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BLAST-RESISTANT TESTING FOR MASS TIMBER EXTERIOR WALLS: FINAL ACCOMPLISHMENT REPORT

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EXECUTIVE SUMMARY

UFC 4-010-01 requires that inhabited Department of Defense (DoD) buildings constructed of mass timber structural systems be analyzed for airblast loads. As of the summer of 2015, there was a lack of test data documenting mass timber system response under the strain rates imposed by airblast loads. Thus, the primary objective of this effort was to perform testing that would demonstrate the capability of mass timber systems to resist airblast loads. To achieve this primary objective, the following project objectives were defined:

- To develop analytical methodologies to analyze mass timber panels for blast loads.
- To conduct static and dynamic testing on mass timber systems as a means to validate and/or improve these developed analytical methodologies.
- To document the developed analytical methodologies and obtained test data in a form that could serve as a reference for structural engineers interested in designing mass timber structural systems to resist blast loads.

Two mass timber systems were investigated as part of this effort: cross-laminated timber (CLT) and nail-laminated timber (NLT). Grades V1, E1, and V4 CLT as well as 2x4 and 2x6 Spruce-Pine-Fir NLT were tested. The general process followed for each entailed the following steps:

- Develop a preliminary resistance function for use in a single-degree-of-freedom analysis model.
- Perform testing to investigate the post-peak response of an individual mass timber panel to a quasi-static, uniformly-applied, out-of-plane load.
- Compare the results of the quasi-static testing with the preliminary resistance function to refine the preliminary resistance function.
- Use this refined resistance function to design test articles for blast demonstration tests.
- Perform blast demonstration tests and document the results of this testing.

The quasi-static panel testing was performed at the University of Maine in Orono for both CLT and NLT panels. The NLT blast demonstration testing was performed at BakerRisk's shock tube facility in La Vernia, Texas, and the CLT blast demonstration testing was performed at Tyndall Air Force Base in Panama City, Florida.

Based on the results of these tests, the following general conclusions are made:

- Mass timber structural systems can effectively resist blast loads in the elastic range with little noticeable damage. Due to the relatively high strength and low stiffness of mass timber panels, significant blast loads can be resisted by mass timber panels in the elastic response range.

- The post-peak response of mass timber panels is relatively brittle. However, for CLT systems, the presence of multiple plies allows for measurable residual strength following initial panel rupture. Additionally, the two-way action inherent in CLT provides a means for load distribution across the panel, thus limiting the damage at the location of peak applied load. NLT systems do not have this advantage of cross lamination and thus do not exhibit these post-peak response benefits.

- Provided fastener penetration is of sufficient depth, significant blast loads can be resisted and transferred through CLT connections that are both simple and quick to install. An added benefit is that dowel-type connection limit states associated with CLT construction are often ductile in nature due to the propensity for wood to crush and/or steel to yield when loaded in shear beyond their respective elastic limits.

- The results of the blast demonstration testing indicated that SDOF dynamic analysis can be used to approximate peak displacements of 3-ply CLT panels without openings within the elastic range. As such, based on CLT characteristic design values and SDOF dynamic analysis calculations, conventional construction standoff distances (CCSDs) for primary gathering / billeting facilities constructed with of CLT can be generated. These CCSDs are shown in the tables below assuming two different claddings and compared with other relevant CCSDs currently defined in UFC 4-010-01.

Conventional Construction Standoff Distances for 3-Ply CLT with EIFS Cladding.

Wall Type	Sections	Span	Min. Static Material Strength	EWI Standoff Distance	EWII Standoff Distance
Reinforced Concrete	≥ 6”	12’ – 20’	3,000 psi	66	16
Reinforced Masonry	8” – 12”	10’ – 14’	1,500 psi	86	30
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade E1**	**120**	**50**
Wood Studs – EIFS	2x4 & 2x6	8’ – 10’	875 psi	207	86
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade V4**	**250**	**95**
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade V1**	**250**	**100**
Steel Studs – EIFS	600S162-43; 600S162-54; 600S162-68	8’ – 12’	50,000 psi	361	151

Conventional Construction Standoff Distances for 3-Ply CLT with Brick Veneer Cladding.

Wall Type	Sections	Span	Min. Static Material Strength	EWI Standoff Distance	EWII Standoff Distance
Reinforced Concrete	≥ 6”	12’ – 20’	3,000 psi	66	16
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade E1**	**75**	**25**
Reinforced Masonry	8” – 12”	10’ – 14’	1,500 psi	86	30
Wood Studs – Brick Veneer	2x4 & 2x6	8’ – 10’	875 psi	105	36
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade V4**	**150**	**45**
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade V1**	**155**	**55**
Steel Studs – Brick Veneer	600S162-43; 600S162-54; 600S162-68	8’ – 12’	50,000 psi	187	75

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LIST OF ACRONYMS

AFB	Air Force Base
AFCEC	Air Force Civil Engineering Center
APA	The Engineered Wood Association
CCSD	Conventional Construction Standoff Distance
CFD	Computational Fluid Dynamics
CLT	Cross-Laminated Timber
DoD	Department of Defense
EOR	Engineer of Record
FE	Finite Element
HFPB	High-Fidelity Physics-Based
K-B	Kingery-Bulmash
MSR	Machine Stress Rated
NDS	National Design Specification for Wood Construction
NLT	Nail-Laminated Timber
PDC	Protective Design Center
PDC-TR	Protective Design Center Technical Report
SDOF	Single-Degree-of-Freedom
SOW	Statement of Work
SPF	Spruce-Pine-Fir
SST	Simpson Strong-Tie
UMaine	University of Maine
USACE	U.S. Army Corps of Engineers

CHAPTER 1

PROJECT OVERVIEW

1.1 BACKGROUND

Inhabited Department of Defense (DoD) buildings must be designed in accordance with the requirements set forth in UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings* [1]. UFC 4-010-01 contains prescriptive analysis assumptions (i.e., Table 2-3 of UFC 4-010-01) and "conventional construction" standoff distances (i.e., Table B-2 of UFC 4-010-01) for several types of construction that, if adhered to, release the engineer of record (EOR) from having to analyze individual exterior wall or roof structural components for airblast loads.

One of the types of construction that is not explicitly addressed by UFC 4-010-01 is mass timber construction such as cross-laminated timber (CLT) and nail-laminated timber (NLT). As such, mass timber structural systems must be analyzed for airblast loads if an EOR intends to use it as part of the exterior wall or roof structural system in an inhabited DoD building. This, coupled with the lack of testing that documents the out-of-plane behavior of mass timber panels and response to dynamic loads, limits the use of mass timber solutions in inhabited DoD buildings.

1.2 PROJECT OBJECTIVES

It follows that the primary objective of this effort was to perform testing that would demonstrate the capability of mass timber systems to resist airblast loads. To achieve this primary objective, the following project objectives were defined:

- To develop analytical methodologies to analyze mass timber panels for blast loads.
- To conduct static and dynamic testing on mass timber systems as a means to validate and/or improve these developed analytical methodologies.
- To document the developed analytical methodologies and obtained test data in a form that could serve as a reference for structural engineers interested in designing mass timber structural systems to resist blast loads.

It is hoped that by accomplishing these objectives, the blast-resistance barrier that limits the inclusion of mass timber structural systems in the DoD market will be removed.

1.3 PROJECT OVERVIEW

Two mass timber systems were investigated as part of this effort: cross-laminated timber (CLT) and nail-laminated timber (NLT). Grades V1, E1, and V4 CLT as well as 2x4 and 2x6 Spruce-Pine-Fir (SPF) NLT were tested. The general process followed entailed the following steps:

- Develop a preliminary resistance function for use in a single-degree-of-freedom (SDOF) analysis model.

- Perform testing to investigate the post-peak response of an individual mass timber panel to a quasi-static, uniformly-applied, out-of-plane load.

- Compare the results of the quasi-static testing with the preliminary resistance function to refine the preliminary resistance function.

- Use this refined resistance function to design test articles for blast demonstration tests.

- Perform blast demonstration tests and document the results of this testing.

The quasi-static panel testing was performed at the University of Maine in Orono for both CLT and NLT panels. The NLT blast demonstration testing was performed at BakerRisk's shock tube facility in La Vernia, Texas, and the CLT blast demonstration testing was performed at Tyndall Air Force Base in Panama City, Florida.

1.4 REPORT OUTLINE

The remainder of this final accomplishment report is divided into five chapters:

- Chapter 2 provides a summary of publicly-available literature relevant to the blast design of mass timber structural systems.

- Chapter 3 describes the process and methodology followed during this effort.

- Chapter 4 summarizes of the major testing efforts conducted during this effort.

- Chapter 5 summarizes the conclusions generated as a result of this effort.

CHAPTER 2

LITERATURE REVIEW

Designing mass timber structural systems for blast loads requires an understanding of how its constituent panels and its associated connections will respond to high-intensity, short-duration loading both at the material and member levels. Of particular relevance is the response of mass timber panels to out-of-plane forces at strain rates associated with blast loads. This chapter summarizes the relevant literature in these areas for CLT and NLT systems.

The sections below provide a summary of testing performed to investigate the effect of high strain rate on timber design, research specific to the out-of-plane static and dynamic response of CLT, and notes concerning the dynamic response of NLT.

2.1 WOOD HIGH STRAIN-RATE TESTING

Research investigating the effect of load duration on timber members has historically served to guide the load duration factor, C_D, specified in the *National Design Specification for Wood Construction* (NDS) [2]. Seminal efforts in this area include:

- Liska [3] performed flexural testing on 1”x1”x14” and 1”x2”x14” samples of two softwoods (i.e., spruce and douglas-fir) and two hardwoods (i.e., maple and birch) species at time-to-failures between 0.32 and 550 seconds. Flexural strength increases (at the proportional limit) of between 25 percent (softwoods) and 10 percent (hardwoods) were observed for the time-to-failures tested. No increase in the modulus of elasticity was observed.

- Gerhards [4] summarized the literature pertaining to rate and duration of load on wood and wood-based materials for bending, compression, tension, and shear force paths. Several of the conclusions made include: (1) duration of load is exponentially related to stress, (2) ultimate stress is exponentially related to loading, and (3) rate of loading has a greater effect on the strength of green wood than dry wood.

Most testing efforts investigating high strain-rate applications have involved impact loads. However, several test series have specifically investigated the response of timber systems to actual blast loads. Divine Buffalo 3, 12, 22, and 23 investigated wood stud wall retrofits for close-in and far-field blast loads [5][6][7][8]. Another test series investigated the response of single-story huts without windows to far-field blast loads [9].

Finally, shock tube tests have been used to investigate wood stud wall components. Notably, a series of tests applied shock-tube-generated blast loads to traditionally framed dimensional lumber walls to determine effective dynamic properties of wood construction [10]. This effort observed a marked increase in stud bending strength under dynamic loads (i.e., an average dynamic increase factor (based on modulus of rupture) of 1.41 for strain rates between 0.1 s^{-1} and 1 s^{-1} as compared to values obtained used a one-minute static load duration). Additionally, this effort indicated a smaller dynamic increase factor of 1.14 should be applied to the modulus of elasticity for the same strain rate range.

2.2 CROSS-LAMINATED TIMBER

Cross-laminated timber is an engineered wood panel that consists of several layers of dimensional lumber boards stacked in alternating directions that are bonded with structural adhesives and pressed. CLT is typically manufactured in 3-ply, 5-ply, and 7-ply widths. Two major grade classifications exist for CLT: (1) “E” or engineered wood that includes machine stress rated (MSR) lumber in the major strength direction and (2) “V” or visually-graded wood that uses only visually-graded wood in its layup.

The references specific to CLT are divided among two subsections. First, standards and their role in governing the structural design of CLT in the United States are summarized. Then an overview of the published research pertaining to the response of CLT panels to out-of-plane static and dynamic loads is documented.

2.2.1 Standards & References

2.2.1.1 ANSI/APA PRG 320-2012

ANSI/APA PRG 320-2012 *Standard for Performance Rated Cross-Laminated Timber* [11], hereafter referred to as PRG 320, provides dimensions and tolerances, performance requirements, test methods, quality assurance, and trademarking for CLT panels. In addition, Annex A of PRG 320 defines the layups for four engineered wood, “E”, grades and three visually-graded, “V”, grades and includes allowable design properties based on the shear analogy model for each grade.

2.2.1.2 2015 NDS

The 2015 edition of the NDS defines adjustment factors for CLT panels.

2.2.1.3 CLT HANDBOOK

While the *CLT Handbook* [12] is not a standard, it serves as a central repository of information related to CLT design and construction. Of particular value to the structural engineer is the static-load design guidance furnished for various load paths in Chapter 3.

2.2.2 Published Research

The published research directly relevant to the design of CLT systems for blast loading can be divided into two sections: (1) quasi-static testing of CLT panels exposed to out-of-plane loads and (2) dynamic testing of CLT components and systems. Details concerning a selection of these efforts are included in Table 2-1.

Table 2-1. CLT Literature Review Summary.

Reference	Specimen				Test Description
	No. of Tests	*Species*	*No. of Plies*	*Variables*	
Ceccotti et al. (2013)	50+	Spruce	3, 5	Joint/anchor assembly; input ground motion; size of openings; shear wall configurations	Connection tests; in-plane monotonic & cyclic wall tests; 1-story pseudo-dynamic; 3-story shake table; 7-story shake table; Connection response; system seismic response
Chen & Lam (2013)	50+	SPF	3, 4	Panel vs. box-based system; panel layup; box configuration	4-Point Bending (L/D > 10); Out-of-plane bending stiffness/strength
Czaderski et al. (2007)		Swiss Spruce	3	Concentrated load configuration	2-Way Simply Supported Spans; Out-of-plane bending strength/stiffness
Dujic et al. (2004, 2006, 2007, 2010)	29+	SPF	3	Wall geometry (w/ & w/o openings), fastener type, bracket type, loading type, vertical load	Monotonic (EN 594) & cyclic (ATC-24) in-plane shear wall testing; Shear wall stiffness/strength
Flatscher & Schickhofer (2011), Flatscher et al. (2013, 2014)	17+		3	Joint/anchor assembly; wall geometry & opening size	Shear tests, tension tests, & shear tests on step joints for connections; monotonic & cyclic in-plane shear wall testing; Connection response; shear wall stiffness/strength
Flores et al. (2015)	9	Chilean Radiata Pine	3		4-Point Bending (L/D > 10); in-plane shear; panel compression; Out-of-plane bending, in-plane shear/bending, and axial compression stiffness
Gavric et al. (2014)	40+	Spruce	5	Joint/anchor assembly	Monotonic & cyclic in-plane shear wall testing; connection response
Hindman & Bouldin (2014)	20+	Southern Pine	5		4-Point Bending (L/D > 10); 3-Point Bending (L/D < 10); AITC T107 Shear; AITC T110 Cyclic Delamination; Out-of-plane bending strength/stiffness, resistance to shear by compression loading, & delamination resistance
Hochreiner et al. (2014)			3,5		Various plate bending tests; Out-of-plane bending strength / stiffness
Oh et al. (2014)	34	Korean Larch	3		Stub Panel Compression; compression strength
Popovski et al. (2010)	32	European Spruce	3	Wall geometry (w/ & w/o openings), fastener type, bracket type, loading type, vertical load	Monotonic (0.2 mm/s ramp loading rate) & cyclic (5 mm/s rate) in-plane shear wall testing; Shear wall stiffness/strength
Vessby et al. (2009)	6	Norway Spruce	5	Joint configuration	In-plane shear; in-plane shear/bending strength/stiffness
Zhou et al. (2014)		Black Spruce	3		3-point bending; out-of-plane bending strength / stiffness

2.2.2.1 OUT-OF-PLANE QUASI-STATIC TESTING

Four-point and three-point bending tests performed in accordance with ASTM D198 [13] or ASTM D4761 [14] are commonly used to assess the bending and shear strength, respectively, of CLT panels. Both one-way [15][16][17] and two-way [18] out-of-plane bending testing has been performed using concentrated loads. Details concerning each of these tests are provided in Table 2-1.

In addition to this testing, each CLT manufacturer must complete flatwise (i.e., that resist out-of-plane forces) testing to be recognized by The Engineered Wood Association (APA) or other third-party certification providers.

It should be noted that while there is a significant volume of testing documenting the response of CLT panels to concentrated loads, no known testing was found that documented the response of CLT panels to uniformly-applied quasi-static loads.

2.2.2.2 DYNAMIC TESTING

Most research investigating the response of CLT to dynamic loads has focused on resisting seismic loads. There are significant testing efforts devoted to investigating the in-plane shear response of CLT panels [19][20][21][22], its associated connections [23][24], and global response of CLT structures to ground shaking [25][26][27]. Details concerning each of these tests are provided in Table 2-1. Also, a thorough summary of seismic testing involving CLT was completed in 2014 [28].

However, dynamic tests investigating the flatwise response of CLT panels is limited. An effort was underway at the time of this project in which a shock tube was used to investigate the out-of-plane response of 3-ply, 5-ply, and 7-ply Grade E1 CLT panels to dynamic loads [29][30]. Also, a series of shock tube tests on CLT panels were also in progress at the University of Ottawa, although results have not yet been published.

2.3 NAIL-LAMINATED TIMBER

Nail-laminated timber is an engineered wood panel that consists of (generally) 2x dimensional lumber nailed together on their broad face with plywood or oriented strand board nailed to one surface. Limited research is specifically devoted to the performance of NLT. However, since NLT is essentially wood studs with no spacing, the research described in Section 2.1 of this chapter is directly relevant to NLT.

One aspect of NLT that is particularly important is the ability of its constituent dimensional lumber pieces to share load. This load sharing of wood construction has been well documented [31] for static loading conditions and is specified in the NDS via the repetitive member factor, C_r. According to the NDS, a C_r factor of 1.15 is applicable provided the wood members are "in contact or spaced not more than 24-inches on center, are not less than three in number and are joined by floor, roof or other load distributing elements adequate to support the design load". As NLT panels are comprised of sheathed dimensional lumber that is nailed together and thus meet these qualifications, the C_r factor appears to be applicable to NLT at first glance. However, it should be noted that the testing cited above does not consider dynamic loading scenarios, which may impact load-sharing phenomenology.

CHAPTER 3

METHODOLOGY

To accomplish the objectives stated in Chapter 1, the effort was divided into three major phases: (1) project planning, (2) quasi-static testing program, and (3) blast demonstration testing. Calculations ranging from simple SDOF to more complex high-fidelity physics-based (HFPB) nonlinear finite element (FE) were performed to assist in test planning and validate the analytical methodologies developed. The following subsections provide an overview of the activities conducted during each phase.

3.1 PHASE 1: PROJECT PLANNING

The initial project planning portion of this effort commenced in June 2015 and concluded in October 2015. The goal of this phase was to plan a path forward that would position mass timber construction as a type of construction that could be effectively and efficiently analyzed for blast resistance to the satisfaction of the Protective Design Center (PDC), the U.S. Army Corps of Engineers (USACE) body responsible for reviewing blast-related questions related to DoD infrastructure. To actively engage the PDC, several conference calls between the PDC and members of the project team were conducted during the months of August and September. In addition, an in-person meeting with the PDC's Chief and Chief of Hardened Structures took place on October 15 at the University of Maine (UMaine). Also in attendance at this meeting was Dr. Ghasan Doudak of the University of Ottawa, who was performing both quasi-static and dynamic testing (in a shock tube) on CLT specimens at the time.

A series of tasks were performed prior to this meeting to prepare a coherent project plan such that the PDC could provide useful feedback. These tasks were: (1) conducting a literature review that focused on the research, testing, and standards devoted to quantifying the out-of-plane response of CLT, (2) performing preliminary scoping analyses to determine appropriate standoff distances for UFC 4-010-01 explosive threats, (3) generating a preliminary static test plan, and (4) generating a preliminary dynamic test plan in order to reach out to test ranges for ROM costs.

The primary outcomes of this meeting were as follows:

- Present and future testing efforts aimed at investigating the dynamic (out-of-plane) performance of CLT and its associated connections were shared to avoid effort duplication and promote synergy. Both the PDC and the University of Ottawa shared the status of their quasi-static and dynamic testing efforts on CLT panels while K&C and WoodWorks shared proposed quasi-static and live blast test plans. Test reports documenting the PDC's testing [29][30] were delivered to K&C and WoodWorks following this meeting. The University of Ottawa has not documented their testing in a publicly-releasable form at this time.

- The quasi-static test matrix was finalized. The test matrix included three CLT grades (i.e., E1, V1, and V4), two CLT panel thicknesses (i.e., 3-ply and 5-ply), two connection types (i.e., a prefabricated Simpson Strong-Tie (SST) bracket and a hot rolled steel angle bracket), and two types of NLT panels (i.e., 2x4 and 2x6 SPF).

- Parameters that should be included in the live blast demonstration testing for CLT were discussed. It was decided to pursue system-level blast demonstration tests rather than panel-only tests for CLT because shock tube testing was already being performed by the PDC and the University of Ottawa.

- Several items were identified as important for the blast-resistant design of mass timber systems but reserved for future testing efforts: (1) progressive collapse, (2) blast testing of 5-ply wall panels to failure, (3) openings, and (4) spline connections. Although openings and spline connections were planned for the live blast demonstration tests, it was thought that a test series specifically devoted to each topic would be necessary.

- The PDC indicated that response limits were the only item necessary to develop conventional construction parameters for mass timber construction.

3.2 PHASE 2: QUASI-STATIC LABORATORY TESTING

Based on the results the October 2015 meeting, the quasi-static laboratory testing program was finalized. This testing began in March 2016 and most of the testing was completed by the end of May 2016. (The Grade V4 CLT tests were performed in August 2017 once SmartLam had achieved APA certification for their Grade V4 panels.)

The purpose of this testing was to investigate the post-peak bending response of CLT and NLT panels in their major strength direction under a uniformly-applied quasi-static load. These tests were used to generate a resistance function capable of being used in SDOF dynamic analysis and to assist in generating a fit for a nonlinear timber material model.

A total of thirty-three panels were tested during this effort. Grade V1 (3-ply and 5-ply) (provided by DR Johnson), Grade E1 (provided by Nordic Structures), and Grade V4 (provided by SmartLam) CLT panels and 2x4 and 2x6 Spruce-Pine-Fir (SPF) NLT panels (provided by StructureCraft) were tested. A test matrix documenting the different configurations tested is included as Table 3-1.

Table 3-1. Quasi-Static Testing Specimen Test Matrix.

ID	Description	Ply No.	L [in]	W [in]	Qty	Bracket Type	Horz. Leg Fasteners	Vert. Leg Fasteners
V1	Gr. V1 CLT	3	126	48	5	No Connection		
5V1	Gr. V1 CLT	5	126	48	5	No Connection		
V1CA	Gr. V1 CLT	3	114	48	4	ABR105 (Simpson)	(14) SD10212 (Simpson)	(10) SD10212 (Simpson)
V1CB	Gr. V1 CLT	3	114	48	3	L4x4x1/4	(6) ASSY SK 5/16x4 (MyTiCon)	(6) ASSY SK 5/16x4 (MyTiCon)
E1	Gr. E1 CLT	3	126	48	4	No Connection		
V4	Gr. V4 CLT	3	126	48	4	No Connection		
4NLT	2x4 NLT	N/A	126	48	4	No Connection		
6NLT	2x6 NLT	N/A	126	48	4	No Connection		

The apparatus utilized for the testing was developed by UMaine and consisted of a series of rubber bladders to be filled with water capable of applying a uniform quasi-static pressure in a controlled fashion. Applied pressure, out-of-plane deflection, and total resisted load were measured and recorded as panels were displaced well beyond the deflection associated with peak panel strength. A pre-test panel photograph is included in Figure 3-1.

