Wall Section (from [3])

This structural system has been experimentally investigated in the U.S., Korea, and Japan. Much of this research, focused primarily on design basis loads, has been incorporated into AISC N690[4]. The specification provides general guidance to design SC structures for impact or impulsive loads. Recent work has begun to improve understanding of SC behavior under these extreme events [3,5–7].

SC structures are well suited for protective construction. Considering local effects from projectile impact, SC walls provide the same level of protection as an with 30% less thickness than an RC wall [8]. Bi-Steel®, a proprietary version of an SC wall, has been shown to provide the same level of protection against blast events as an RC wall which is twice as thick [9]. Considering SC and RC walls of the same thickness and reinforcement ratios, the mid-span displacement of an SC wall section was around 75% of the displacement of an equivalent RC wall section for a variety of lengths and blast loads [7].

This paper summarizes the design provisions provided in AISC N690 and explains experimentally validated methods to design SC structures to resist the effects of impact (such as tornado- or hurricane-generated projectiles, aircraft impact, or turbine failure fragments) and impulsive loads (such as from accidental explosions or terrorist threats).

SUMMARY OF AISC N690 APPENDIX N9

The behavior of SC walls in flexure, in-plane shear, and out-of-plane shear has been studied, clearly described, and developed into a design specification as Appendix N9 of AISC N690s1 (Draft) [4] (Supplement No. 1 to AISC N690-12[10], due to be published in 2015). The commentary of the supplement contains a flowchart to facilitate use of Appendix N9. The contents, organization, and utilization of the appendix are summarized in [11].

AISC N690s1 Section N9.1.6. provides general guidance for the design of SC structures to resist impact and impulse loads. Maximum values for dynamic increase factors (DIF) used in analysis of impact or impulsive loads are given as are deformation ductility limits for flexure- and shear-controlled SC walls and for those with axial compressive loads. In designing an SC wall to resist an impact load, criteria for both local effects and overall structural response must be satisfied and the steel faceplate thickness must be at least 1.25 times the calculated thickness. To calculate the response of an SC wall to impulsive loads, the appendix provides three general options:

- (1) Account for dynamic effects of the impulsive load by calculating a dynamic load factor (DLF),
- (2) Use impulse, momentum, or energy balance methods to calculate the dynamic effects of the impulsive loads, or
- (3) Perform a time-history dynamic analysis including mass and inertial properties and nonlinear resistance behavior of the structural members.

The commentary of AISC N690s1 Appendix N9 provides recommendations for analysis of impact loads on SC walls using single-degree-of-freedom (SDOF) or finite element (FE) analysis. The commentary also describes a rational method to calculate the steel faceplate thickness required to prevent perforation by projectile impact. The method they recommend is described in the following section of this paper. Provided in the commentary are a series of figures that show the required SC wall thickness required to prevent perforation by a variety of projectile diameters, weights, and initial velocities.

IMPACT RESISTANT DESIGN OF SC WALLS

There are two primary design concerns or an SC wall required to resist the effects of impact loads (e.g. tornado- or hurricane-generated missiles, turbine fragments, heavy load drops, or aircraft impact): (1) ensure the wall is not perforated by the impact and (2) confirm that the wall remains stable after impact by ensuring a ductility or end rotation limit is not exceeded. While related, these two design concerns may be considered in two separate analyses.

Design SC Wall to Prevent Perforation

Bruhl et al. [5] provided a three-step rational method to design an individual SC wall against local failure due to a specific missile. The local failure mechanism for missile impact on SC walls is shown in Figure 2. As shown, the impact side steel plate is conservatively neglected. The missile dislodges a conical concrete plug which, in turn, impacts the rear steel plate. The rear steel plate is impacted by the concrete plug and original missile as they move together. The three steps to design the rear steel plate are:

- (1) Select an initial concrete wall thickness. This may be from previous design calculations or from practical limits. For new designs, this thickness may be assumed to be 70% of the required thickness of an RC wall calculated using DOE-STD-3014[12] or NEI 07-13[13].

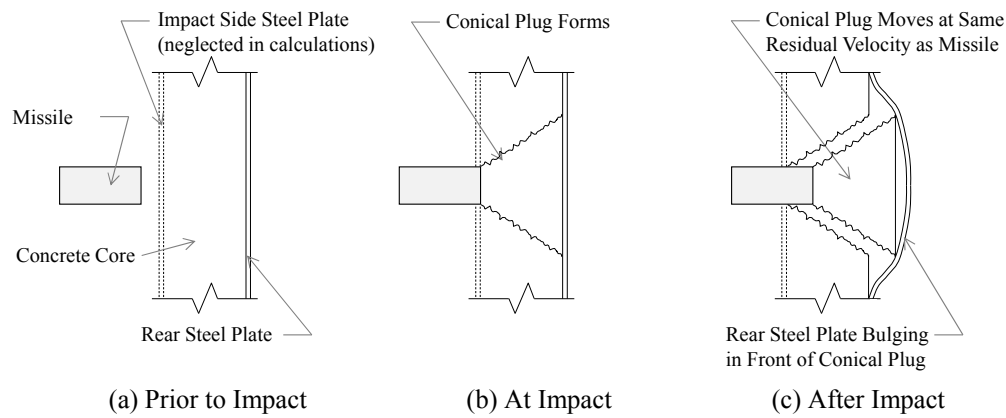


Figure 2. Impact Sequence of Missile on SC Wall (from [5])

- (2) Estimate the residual velocity of the concrete plug as it dislodges from the surrounding concrete. This requires calculating the weight of the concrete plug and the minimum missile velocity needed to form the plug.
- (3) Calculate the required plate thickness to prevent perforation by the impacting concrete plug and missile. Use this thickness for both steel plates.

Equations and additional explanation for each step are provided in [5]. The method was validated against the complete database of 130 missile impact tests conducted on SC walls by a variety of researchers over a 30-year period and the authors included a factor in the equations to account for statistical variation. The authors also provided details of a benchmarked finite element modeling method for use in more detailed analyses of local effects due to projectile impact on SC walls.

Others [14–16] have suggested that the rear steel plate be considered as an equivalent concrete thickness. Combining this equivalent thickness with the actual concrete thickness to determine a total equivalent thickness, conventional RC equations can be used to evaluate the performance of a given design. These methods are most useful for investigating the perforation resistance of an existing but can be manipulated to solve for required steel plate thickness. As explained in [5] results from these equivalent thickness methods tend to be less conservative than the more direct three-step method described above.

Evaluate SC Wall Global Response due to Missile Impact

While a wall may prevent perforation by a specific missile, the global deformation must be within certain limits to maintain structural stability. Two idealized methods exist for conducting a global analysis as shown in **Error! Reference source not found.**: (1) a SDOF model in which the entire wall section is idealized as a single effective mass with an inelastic spring or (2) a two-degree-of-freedom (TDOF) model which separates the mass into global and local effective masses with corresponding inelastic springs defined for each. Equivalent viscous damping can be applied to the effective mass but is often conservatively neglected.

SINGLE-DEGREE-OF-FREEDOM ANALYSIS

Johnson et al.[3] suggested that for cases in which the projectile is stopped by the wall and the response is dominated by global bending, an inelastic SDOF model can be used to estimate the maximum mid-span displacement (see Figures 3(a) and (c)). This requires solving the equation of motion (Equation (1)) for the displacement-time history, $y(t)$.

$$M\ddot{y}(t) + C\dot{y}(t) + R(y)y(t) = f(t) \quad (1)$$

The effective mass, M , can be calculated using the mass transformation factor, K_M , which is 0.14 for elastic response and 0.10 for plastic response of a square fixed-edge panel with a central concentrated load [6]. Because the value of primary interest is the first peak of the displacement-time history, damping, C , is often conservatively neglected [17]. Bruhl et al. [6] provided prescriptive equations to calculate an idealized static resistance function, $R(y)$, for square two-way SC panels with four fixed or four simply supported edges subjected to a central concentrated load. The forcing function, $F(t)$, depends on the type of impact in question. The modified Riera method may be used for aircraft impact [18]. Methods for idealizing $F(t)$ for other types of impact are described in [19].

TWO-DEGREE-OF-FREEDOM ANALYSIS

SDOF analysis cannot account for local deformation. For these cases, a TDOF analysis will yield more reasonable results and provide an understanding of how much of the total displacement is due to global bending and local deformation. For this analysis, two equations of motion are written – one for each degree of freedom – and solved simultaneously. Bruhl et al.[20] described a method to develop idealized resistance functions for both inelastic springs (global and local), how much of the mass to associate with both DOFs, and how to account for damping (particularly of the local DOF). Their work was not prescriptive. Additional work is necessary to fully develop this method of analysis for SC walls.