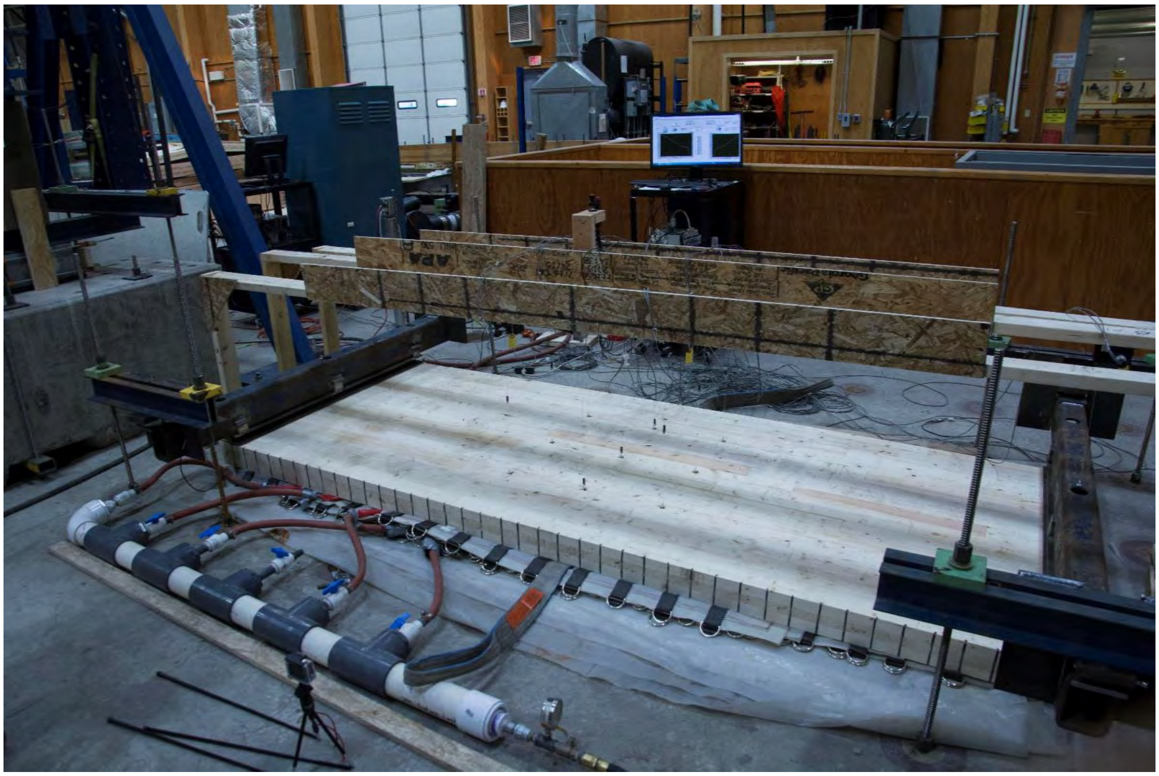
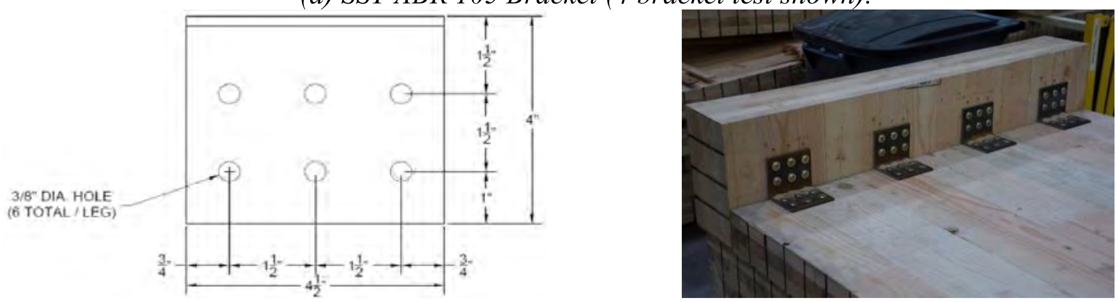


Figure 3-1. Pre-Test Photograph of Panel Test (w/o Connections).

While most of the panels were tested with end conditions that did not restrain panel rotation, six 3-ply Grade V1 CLT panels were tested with connections meant to represent those that might be used to attach a wall to a floor and ceiling in a building designed to resist significant out-of-plane wall loading. Two types of angle brackets were used (Figure 3-2): (a) 11-gauge SST ABR105 brackets and (b) 4.5-inch lengths of pre-drilled ASTM A36 L4×4×1/4 angle. The SST brackets were secured using SD10212 (i.e., #10 x 2-1/2") self-tapping screws also manufactured by SST and the 4"x4"x1/4" steel angle brackets (L4x4x1/4) were secured using ASSY SK 5/16x4 self-drilling screws manufactured by MyTiCon. The number of angle brackets was varied between two and four between tests to ascertain the impact of the number of angle brackets on the ultimate flexural resistance of the panels.

The complete testing report in Appendix B documents the test setup details for this testing program.

(a) SST ABR 105 Bracket (4 bracket test shown).



(b) L4x4x1/4 Bracket (4 bracket test shown).

Figure 3-2. Connection Types.

3.3 PHASE 3: LIVE BLAST DEMONSTRATION TESTING

3.3.1 NLT Shock Tube Testing

Following the quasi-static testing, a small number of shock tube tests on NLT panels at BakerRisk’s shock tube facility in La Vernia, Texas were performed. This testing:

1. Evaluated the ability of the SBEDS [32] wood beam module to predict peak NLT displacement across a range of displacement ductility (μ) values.

2. Demonstrated the ability of NLT to resist blast loads.

A total of six panels were tested during this effort – three No. 2 or better 2x4 SPF and three No. 2 or better 2x6 SPF. The panels were manufactured by StructureCraft, were 4-feet wide by 10-feet high, and had ½-inch thick plywood nailed to the compression face of the specimen. Single-degree-of-freedom dynamic analyses were performed using SBEDS to determine a reflected pressure, P_r, and reflected impulse, i_r, combination (pressure was held constant) needed to meet target displacement ductility values, μ_{target}. These blast loads are included in the test matrix shown in Table 3-2.

The complete testing report in Appendix C documents the test setup details for this testing program.

Table 3-2. NLT Shock Tube Testing Specimen Test Matrix.

Specimen	Specimen Description	μ_{target}	P_r [psi]	i_r [psi-ms]
1	2x4 NLT	1.00	15.0	75
2	2x4 NLT	0.75	15.0	60
3	2x4 NLT	1.25	15.0	100
4	2x6 NLT	0.75	15.0	80
5	2x6 NLT	1.00	15.0	120
6	2x6 NLT	1.25	15.0	200

3.3.2 CLT Open-Air Blast Testing

Preparation for the live blast testing on CLT construction began in the fall of 2015. K&C engaged the Air Force Civil Engineer Center (AFCEC) based at Tyndall Air Force Base (AFB) in Panama City, Florida, by generating a statement of work to discuss the potential for partnering in such testing.

A series of live blast tests was planned for three two-story, single-bay CLT structures at Tyndall AFB. These structures are shown in Figure 3-3. The structures, included anchorage to an existing concrete slab, were constructed in full over a period of eight days by Lend Lease. Each structure was constructed using a different grade of CLT (i.e., grade designations V1, E1, and V4 provided by DR Johnson, Nordic Structures, and SmartLam, respectively) and included window and door openings consistent with an actual building. Actual doors were provided by American Direct and manufactured by Ambico. Self-tapping screws (provided by MyTiCon) and adhesive anchors (provided by Hilti) were utilized in concert with steel angles to connect the constituent panels of each structure.

Figure 3-3. CLT Test Structures.

Three shots were performed to demonstrate the effectiveness of CLT over a spectrum of blast loads. The first two shots were designed to stress the CLT structures within their respective elastic limits. The third shot was designed to push the structures beyond their elastic limits such that post-peak response could be observed. Reflected pressure and peak deflections were recorded at front, side, and roof faces using a total of sixty-two gages to thoroughly document the response of the structure in time.

Prior to formalizing this test plan, the project team traveled to Omaha, Nebraska, to meet with the PDC a second time, present a draft test plan, and solicit their input. Based on the results of this meeting, the test plan was formalized and final structure design commenced.

The structures were designed using SDOF dynamic analysis models and connection detailing was based on the forces computed for Test 2. Information obtained via the quasi-static testing program and previous shock tube testing performed by the PDC [29][30] was used to inform the resistance function ultimately used for SDOF dynamic analysis. Based on the results of these analyses, final construction drawings were created.

To verify the SDOF design methodology, a series of HFPB models were created:

- Nonlinear FE model of panels from quasi-static test program. This model was created to generate a fit that could be used with LS-DYNA's wood material model (i.e., MAT 143). Because the quasi-static testing indicated that CLT panel rupture generally occurred near a finger joint, it was important that the details of the CLT panel were explicitly modeled. Thus, individual boards, the gap between boards, and finger joint locations were included in the final model. A screenshot from this analysis is included as Figure 3-4. More details concerning these analyses and of the models created can be found in the pre-test briefing included as Appendix E.

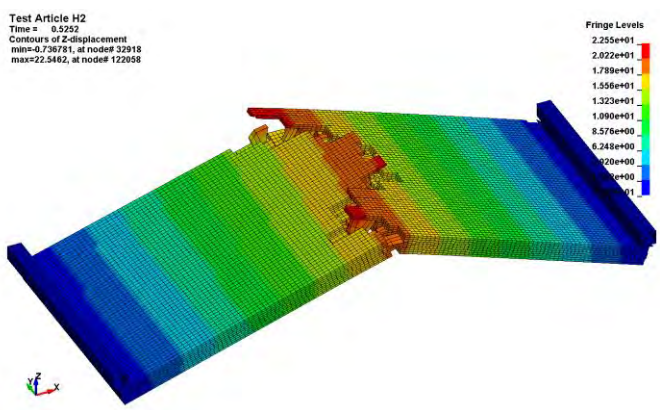


Figure 3-4. FE Model of CLT Panel from Quasi-Static Testing.

- Computational fluid dynamics (CFD) for test structure configuration. Due to the finite surface area of the existing reinforced concrete slab upon which the CLT structures were constructed, the structures were necessarily close to each other (i.e., within 11 feet at their closest point). Thus, a CFD analysis was performed to ensure there would be minimal shock wave reflections on adjacent structures. A screenshot from this analysis is included as Figure 3-5.

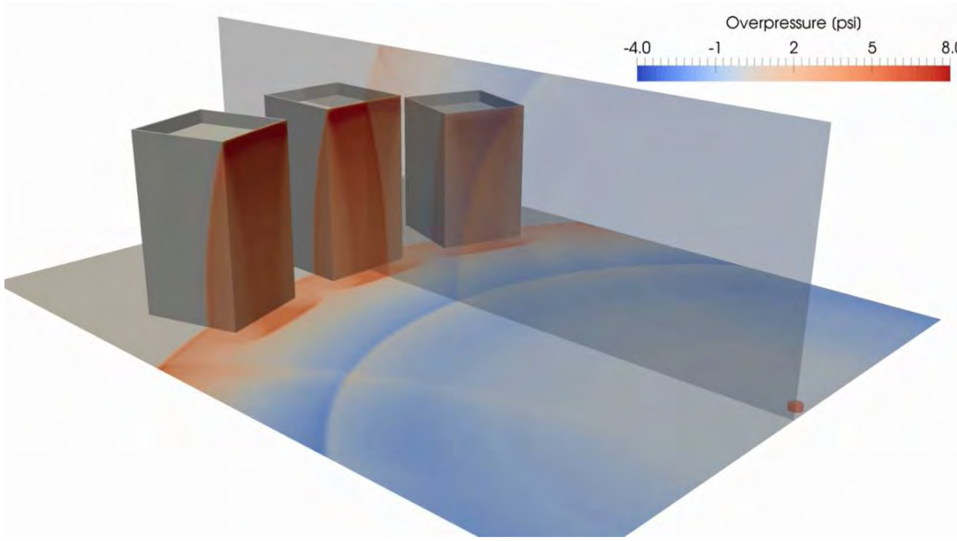


Figure 3-5. CFD Analysis Screenshot.

- Nonlinear finite element analysis of Grade E1 test structure for Shots 2 and 3. These analyses were conducted to verify the forces in the existing slab anchorage would not exceed their capacity. The material model fit generated from the panel FE model was used in this model. To limit the size of the model, a half-symmetry model was constructed. (It should be noted that the structure is not truly symmetrical as a result of the door and window openings.) The final FE model constructed is shown in Figure 3-6.

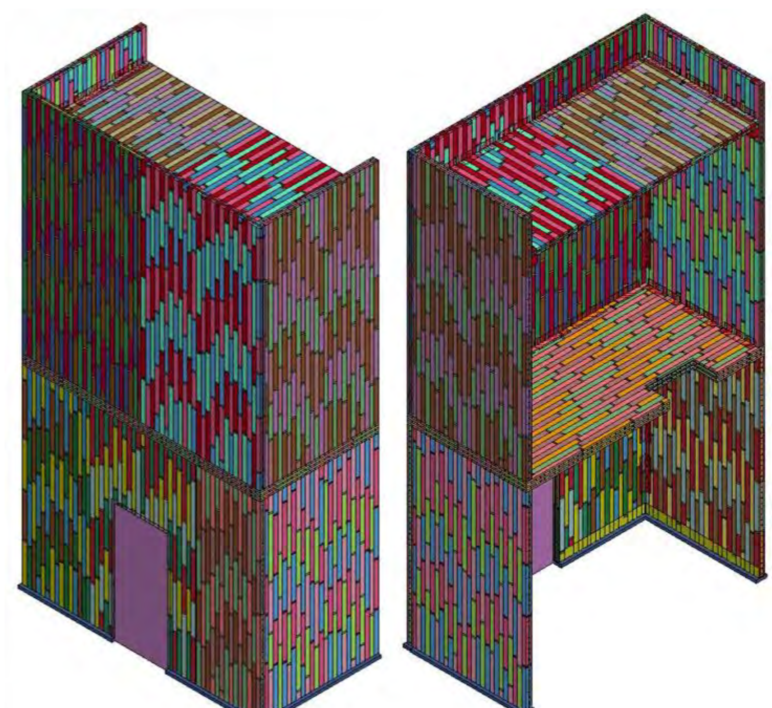


Figure 3-6. FE Model of Grade E1 Test Structure (Half-Symmetry).

Once the test structures were designed and constructed, final preparations for the blast testing commenced. A pre-test briefing was delivered prior to the first shot. This briefing is included as Appendix E.

Further details concerning these tests, including test structure drawings, instrumentation plan, and charge sizes is included in Appendix F.

CHAPTER 4

RESULTS / DISCUSSIONS / FINDINGS

This chapter provides an overview of the results generated during the testing programs completed as a part of this effort. The results are divided between the three testing programs (e.g., quasi-static laboratory panel testing on CLT and NLT panels, shock tube testing on NLT panels, and live blast testing on CLT structures) executed as part of this effort.

4.1 QUASI-STATIC LABORATORY TESTING

Complete results from the quasi-static testing are included in the final test report in Appendix B. The subsections that follow summarize these results and their applicability to the subsequent dynamic tests performed under this effort.

4.1.1 Results Overview

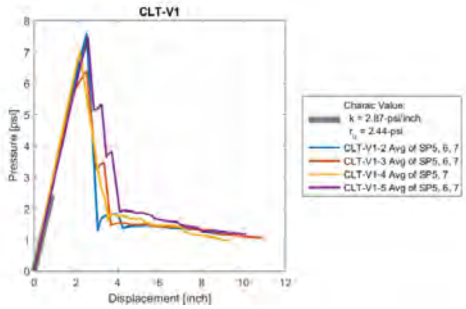
Panel tests without connections typically failed due to rupture of the outermost tension ply at finger joints, knots, or sloped grain. The typical failure pattern observed for a CLT and NLT panel is shown in Figure 4-1.

(a) CLT Panel.

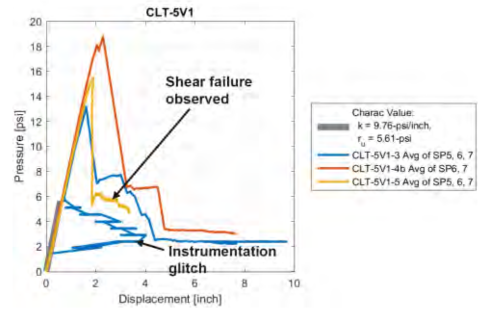
(b) NLT Panel.

Figure 4-1. Photo of Typical 3-ply CLT Panel at End of Test.

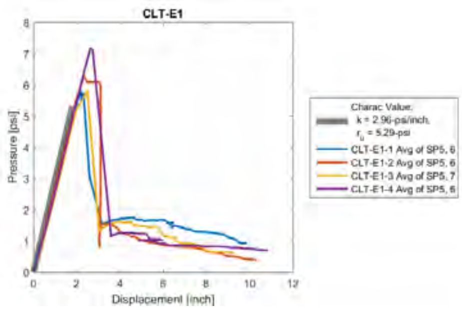
Also, load-displacement plots derived from each type of panel without connections are shown in Figure 4-2.



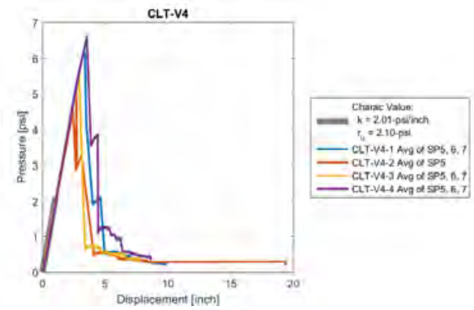
(a) 3-Ply Grade V1.



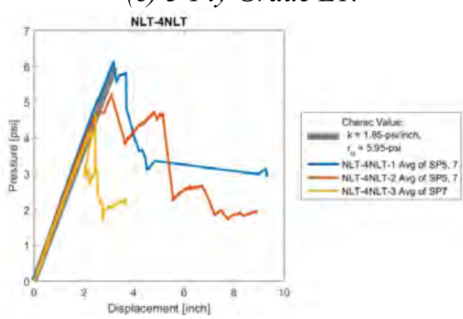
(b) 5-Ply Grade V1.



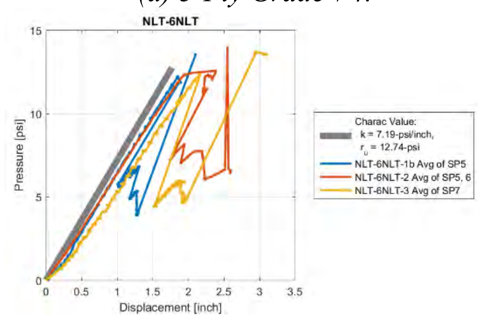
(c) 3-Ply Grade E1.



(d) 3-Ply Grade V4.



(e) 2x4 NLT.



(f) 2x6 NLT.

Figure 4-2. Load-Displacement Plot Results.

4.1.2 CLT Conclusions

The quasi-static laboratory testing generated the following observations and conclusions that were used for the CLT live blast test planning:

- When CLT panels ruptured due to flexure, negligible shear slip between panel plies away from the location of panel rupture was observed. This observation lends credence to a fully-composite panel, at the core of the shear analogy method.

- The shear analogy method can be employed with the characteristic values show in Table 1 of PRG 320 to faithfully reproduce the observed elastic bending stiffness for the grades and ply numbers tested. Figure 4-2 shows this computed stiffness as a dark gray line. As can be observed from Figure 4-2, CLT panel response was essentially linear elastic prior to panel rupture.

- The shear analogy method, with the 0.85 reduction factor included in PRG 320, can be used with the characteristic values shown in Table 1 of PRG 320 to generate a lower-bound panel strength for the grades and ply numbers tested. For the grade with MSR lumber in the major strength direction (i.e., Grade E1), the computed strength and average tested strength were within roughly 20-percent of each other. On the other hand, the grades with only visually graded lumber (i.e., Grades V1 and V4) had a tested strength almost three times of that computed using characteristic values.

- Upon panel rupture, there is a relatively sudden drop in panel strength to a residual panel strength plateau. The value of this residual strength plateau always exceeded the strength computed assuming the shear analogy model and ignoring the ruptured ply. For example, the residual strength of a 3-ply after rupture of the tension lamination would typically be calculated as the bending strength of a single ply.

- Panels that are not continuously supported are susceptible to top board disengagement at high deformations.

- Fastener length and the corresponding number of plies that are engaged can impact the ultimate failure mode observed. Although more testing would be needed to corroborate this conclusion, it appears where the fasteners were long enough to engage all panel plies, the fasteners served to act as shear reinforcement and resist the augmented shearing forces associated with discrete support points.

4.1.3 NLT Conclusions

The quasi-static laboratory testing generated the following observations and conclusions.

- The NLT panels failed in flexure near panel mid-span at values very close to that predicted by SBEDS. Typically, one stud would rupture first and then its neighbors would fail in a zippering fashion. This response led to highly unbalanced panels where one half was nearly intact and showed minimal deflection while the other half would be completely ruptured and have a mid-span deflection upwards of 10 inches. This result is in direct contrast with the failure observed in CLT panels, which was not localized in one portion of the panel. This observation indicates there is not a high distribution of load in NLT panels as compared to CLT panels.

- The average NLT bending stiffness and strength values obtained via testing are consistent with values computed using the NDS that ignore the impact of the plywood sheathing. Thus, the dimensional lumber model used in SBEDS was deemed a good approximation for a starting resistance function.

4.2 NLT SHOCK TUBE TESTING

Complete results from the NLT shock tube testing are included in the final test report in Appendix C. Additionally, a briefing given at the 2016 Shock and Vibration Exchange concerning these tests is included as Appendix D.

4.2.1 Results Overview

A summary of the shock tube testing results is included as Table 4-1. Also, photographs of the panels following each test is included as Figure 4-3.

Table 4-1. NLT Shock Tube Testing Results Summary.

ID	Description	Target Load		Pre-Test Computed[3]			Recorded				Post-Test Computed[3]	
		p_r	i_r	μ	Δ	R_{flex}	p_r	i_r	Δ	R_{test}	μ	Δ
		[psi]	[psi-ms]	-	[in]	[lb/in]	[psi]	[psi-ms]	[in]	[lb/in]	-	[in]
1	2x4 NLT	15.0	75	1.00	3.72	460	15.4	81	4.00	500	1.11	4.14
2			50	0.68	2.53		12.5	60	3.30	340	0.88	3.17
3			100	1.29	4.82		14.7	93	20+	520	1.17	4.37
4	2x6 NLT		75	0.72	1.48	990	13.8	77	1.78	790	0.63	1.30
5			120	0.99	2.03		14.6	128	2.46	930	0.93	1.92
6			200	1.24	2.55		14.4	199	2.74	1080	1.11	2.29

1. Panels were 8'(W) x 10'(L) with the 2x members running in the 10' direction. No. 2 or better Spruce Pine Fir was used for the 2x members. 1/2" plywood was nailed to the compression face of the NLT specimens.
2. Variable definitions
   - p_r = average reflected pressure.
   - i_r = peak reflected impulse; impulse measured up to time pressure first transitions from positive to negative.
   - μ = computed displacement ductility.
   - Δ = peak out-of-plane panel deflection.
   - R_{flex} = peak reactions based on ultimate flexural resistance from SBEDS output.
   - R_{test} = peak reactions based on summing the data obtained via three load cells placed at one end of the panel and dividing by the panel width.
3. Computed using wood beam model included in SBEDS v5.1. Span for analysis was idealized a simple span of 9.625' based on the boundary conditions associated with the test. Plywood was assumed to only act as supported weight (i.e., it was ignored for purpose of computing ultimate resistance and stiffness).

(a) Test 2: 2x4 NLT
μ_{target} = 0.75.

(b) Test 1: 2x4 NLT
μ_{target} = 1.00.

(c) Test 3: 2x4 NLT
μ_{target} = 1.25.

(d) Test 4: 2x6 NLT
μ_{target} = 0.75.

(e) Test 5: 2x6 NLT
μ_{target} = 1.00.

(f) Test 6: 2x6 NLT
μ_{target} = 1.25.

Figure 4-3. Post-Test Photographs of NLT Shock Tube Testing.

4.2.2 Conclusions

The following general conclusions were made from these tests:

- Significantly different qualitative responses were observed between the 2x4 and 2x6 NLT panels for the same target displacement ductility. This result is most clearly illustrated when target ductility was set equal to 1.25 (i.e., tests 3 and 6) where the 2x4 NLT panel completely was blown out while the 2x6 NLT panel simply exhibited minimal rupture and remained completely supported by the frame. This result calls into question the use of ductility as the sole factor in dictating response limit.

- Blow out occurred at a ductility response limit of 1.25 for the 2x4 NLT. As the blowout response limit for non-load bearing wood studs is displacement ductility of 4 according to PDC-TR 06-08 [33], these shock tube tests indicate that the ductility response limits for wood may require modification but more testing would be required to verify this.

- The analytical model for 2x6 NLT in SBEDS consistently underpredicted (by as much as 30 percent) the peak response observed in the shock tube testing. As these panels generally exhibited elastic response (except for Test 6), it is possible that the either the computed panel stiffness was underestimated or the assumed panel weight was overestimated.

4.3 CLT LIVE BLAST TESTING

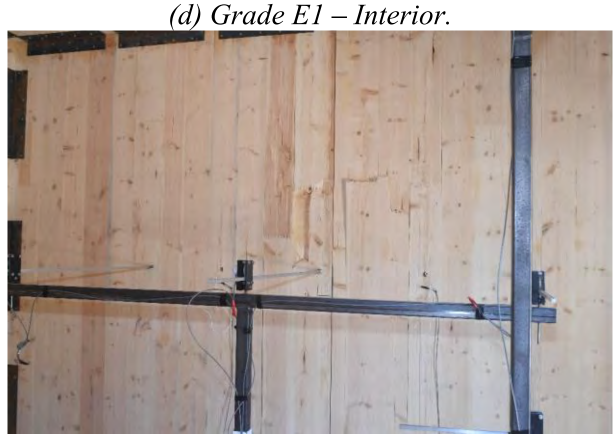
Complete results from the CLT live blast testing are included in the final test report in Appendix F. Additionally, a briefing given prior to Shot 1 is included as Appendix E of this report.

4.3.1 Results Overview

The tests proceeded as planned and the CLT structures responded as expected. For the first two tests, peak recorded deflections were consistent with pre-test predictions indicating the efficacy of the design assumptions and methodology in predicting elastic response of CLT to dynamic loads. Furthermore, results from the third test indicated a controlled response in which localized panel rupture was observed but connection integrity and load carrying ability were not compromised for each of the three structures tested. Photographs showing the front and back face of the first-floor panel that ruptured in all three test structures is included as Figure 4-4.

(a) Grade V1 – Exterior. *(b) Grade V1 – Interior.*

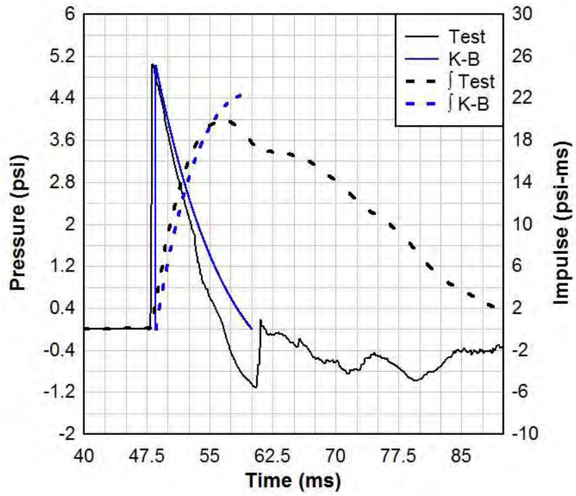
(c) Grade E1 – Exterior. *(d) Grade E1 – Interior.*



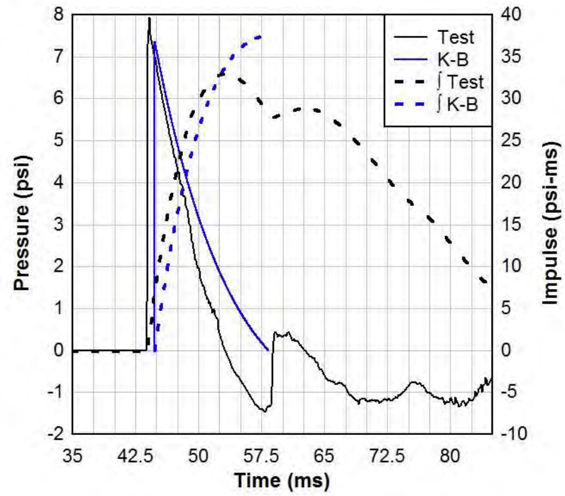
(e) Grade V4 – Exterior. *(f) Grade V4 – Interior.*

Figure 4-4. Shot 3 Post-Test Photographs of 1st Floor Front Panel.

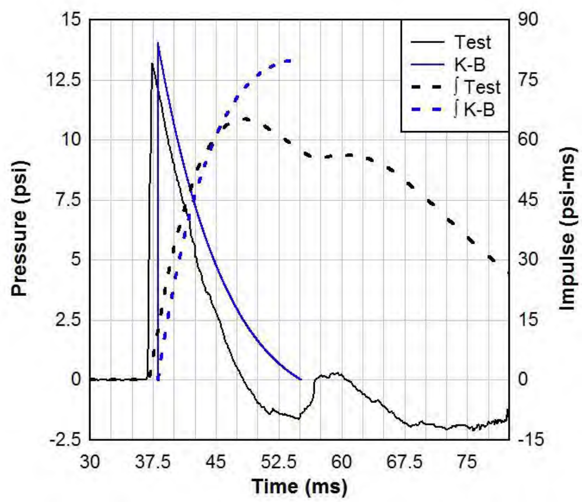
Measured pressure and displacement data generally compared well those computed using the Kingery-Bulmash (K-B) equations (pressure) and SDOF dynamic analysis (displacement). Comparisons for the first-floor front panel are shown in Figure 4-5 for pressure and in Figure 4-6 for displacement. Other pressure and displacement comparisons are included in the final test report.



(a) Shot 1.

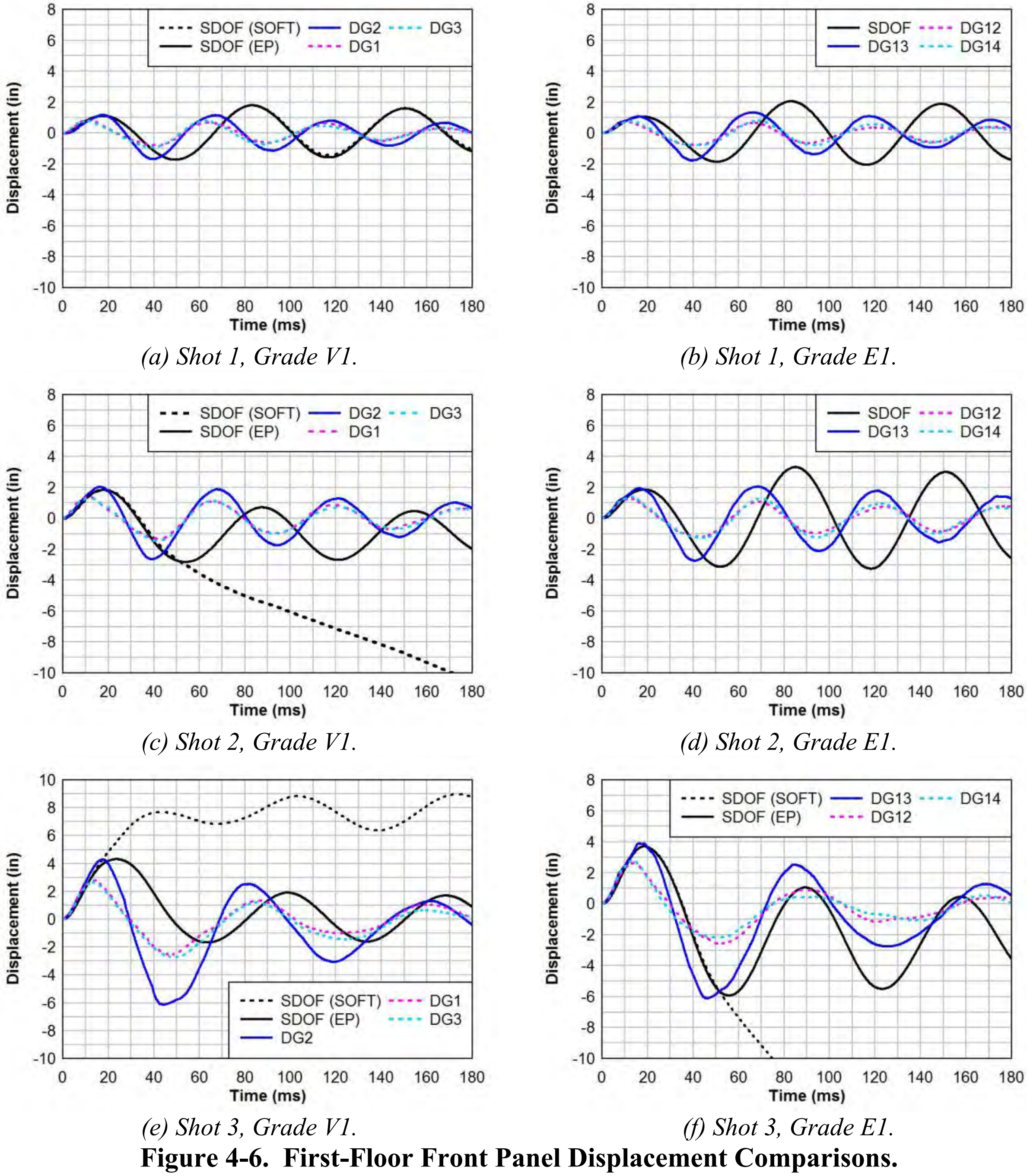


(b) Shot 2.



(c) Shot 3.

Figure 4-5. Reflected Pressure Data Comparisons at First-Floor Front Panels.



(a) *Shot 1, Grade V1.*

(b) *Shot 1, Grade E1.*

(c) *Shot 2, Grade V1.*

(d) *Shot 2, Grade E1.*

(e) *Shot 3, Grade V1.*

(f) *Shot 3, Grade E1.*

Figure 4-6. First-Floor Front Panel Displacement Comparisons.

4.3.2 Conclusions

Based on the results of this testing effort, the following general conclusions can be made:

- The rebound response of CLT often controls over its inbound response, thus underlying the importance of considering the negative phase of the airblast loading when designing CLT components and systems for airblast loading.

- Visually graded CLT panels demonstrate significantly greater out-of-plane bending strength than that associated with the characteristic values defined in PRG 320.

- Localized CLT panel rupture can be sustained without adverse consequences to the CLT system's connections and load carrying ability. Further testing can be used to investigate the impact of localized CLT panel rupture for different conditions (e.g., different in-plane axial loads, different connection configurations, etc.).

- An SDOF dynamic analysis can be used to approximate peak displacements in 3-ply CLT panels without openings provided the mean out-of-plane strength of the CLT panel can be approximated.

- An SDOF dynamic analysis is not well-suited to approximate peak displacements in CLT panels with openings. A more refined analytical model with more degrees of freedom is necessary to approximate peak displacements in these circumstances.

- The minor strength direction bending strength values for 3-ply CLT panels in Annex A of PRG 320 may be too conservative from an ultimate response perspective. Further testing to justify more representative peak bending strengths in the minor strength direction may allow for airblast-loaded structures to be designed more economically.

CHAPTER 5

SUMMARY / CONCLUSIONS / RECOMMENDATIONS

5.1 SUMMARY & CONCLUSIONS

UFC 4-010-01 requires that inhabited DoD buildings constructed of mass timber structural systems be analyzed for airblast loads. In summer of 2015, there was a lack of test data documenting mass timber system response under the strain rates imposed by airblast loads. Thus, the primary objective of this effort, to perform a series of tests to demonstrate the capability of mass timber systems to resist airblast loads, was achieved. The project succeeded in:

- Developing a preliminary resistance function capable for use in SDOF dynamic analysis.
- Performing testing to investigate the post-peak response of an individual mass timber panel to a quasi-static, uniformly-applied, out-of-plane load.
- Comparing the results of the quasi-static testing with the preliminary resistance function to refine the preliminary resistance function.
- Using this refined resistance function to design test articles for blast demonstration tests
- Performing demonstration tests and document the results of this testing.

Based on the results of these tests, the following general conclusions can be made:

- Mass timber structural systems can effectively resist blast loads in the elastic range with little noticeable damage. Due to the relatively high strength and low stiffness of mass timber panels, significant blast loads can be resisted by mass timber panels in the elastic response range.
- The post-peak response of mass timber panels is relatively brittle. However, for CLT systems, the presence of multiple plies allows for measurable residual strength following initial panel rupture. Additionally, the two-way action inherent in CLT provides a means for load distribution across the panel, thus limiting the damage at the location of peak applied load. NLT systems do not have this advantage of cross lamination and thus do not exhibit these post-peak response benefits.
- Provided fastener penetration is of sufficient depth, significant blast loads can be resisted and transferred through CLT connections that are both simple and quick to install. An added benefit is that dowel-type connection limit states associated with CLT construction are often ductile in nature due to the propensity for wood to crush and/or steel to yield when loaded in shear beyond their respective elastic limits.
- The results of the blast demonstration testing indicated that SDOF dynamic analysis can be used to approximate peak displacements of 3-ply CLT panels without openings within

the elastic range. As such, based on CLT characteristic design values and SDOF dynamic analysis calculations, conventional construction standoff distances (CCSDs) for primary gathering / billeting facilities constructed with of CLT can be generated. These CCSDs are shown in the tables below assuming two different claddings and compared with other relevant CCSDs currently defined in UFC 4-010-01.

Table 5-1. 3-Ply CLT with EIFS Cladding CCSD Comparison.

Wall Type	Sections	Span	Min. Static Material Strength	EWI Standoff Distance	EWII Standoff Distance
Reinforced Concrete	≥ 6”	12’ – 20’	3,000 psi	66	16
Reinforced Masonry	8” – 12”	10’ – 14’	1,500 psi	86	30
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade E1**	**120**	**50**
Wood Studs – EIFS	2x4 & 2x6	8’ – 10’	875 psi	207	86
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade V4**	**250**	**95**
CLT – EIFS	**3-ply**	**10’ – 12’**	**Grade V1**	**250**	**100**
Steel Studs – EIFS	600S162-43; 600S162-54; 600S162-68	8’ – 12’	50,000 psi	361	151

Table 5-2. 3-Ply CLT with Brick Veneer Cladding CCSD Comparison.

Wall Type	Sections	Span	Min. Static Material Strength	EWI Standoff Distance	EWII Standoff Distance
Reinforced Concrete	≥ 6”	12’ – 20’	3,000 psi	66	16
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade E1**	**75**	**25**
Reinforced Masonry	8” – 12”	10’ – 14’	1,500 psi	86	30
Wood Studs – Brick Veneer	2x4 & 2x6	8’ – 10’	875 psi	105	36
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade V4**	**150**	**45**
CLT – Brick Veneer	**3-ply**	**10’ – 12’**	**Grade V1**	**155**	**55**
Steel Studs – Brick Veneer	600S162-43; 600S162-54; 600S162-68	8’ – 12’	50,000 psi	187	75

It should be noted that the CCSDs shown in Table 5-1 and Table 5-2 are based on characteristic, or 5-percent exclusion, out-of-plane panel strengths. Both the quasi-static panel and live blast demonstration testing indicated that visually graded CLT panels have significantly higher out-of-plane strengths than their 5-percent exclusion values. Additionally, Table 5-1 and Table 5-2 assume the CLT panels remain elastic. The live blast demonstration tests indicated that CLT panels can exhibit rupture and still pose negligible threat to building inhabitants. Thus, it is expected that the CCSDs shown in Table 5-1 and Table 5-2 have the potential to be reduced.

5.2 NEXT STEPS

At the meeting with the PDC in Omaha in July 2016, the PDC indicated the importance of having test data documenting the response of axially-loaded mass timber, and specifically CLT, systems to blast loads. An effort is currently underway to reuse the CLT test structures created during this effort to test CLT construction under axial load. This effort is currently scheduled to be completed in 2017.

In addition, K&C is currently working with the PDC to develop a Protective Design Center Technical Report (PDC-TR). A meeting was held in Omaha in January 2017 to review a first draft of this PDC-TR. This PDC-TR will offer design guidance to allow engineers to design CLT systems to resist blast loads. Information such as methods to compute panel stiffness and panel strength, response limits, and connection detailing requirements will be included in this PDC-TR. The PDC-TR is scheduled to be completed in 2017 following the completion of the blast testing on axially-loaded CLT structures.

APPENDIX A

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APPENDIX B

QUASI-STATIC TESTING FINAL REPORT

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TR-16-42.1

QUASI-STATIC OUT-OF-PLANE TESTING OF CLT AND NLT PANELS

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CHAPTER 1

INTRODUCTION

The University of Maine (UMaine) in conjunction with WoodWorks and Karagozian & Case, Inc. (K&C) performed a testing program aimed at investigating the bending response of cross-laminated timber (CLT) and nail laminated timber (NLT) panels in their major strength direction under a uniformly-applied quasi-static load. The apparatus utilized for the testing was developed by UMaine and consisted of a series of rubber bladders filled with water capable of applying a uniform quasi-static pressure in a controlled fashion. Applied pressure, out-of-plane deflection, and total resisted load were measured and recorded as panels were displaced well beyond the deflection associated with peak panel strength. While most of the panels were tested with end conditions that did not restrain panel rotation, six were tested with connections meant to represent those that might be used to attach a wall to a floor and ceiling in a building designed to resist significant out-of-plane wall loading. Panels following testing are shown in Figure 1-1.

Figure 1-1. Panels Following Testing.

CHAPTER 2

TEST SPECIMENS

A total of 25 CLT and 6 NLT panels were tested during this effort. In general, the panels were 10’-6” long (major strength direction) and 4’-0” wide (minor strength direction), although panels that were tested with angle bracket connections were only 9’-6” in length. Different panel and connection types were tested with between three and five specimens being tested for each configuration. A test matrix documenting the different configurations tested is included as Table 2-1. Additionally, attributes of each panel tested, including panel weight, moisture content at time of testing, connection type, test date, and miscellaneous notes, are recorded in Table 2-2.

Table 2-1. Specimen Test Matrix.

ID	Description	Ply No.	L [in]	W [in]	Qty	Bracket Type	Horz. Leg Fasteners	Vert. Leg Fasteners
V1	Gr. V1 CLT	3	126	48	5	No Connection		
5V1	Gr. V1 CLT	5	126	48	5	No Connection		
V1CA	Gr. V1 CLT	3	114	48	4	ABR105 (Simpson)	(14) SD10212 (Simpson)	(10) SD10212 (Simpson)
V1CB	Gr. V1 CLT	3	114	48	3	L4x4x1/4	(6) ASSY SK 5/16x4 (MyTiCon)	(6) ASSY SK 5/16x4 (MyTiCon)
E1	Gr. E1 CLT	3	126	48	4	No Connection		
V4	Gr. V4 CLT	3	126	48	4	No Connection		
4NLT	2x4 NLT	N/A	126	48	4	No Connection		
6NLT	2x6 NLT	N/A	126	48	4	No Connection		

Table 2-2. Attributes of Individual Panels.

Panel ID	Panel Type	Weight[1] (lb)	Average MC[2] (%)	End Cond. Type[3]	Test Date	Notes
V1-1	Gr. V1, 3-ply CLT	500	13.5	S	3/1/16	Shakedown test for instrumentation
V1-2	Gr. V1, 3-ply CLT	490	14.5	S	3/9/16	Tested twice (hose detached)
V1-3	Gr. V1, 3-ply CLT	490	13.3	S	3/9/16	Small water bag rupture
V1-4	Gr. V1, 3-ply CLT	490	14.0	S	3/9/16	
V1-5	Gr. V1, 3-ply CLT	470	12.2	S	3/10/16	
5V1-1	Gr. V1, 5-ply CLT	780	17.4	S	3/11/16	Multiple tests (end rotation); stopped early due to bag leak
5V1-2	Gr. V1, 5-ply CLT	740	N/A	S	3/11/16	Bladder failed post-peak
5V1-3	Gr. V1, 5-ply CLT	920	11.5	S	4/19/16	
5V1-4	Gr. V1, 5-ply CLT	880	11.7	S	5/3/16	Tested twice (crossbar yield)
5V1-5	Gr. V1, 5-ply CLT	740	8.1	S	5/3/16	Stopped early due to bag leak
V1CA-1	Gr. V1, 3-ply CLT	440	N/A	SR-A	3/18/16	Shakedown test for “semi-rigid” test setup (end rotation); 4 brackets at each end of panel
V1CA-2	Gr. V1, 3-ply CLT	660[4]	7.2	SR-A	5/18/16	4 brackets at each end of panel
V1CA-3	Gr. V1, 3-ply CLT	425	7.6	SR-A	5/19/16	3 brackets at each end of panel
V1CA-4	Gr. V1, 3-ply CLT	420	7.0	SR-A	5/24/16	2 brackets at each end of panel
V1CB-1	Gr. V1, 3-ply CLT	420	7.0	SR-B	5/18/16	4 brackets at each end of panel
V1CB-2	Gr. V1, 3-ply CLT	430	8.0	SR-B	5/23/16	3 brackets at each end of panel
V1CB-3	Gr. V1, 3-ply CLT	410	7.2	SR-B	5/23/16	2 brackets at each end of panel
E1-1	Gr. E1, 3-ply CLT	480	14.5	S	3/22/16	
E1-2	Gr. E1, 3-ply CLT	480	13.3	S	3/22/16	
E1-3	Gr. E1, 3-ply CLT	480	15.5	S	3/22/16	
E1-4	Gr. E1, 3-ply CLT	470	13.4	S	3/24/16	
V4-1	Gr. V4, 3-ply CLT	400	9.2	S	8/5/16	
V4-2	Gr. V4, 3-ply CLT	410	9.2	S	8/5/16	
V4-3	Gr. V4, 3-ply CLT	395	9.9	S	8/5/16	
V4-4	Gr. V4, 3-ply CLT	405	10.6	S	8/5/16	
4NLT-1	2x4 NLT	480	7.0	S	5/9/16	
4NLT-2	2x4 NLT	500	6.9	S	5/9/16	
4NLT-3	2x4 NLT	420	6.9	S	5/10/16	
6NLT-1	2x6 NLT	680	6.5	S	5/10/16	Tested twice (nut backed off)
6NLT-2	2x6 NLT	700	6.6	S	5/17/16	
6NLT-3	2x6 NLT	660	7.2	S	5/17/16	

1 Weight measured with crane scale, reported to nearest 10 pounds.
2 Moisture Content (MC) measured at two to five points on the panel immediately following test with Delmhorst BD-2100 hand held moisture meter.
3 End condition type: S: simple; SR-A: semi-rigid with Simpson bracket; SR-B: semi-rigid with L4x4x1/4 bracket.
4 Panel weighed 660 pounds with end pieces attached; panel alone not weighed.

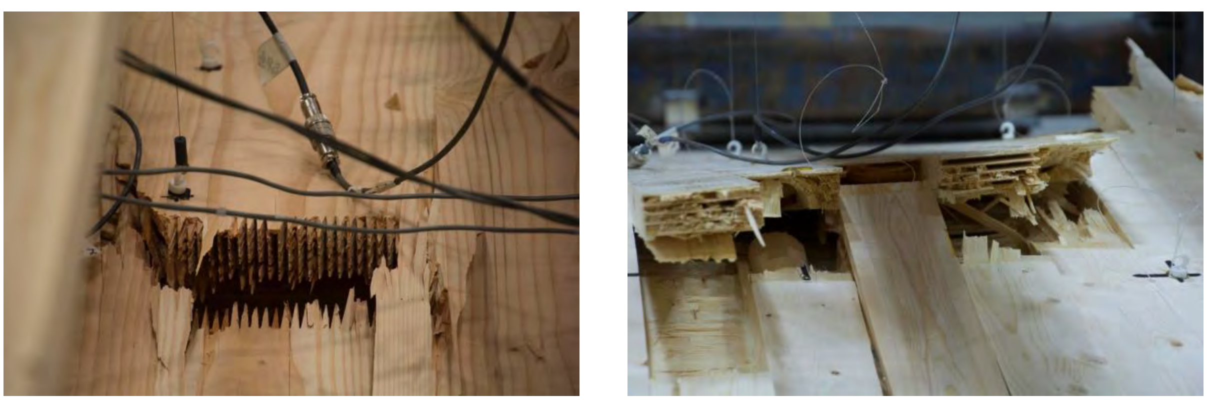
2.1 PANEL DESCRIPTION

Three different grades of CLT were tested during this effort:

- Grade V1 panels as manufactured by DR Johnson Lumber Company. Both 3-ply and 5-ply thick Grade V1 panels were tested. According to [1], Grade V1 CLT consists of No. 2 Douglas fir-Larch (DFL) lumber in the major strength direction and No. 3 DFL lumber in the minor strength direction. All Grade V1 panels were made from planed 2×8 (nominal) lumber. In general, the constituent lumber pieces did not appear to be intentionally edge glued, although adhesive was observed intermittently between boards along some edges. While the 3-ply panels had vertical finger joints that were visible on the broad face of the board in the major strength direction (see Figure 2-1), no finger joints were visible in the 5-ply panels.

- Grade E1 panels as manufactured by Nordic Structures. Only 3-ply Grade E1 panels were tested. According to [1], Grade E1 CLT consists of 1950f-1.7E Spruce-Pine-Fir (SPF) Machine Stress Rated (MSR) lumber in the major strength direction and No. 3 SPF lumber in the minor strength direction. All grade E1 panels were comprised of relatively short (i.e., approximately 2’-0” to 4’-0”) segments of 2×4 (nominal) lumber in the major strength direction. No adhesive was observed between adjacent board edges. Post-test observations indicated that board segments were joined via horizontal finger joints (see Figure 2-1).

- Grade V4 panels as manufactured by SmartLam. Only 3-ply thick Grade V4 panels were tested. According to [2], Grade V4 CLT consists of No. 2 SPF (South) lumber in both the major and minor strength directions. The Grade V4 panels were visually similar to the Grade V1 panels in terms of constituent board dimensions, finger joints, and observable edge gluing.



(a) Grade V1 (Grade V4 similar). *(b) Grade E1.*

Figure 2-1. Finger Joint Orientation in Panels.

Additionally, 2x4 and 2x6 NLT panels were tested during this effort. The NLT panels were manufactured by StructureCraft Builders, Inc. using No. 2 or better SPF. Boards were aligned broad face to broad face and nailed with two 0.121-inch diameter by 3-inch long nails at 16 inches on center. The 2x4 NLT specimens were backed by 3-ply, 4-layer 15/32-inch thick plywood. The 2x6 NLT specimens were backed by 5-ply, 5-layer 15/32-inch thick plywood. The plywood was used as the compression face of the panels in testing.