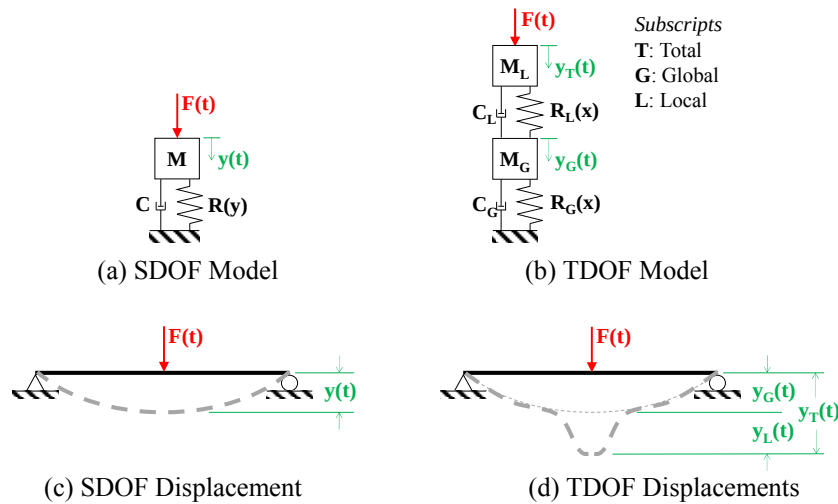


Figure 3. SDOF and TDOF Models and Displacements

BLAST RESISTANT DESIGN OF SC WALLS

There is limited experimental data available in the literature regarding the blast response of SC walls. A series of blast tests on fully enclosed steel-concrete-steel panels [21] and on thin profiled steel sheet cast compositely with concrete [22] have been conducted. These do not include shear studs and tie bars. Tests of two other steel-concrete composite panel configurations, one with J-hooks in place of headed stud anchors and tie bars and the other with internal longitudinal stiffeners have also been reported [23]. Results from tests of the Bi-Steel® system were described qualitatively [24,25]. This structural system tends to have larger flexural and shear reinforcement ratios than typical SC walls and Bi-Steel® does not include stud anchors. While results from each of these test series are instructive, none can be used to evaluate general performance of SC walls to blast loads.

Numerical experiments of the far-field blast response of SC wall sections have been conducted and a SDOF model accurately estimated the maximum mid-span deflection [7]. From this study, the authors concluded that tie bar design was a critical component to ensuring flexural behavior particularly for large blast loads (scaled distance, Z , of less than $3\text{-ft/lb}^{1/3}$). The authors of this study described a rational method to design an SC wall to resist a given blast threat by adapting the provisions for RC wall blast resistant design from UFC 3-340-02 [17] to account for the differences between SC and RC. One conclusion from this study was the need to space tie bars more closely than the AISC N690 recommended $t_{sc}/2$.

Recent experimental tests by the authors of one-third scale SC wall sections have demonstrated that the SDOF model provides conservative estimates of the maximum mid-span displacement of SC panels subjected to large blast loads. The study included 12 specimens of various designs subjected to different loads. Section parameters varied were steel plate thickness, steel plate strength, tie bar diameter, and tie bar spacing. Lower pressure ($Z \approx 11.6\text{-ft/lb}^{1/3}$) and higher pressure shots ($Z \approx 6.8\text{-ft/lb}^{1/3}$) were included in the study. The tests were conducted in the U.S. Army Corps of Engineers – Engineering Research and Development Center (USACE-ERDC) Blast Load Simulator (BLS) in Vicksburg, MS. These test results will be used to develop guidance to design SC structures to resist blast loads.

CONCLUSIONS

SC structures are well suited for protective construction. A rational method to design an SC wall to prevent projectile perforation is available in the literature. SDOF or TDOF analysis methods to quantify global deformation due to projectile impact which does not perforate the wall are in development. A rational method to design an SC wall to resist blast loads is also under development based on a recent series of experiments conducted by the authors.

Additional research is recommended for the following:

- Experimentally confirm the static resistance function of a two-way SC panel subjected to a central concentrated load,
- Experimentally confirm the accuracy of SDOF and TDOF methods to estimate displacement / deformation of a two-way SC panel subjected to a central impact load, and

- Develop understanding of connection behavior under blast and impact loads.

With these additional experimental results to inform future numerical studies, comprehensive design of SC structures for a variety of extreme events will be possible. The ultimate goal is to provide more effective protection for critical infrastructure, personnel, and equipment.

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SUMMARY OF THE BEHAVIOR OF STEEL-PLATE COMPOSITE (SC) WALLS SUBJECTED TO IMPULSIVE LOADS

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ABSTRACT

Steel-plate composite (SC) walls are a viable alternative to reinforced concrete (RC) for protective structures and offer several advantages over RC. Current blast resistant design standards describe methods to design protective structures using RC, structural steel, or masonry in part because of the available experimental database for these materials to validate design methods. While there are blast test results of steel-concrete composite panels available in the literature, they are few and the majority are of specific configurations making it difficult to extrapolate to general behavior. This paper reports representative experimental results of simply supported one-way steel-plate composite (SC) wall sections, designed in accordance with AISC N690s1 Appendix N9 (AISC, 2014), and loaded with short duration uniform pressure pulses representative of far-field blast effects. Two numerical methods to assess the performance of SC walls subjected to blast loads are discussed and comparisons of experimental to numerical results are provided.

INTRODUCTION

Steel-plate composite (SC) walls consist of two steel faceplates and a plain concrete infill as shown in Figure 1. The faceplates provide flexural reinforcement and also serve as formwork. Composite action is provided by steel headed stud anchors on both faceplates. Positioning the faceplates at the extreme fiber of the cross-section maximizes their influence on flexural resistance and tension reinforcement is provided in all planer directions (Varma et al., 2014). Because the faceplates serve as formwork the need for temporary formwork structures is eliminated which reduces construction time. Transverse shear reinforcement is provided by steel bars, structural steel sections, or rectangular plates.

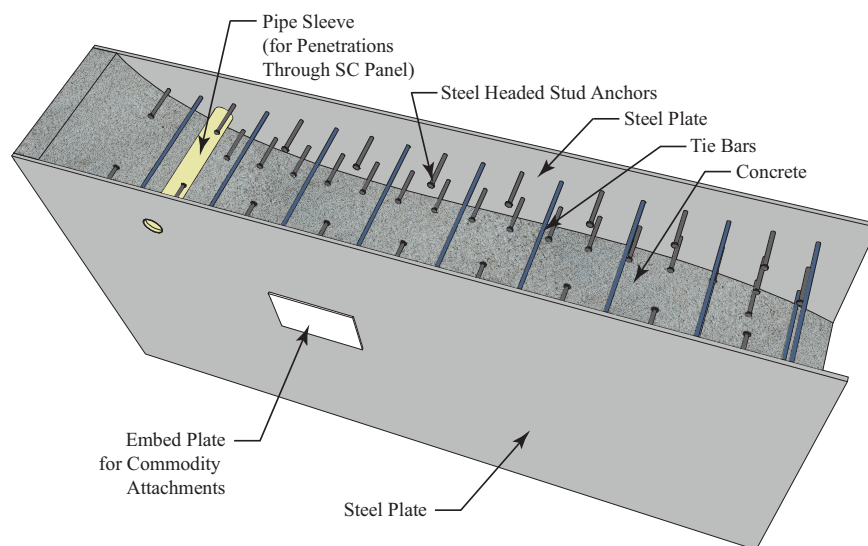


Figure 1. Typical SC Wall Section (from Johnson et al. (2014))

Reinforced concrete (RC) has historically been the preferred choice for blast resistant structures. SC walls are a viable alternative and offer several advantages over RC: (i) steel plates on the outside of the concrete maximize flexural resistance of the section, (ii) the plates serve as formwork which reduces construction time, and (iii) SC modules can be prefabricated which improves construction efficiency. Similar to RC, SC structures combine tensile strength and ductility of steel with compressive strength of concrete.

A specification for the design of SC walls for flexure, in-plane shear, and out-of-plane shear has recently been published as Appendix N9 of AISC N690s1 (AISC, 2014), a supplement to AISC N690-12 (AISC, 2012). Methods to design SC structures to resist the local effect of impacts, such as from an aircraft or tornado or hurricane-borne projectile, have been experimentally validated (Bruhl et al., 2015b; Grisaro & Dancygier, 2014). Quantifying the total and global response of SC structures subjected to impact loads can be accomplished using dynamic analysis methods such as single-degree-of-freedom (SDOF) models or a two-degree-of-freedom (TDOF) model which accounts for both the local and global contributions to total response (Bruhl et al., 2015a; Bruhl et al., 2015c).