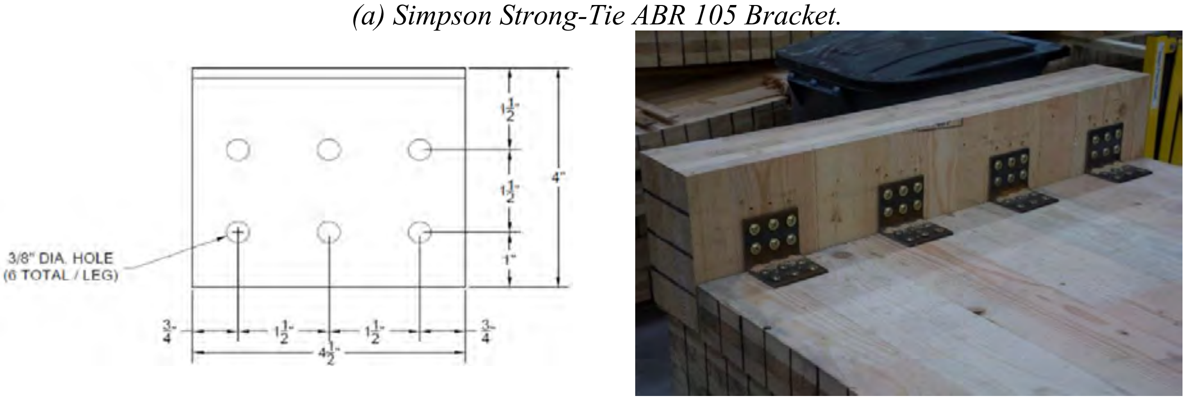
2.2 CONNECTION DESCRIPTION

Most panels were tested without connections. The test setup in this case allowed for free rotation of the panel at its ends. This panel end condition is referred to as “simple” in this report.

A total of seven panels were tested with angle bracket connections, although the first test was a shakedown test and prompted modifications to the test setup for the remaining tests. Two types of angle brackets were used and are shown in Figure 2-2: (a) 11 gauge Simpson Strong-Tie (SST) ABR105 brackets and (b) 4.5-inch lengths of pre-drilled ASTM A36 L4×4×1/4 angle. Bracket (a) was secured using fourteen SD10212 (i.e., #10 x 2-1/2”) self-drilling screws by SST in the horizontal leg (i.e., leg attached to panel) and twelve SD10212 screws in the vertical leg; both screws were manufactured by SST. Bracket (b) was secured using six ASSY SK 5/16x4 self-drilling screws by MyTiCon in each leg of the angle bracket. The number of angle brackets used was varied between tests and is indicated in Table 2-2. Angle brackets were evenly spaced along the width of the panel. To simulate an actual floor/roof that would not rotate, end supports for the angle bracket connections were cut from 5-ply Grade V1 panels. This panel end condition is referred to as “semi-rigid” in this report.

Details concerning the “simple” and “semi-rigid” end condition test setups are documented in Chapter 3.

(a) Simpson Strong-Tie ABR 105 Bracket.



(b) L4x4x1/4 Bracket.

Figure 2-2. Angle Bracket Connection Types.

CHAPTER 3

TEST APPARATUS

As mentioned in Chapter 2, two different test setup configurations were used to apply the quasi-static uniformly applied pressure load to the CLT and NLT panels. The first configuration was used to simulate "simple" end conditions while the second configuration was used to simulate end conditions associated with platform framing of a multi-story building (i.e., referred to as the "semi-rigid" configuration). The final test setup configurations and the changes that were made during testing to each configuration are described below. Additionally, the type and layout of instrumentation is documented.

3.1 TEST SETUP

3.1.1 "Simple" Configuration

A schematic diagram of the "simple" test setup configuration is included as Figure 3-1.

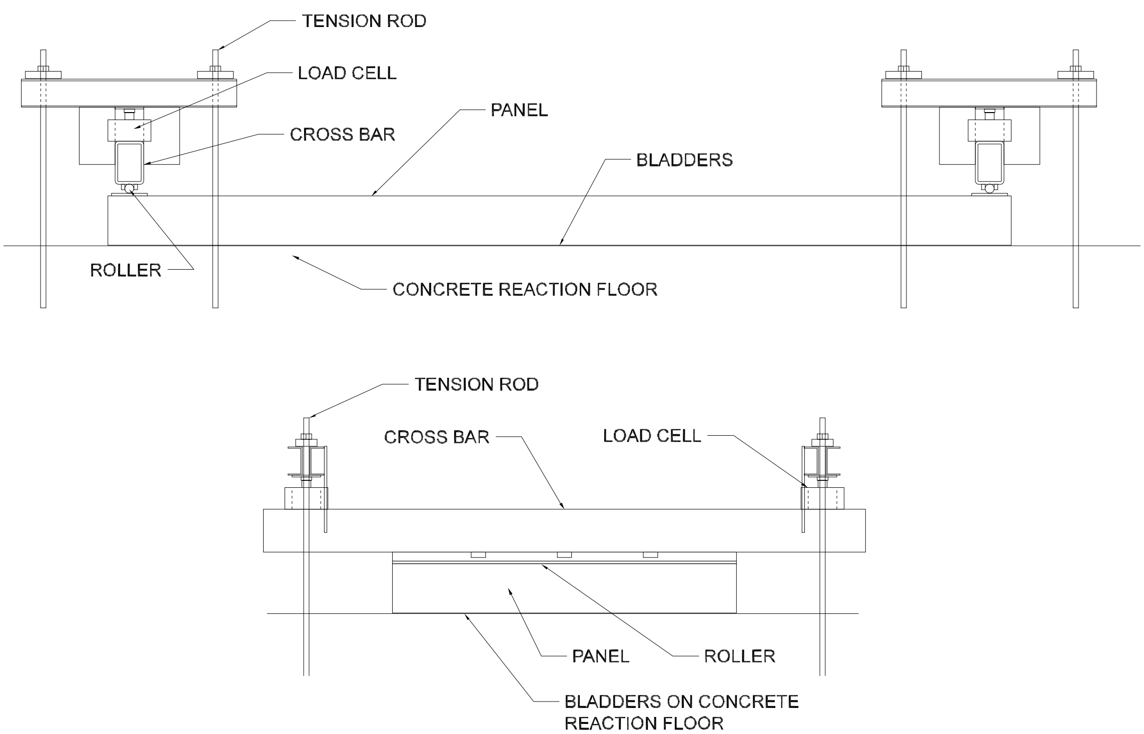


Figure 3-1. Schematic Diagram of "Simple" Test Setup Configuration.

The "simple" test setup configuration consisted of two crossbars, one at each end of the panel, secured to a concrete reaction floor by eight tension rods, two at each end of each crossbar. The tension rod nuts were hand-tightened for these tests providing a pre-tension on the order of a few hundred pounds.

Several crossbars were used during the testing. The initial crossbar consisted of back-to-back channels with one tension rod at each end of each crossbar. However, the force at each panel end was applied with a large enough eccentricity (i.e., with respect to the channels' shear center) to cause the channels to rotate and the tension rods to yield (see Figure 3-2). As such, the back-to-back channels were replaced with a tube with two tension rods at each end of each crossbar early in the testing program.

Figure 3-2. Failure of Initial Crossbar.

In addition, other errors associated with faulty test setup components also caused several panels to be tested more than once. These errors are documented in Table 2-2.

Prior to each test, a series of rubber bladders were placed on the concrete reaction floor directly underneath the panel. The bladders were centered on the panel and piled on top of each other in order of decreasing length (i.e., the longest was placed on the reaction floor and the shortest was placed on the top). Bladders were 3'-6", 5'-0", 5'-10", 6'-2", 7'-4" and 10'-0" long and 4'-0" wide in all circumstances. After the 10'-0" bladder burst (see test 5V1-2 in Table 2-2), it was replaced with 3'-6" and 7'-4" long bladders at the lowest level. Each bladder was encased in a woven bag to minimize outward deformation. The bladders in place prior to panel placement can be seen in Figure 3-3.

Figure 3-3. Bladders in Place Prior to Panel Placement.

Filling the bladders with water caused the panels to deflect upwards. The bladders were filled with tap water from a 3-inch diameter PVC distributor that ensured that the water in all of them had the same pressure. Pressure was measured via a pressure transducer (PT 1) at one bladder away from the water flow and with an analog meter where the hose enters the distributor. The ends were free to rotate due to a 48-inch-long, 1.25-inch-diameter steel cylindrical bar between the crossbar and the panel. The simply supported panels had a structural span of 10'-0" from roller to roller. A photograph of the final configuration for the "simple" test setup configuration is included as Figure 3-4.

Figure 3-4. "Simple" Test Setup Configuration.

3.1.2 "Semi-Rigid" Configuration

A schematic diagram of the "semi-rigid" test setup configuration is included as Figure 3-5.

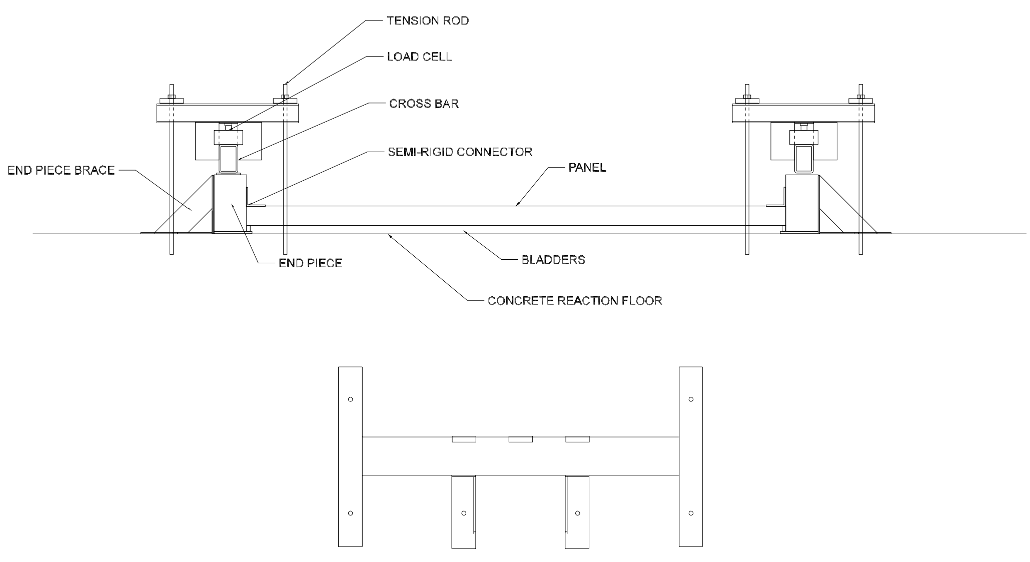


Figure 3-5. Schematic Diagram of "Semi-Rigid" Test Setup Configuration.

The “semi-rigid” configuration utilized the crossbar and tension rod system of the “simple” configuration, but replaced the cylindrical bar with stub end pieces of a 5-ply CLT panel to which angle brackets were secured. The resulting span from inside of end piece to inside of end piece was 9’-6” (i.e., the length of the panel). To prevent the end pieces from sliding or rotating during testing, a steel brace was installed on the back side of the end pieces and the tension bars were wrench-tightened to provide a pretension force of between one and two thousand pounds. A photograph of the “semi-rigid” test setup configuration is included as Figure 3-6.

Figure 3-6. “Semi-Rigid” Test Setup Configuration.

For the “semi-rigid” test setup configuration, loose straps were placed over the panel and connected in a loop beneath the structural floor to restrain the panels in case of a catastrophic angle bracket failure. These restraining straps are shown in Figure 3-7. While the restraining straps applied a force to panels, this force was negligible compared to that associated with panel rupture.

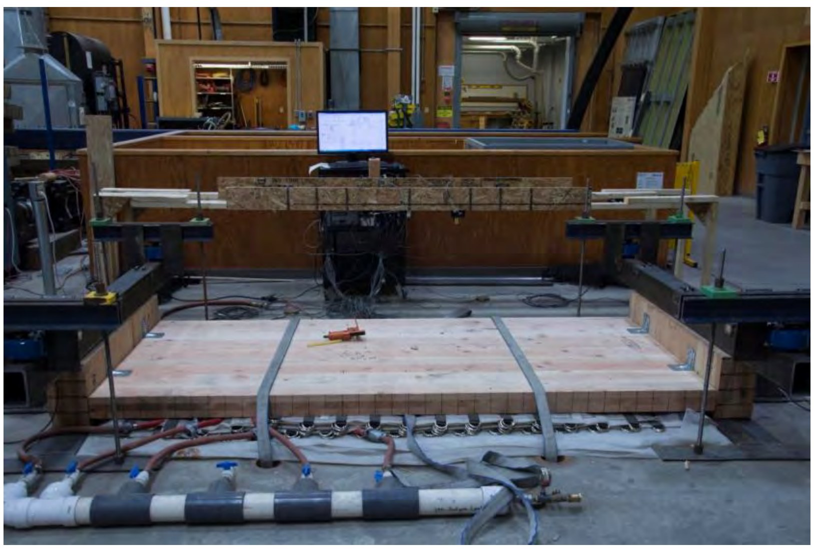


Figure 3-7. Restraining Straps in “Semi-Rigid” Test Setup Configuration.

3.2 INSTRUMENTATION

A list of instrumentation used during this test program is listed in Table 3-1. Additionally, the layout of the instrumentation is shown schematically in Figure 3-8. The data recorded by all instrumentation was read during the tests with LabView through a DAQ card and written concurrently to a CSV file. For the initial tests, data was collected at a 10 Hz frequency. However, this frequency was later increased to 100 Hz. The reason for this change was to record enough data to allow for the filtering out of signal noise (i.e., typically at approximately 3.5 Hz) that was most noticeably present in the pressure data. A 50-point running average filter was used to remove this signal noise from the pressure data.

Table 3-1. Instrumentation List.

ID	Description	Make	Serial No.	Service Dates
PT 1	50 psi pressure transducer	N/A	N/A	Pre- 3/18/16
PT 1	30 psi pressure transducer	Omegadyne	N/A	3/18/16 onwards
LC 1	25K washer load cell	Omega	345390	Pre- 3/30/16
LC 1	50K pancake load cell	Interface	1220AF-50K / AS1955	4/19/16 onwards
LC 2	20K washer load cell	Omega	345393	Pre- 3/30/16
LC 2	50K pancake load cell	Interface	1220AF-50K / AS1976	4/19/16 onwards
LC 3	20K washer load cell	Omega	345395	Pre- 3/30/16
LC 3	50K pancake load cell	Interface	1220AF-50K / AS1977	4/19/16 onwards
LC 4	20K washer load cell	Omega	345400	Pre- 3/30/16
LC 4	50K pancake load cell	Interface	1220AF-50K / AS1978	4/19/16 onwards
SP 1	10” string pot	Celesco	L1505710C	Entire test cycle
SP 2	10” string pot	Celesco	L1505723C	Entire test cycle
SP 3	10” string pot	Celesco	L1505726C	Pre- 4/26/16
SP 3	10” string pot	Celesco	N/A	4/26/16 onwards
SP 4	10” string pot	Celesco	L1505728C	Entire test cycle
SP 5	25” string pot	Celesco	3113	Pre- 4/26/16
SP 5	25” string pot	Celesco	N/A	4/26/16 onwards
SP 6	25” string pot	Celesco	N/A	Entire test cycle
SP 7	25” string pot	Celesco	2906	Pre- 4/26/16
SP 7	25” string pot	Celesco	N/A	4/26/16 onwards
SP 8	10” string pot	Celesco	L1505721C	Entire test cycle
SP 9	10” string pot	Celesco	L1505716C	Pre- 4/26/16
SP 9	10” string pot	Celesco	N/A	4/26/16 onwards
SP 10	10” string pot	Celesco	L1505719C	Entire test cycle
SP 11	10” string pot	Celesco	L1505737C	Entire test cycle

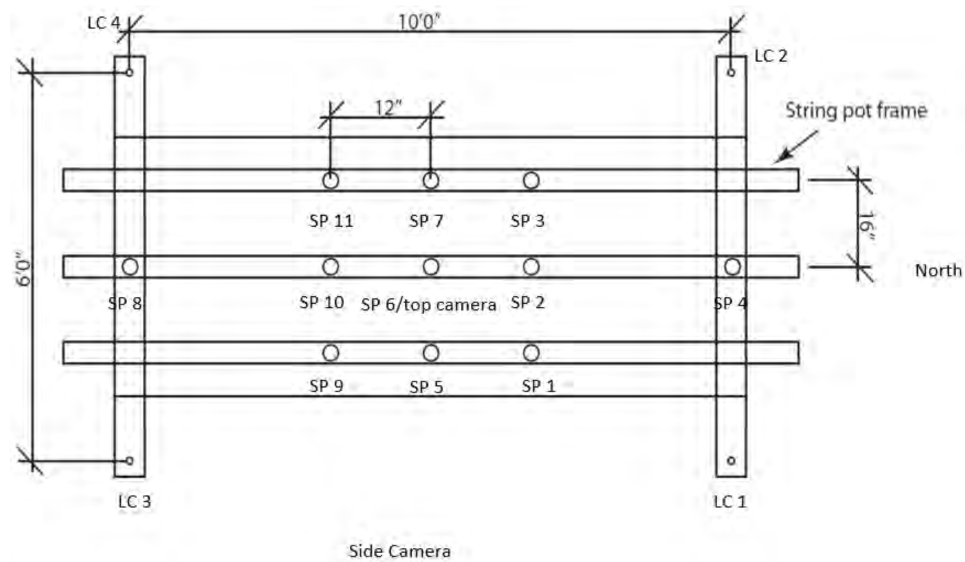


Figure 3-8. Instrumentation Key Plan.

Notes concerning the instrumentation are recorded below:

- Pressure transducer
  - Pressure in the rubber bladders was measured using one pressure transducer (PT 1) that was connected by a 1-inch diameter hose to the southwest corner of the lowest bladder.
- Load cell (see Figure 3-8 for location in plan)
  - Initially, washer load cells were used to measure the force in the tension rods. During the course of the testing program, the washer load cells were found to produce non-repeatable data. As such, washer load cells were replaced with pancake load cells for the remainder of the tests.
  - Assuming negligible tension rod pre-tension, the sum of the forces recorded by all load cells (i.e., LC 1 through LC 4) should be equal to the total pressure in the bladders, as recorded by PT 1, multiplied by the loaded area of the panel. In some cases, the PT1 and load cell sum curves are noticeably different in magnitude. The primary reason for this deviation is thought to be due to the rubber bladder not always being in contact with the entire surface area of the panel, particularly in the width direction.
- String potentiometer (see Figure 3-8 for location in plan)
  - Panel deflection was measured at eleven locations using string potentiometers (SPs) with 10-inch and 25-inch strokes. One 10-inch SP was attached at the mid-span of each crossbar to measure crossbar displacement (i.e., SP 4 and SP 8). The remaining SPs were placed in a 3×3 grid centered at panel mid-span. SP 5, 6, and 7 had a 25-inch stroke and SP 1, 2, 3, 9, 10, and 11 had a 10-inch stroke.

- The string potentiometers were hung from a timber frame that was supported by legs resting directly on the concrete reaction floor. There was no connection between the test fixture and the timber frame other than the concrete floor.

- Hooks were glued to the panels and a loop at the end the string was pulled over the hook (see Figure 3-9).

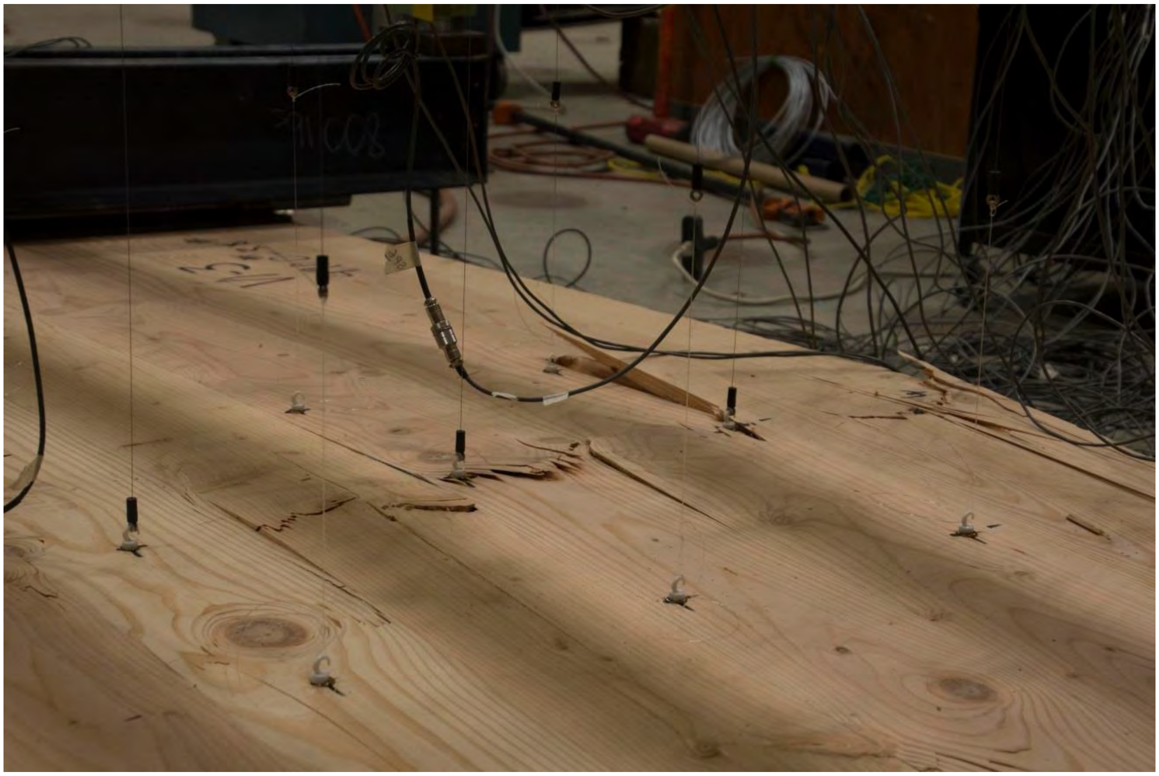


Figure 3-9. Connection of String Potentiometers to Panels.

- Video / Photo

  - Two digital video cameras were used to record the tests. One was supported above the panel near the center and the other mounted on a tripod to capture the long edge of the panel. The side video recorded the edge that had the lines traced on it at 3-inches on center.

  - Still photographs were taken of the set-up and of each panel before, during, and after testing.

3.3 TEST PROCEDURE

The test procedure was the same regardless of which test setup configuration was used – panels were set in place, the fixture elements installed, plumbed, leveled, and tightened, and instrumentation attached. Data and video recording were started and then the ball valve between the hose and PVC distributor was full opened. Water pressure at the tap was between 80 and 90 psi. Panels generally reached peak bending strength within three to five minutes, but testing continued until it was apparent that no additional useful deformation data could be gathered.

Typically, one or more SP would detach during the initial violent failure, occasionally more would come off in subsequent failures, and eventually, in general, the remaining 10-inch SP would reach the end of their stroke. Testing was typically complete within ten minutes. Figure 3-10 shows a typical 3-ply CLT panel near the end of its test.

Figure 3-10. Photo of Typical 3-ply CLT Panel at End of Test.

During the testing of the 5-ply panels, flexure of the crossbars was visually evident near peak panel bending strength. This deformation was tracked by SP 4 and 8.

CHAPTER 4

TEST RESULTS

This chapter presents the results from the testing program. This first part of the chapter provides qualitative observations concerning panel out-of-plane response through failure. Photographs documenting the results of each test are included in Appendix A. The second part of the chapter describes the test data recorded by instrumentation and presents representative plots for a single test. Similar plots for all tests are included in Appendix B.

4.1 QUALITATIVE OBSERVATIONS

Table 4-1 records the observed limit state and the peak panel strength for each test. With one notable exception, all CLT panels without connections (i.e., "simple" test setup configuration) failed near panel mid-span, presumably due to flexural stress. The location of panel rupture typically centered on knots, sloped grain, and finger joints (see Figure 4-1). No shear slip between panel plies away from the location of panel rupture was observed.

(a) Sloped Grain. *(b) Finger Joint.*

Figure 4-1. Typical Panel Failure Locations.

One 5-ply Grade V1 CLT panel failed in shear near the supports. Failure was initiated at a point internal to the panel, thus making it difficult to observe failure progression in time. Figure 4-2 shows photographs of the 5-ply panel that failed in shear.

(a) Side View. *(b) Top View.*

Figure 4-2. CLT Panel with Shear Failure at Support.

Table 4-1. Limit State & Peak Pressure Test Results.

Panel ID	Panel Type	Limit State Type[1]	Peak Strength[2] (psi)	Residual Strength 1[3] (psi)	Residual Strength 2[4] (psi)
V1-1	Gr. V1, 3-ply CLT	F	-	-	-
V1-2	Gr. V1, 3-ply CLT	F	7.6	1.5	-
V1-3	Gr. V1, 3-ply CLT	F	6.4	1.5	-
V1-4	Gr. V1, 3-ply CLT	F	7.0	1.7	-
V1-5	Gr. V1, 3-ply CLT	F	7.5	1.8	-
5V1-1	Gr. V1, 5-ply CLT	F	-	-	-
5V1-2	Gr. V1, 5-ply CLT	F	-	-	-
5V1-3	Gr. V1, 5-ply CLT	F	13.2	7.4	2.3
5V1-4	Gr. V1, 5-ply CLT	F	18.6	6.8	3.1
5V1-5	Gr. V1, 5-ply CLT	V	13.9	6.0	-
V1CA-1	Gr. V1, 3-ply CLT	N/A	-	-	-
V1CA-2	Gr. V1, 3-ply CLT	V	7.1	4.2	-
V1CA-3	Gr. V1, 3-ply CLT	V/TB	7.5	3.7	-
V1CA-4	Gr. V1, 3-ply CLT	V/TB	6.7	4.5	-
V1CB-1	Gr. V1, 3-ply CLT	F	7.4	3.5	-
V1CB-2	Gr. V1, 3-ply CLT	F	6.5	2.8	-
V1CB-3	Gr. V1, 3-ply CLT	F/TB	7.6	2.4	-
E1-1	Gr. E1, 3-ply CLT	F	5.8	1.6	-
E1-2	Gr. E1, 3-ply CLT	F	6.3	1.2	-
E1-3	Gr. E1, 3-ply CLT	F	5.7	1.5	-
E1-4	Gr. E1, 3-ply CLT	F	7.2	1.2	-
V4-1	Gr. V4, 3-ply CLT	F	6.2	0.5	-
V4-2	Gr. V4, 3-ply CLT	F	5.0	0.5	-
V4-3	Gr. V4, 3-ply CLT	F	5.5	0.6	-
V4-4	Gr. V4, 3-ply CLT	F	6.6	0.6	-
4NLT-1	2x4 NLT	F	6.2	-	-
4NLT-2	2x4 NLT	F	5.3	-	-
4NLT-3	2x4 NLT	F	4.7	-	-
6NLT-1	2x6 NLT	F	13.9	-	-
6NLT-2	2x6 NLT	F	14.0	-	-
6NLT-3	2x6 NLT	F	13.9	-	-

1 F: Flexural. Rupture due to flexural stress near mid-span localized near finger joints, sloped grain, and/or knots.
V: Shear. Rupture due to rolling shear stress near support.
F/TB: Flexural with top board disengagement. Rupture due to flexural stress near mid-span localized near finger joints, sloped grain, and/or knots combined with top boards not supported by angle brackets popping upwards.
V/TB: Shear with top board disengagement. Rupture due to rolling shear stress near support combined with top boards not supported by angle brackets popping upwards.
N/A: Test apparatus failure prior to panel failing.

2 Sum of peak forces recorded by load cells divided by 5,760 for “simple” end conditions and 5472 for “semi-rigid” end conditions.

3 Sum of (approximate) first plateau forces recorded by load cells divided by 5,760 for “simple” end conditions and 5472 for “semi-rigid” end conditions.

4 Sum of (approximate) second plateau forces recorded by load cells divided by 5,760 for “simple” end conditions and 5472 for “semi-rigid” end conditions.

Excluding the Grade V4 panels, which arrived to the lab much later, it is worth noting that the 5-ply CLT panel that failed in shear was the final CLT panel tested in the "simple" test setup configuration, and thus had the lowest moisture content (see Table 2-2). It was noted that as panels dried, significant inter-layer stresses developed leading to checking and some joint failure along the panel edges (see Figure 4-3). Thus, the different limit state observed for the final 5-ply test could be related to panel moisture content and the imperfections that resulted, but in the absence of further testing this theory is conjecture.

(a) Checking. *(b) Bond Failure.*

Figure 4-3. Localized Deformation in CLT Panels.

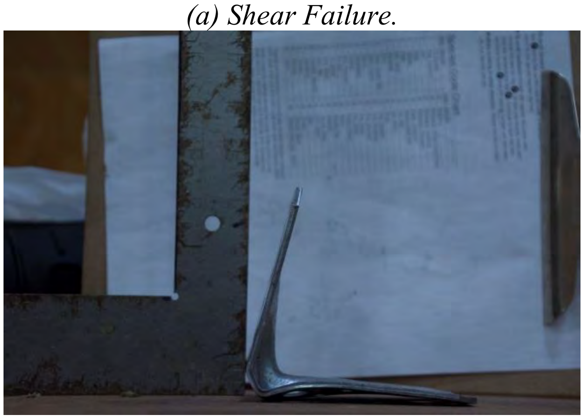
The first test on a panel with connections (i.e., V1CA-1) resulted in one of the end pieces rotating, causing significant angle bracket deformation but minimal panel deformation (see Figure 4-4). Based on this test, modifications were made to the "semi-rigid" test setup configuration to constrain the end pieces. It should be noted that by constraining the end pieces, it is possible that panel arching between supports could occur, thus potentially augmenting the panel strength from a purely simply-supported condition.

Figure 4-4. Deformation of Simpson Bracket in V1-CA1.

The panels with the Simpson Strong-Tie (SST) angle brackets (i.e., V1CA-2 through V1CA-4) typically exhibited shear failures near one end of the panel (see Figure 4-5(a)). For the test in which four SST brackets were installed at each end (i.e., V1CA-2), the shear failure precipitated a flexural failure near panel mid-span. For the tests in which two and three SST brackets were installed at each end (i.e., V1CA-3 and V1CA-4), top boards not directly supported by angle brackets pulled away from those that were (see Figure 4-5(b)). In general, the SST brackets were capable of deforming significantly while still being able to support their respective loads (see Figure 4-5(c)). Additionally, both a small amount of wood crushing beneath the SST angle brackets and fastener pullout was observed (see Figure 4-5(d)).

(a) Shear Failure.

(b) Top Board Disengagement.



(c) Bracket Deformation.

(d) Wood Crushing & Fastener Pullout.

Figure 4-5. Failure Modes Associated with Simpson Brackets.

By contrast, the panels with L4x4x1/4 angle brackets typically exhibited a flexural failure response near mid-span (see Figure 4-6(a)) similar to the predominant failure mode observed for panels tested in the "simple" test setup configuration. For the tests in which two brackets were installed at each end (i.e., V1CB-4), a few of the top boards not directly supported by angle brackets pulled away from those that were. As with the SST brackets, the L4x4x1/4 brackets yielded (see Figure 4-6(b)), but unlike the SST brackets, no fastener pullout or wood crushing was observed.

(a) Flexural Failure. *(b) Bracket Deformation.*

Figure 4-6. Failure Modes Associated with L4x4x1/4 Brackets.

The NLT panels failed in flexure near panel mid-span. Typically, one stud would rupture first and then its neighbors would fail in a zippering fashion. This response led to highly unbalanced panels where one half was nearly intact and showed minimal deflection while the other half would be completely ruptured and have a mid-span deflection upwards of 10 inches. Figure 4-7 shows a typical example of this response.

Figure 4-7. Asymmetric Flexural Failure of NLT Panel.

Significant rotation of the boards relative to one another occurred due to the asymmetric deformation. Nail pullout and yielding was observed as well (see Figure 4-8). Prior to failure, panels were uniformly deflected. As some of the boards broke, the others rebounded as the pressure diminished and the water shifted to the failed portions of the panel.

Figure 4-8. Details of NLT Panel Failure.

4.2 RECORDED DATA

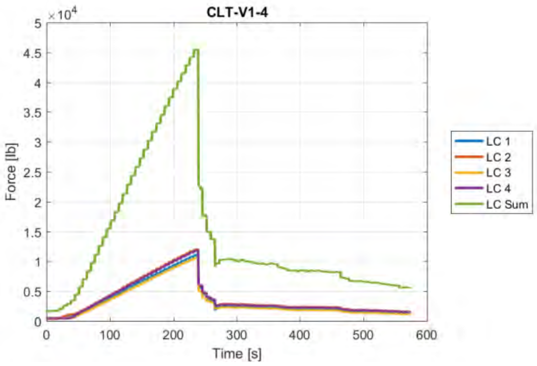
Typical failure pressures for 3-ply CLT and all NLT panels were between 5 and 8 psi, corresponding to a total load of between 28,000 and 46,000 pounds of applied load. The 5-ply CLT panels failed with a pressure of around 15 psi or about 86,000 pounds of applied load. The peak pressure resisted by each panel is recorded in Table 4-1.

Figure 4-9 includes plots of the data recorded for CLT panel test V1-4. These plots are representative of those obtained for 3-ply CLT panel flexural failures observed in the "simple" test setup configuration. Plots are included in Appendix B for the remainder of the tests. The text that follows describes how the plots were created and typical patterns observed.

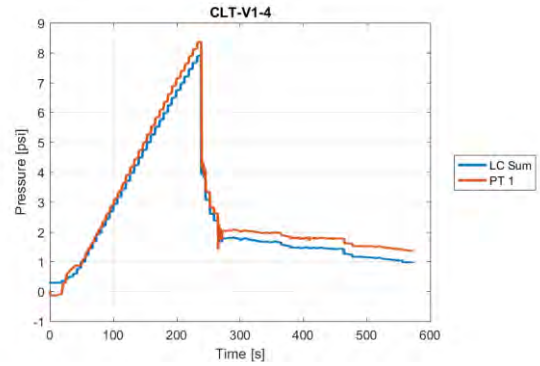
a) **LC**: This figure plots the force data from load cells 1 through 4 in time. Additionally, the sum of the load cell forces is plotted. In general, recordings from the individual load cells were relatively consistent, thus indicating the symmetric application of load by the rubber bladders.

b) **LC vs. PT 1**: This figure plots the sum of the load cell force data in time divided by 5,760 in^2 (i.e., panel width multiplied by panel span) against the data recorded by the pressure transducer in the rubber bladder (i.e., PT 1). A 50-point running average filter was used to filter the pressure data.

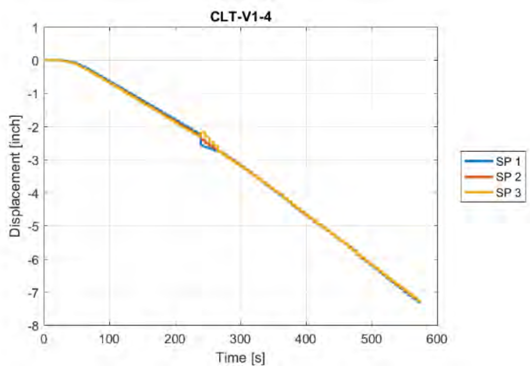
c) **SP 1 – SP 3**: This figure plots the displacement data in time from string potentiometers (SPs) 1 through 3, or as shown in Figure 3-8, the SPs offset 12 inches to the north of panel mid-span.



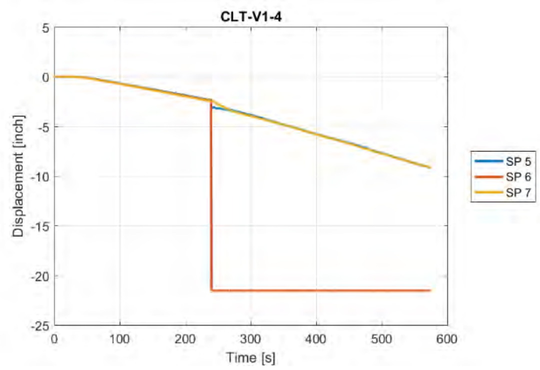
(a) LC.



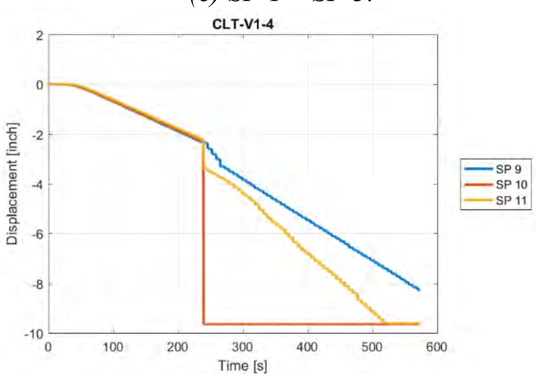
(b) LC vs. PT 1.



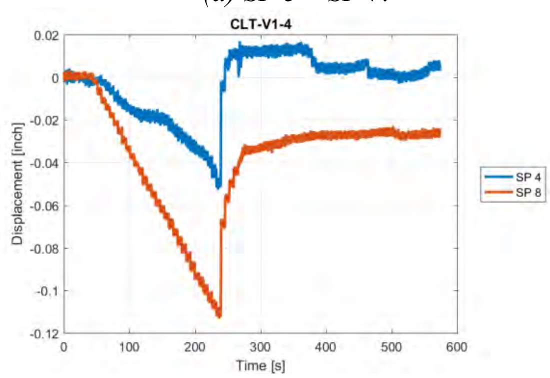
(c) SP 1 – SP 3.



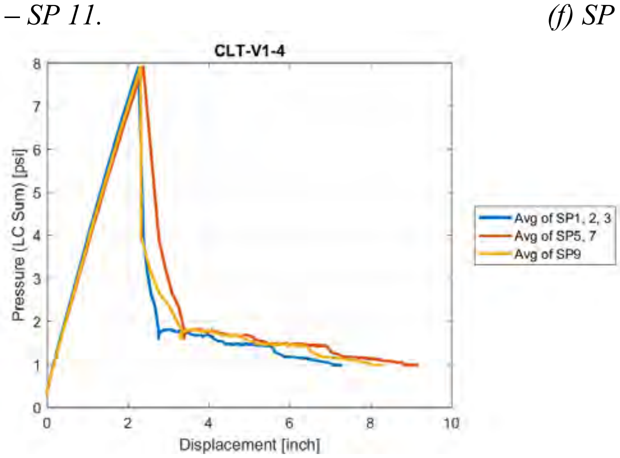
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.



(f) SP 4 & SP 8.



(g) Average SP.

Figure 4-9. Typical Data Recorded for "Simple" Test Setup Configuration (Test V1-4).

d) **SP 5 – SP 7**: This figure plots the displacement data in time from SPs 5 through 7, or as shown in Figure 3-8, the SPs at panel mid-span. As mentioned in Chapter 3, some SPs were lost when the panel ruptured suddenly. This gauge failure is indicated by an instantaneous drop in the displacement data. The SP 6 data shown in the plot is an example of this phenomenon.

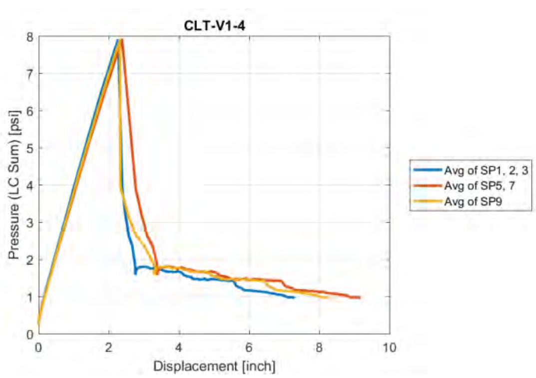
e) **SP 9 – SP 11**: This figure plots the displacement data in time from SPs 9 through 11, or as shown in Figure 3-8, the SPs offset 12 inches to the south of panel mid-span. As with SP 6, SPs 10 and 11 were compromised in the process of panel rupture for this test.

f) **SP 4 & SP 8**: This figure plots the displacement data in time from SPs 4 and 8, or as shown in Figure 3-8, the SPs located at the mid-span of the crossbar at each end of the panel. As observed from this plot, the deflection of the crossbar through the range of applied force is negligible.

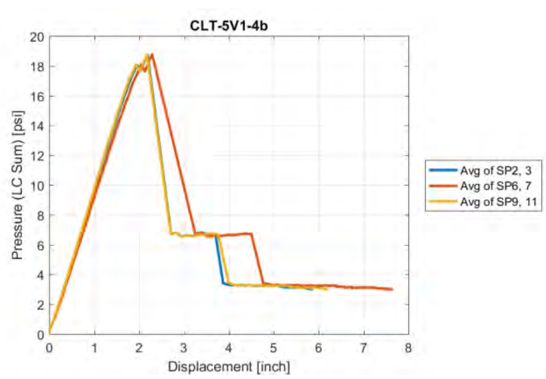
g) **Average SP**: This figure plots the average of the SP displacement data from each row for gauges that continuously recorded data for the duration of the test. The pressure values are based on the sum of the load cell force data (i.e., "LC Sum"), as this value is thought to better represent the actual force resisted by the panel.

As shown in Figure 4-9(g), 3-ply CLT panel response is essentially linear elastic prior to panel rupture. Upon panel rupture, there is a relatively sudden drop in panel strength to a residual panel strength. This residual panel strength remains relatively constant through a large deflection. Generally, the stroke of the SP was reached prior to the panel's strength dropping to zero.

For the 5-ply CLT panel, two residual strength plateaus are discernable in the recorded force-displacement data. The first plateau corresponds to the rupture of the outermost ply in tension and the second plateau corresponds to the rupture of the middle ply. This phenomenon is shown in Figure 4-10. It is interesting to note that the first residual strength plateau is relatively consistent in magnitude with that of the peak strength of the 3-ply CLT panel. This observation can be gleaned by comparing the data recorded in Table 4-1.



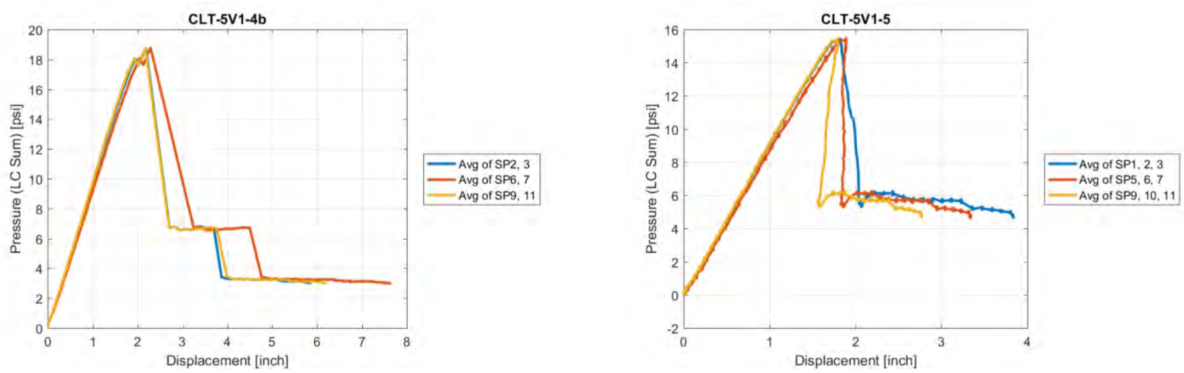
(a) 3-Ply.



(b) 5-Ply.

Figure 4-10. Ply Number Impact on Panel Response.

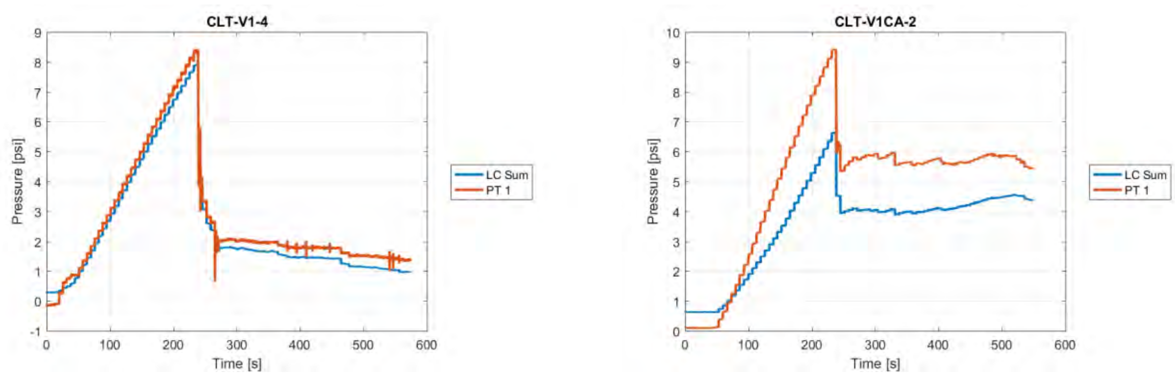
For the shear failure observed in test 5V1-5, the post-peak measurements are somewhat different. Figure 4-11 shows how instead of having two well defined plateaus, the shear failure shows one drop. However, it should be noted that test 5V1-5 was terminated at a much smaller deflection than tests 5V1-3 and 5V1-4.



(a) Flexure. *(b) Shear.*

Figure 4-11. Limit State Impact on Panel Response.

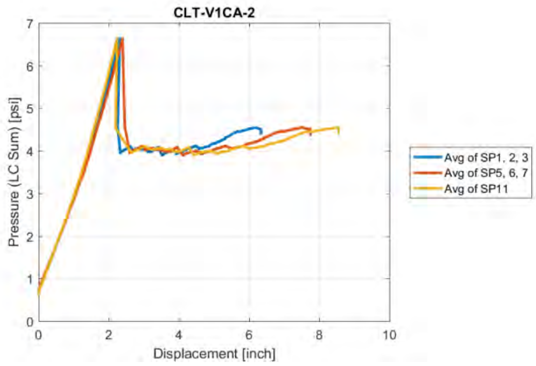
For several tests, particularly for those with connections, the LC Sum and PT 1 values diverge relatively significantly (see Figure 4-12). This divergence is attributed to: (1) the pre-tension forces placed on the threaded rods tying down the crossbar and (2) the rubber bladders not being in contact with the entire surface area of the panel.



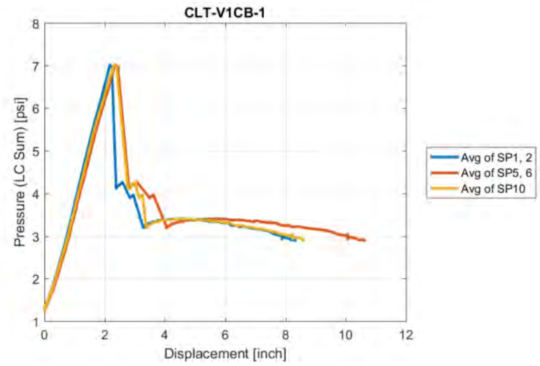
(a) "Simple". *(b) "Semi-Rigid".*

Figure 4-12. Test Setup Configuration Impact on Load Measurement Data.

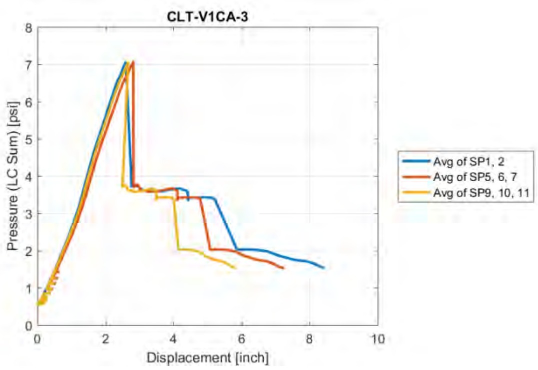
Finally, it is interesting to note the difference that the number of brackets might have had on the response of the panel. Qualitative observations are listed in the previous section, but the plots that quantify these differences are included in Figure 4-13.



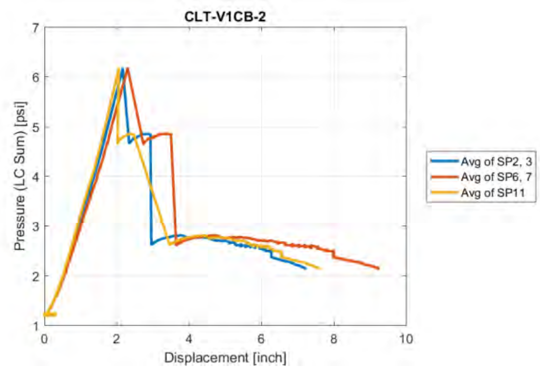
(a) 4 Simpson Brackets.



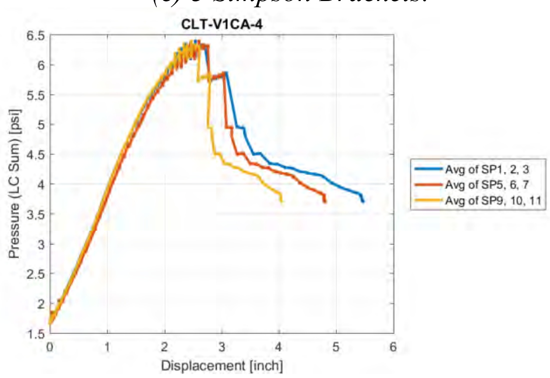
(b) 4 L4x4x1/4 Brackets.



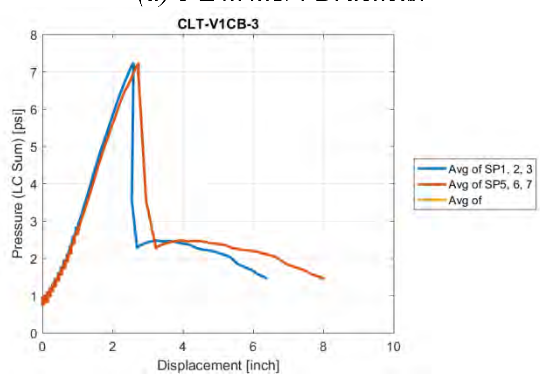
(c) 3 Simpson Brackets.



(d) 3 L4x4x1/4 Brackets.