Steel-concrete composite panels of various configurations have been subjected to blast loads (Heng et al., 1995; Hulton & Gough, 1999; Hulton, 1995; Lan et al., 2005; Liew & Wang, 2011). With the exception of methods to design Bi-Steel™ walls (Coyle and Cormie 2009), no blast resistant design guidance for SC walls has been developed. This paper describes the behavior of simply supported one-way SC wall sections subjected to short-duration pressure pulses. These pulses were generated by a blast load simulator (BLS) operated by the US Army Corps of Engineers, Engineering Research and Development Center (USACE-ERDC) in Vicksburg, MS. The pressure was approximately uniform across the face of the specimens, simulating far-field blast effects. Time histories of displacement at two locations, mid-span acceleration, faceplate strains, and pressure were recorded for each specimen.

Two numerical modelling methods were benchmarked to these experimental results: (1) a lumped mass, single-degree-of-freedom (SDOF) model using an experimentally developed static resistance function, and (2) a detailed non-linear finite element (FE) model analyzed using the commercial software LS-DYNA. These benchmarked models can be used to better understand the influence of design and loading parameters making them particularly useful in analysis of existing or new SC structures.

BACKGROUND

Blast tests of various configurations of steel-composite panels have been conducted by other researchers. Heng et al. (1995) described results from near-field blast tests of one-third scale fully enclosed steel-concrete-steel (SCS) sandwich panels. Their panel design contained no internal components (shear studs or tie bars) and the concrete core was enclosed by steel plates on all sides. Lan et al. (2005) described results from a series of blast tests of three different panel types: (i) steel fiber reinforced concrete (SFRC), (ii) profiled steel sheeting reinforced concrete (PSSRC), and (iii) SCS panels similar to those by Heng et al. (1995). Tests of SCS panels investigated the influence of concrete infill (they tested some panels with no concrete), concrete thickness, and steel plate thickness for a variety of charge weights at a constant stand-off distance. Liew & Wang (2011) conducted blast tests on a unique configuration of SCS and a type of panel they called cellular stiffened plate (SP). The SCS panels had j-hooks in place of headed stud anchors and tie bars. The SP configuration connected steel faceplates with internal steel stiffeners. They tested panels with different steel plate thicknesses subjected to blasts of the same charge weight and stand-off distance. They concluded that increased steel faceplate thickness improved blast resistance of the structure and the type of concrete infill influenced the type of failure experienced by the panel. Hulton (1995) provided a qualitative summary of blast tests conducted by the United Kingdom Defence Research Agency of steel-backed reinforced concrete - some with steel plates on the back-face alone and others with steel plates on both front- and back-faces. The author concluded that panels with plates on both sides

performed better than those with only back-plates but that front-plates were inefficient because of fragment damage or buckling.

Hulton & Gough (1999) discussed development of Bi-Steel™, a proprietary product of British Steel PLC (now Tata Steel). The authors described various configurations of SCS composite sections and explained how the Bi-Steel™ design overcame weaknesses of each. Bi-Steel™ walls consist of two steel faceplates connected by friction welded transverse bar connectors. These modular sections could be cast with concrete in the factory and shipped to their intended location or set in place, welded to each other, and filled with concrete on site. The authors reported general behavior of the panels in blast tests but provided no details on test specimens, blast loads applied, or observed damage. Coyle & Cormie (2009) described blast resistant design of SCS composite sections, addressed similarities and differences between RC and SCS, and discussed computation of strength and stiffness. They focused on Bi-Steel™ and the information was not general enough to be of use for other configurations of SC walls.

Of the different configurations that have been tested, Bi-Steel™ is most similar to the general SC system described in this paper but is proprietary and has key differences from SC structures designed in accordance with AISC N690s1 Appendix N9. For example, Bi-Steel™ does not include headed stud anchors and its flexural reinforcement is often larger than the 5% maximum permitted by N690s1 N9.

EXPERIMENTAL PERFORMANCE OF SC WALLS SUBJECTED TO BLAST LOADS

An experimental investigation by Bruhl (2015) of one-third scale SC wall sections subjected to blast loads studied the influence of design and blast load parameters on SC wall performance (see Table 1).

Table 1 Experimentally Varied Parameters

Design Parameter	Range Considered
Steel Plate Strength	A1011 HSLA Grade 50 to 80
Steel Plate Thickness, t_p	14 to 12 Gage
Flexural Reinforcement Ratio, ρ	3.5% to 5.6%
Shear Reinforcement Ratio, ρ_t	0.37% to 1.23%
Shear Reinforcement Spacing, S	$t_{sc}/2$ to t_{sc}
Peak Reflected Pressure, p_r	18.9 to 61.6 psi
Peak Reflected Impulse, i_r	260 to 1110 psi·msec

Specimen Design

Specimens were designed in accordance with AISC N690s1 Appendix N9 and current blast resistant design methods for RC explained in UFC 3-340-02 Chapter 4, to account for dynamic shear demand. Because the specimens were one-third scale and faceplate strength was intentionally varied, they were not in accordance with every aspect of N690s1 N9:

- The smallest plate thickness permitted is 0.25-in; these specimens used 12 and 14 gage sheet (nominally 0.105-in or 0.075-in respectively),
- Grade 80 steel is not permitted, and
- The maximum permitted flexural reinforcement ratio of 5.0% was exceeded for three specimens; actual plate thickness of the 12 gage Grade 50 sheet was slightly larger than nominal.

As shown in Figure 2, each specimen had identical global dimensions: 4-in total thickness, t_{sc} ; 11-in width, b ; 64-in total length; and 52-in unsupported length, L . A1011 HSLA hot-rolled steel sheets were used for top and bottom faceplates. Steel sheet grade and gage varied between specimens. All specimens

were cast from the same concrete batch. Concrete aggregate was pea gravel with a maximum diameter of 0.375-in. Day-of-test concrete cylinder strengths, f'_c , were between 6.2 and 6.5-ksi. Threaded rods provided transverse shear reinforcement. Diameter of the rods, d_t , varied from 0.138-in to 5/16-in. Holes were drilled in the faceplates 1/32-in larger than the tie bar diameter. Tie bars extended through the holes and were secured with hex nuts on both sides of the faceplate. All specimens included 0.25-in diameter (d_s) 1.125-in long, headed shear stud anchors spaced, s , at 2-in on-center.

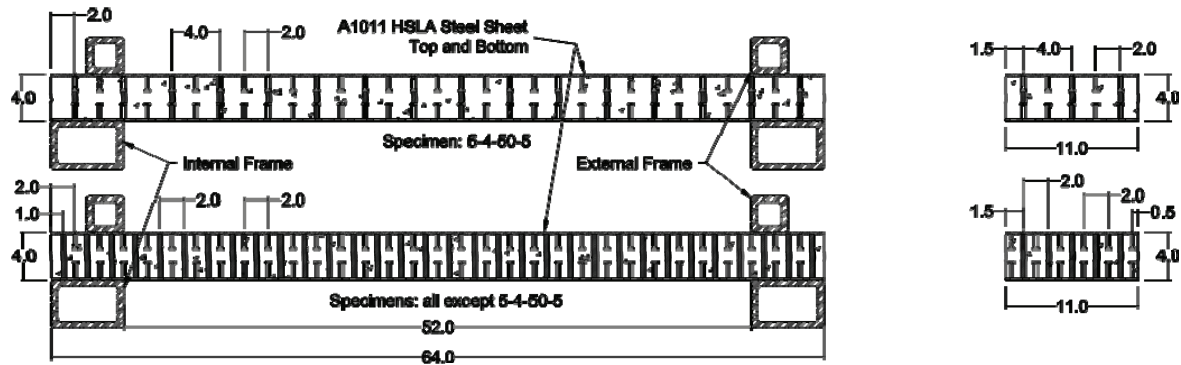


Figure 2 Blast Specimen Design Schematic (units are inches)

Test Setup

The BLS, shown in Figure 3, is a compressed-gas-driven shock tube which can be tuned to simulate far-field blast waveforms. It consists of: (i) a driver and pressure vessel which is pressurized with a combination of helium and air, (ii) vented cone and transition components which shape the pressure wave, and (iii) a target vessel to support the tested specimens and contain any debris (Johnson 2013). The BLS permits extensive instrumentation of specimens and there is no fireball or debris cloud from an explosion to obscure video recordings of the tests.

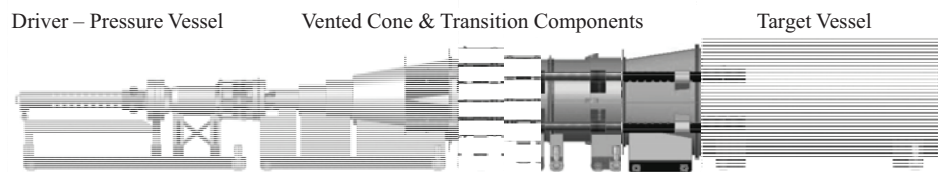


Figure 3 Schematic of USACE-ERDC BLS (after C. F. Johnson (2013))

Three specimens were tested simultaneously during each of four setups (see Figure 4). The frame installed in the target vessel provided approximately simple supports of the specimens. In each of the first two setups, two shots were conducted. The first shot was designed to result in elastic response of the specimens and the second, larger pressure, shot was expected to result in plastic deformation. Each of the last two setups subjected the specimens to the larger pressure shot.

The layout and type of sensors was identical for each of the four tests (Figure 4). Eight pressure gauges around the perimeter of the frame recorded the pressure-time history. A ninth pressure gauge recorded the pressure inside the target vessel. Potentiometers recorded mid-span deflection of each. As an alternate measure, mid-span deflection was calculated from mid-span accelerometer data. Cable-extension transducers recorded deflection 8.75-in away from mid-span. Strains were measured at mid-span on the compression (blast) face and the quarter-points on the tension (non-blast) face of each specimen.

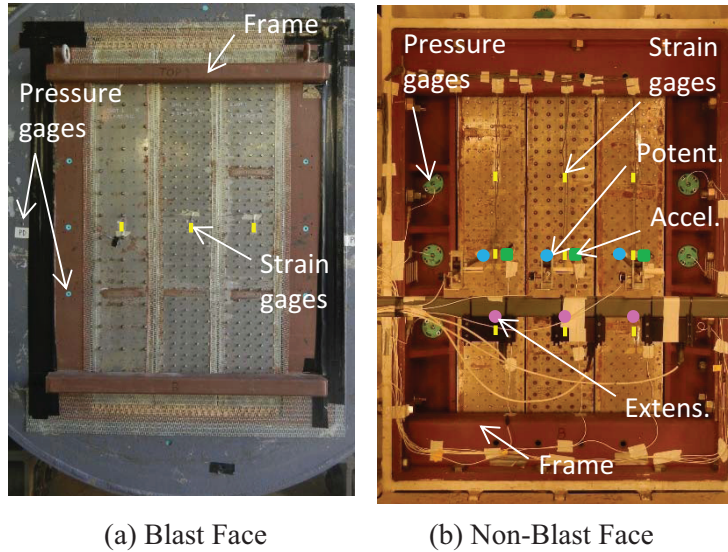


Figure 4 Specimen Setup in BLS (Pre-test Photos) With Sensor Layout

Three high-speed video cameras documented the specimen responses (left, right, and rear angles) and pre- and post-test still photos were taken. Residual displacements were measured prior to removing specimens from the target vessel. After each test was complete, the three specimens were removed from the frame and cracks along the sides were marked and photographed.

Experimental Results

For brevity, representative results from one specimen from the second setup are discussed in this paper. Complete experimental results from all sensors for all six shots are provided by Bruhl (2015). This representative specimen was comprised of 12 gage Gr50 A1011 HSLA steel sheets, 0.25-in tie bars spaced at 2-in on-center, and 6.3-ksi concrete. Measured stress-strain curves of the steel components are shown in Figure 5. Included on this figure are idealizations of the measured curves which were used as input for numerical models.

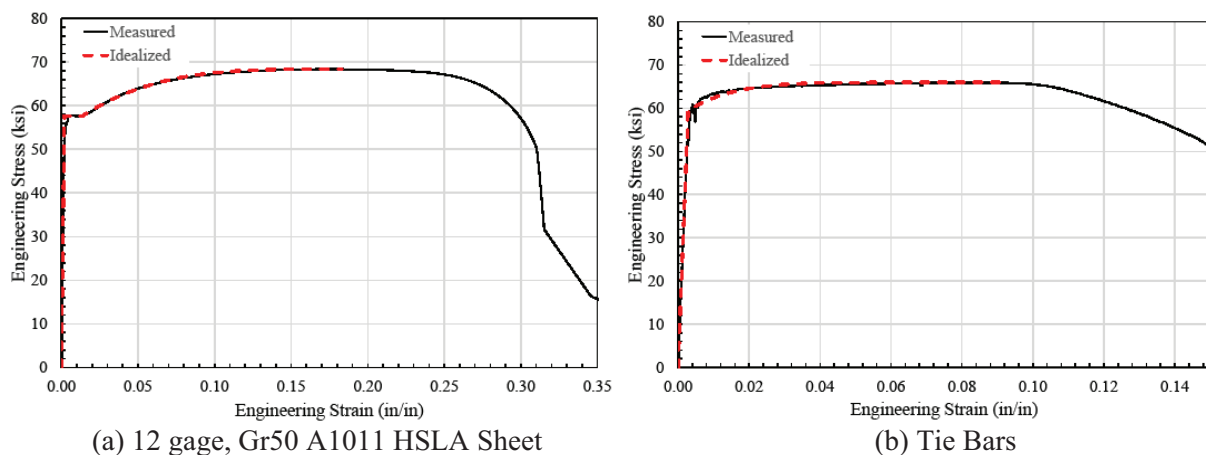


Figure 5 Measured Steel Stress-Strain Curves

This specimen was subjected to two pressure loads. Representative pressure time-histories for each shot are provided in Figure 6. Overlaid on the measured data in this figure is an idealized exponential decay function which served as input for numerical models. The idealization modified the Friedlander waveform (Baker et al., 1983) to include a finite rise time to peak pressure, p_r . The average maximum measured pressure from the eight pressure gages on the face of the target vessel for each of the two shots is provided with the average peak impulse in the captions of Figure 6.

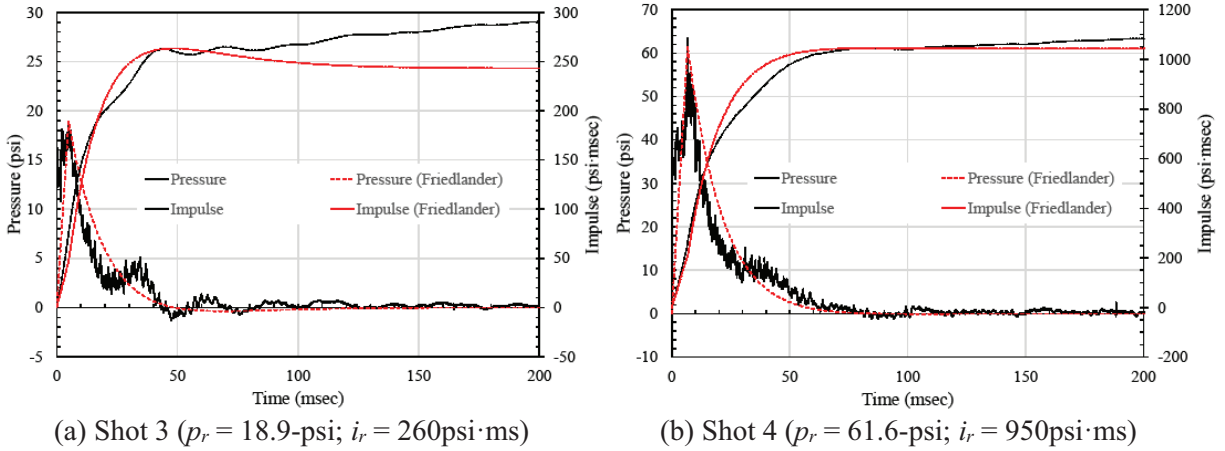


Figure 6 Measured and Idealized Pressure Time-History and Calculated Impulse

The displacement time history for the representative specimen for both shots is provided in Figure 7. The measured value from the extensometer was at a location away from mid-span and expected to be less than mid-span displacement. Because of numerical errors, only the first peak displacement value calculated from accelerometer data is valuable but the potentiometer and extensometer transient displacement data is valuable. Despite being post-tensioned, the frame displaced as a rigid body during each test and maximum displacement of the frame was determined from analysis of high-speed video recordings. Corrected values accounting for frame displacement are provided in the captions of Figure 7 for maximum and residual displacement, X_M and X_r respectively.