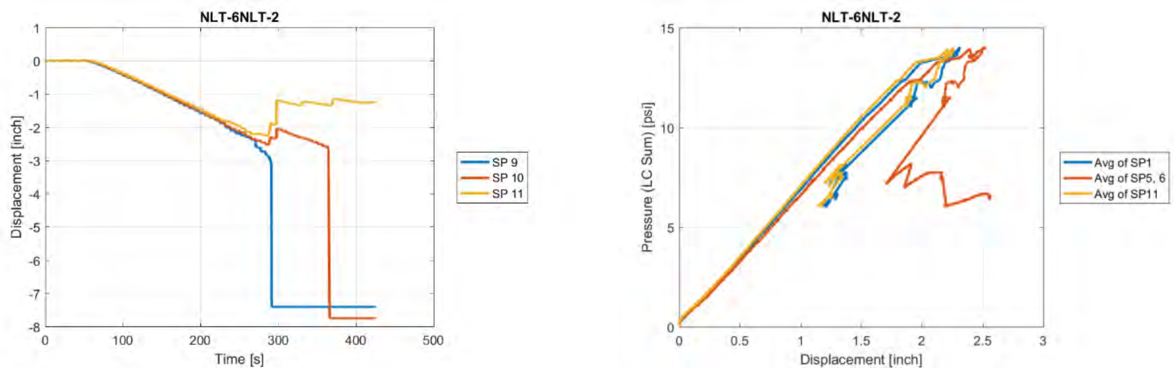
(e) 2 Simpson Brackets.



(f) 2 L4x4x1/4 Brackets.

Figure 4-13. Bracket Type and Number Impact on Load-Displacement Data.

Reviewing the NLT panel data resulted in similar observations made for the 3-ply CLT panels in the "simple" test setup configuration for the elastic range. However, following peak strength, due to the unbalanced failure pattern the gauges in the portion of the panel that did not fail rebounded (see SP 11 of Figure 4-14(a)). Also, those in the portion of the panel that failed broke in the violent rupture of the panel in many cases. Thus, the post peak response of the NLT panels as shown in Figure 4-14(b) is not representative of how the panel responded. It is interesting to note that a similar failure did not happen in the CLT panels, presumably due to the layer in the secondary direction that distributed the failure across the panel more evenly.

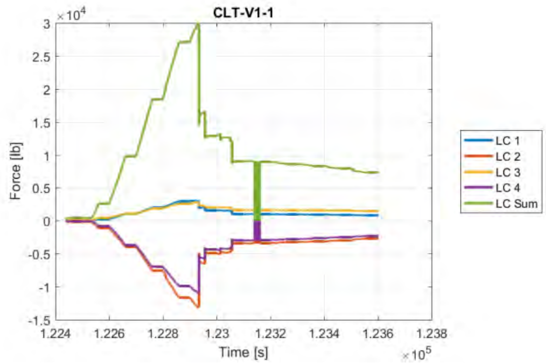


(a) SP 9 – SP 11. *(b) Average SP.*

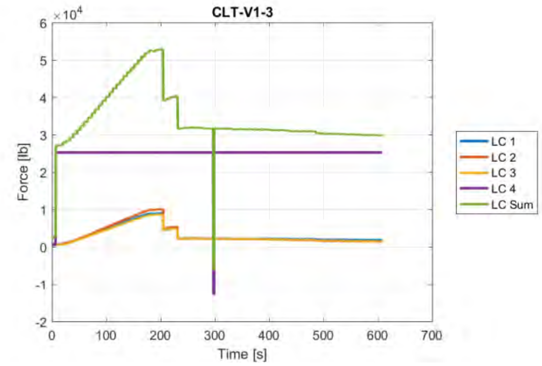
Figure 4-14. Typical NLT Panel Results.

In addition to the observations made above, several tests suffered from abnormalities in the instrumentation or test setup that impaired data measurement:

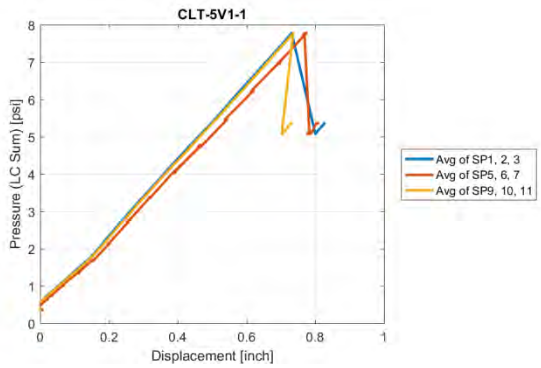
- The shakedown test for the "simple" test setup configuration (i.e., V1-1) indicated issues with the load cell measurements (see Figure 4-15(a)).
- For test V1-3, LC 3 was defective (see Figure 4-15(b)).
- Leaking of the rubber bladders caused an early termination of test 5V1-1 (see Figure 4-15(c)).
- The SP data for test 5V1-3 recorded a sudden jump and then rebound in the elastic range (see Figure 4-15(d)). The cause of this jump/rebound is unknown and cannot be corroborated by video.



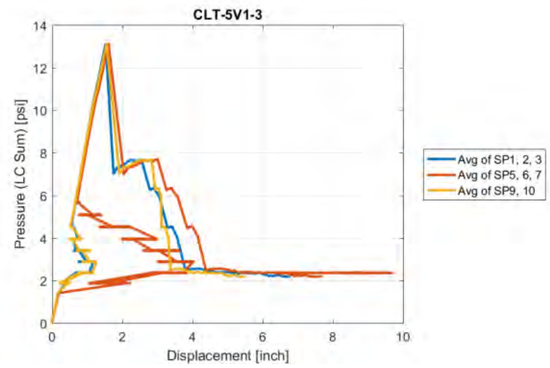
(a) V1-1 LC.



(b) V1-3 LC.



(c) 5V1-1 Average SP.



(d) 5V1-3 Average SP.

Figure 4-15. Instrumentation Abnormalities.

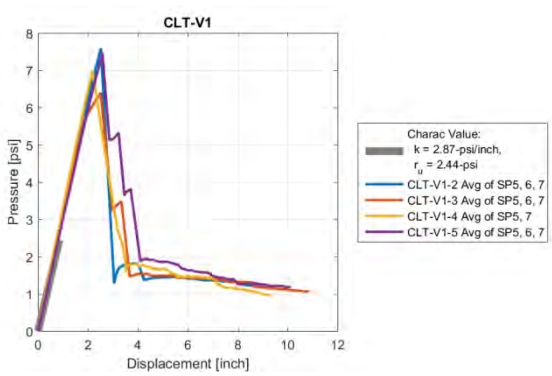
CHAPTER 5

CONCLUSIONS

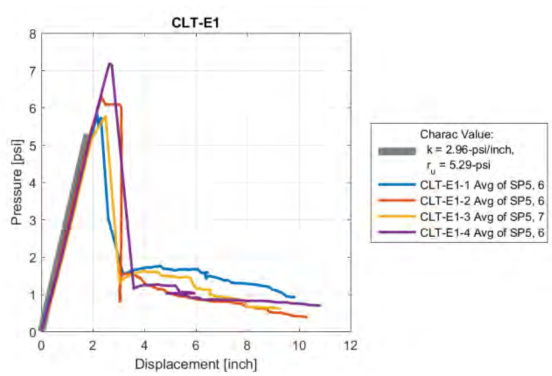
The out-of-plane response of a selection of CLT and NLT panels was investigated using a test apparatus capable of applying a uniform quasi-static load. Grade, ply number, and connections were varied in the CLT panel testing and thickness was varied in the NLT panel testing. In this chapter, the applied load and panel displacement measurements are plotted together and compared with characteristic bending strength and stiffness values computed according to applicable standards. Additionally, observations made as a result of this testing are summarized.

5.1 DESIGN VALUE COMPARISON

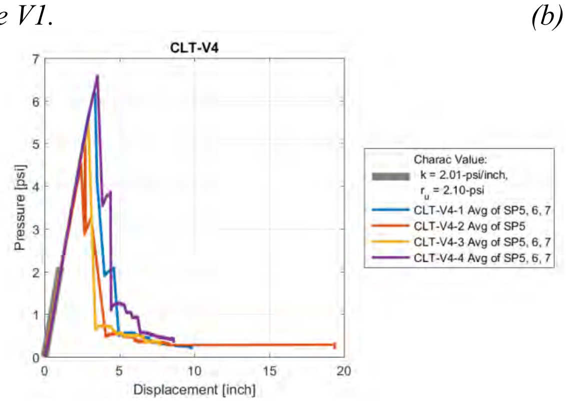
The design bending strength (i.e., $F_bS_{eff,0}$) and effective bending stiffness (i.e., $EI_{eff,0}$) of CLT panels in the major strength direction is specified in Table A2 of ANSI/APA PRG 320-2012 [1] for Grades V1 and E1 CLT and in an APA supplement [2] for Grade V4 CLT. To obtain the characteristic values, $F_bS_{eff,0}$ was multiplied by 2.1 (i.e., consisting of a load duration factor, C_D, of 1.6 multiplied by a safety factor of 1.3 [3]). Figure 5-1 plots these characteristic bending strength and stiffness values for the 3-ply CLT panels against load-displacement plots created by using the average of the SP 5 through SP 7 (i.e., SP located at panel mid-span) displacement data and the sum of the load cell force data.



(a) *Grade V1.*



(b) *Grade E1.*



(c) *Grade V4.*

Figure 5-1. 3-Ply CLT Panel Load-Displacement Plots.

In general, the characteristic bending stiffness values are consistent with those obtained via testing. The bending strength values from testing are always greater, and in some cases, much greater than the characteristic bending strength values. For the Grade E1 CLT panels, which use MSR lumber in the major strength direction, the testing values are 5 to 40 percent greater than the characteristic values. However, for the Grades V1 and V4 CLT panels, which use visually graded lumber in the major strength direction, the testing values are approximately three times greater than the characteristic values.

Figure 5-2 creates a similar load-displacement plot for the 5-ply Grade V1 CLT panel tests. Again, the testing and characteristic bending stiffness values are consistent while the testing strength value is on the order of three times larger than the characteristic strength value.

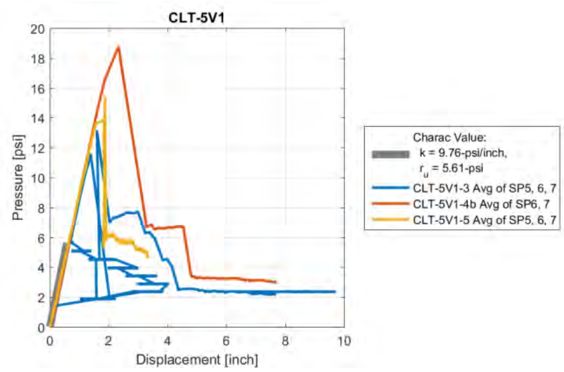
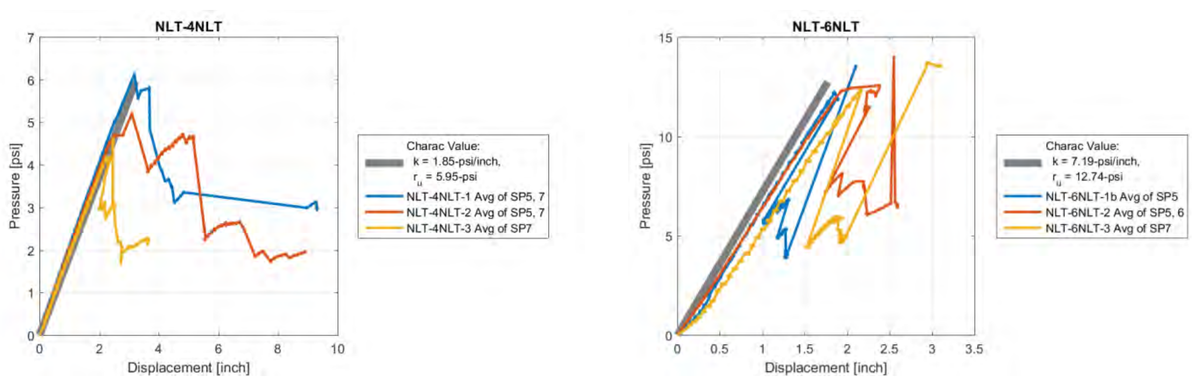


Figure 5-2. 5-Ply Grade V1 CLT Panel Load-Displacement Plot.

The design bending strength and effective bending stiffness of NLT panels is computed using guidance for sawn lumber from ANSI/AF&PA NDS-2005 [4]. Actual, rather than nominal, dimensions are used to compute section properties. The plywood on the compression face of the panel is ignored for the purpose of computing the strength of the panel. Load duration (i.e., C_D = 1.6) and size (i.e., C_F = 1.5 for 2x4 NLT, C_F = 1.3 for 2x6 NLT) factors are applied to the bending strength value. Additionally, the design bending strength value is multiplied by 2.5 to transform the design value to the expected average value. This 2.5 factor is used in SBEDS [5], which is consistent with Breyer et al. (2007) [6]. Figure 5-3 plots these expected average bending strength and stiffness values against the tested values.



(a) 2x4 NLT. *(b) 2x6 NLT.*

Figure 5-3. NLT Panel Load-Displacement Plots.

Initially, the expected average and tested bending stiffness values are consistent for the NLT panels. However, while the 2x4 NLT exhibits essentially linear response in the pre-peak region, the 2x6 NLT appears to exhibit a measure of nonlinearity. Whether this nonlinearity is related to the test setup or the panel itself is not known. Concerning strength, the expected average value is remarkably consistent for the 2x6 NLT. However, for the 2x4 NLT, the expected average value exceeds one of the tested values by approximately 25 percent.

Table 5-1 compares the average peak and residual strengths tabulated in Table 4-1 with those computed in this chapter using the applicable design standard. R_{test} values refer to testing values while R_{char} values refer to characteristic values. Characteristic residual strengths for the CLT panels were computed ignoring the ruptured major strength ply and minor strength ply directly below it. Thus, the "Residual Strength 1" value for a 5-ply CLT panel is assumed to be equivalent to that of a pristine 3-ply panel for the purpose of comparison. For the single ply condition, a flat use increase factor (i.e., 1.1 for Grade E1 CLT due to 2x4 boards and 1.15 for Grades V1 and V4 CLT due to 2x8 boards) was used in addition to a load duration factor of 1.6 and safety factor of 1.3 to augment the characteristic strength value.

Table 5-1. Panel Strength Summary.

ID	Description	Peak Strength			Residual Strength 1			Residual Strength 2		
		R_{test} (psi)	R_{char} (psi)	R_{test} / R_{char}	R_{test} (psi)	R_{char} (psi)	R_{test} / R_{char}	R_{test} (psi)	R_{char} (psi)	R_{test} / R_{char}
V1	Gr. V1 CLT	7.1	2.44	291%	1.6	0.38	421%	-	-	-
5V1	Gr. V1 CLT	15.2	5.61	271%	6.7	2.44	275%	2.7	0.38	711%
V1CA	Gr. V1 CLT	7.1	2.70	263%	4.1	0.42	976%	-	-	-
V1CB	Gr. V1 CLT	7.2	2.70	267%	2.9	0.42	690%	-	-	-
E1	Gr. E1 CLT	6.3	5.29	119%	1.4	0.79	177%	-	-	-
V4	Gr. V4 CLT	5.8	2.10	276%	0.55	0.33	167%	-	-	-
4NLT	2x4 NLT	5.4	5.95[1]	91%	-	-	-	-	-	-
6NLT	2x6 NLT	13.9	12.74[1]	109%	-	-	-	-	-	-

[1] These values are expected average values rather than characteristic values. Characteristic values are meant to represent the 5% exclusion limit while expected average values are to be considered as mean values. Mean values were used for NLT to align with the current methodology used in SBEDS [5].

5.2 SUMMARY

5.2.1 CLT Panel without Connections

The CLT panel testing without connections yielded the following general observations:

- With one notable exception, all CLT panels without connections failed near panel mid-span, presumably due to flexural stress. The location of panel rupture typically centered on knots, sloped grain, and finger joints.

- CLT panel response was essentially linear elastic prior to panel rupture. The tested stiffness of the panel was consistent with that specified in the applicable design standard. Upon panel rupture, there is a relatively sudden drop in panel strength to a residual panel strength plateau.

- The peak tested strength of Grade E1 panels with MSR lumber in the major strength direction was roughly 20 percent greater than the characteristic strength. On the other hand, the peak tested strength of Grade V1 and V4 panels with visually graded lumber in the major strength direction was roughly three times greater than the characteristic strength.

- The average residual panel strength plateau of the Grades E1 and V4 panels were roughly two times greater than the characteristic strength of a single ply member while the average residual panel strength plateau of the Grade V1 panel was at least four times greater than the characteristic strength of a single ply member.

- When CLT panels ruptured due to flexure, negligible shear slip between panel plies away from the location of panel rupture was observed.

- As panels dried, significant inter-layer stresses developed leading to checking and some joint failure along the panel edges. The shear failure observed for the final 5-ply test could be related to panel moisture content and the imperfections that resulted, but in the absence of further testing this theory is conjecture.

- No compression failure was observed in the panel, although for some panels a flexural failure of the lowest ply was observed towards the end of the test.

5.2.2 CLT Panel with Connections

Similarly, the CLT panel testing with connections yielded the following general observations:

- The peak strength of the CLT panel was independent of the number of angle brackets.

- In several tests, particularly for tests in which only two or three angle brackets were used at each end of the panel, top boards not directly supported by angle brackets pulled away from those that were.

- The panels with the Simpson Strong-Tie (SST) angle brackets typically exhibited shear failures near one end of the panel while those with the L4x4x1/4 angle brackets typically exhibited flexural failures near panel mid-span. One reason for this pattern could be that the SST brackets were connected by fasteners that penetrated only two of the three plies while the L4x4x1/4 bracket fasteners penetrated all three plies; however, this theory would need to be substantiated by additional testing or modeling.

5.2.3 NLT Panel without Connections

Finally, the NLT panel testing yielded the following general observations:

- The NLT panels failed in flexure near panel mid-span. Typically, one stud would rupture first and then its neighbors would fail in a zippering fashion. This response led to highly unbalanced panels where one half was nearly intact and showed minimal deflection while the other half would be completely ruptured and have a mid-span deflection upwards of 10 inches.

- The average NLT bending stiffness and strength values obtained via testing are consistent with values computed using the NDS that ignore the impact of the plywood sheathing.

CHAPTER 6

REFERENCES

[1] ANSI/APA PRG 320-2012, "Standard for Performance-Rated Cross-Laminated Timber", The Engineered Wood Association, 2012.

[2] B.J. Yeh (personal communication, July 13, 2016).

[3] B.J. Yeh (personal communication, April 13, 2016).

[4] ANSI/AF&PA NDS-2005, "National Design Specification for Wood Construction with Commentary and Supplement: Design Values for Wood Construction", American Forest & Paper Association, 2005.

[5] PDC-TR 06-01 Rev 2, "Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)", US Army Corps of Engineers, December 2012. (*Export Controlled – Distribution Statement C*)

[6] Breyer et al., "Design of Wood Structures ASD/LRFD, 6th Edition", New York: McGraw Hill, 2007.

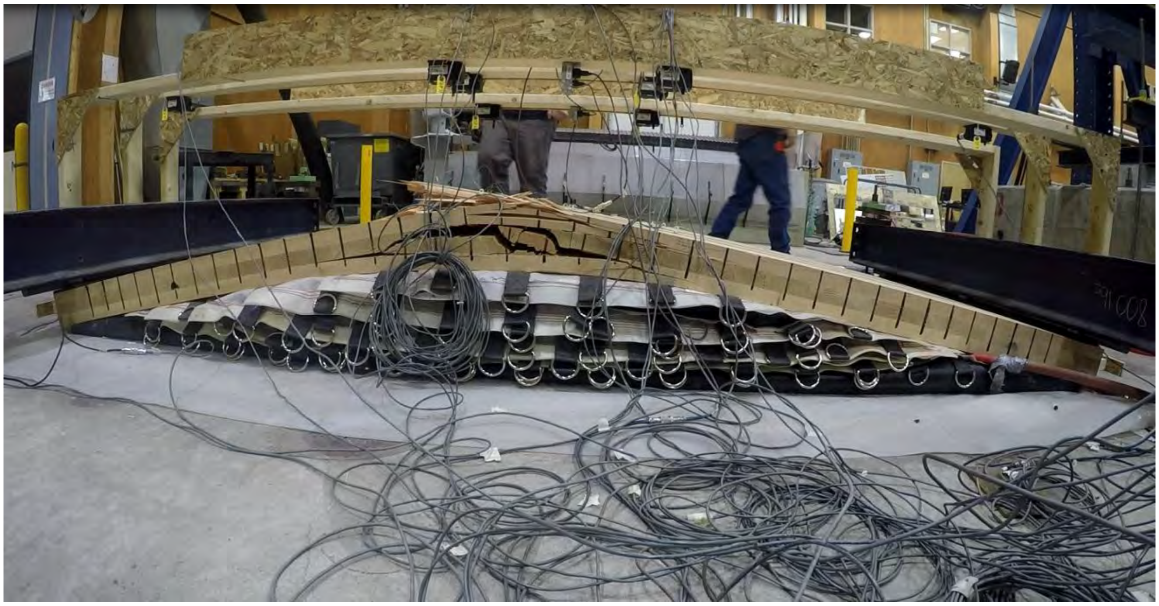
APPENDIX A

PHOTOGRAPHS OF EACH TEST

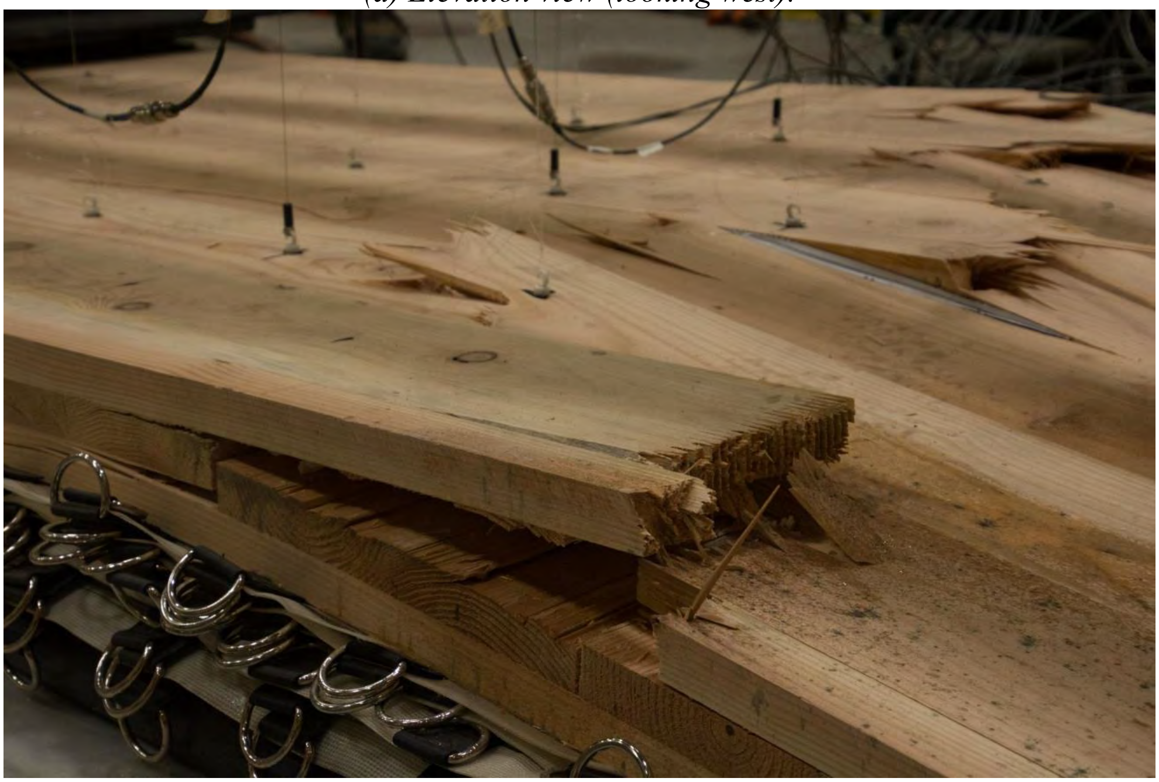
(a) Elevation view (looking west).

(b) Detail view (looking east).

Figure A-1. 3-Ply Grade V1 CLT – Test 1 (V1-1).