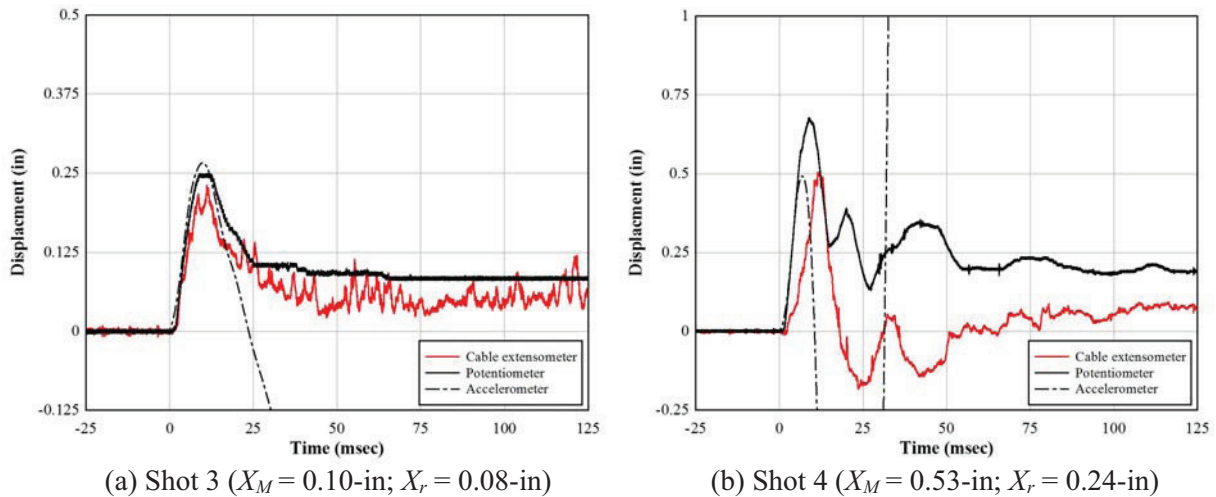


Figure 7 Measured Displacement Time-History

Measured strains of the specimen for the two shots are shown in Figure 8. The black lines are the strains at the quarter points of the non-blast (tension) faceplate and the red line is the strain at mid-span of the blast (compression) faceplate. The strains during Shot 3 were below yield ($\sim 2000\text{-}\mu\epsilon$) demonstrating that the specimen responded elastically. Results from Shot 4 show that the larger load resulted in plastic deformation – maximum tensile strain at mid-span was beyond yield. The peak compressive strain of the blast face plate was also close to yield. There was no evidence of concrete crushing.

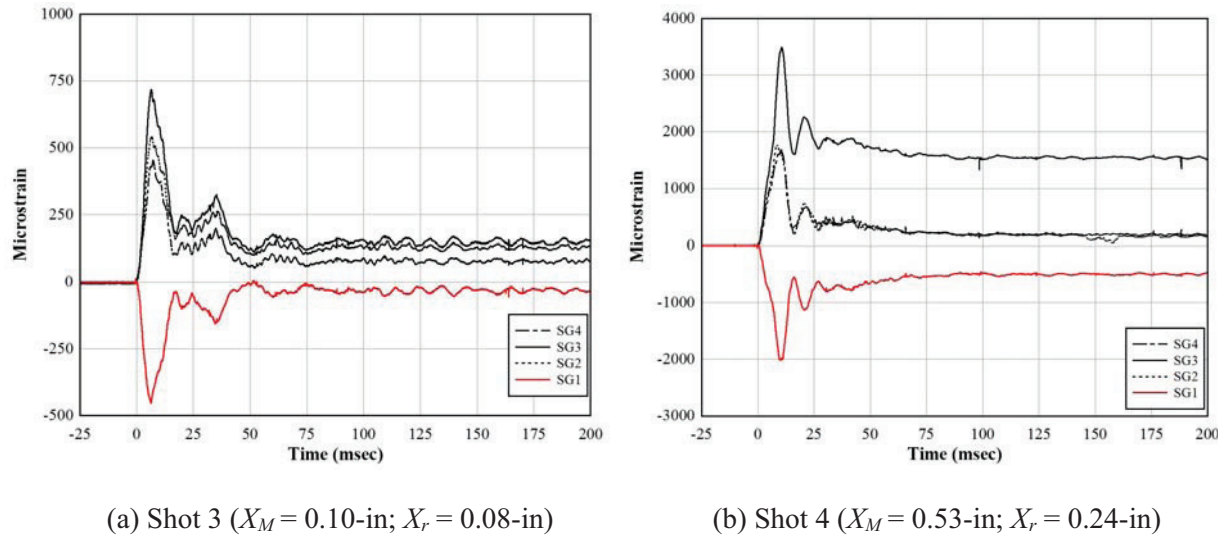


Figure 8 Measured Strain Time-History

NUMERICAL MODELS

Two numerical models were benchmarked to the experimental results: (1) a lumped mass SDOF model and (2) a detailed non-linear FE model. For brevity, a summary of each modelling method is provided below. Detailed information about input parameters for both models is provided by Bruhl (2015).

SDOF Model

An idealized lumped mass SDOF model is a common tool in blast resistant design of structural elements (ASCE 59-11; Smilowitz, et al., 2010; UFC 3-340-02). The SDOF model for this analysis included:

- The idealized pressure time-history shown in Figure 6,
- An idealized experimentally validated bi-linear static resistance function computed from section geometric and material properties, including dynamic increase factors (DIF) (Bruhl 2015), and
- The mass of the unsupported length estimated from the measured mass of the entire specimen.

The effect of damping is often neglected in analysis supporting blast resistant design. For comparison purposes, the model was analyzed with 0% damping and 5% damping. The equation of motion for the SDOF model was solved numerically using the constant acceleration method.

FE Model

A detailed non-linear FE model was analyzed using LS-DYNA (Hallquist 2006). The model, shown in Figure 9, consisted of solid elements for the concrete infill, steel faceplates, and supports and beam elements for stud anchors and tie bars. The concrete core was modelled using the Winfrith concrete model. When the strain rate effects were included in the Winfrith model inaccurate results were obtained.

Instead, DIFs for concrete compressive strength, tensile strength, and fracture energy were applied directly to input values. Steel properties were input using the idealized piecewise curves from Figure 5. DIFs were applied to steel yield strength. Studs were connected to faceplates using zero-length connector elements which accounted for load-slip behavior (Zhang et al., 2014).

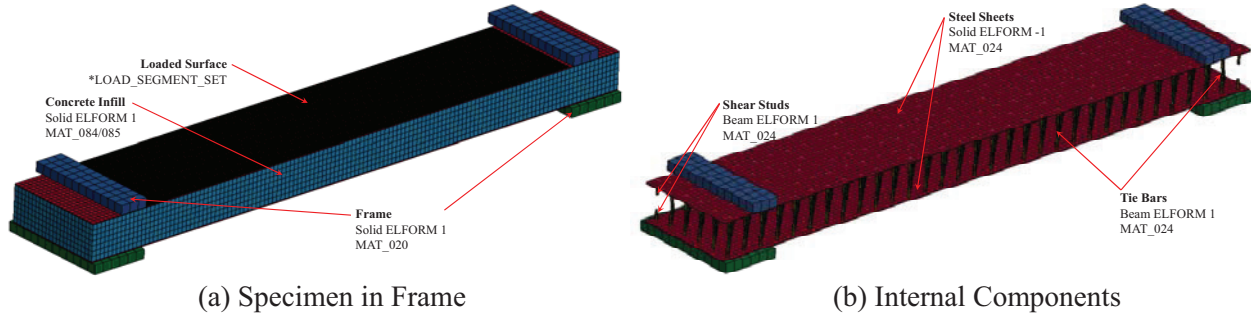


Figure 9 FE Model Overview

Comparison of Results

A comparison of experimentally measured displacements to those computed from the numerical methods is provided in Table 2. The SDOF model with 0% damping provided a conservative estimate of maximum response of this specimen for both shots and the FE model was nearly exact for this specific case. Including 5% damping in the SDOF analysis had small influence on the estimated response in the elastic region but decreased the maximum displacement for the larger shot by 17%.

Table 2 Representative Blast Tests Results

Setup	Shot	SDOF						FE	
		Experiment		0% Damping		5% Damping		X_M (in)	X_r (in)
		X_M (in)	X_r (in)	X_M (in)	X_r (in)	X_M (in)	X_r (in)		
2	3	0.10	0.08	0.15	0.00	0.14	0.00	0.10	0.00
	4	0.53	0.24	0.56	0.25	0.46	0.10	0.53	0.16

The displacement time-history from the SDOF model with 0% and 5% damping considered and the FE model are shown in Figure 10. A direct comparison to the experimental values in Figure 7 is difficult because the measured data included the rigid body displacement but the shape of the transient response is most similar between the FE model results and the experimental response.