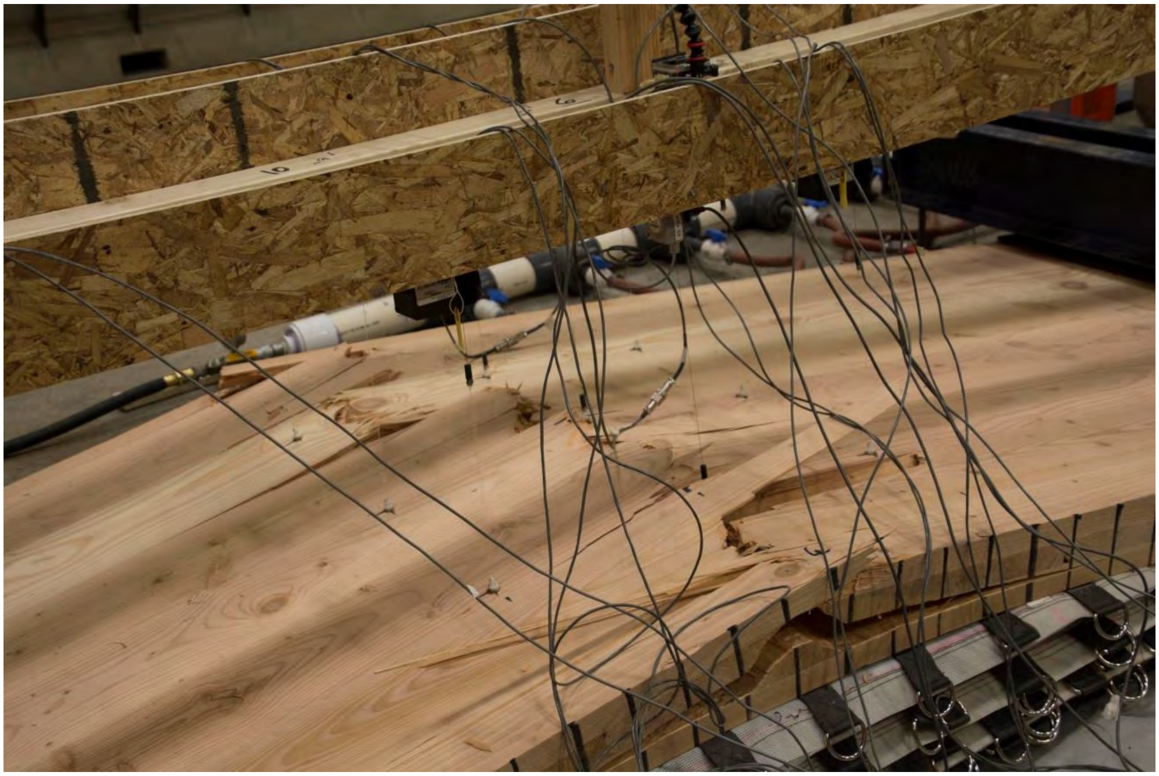


(a) Elevation view (looking west).



(b) Detail view (looking east). (Note white plastic between adjacent boards.)

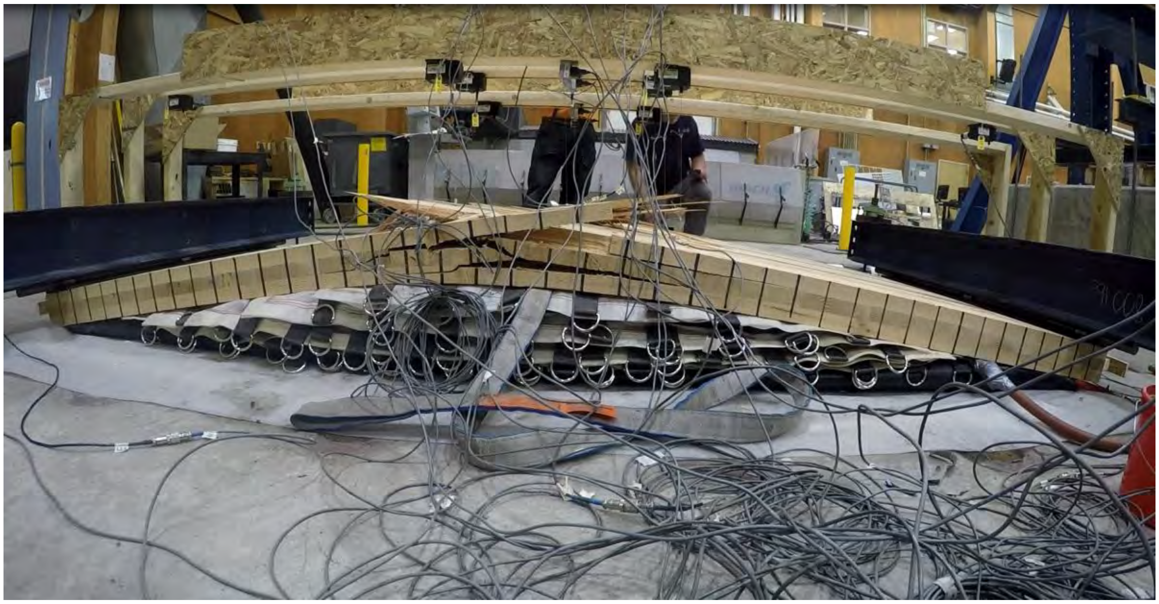
Figure A-2. 3-Ply Grade V1 CLT – Test 2 (V1-2).



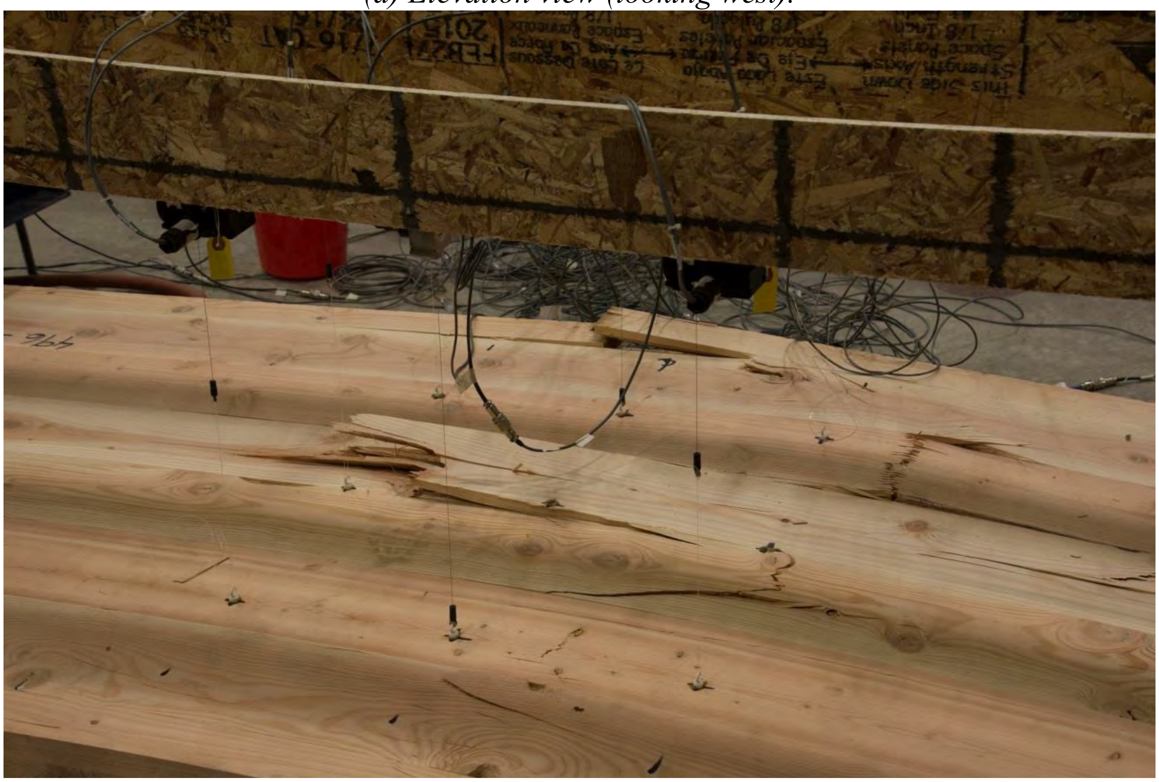
(a) Top view (looking west).

(b) Detail view (looking east).

Figure A-3. 3-Ply Grade V1 CLT – Test 3 (V1-3).

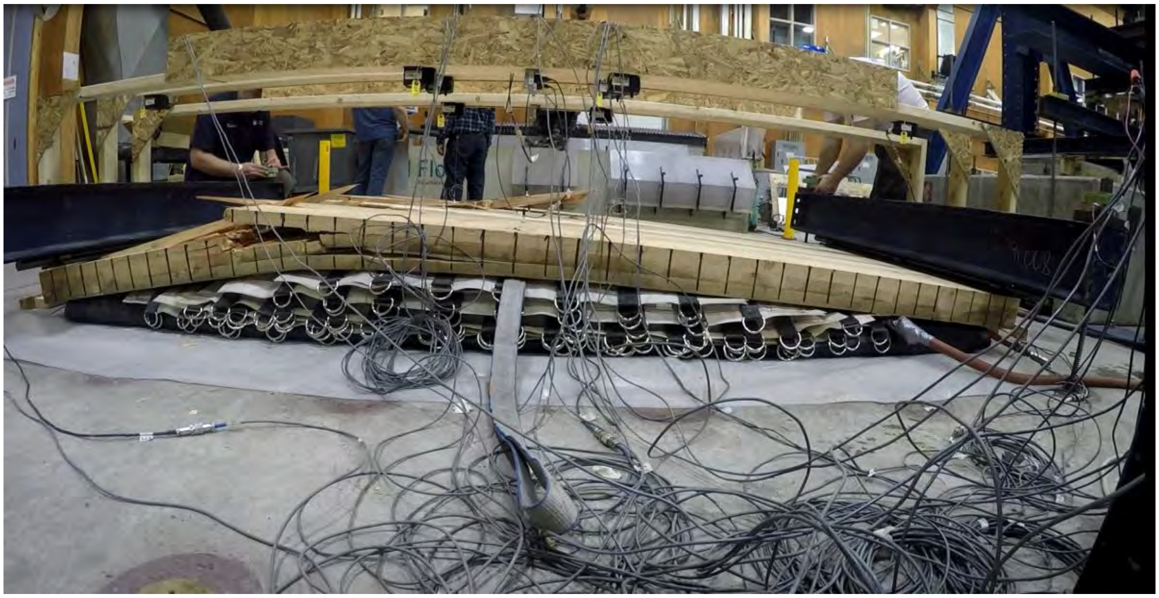


(a) Elevation view (looking west).



(b) Initiation of failure (looking east). (Note failure at finger joint outside middle two feet.)

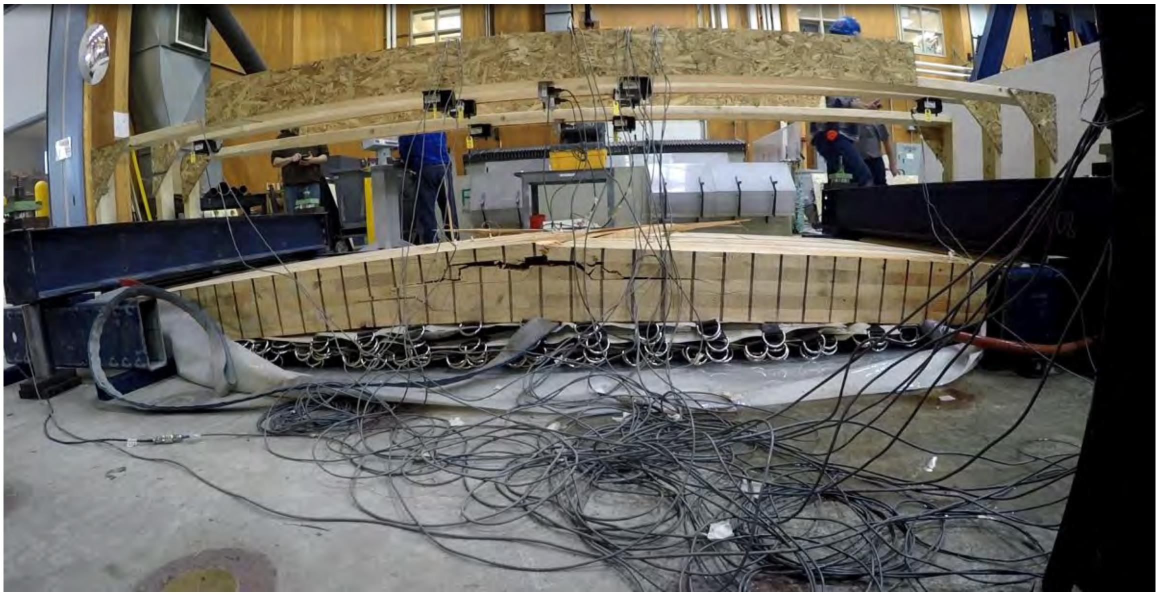
Figure A-4. 3-Ply Grade V1 CLT – Test 4 (V1-4).



(a) Elevation view (looking west).

(b) Top view (looking east). (Note failure at finger joints near support, no finger joints located near panel mid-span.)

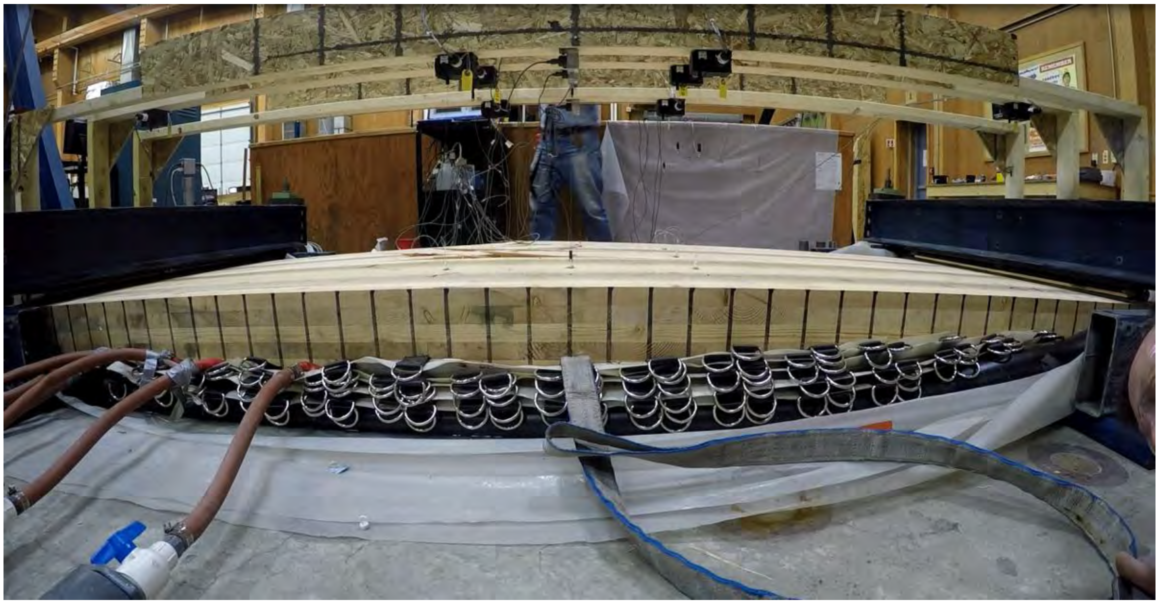
Figure A-5. 3-Ply Grade V1 CLT – Test 5 (V1-5).



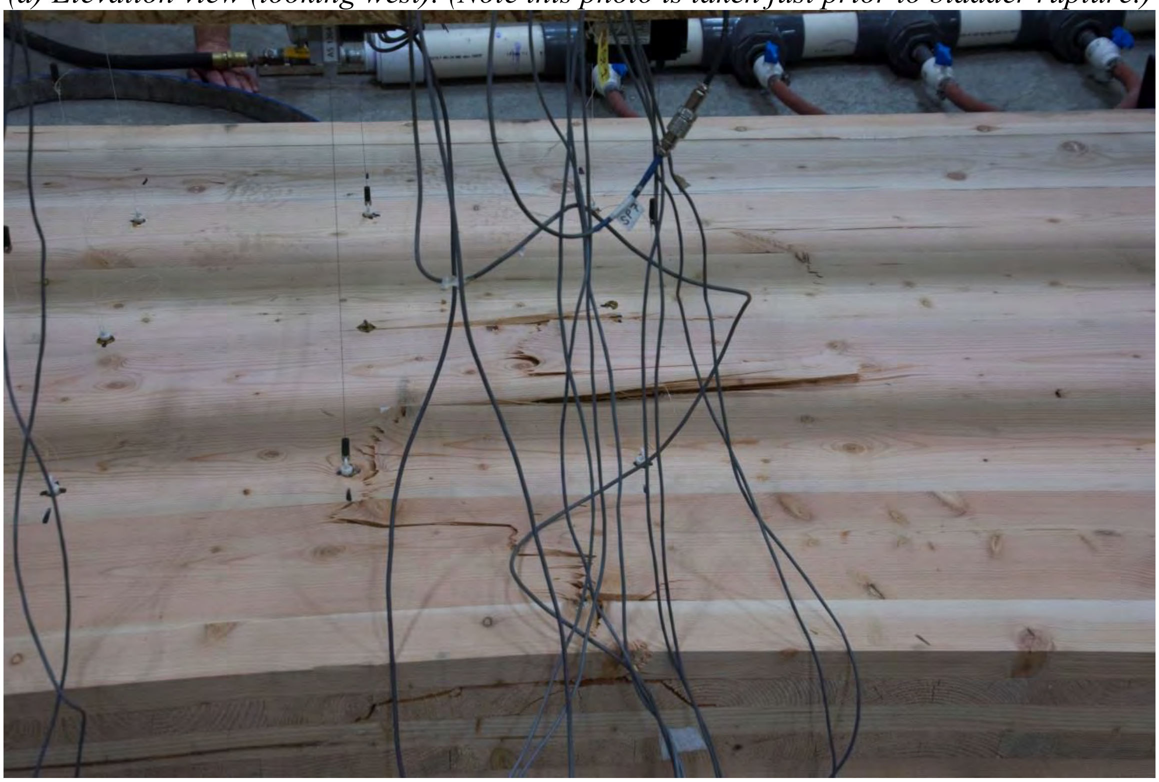
(a) Elevation view (looking west). (Note bladder leakage at lower right.)

(b) Top view (looking east).

Figure A-6. 5-Ply Grade V1 CLT – Test 1 (5V1-1).



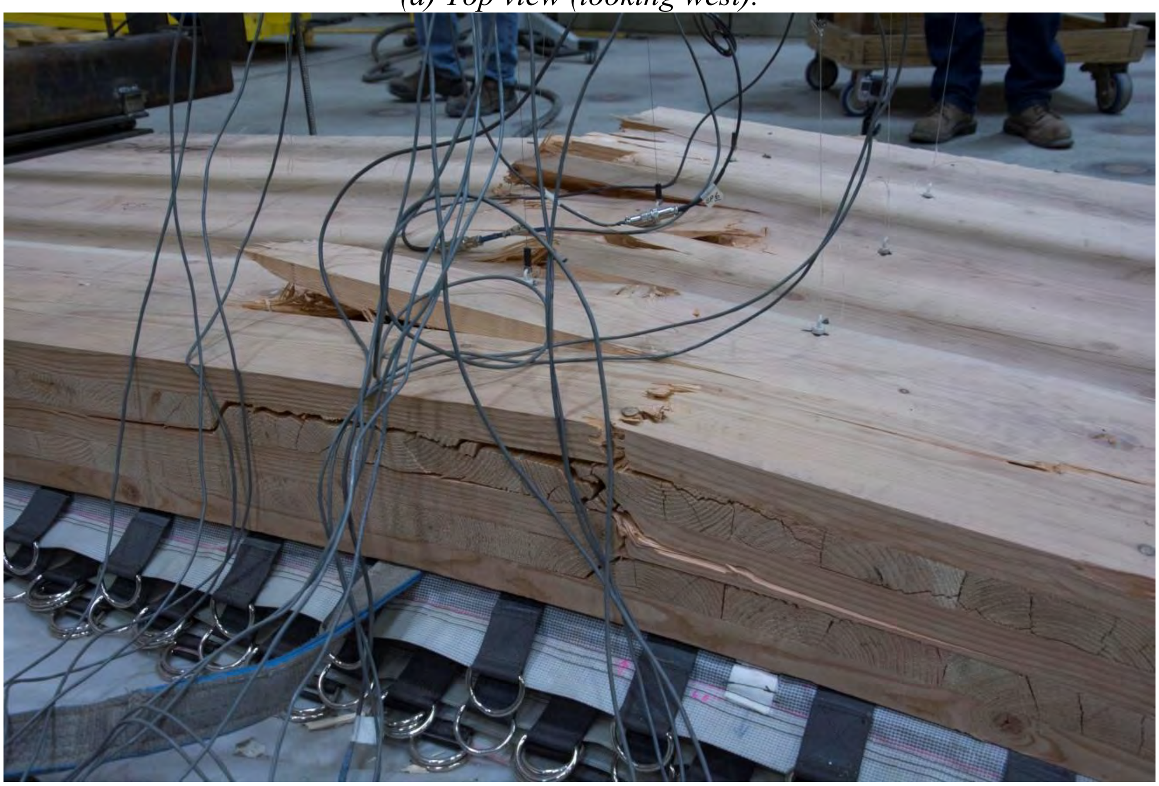
(a) Elevation view (looking west). (Note this photo is taken just prior to bladder rupture.)



(b) Top view (looking east).

Figure A-7. 5-Ply Grade V1 CLT – Test 2 (5V1-2).

(a) Top view (looking west).



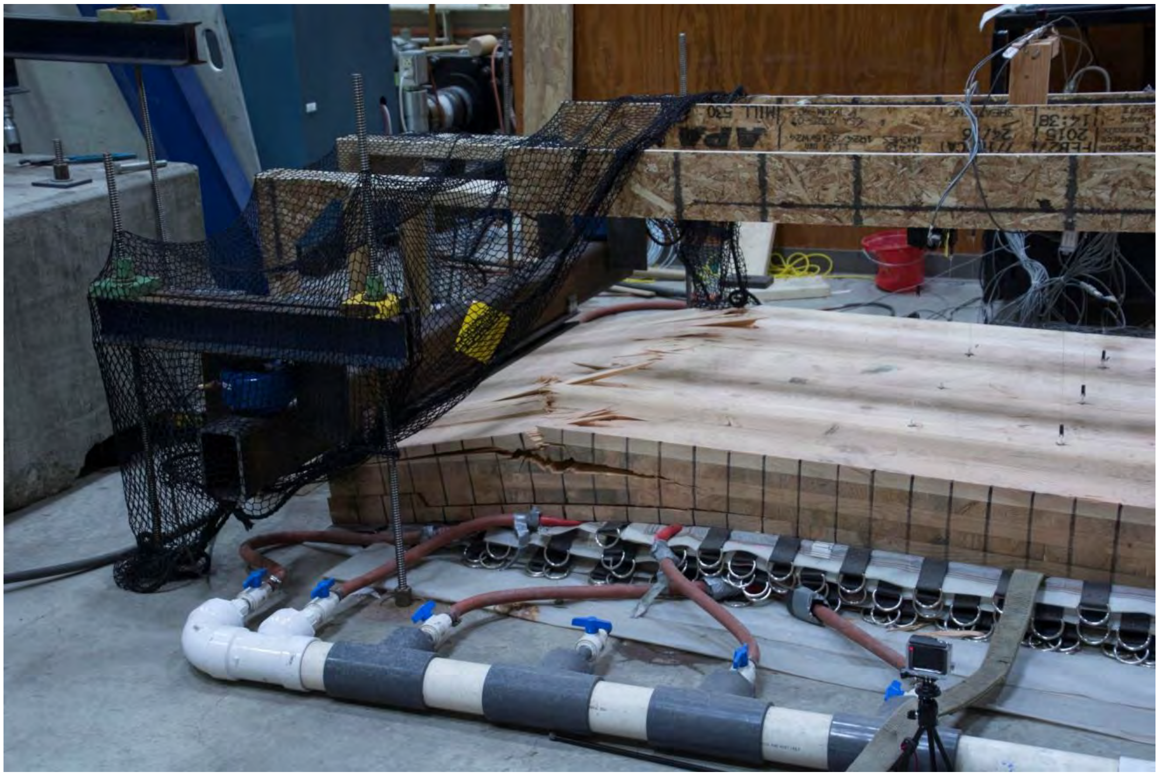
(b) Detail view (looking east).

Figure A-8. 5-Ply Grade V1 CLT – Test 3 (5V1-3).