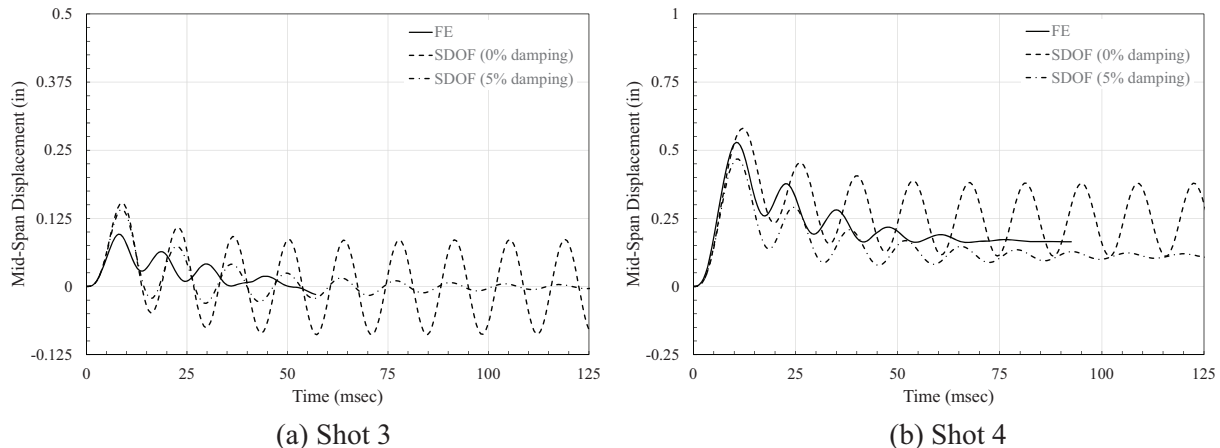


Figure 10 Numerical Analysis Displacement Time-History

The strain time-history from the FE model for both shots of this representative specimen are shown in Figure 11. Neither analysis was run for the same length of time as shown in the experimental data but the horizontal scale for these figures was the same as Figure 8 for ease of comparison. Strains for the elastic response were similar between the FE model and experimental results. For the plastic response, the FE model predicted larger tensile strains and smaller compressive strains at mid-span than experimentally observed indicating the neutral axis was closer to the top of the cross section in the numerical model than in the experiment.

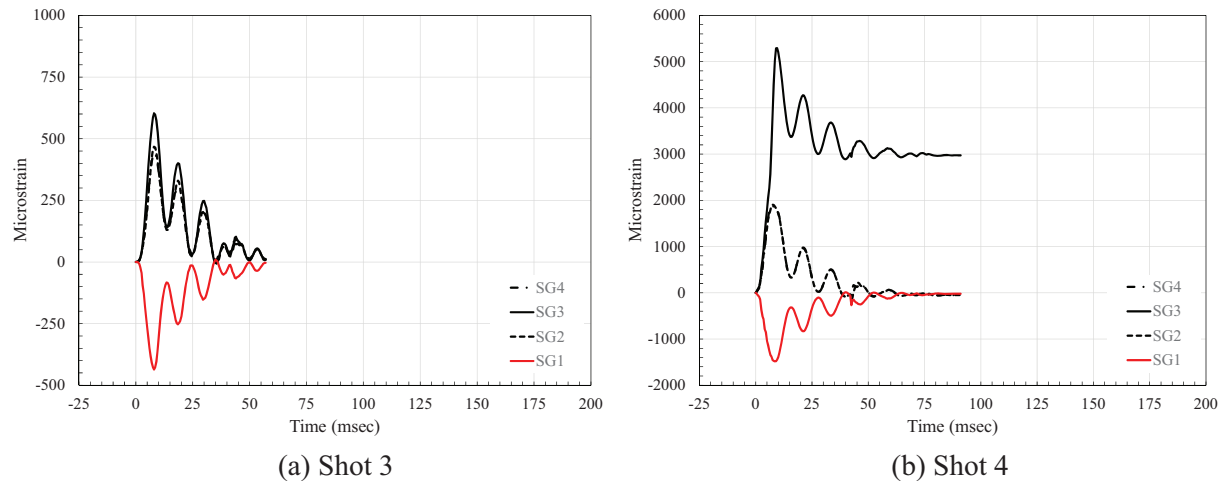


Figure 11 Numerical Analysis Strain Time-History

CONCLUSIONS

This paper described a recent experimental investigation of structural performance of simply supported one-way SC walls subjected to short duration uniform pressure loads using the USACE-ERDC BLS. The specimens tested were designed as flexure-controlled in accordance with provisions for SC walls in safety related nuclear facilities (AISC N690s1 N9) and accounted for dynamic shear demands.

Detailed results were provided for a representative specimen subjected to two loads: a lower pressure designed to result in elastic response and a higher pressure designed to yield the section. Two numerical modelling methods were presented and, for the representative specimen, the SDOF and non-linear FE analyses provided good estimates of maximum and residual displacement. The SDOF model was generally conservative when 0% damping was considered and underestimated the response when 5% damping was included in the solution.

ACKNOWLEDGEMENTS

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Experimentally Validated Analysis Methods for Steel-Plate Composite (SC) Walls Subjected to Blast and Impact Loads

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ABSTRACT

This paper presents analytical tools to assess the performance of steel-plate composite (SC) walls subjected to blast and impact loads. These methods include design charts, single-degree-of-freedom (SDOF) models, P-I diagrams, and nonlinear finite element (FE) analysis. Each analytical method has been benchmarked to an experimental database which includes a wide range of design and load parameters. The paper also includes examples and recommendations to use these tools to assist in the design of protective structures.

INTRODUCTION

Steel-plate composite (SC) structures consisting of exterior steel faceplates, a concrete core, headed stud anchors and transverse tie bars have been used in submerged tube tunneling, offshore construction, storage vessels, safety-related nuclear power plant structures, and as protective structures. As Malushte and Varma (2015) describe, SC structures have distinct advantages when compared to reinforced concrete (RC). SC wall modules can be prefabricated which can expedite construction. SC structures have gained increased interest in the nuclear power plant industry because of the advantages of modular construction, reduced construction time and demonstrated resilience during extreme events such as seismic, blast, and impact loads.

SC structures have been shown to provide a higher level of protection against projectile perforation and blast loads than RC structures. Experimental tests have demonstrated improved ductility and reduced risk of direct shear failure of concrete filled tubes (CFT) when compared to the behavior of steel-jacketed RC columns subjected to blast loads (Fujikura and Bruneau 2011; Fujikura et al. 2008). Experimental programs have investigated the performance of a variety of SC configurations subjected to near-field and far-field explosions (Bruhl 2015; Heng et al. 1995; Hulton 1995; Lan et al. 2005; Liew and Wang 2011). Mizuno et al. (2005) concluded that an SC wall provides the same level of protection against projectile impact as an RC wall which is 30% thicker. Other experiments of SC walls subjected to rigid and deformable projectile impacts have substantiated this conclusion and a database of 130 impact tests on SC walls available in the literature was compiled by Bruhl et al. (2015).

In order to justify the use of SC structures in new designs, engineers require experimentally validated analysis methods. This paper describes analytical tools to assess the performance of SC walls subjected to blast and impact loads.

BLAST ANALYSIS METHODS

Blast loads are often described as close-in, near-field, or far-field, depending on the magnitude of the scaled distance of the explosion, Z . This term is defined as the stand-off distance, R , divided by the cube root of the charge weight, W , and aids in characterizing the pressure distribution over the face of the structural element. Far-field effects can be assumed as uniform across the face of the element; this is not a reasonable approximation for near-field or close-in explosions. An important consideration for selecting appropriate blast analysis methods is the structural response: whether the response depends on the pressure only, the pressure-time history, or the impulse only. These three categories relate to low, intermediate, and high-pressure loadings and are also described as quasi-static, dynamic, and impulsive responses, respectively. For the first two categories, numerical methods to solve the equation of motion for a SDOF model are appropriate. For the third, impulse category, energy methods or design charts are required. UFC 3-340-02 (U.S. DOD 2008) describes details of analysis methods for blast resistant design. Other useful references which describe general blast response analysis methods include Dusenberry (2010) and Krauthammer (2008).

Close-In and Near-Field Explosions

There are few experimental investigations of close-in and near-field explosive effects on SC structures. Fujikura et al. (2008) tested CFTs subjected to near-field explosions and Heng et al. (1995) tested concrete filled steel panels subjected to close-in explosions. In each of these cases, results from impulse analyses were compared to experimental results. Additionally, Fujikura et al. (2012) described a variety of analytical methods such as SDOF, fiber models, and finite element (FE) models to estimate the behavior of CFTs subjected to blast loads. These models were benchmarked to the experiments in Fujikura et al. (2008).

Far-Field Explosions

A series of experiments investigating the behavior of far-field explosive effects on SC walls designed in accordance with AISC N690s1-15 Appendix N9 was completed by Bruhl (2015). These experiments included twelve specimens which varied design parameters including: steel faceplate strength, flexural reinforcement ratio, shear reinforcement ratio, and tie bar spacing. The one-way bending specimens were tested with short duration uniform pressure pulses generated by the Blast Load Simulator (BLS) owned and operated by the U.S. Army Engineer Research and Development Center in Vicksburg, MS. Identical specimens were tested statically at Purdue University's Bowen Laboratory for Large Scale Civil Engineering Research to characterize the static resistance function for use in a benchmarked SDOF model for far-field effects on SC walls.

From these experiments, SDOF and FE models were benchmarked. The SDOF model neglected damping, used an idealized bi-linear static resistance function developed from static experimental results (modified to include dynamic increase factors (DIF) on steel and concrete strength), and idealized the pressure-time history using the modified Friedlander waveform fit to the experimentally measured pressure-time history records. The equation of motion was solved numerically using the constant acceleration method. FE models of the statically loaded and pressure pulse loaded specimens were analyzed using LS-DYNA. The only difference between the models was the inclusion of DIFs on the concrete and steel material properties for the pressure pulse loaded models. These FE models used measured stress-strain curves for the steel faceplates and tie bars as inputs. Additional details of these modeling methods are provided in (Bruhl 2015).

Results from the SDOF and FE models are compared to experimental results in Figure 1. In each graph, the horizontal axis is the measured mid-span displacement from the tests completed in the BLS and the vertical axis is the calculated mid-span displacement of the specimen using the SDOF model (Figure 1(a)) or the FE model (Figure 1(b)). As these comparisons demonstrate, both models provided reasonable estimates of the measured behavior and can be used to predict the behavior of specific SC wall dimensions for various blast threats.

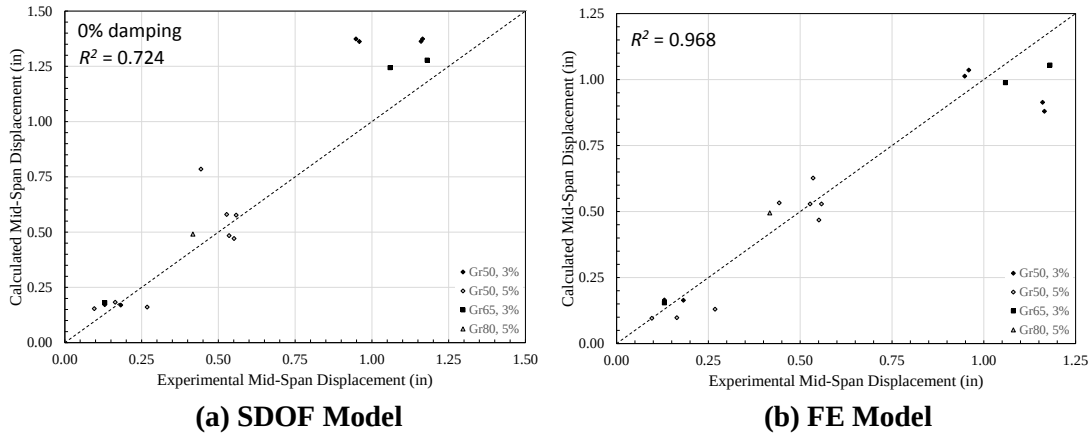


Figure 1 Comparison of Analytical to Experimental Results

An efficient tool to analyze the performance of a structural element for a given blast threat is a total force vs total impulse (P-I) diagram. These diagrams normalize the blast load and section property parameters using (1 and (2 (Krauthammer et al. 2008). In these equations, p_r is the peak reflected pressure, i_r is the peak reflected impulse (area under the pressure-time history curve), R_y is the yield resistance of the cross-section, ω_n is the natural frequency of the structural element, and b and L are the width and length of the structural element, respectively. Contours on P-I diagrams are created for deflection ductility limits, which is the ratio of maximum deflection to the elastic deflection of the structural element.

$$P = \frac{2p_r bL}{R_y} \quad (1)$$

$$I = \frac{i_r b L}{(R_y / \omega_n)} \quad (2)$$

P-I ductility contours for one-way SC wall structures were generated using the benchmarked SDOF model. Using computer methods described by Krauthammer et al. (2008), the required p_r and i_r combinations to achieve a maximum displacement associated with a specific ductility were calculated and plotted. This was done for a variety of ductility limits as shown in Figure 2. This figure also labels the portions of a P-I diagram which correspond to the quasi-static, dynamic, and impulsive structural response as described above. Shown on Figure 2 is also the measured ductility from each of the BLS tests. This measured ductility is the ratio of the measured maximum mid-span displacement to the calculated yield deflection of the specimen. As shown, measured ductility falls within the regions of the P-I diagram as expected, with three exceptions. The point near the bottom left labeled 0.780 is an outlier. Two points near the top right labeled 4.028 and 4.045 represent two specimens with larger experimental ductility than expected. Both of these specimens had been previously loaded by a lower pressure blast wave and sustained minor damage which reduced their resistance to the second larger blast wave.

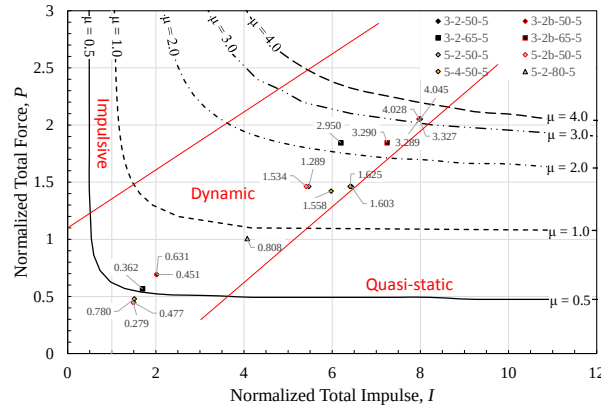


Figure 2 P-I Diagram with Experimentally Measured Ductility

As seen in Figure 2, the experiments were all within the dynamic response domain. To expand the database and validate the P-I diagram in the other two response domains, blast load parameters on the benchmarked FE models were changed and the responses were used to create ductility contours. These confirmed accuracy in all regions of the P-I diagram.

Figure 3 was prepared to depict ductility limit states from ASCE 59-11 and AISC N690s1-15. The limits from ASCE 59-11 were for RC walls with compressive loads (as in a conventional building structure) and the one from AISC N690s1-15 is for an SC wall for safety related nuclear power plant construction. It is beyond the scope of this paper recommend which limit states to use; rather the purpose is to provide design and analysis tools for use in engineering practice. For example, consider a case in which the calculated normalized total pressure was 2 and normalized total impulse

was 3. In this scenario the given wall would be expected to meet the ASCE 59-11 Heavy damage criteria but exceed the Superficial and Moderate limit states.

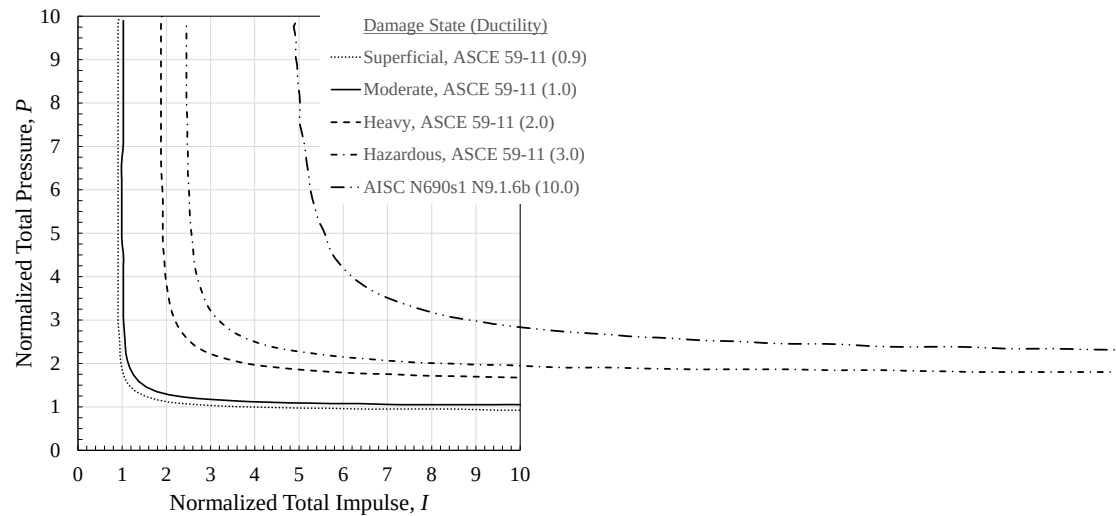


Figure 3 P-I Diagram with Published Limit States for RC and SC Walls

IMPACT ANALYSIS METHODS

Kim et al. (2015) summarized methods to analyze and design SC walls subjected to impact loads. The authors described methods to address each of the three primary design considerations: (i) prevent wall perforation, (ii) limit the maximum displacement to avoid wall contact with internal components, and (iii) limit end rotations to maintain structural stability and meet desired ductility limits.

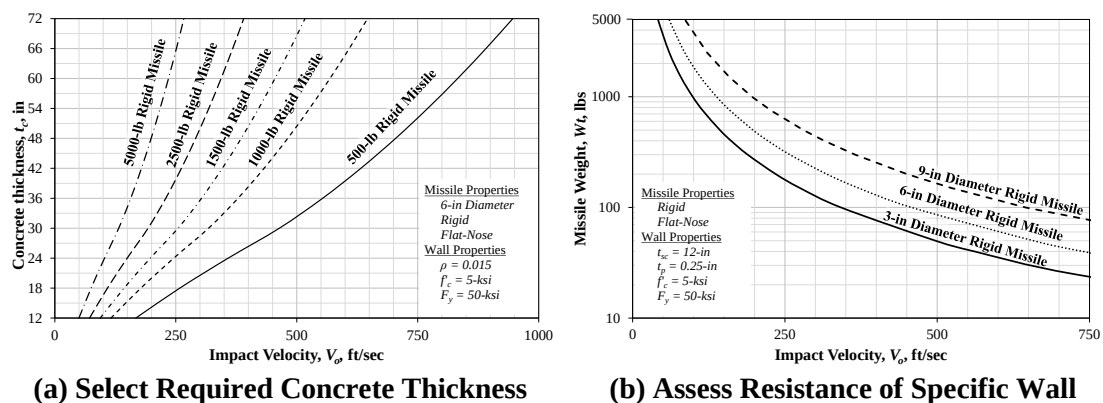
Prevent Wall Perforation

Bruhl et al. (2015) presented a rational method to design the faceplate thickness of an SC wall to prevent perforation by a selected projectile. This method consists of three steps: (i) select the concrete wall thickness based on existing design requirements or 70% of the required RC wall thickness, (ii) estimate the weight and velocity of a dislodged concrete frustum which, with the original impacting projectile, impacts the rear steel faceplate, and (iii) calculate the required rear steel faceplate thickness required to resist perforation. This method is also applicable as an analysis tool by comparing the required steel faceplate thickness to the specified faceplate thickness. If the specified thickness is larger than required, the design is safe. The three-step design method was validated against a database of 130 impact tests on SC walls and was found to be generally conservative for the wide range of projectile and wall parameters included in the database.

The method can be used to create design curves to enable efficient preliminary design or assess the protective capacity of existing SC walls. Two examples of such design curves are shown in Figure 4. Figure 4(a) was created for a 6-in diameter, flat-nosed rigid missile. The vertical axis shows the concrete thickness required to prevent perforation by a projectile with a specific impact velocity (shown on the horizontal

axis) and weight (the curved lines on the figure). For example, if the specific projectile of concern is 500-lbs with an impact velocity of 350-ft/sec, a concrete thickness of 24-in is required. This figure was created for specific material properties as listed on the figure and a reinforcement ratio of 1.5%; the required plate thickness for this SC wall would be 0.18-in on both faces.

Figure 4(b) was created to assess the perforation resistance of a specific wall configuration, as listed on the figure, for a variety of projectile threats. The vertical and horizontal axes show the projectile weight and impact velocity, respectively, and the curved lines on the figure are for various projectile diameters. If the projectile in question falls below the line the wall will resist perforation. For example, if the projectile of concern is a 200-lb, 6-in diameter, missile with an impact velocity of 250-ft/sec, the wall will stop the projectile, but if the velocity were increased to 400-ft/sec, the projectile would be expected to perforate the wall.



**Figure 4 Design and Assessment Curves for Perforation Resistance of SC Walls
(from AISI N690s1-15 (AISI 2015))**

Other methods to assess the perforation resistance of SC walls recommend converting the steel faceplate thickness to an equivalent concrete thickness. Walter and Wolde-Tinsae (1984) and Tsubota et al. (1993) proposed simple equations based on specific sets of experiments. Use of these equations for ranges of parameters beyond these experiments is not recommended. Grisaro and Dancygier (2014) proposed a more general methodology to convert steel faceplates to equivalent concrete thicknesses which is applicable for a wider range of design parameters. These methods are useful for analysis of existing designs.

Limit Maximum Displacement

A wall designed to prevent perforation may have excessive deflection which could damage internal components if the wall comes into contact with them. Deflection of a wall during missile impact consists of global bending and localized deflection in the vicinity of the impact location. Figure 5 defines deflection terminology: total mid-span displacement, x_T , and its local and global components, x_L and x_G respectively.

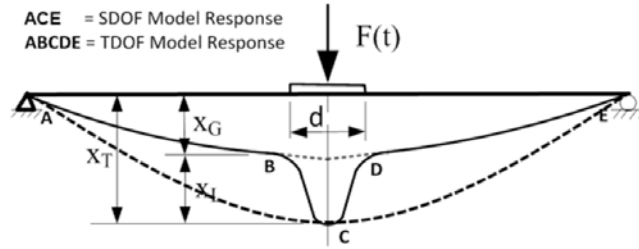


Figure 5 Definition of Local, Global, and Total Displacements (from Kim et al. (2015))

The maximum total deflection, x_T , of SC walls subjected to a projectile impact which did not perforate the wall can be estimated using an SDOF model. The damping term in the equation of motion can be neglected. An idealized bi-linear static resistance function for two-way behavior of SC walls with central concentrated loads was developed by Bruhl et al. (2015b). This SDOF model is only valid when the governing failure mode of the SC wall is flexure. Flexure-controlled SC walls must be properly detailed in accordance with the specifications in Appendix N9 of AISC N690s1 (2015).

Limit End Rotations

A wall designed to prevent perforation and not contact internal components as it deflects may still result in excessive global deflection and end rotations which could lead to structural stability concerns. In order to address this design consideration a two-degree-of-freedom (TDOF) model has been proposed by Bruhl et al. (2015c). In this model, the local displacement, x_L , is extracted from the total displacement, x_T , to calculate the global displacement, x_G . The global displacement, x_G , is the appropriate displacement to use in estimating end rotation or deflection ductility based on an impact force. Initial analysis has been completed but additional work is necessary to create reliable idealized resistance functions from which the global displacement, x_G , can be estimated.

A method proposed by Grisaro and Dancygier (2015) estimates residual displacement of the rear steel faceplate after projectile impact which is related to the maximum local displacement, x_L , during an impact event. In this method, the rear steel liner with the ejected concrete fragments and impacting projectile are represented together as a SDOF by decoupling it from the response of the remainder of the concrete wall. This methodology requires estimation of the mass and energy of the concrete fragments with or without the projectile which is used as the loading function on the rear faceplate. This SDOF model was verified using experimental results from the database of SC wall impact tests. This method provides reasonable estimates of the local displacement but does not account for the global bending contribution of the total maximum displacement.

Finite Element Analysis for Projectile Impact of SC Structures

For cases in which simplified models are not adequate or acceptable, detailed, 3D nonlinear finite element models may be required. Depending on the design consideration which is the focus of the analysis, various benchmarked models exist in

the literature. To investigate local perforation resistance, a very fine mesh in the impacting region is required and Bruhl et al. (2015a) described a benchmarked finite element modeling method using LS-DYNA. To understand the global behavior of a wall which prevents perforation, a much courser mesh is acceptable. For such cases, Bruhl et al. (2015b) provide benchmarked models of SC wall panels using LS-DYNA.

Jiang and Chorzepa (2014) provided a summary of methods to numerically model aircraft impact and include a variety of references to experimental results which can be used for model validation. These references include modeling methods using a variety of finite element software including LS-DYNA, AUTODYN, and ABAQUS-E, among others, and describe examples of detailed models of aircraft and containment structures.

CONCLUSIONS

Experimentally validated methods to analyze the behavior of SC walls for blast loads and projectile impact were presented. These included methods to estimate the behavior of one-way SC walls subjected to far-field blast effects. Methods were also presented to address the three design considerations for impact loads: (i) prevent perforation, (ii) limit total deflection, and (iii) limit end rotations to ensure structural stability.

There is need for research concerning close-in and near-field blast effects on SC walls. Existing work in this field is rather limited and does not address performance of SC walls designed in accordance with AISC N690s1-15. Additional experimental and numerical work is recommended to improve understanding of behavior and develop tools to assist in the design of protective structures. There is also need for additional research in separating the global from local deformations during an impact event. Additionally, there is need to evaluate the performance of structural connections during blast and impact events.

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