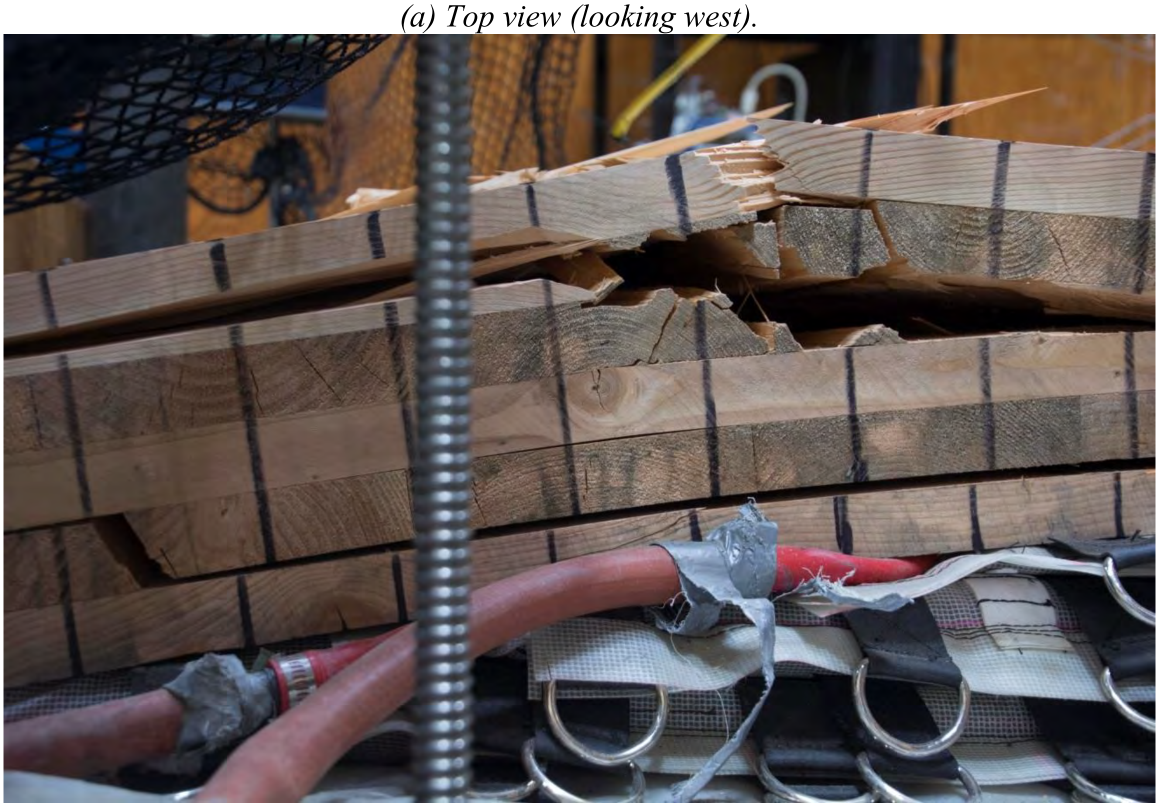
(a) Elevation view (looking west).

(b) Detail view (looking west).

Figure A-9. 5-Ply Grade V1 CLT – Test 4 (5V1-4).



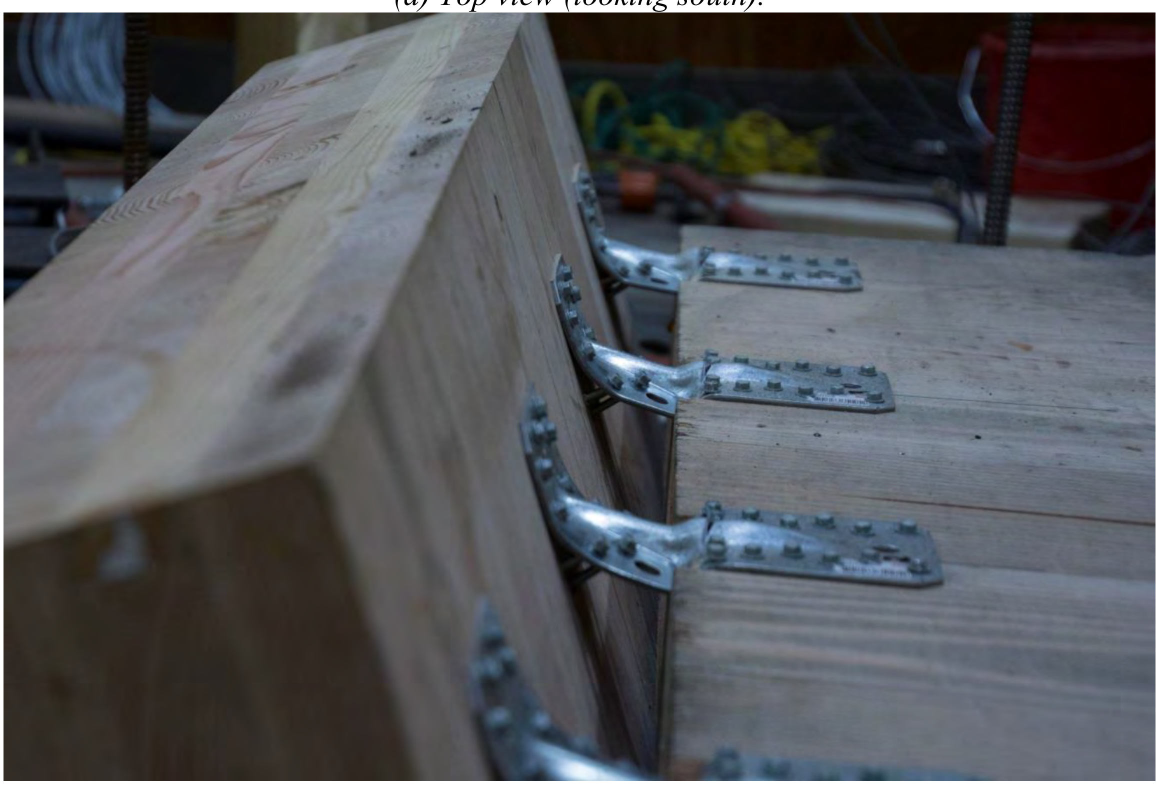
(a) Top view (looking west).



(b) Detail view (looking west). (Note slip between individual CLT plies.)

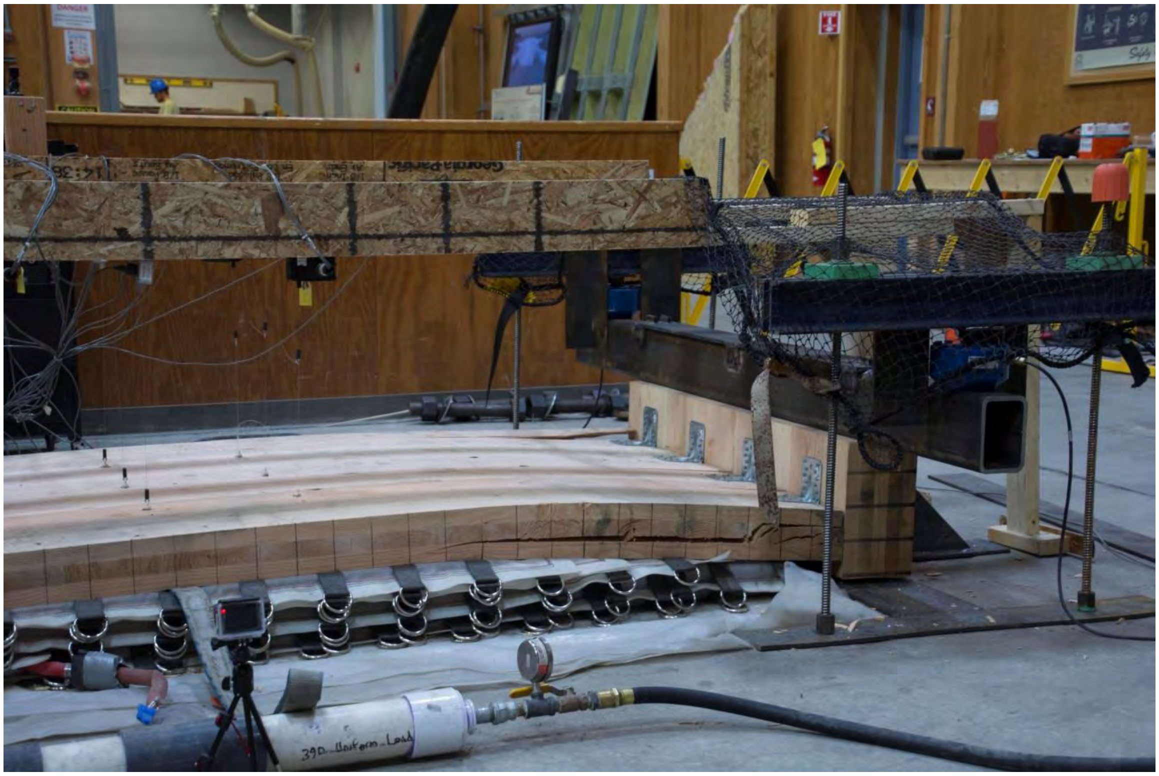
Figure A-10. 5-Ply Grade V1 CLT – Test 5 (5V1-5).

(a) Top view (looking south).

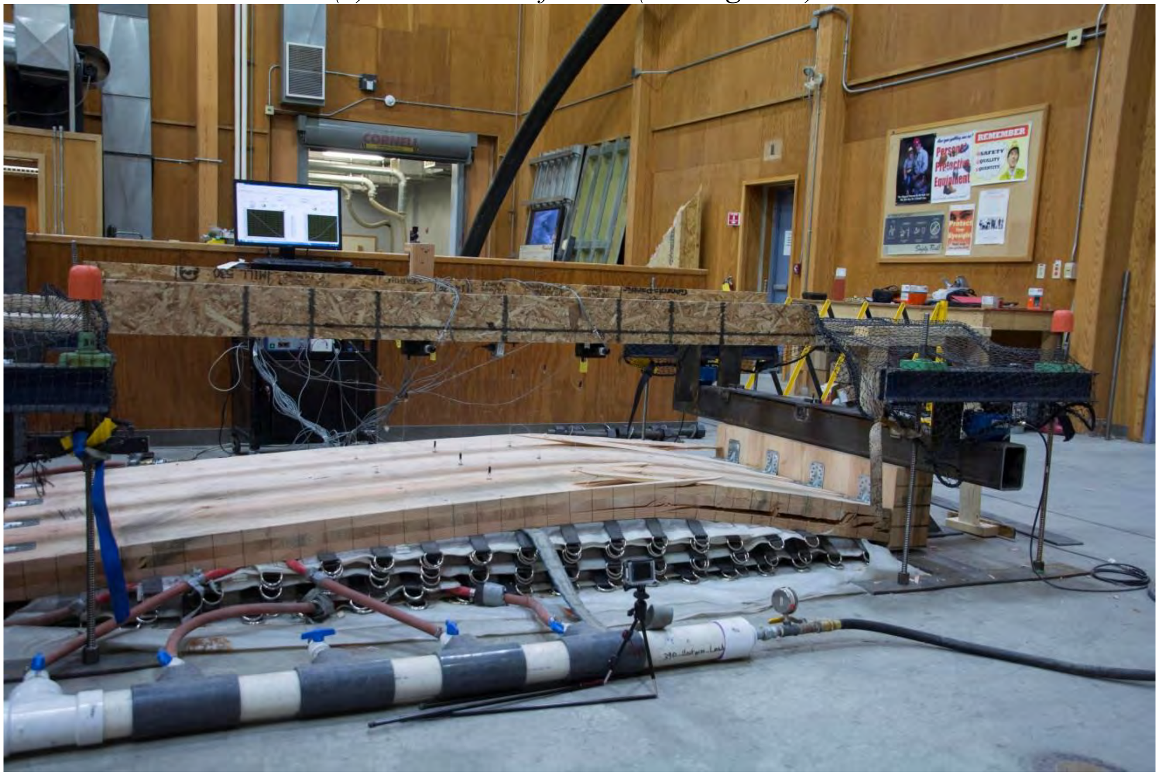


(b) Detail view (looking west).

Figure A-11. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 1 (V1CA-1).

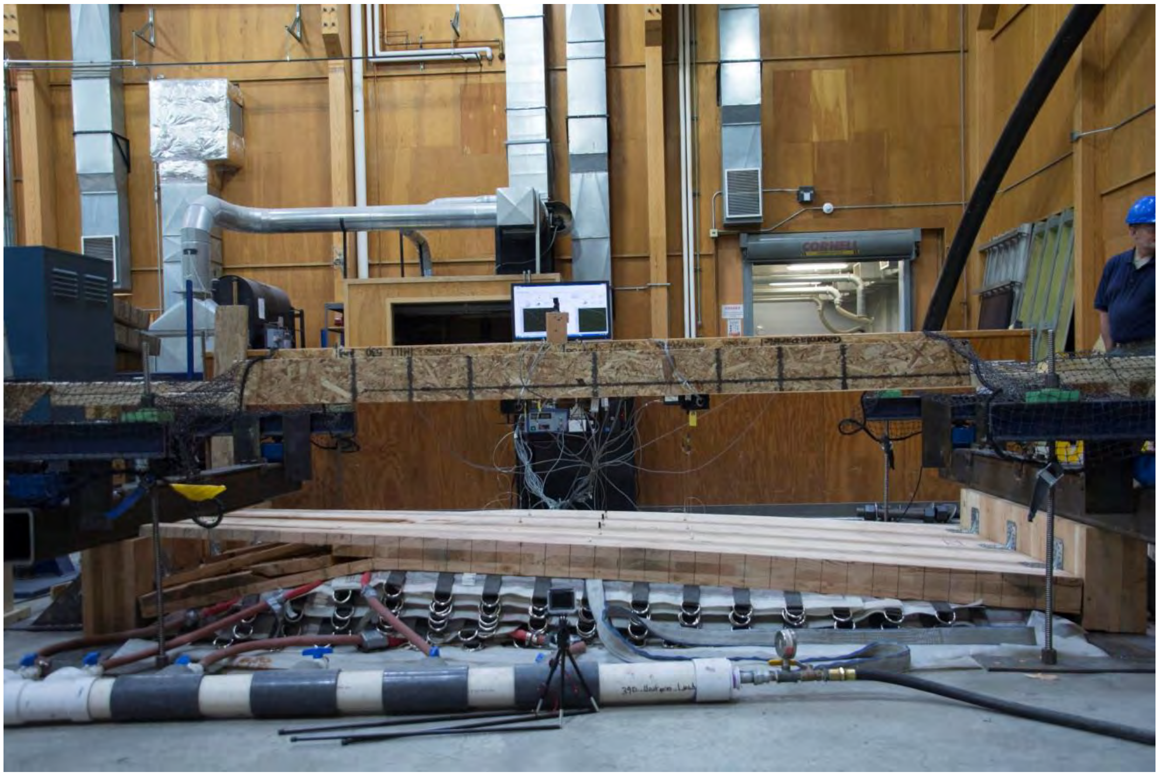


(a) Initial shear failure (looking west).

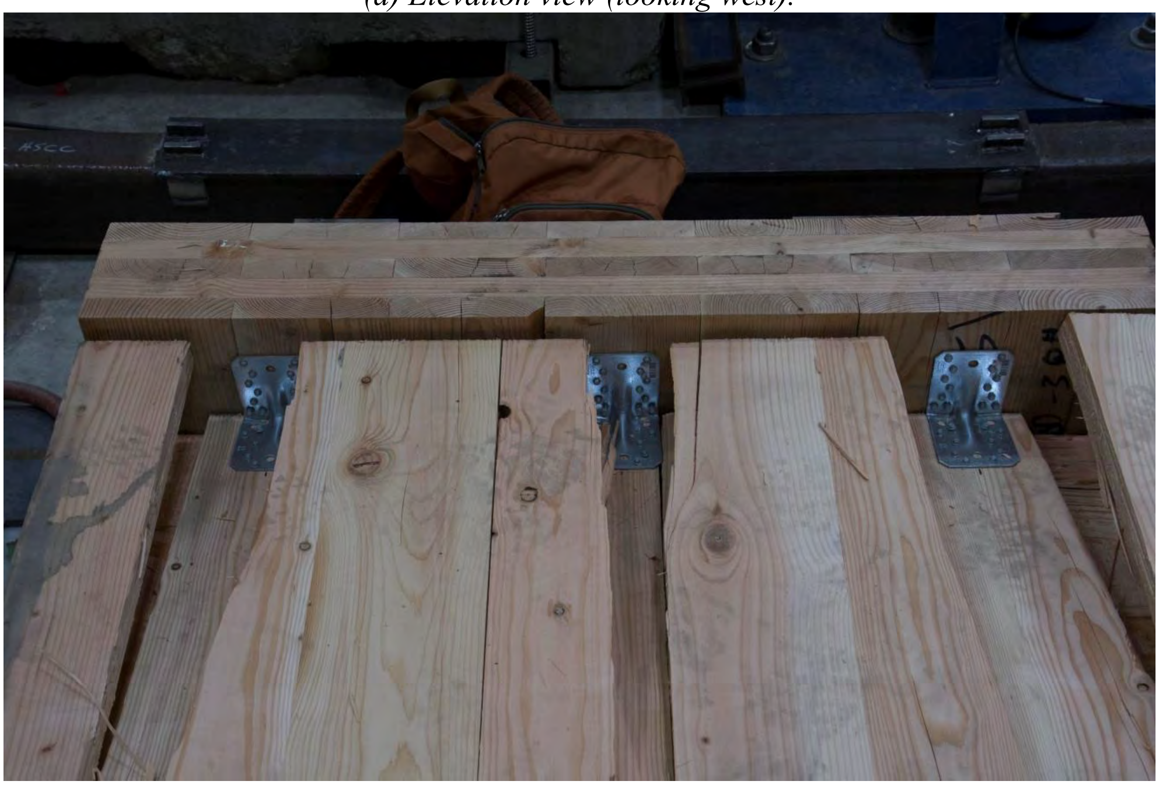


(b) Subsequent flexural failure (looking west).

Figure A-12. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 2 (V1CA-2).

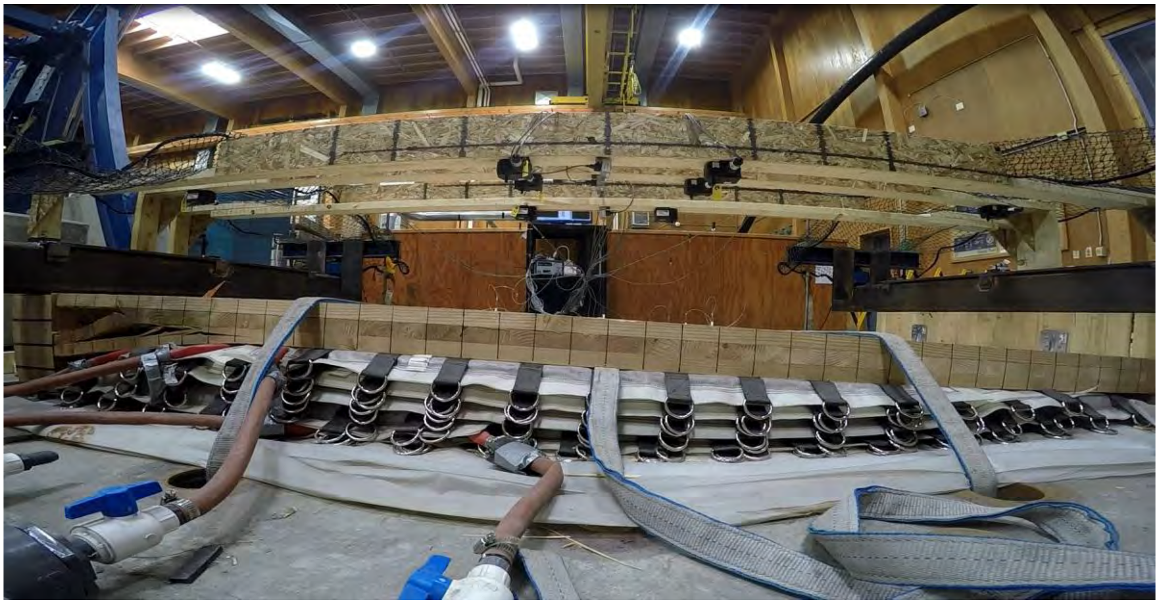


(a) Elevation view (looking west).



(b) Top view.

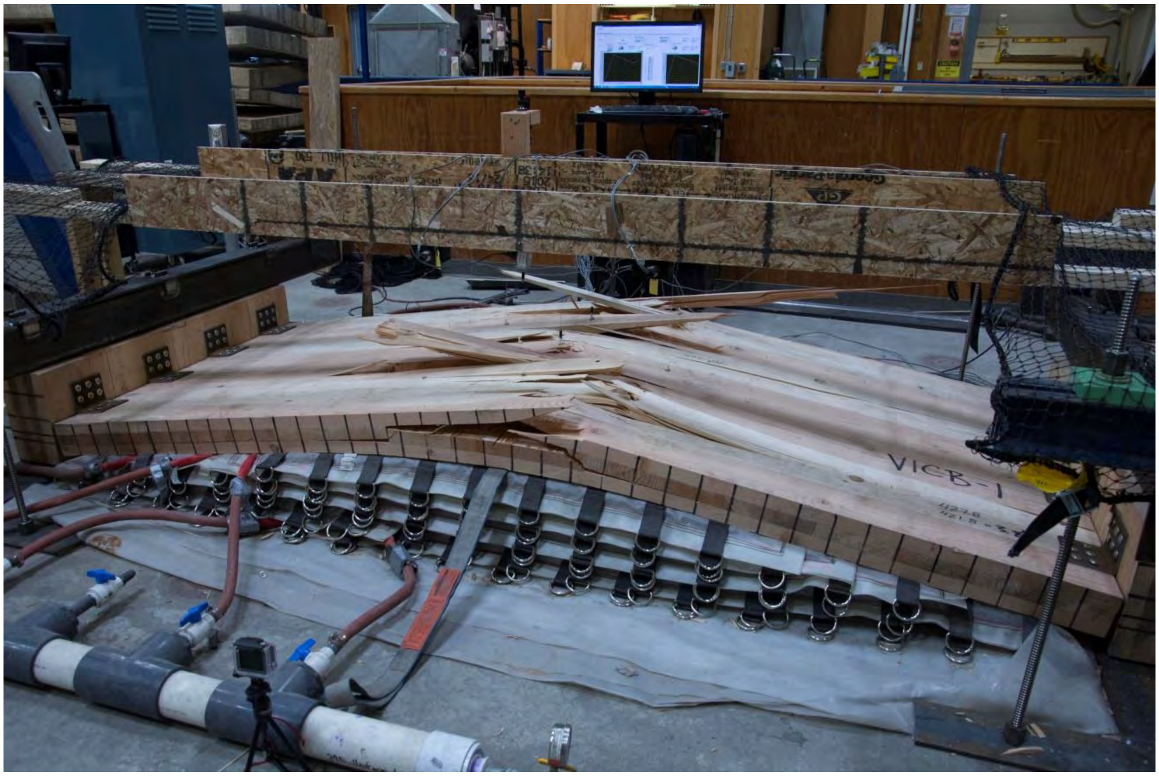
Figure A-13. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 3 (V1CA-3).



(a) Elevation view (looking west).

(b) Top view (looking west).

Figure A-14. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 4 (V1CA-4).

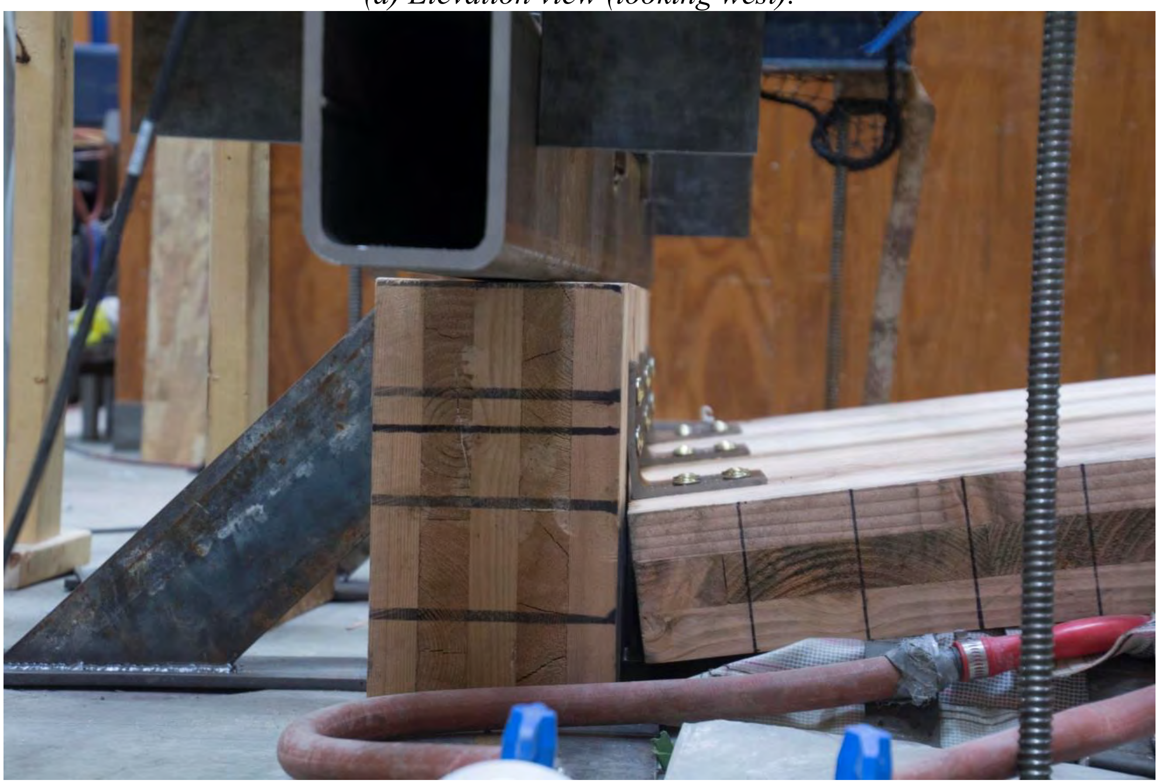


(a) Top View (looking west).

(b) Detail view – bracket deformation (looking west).

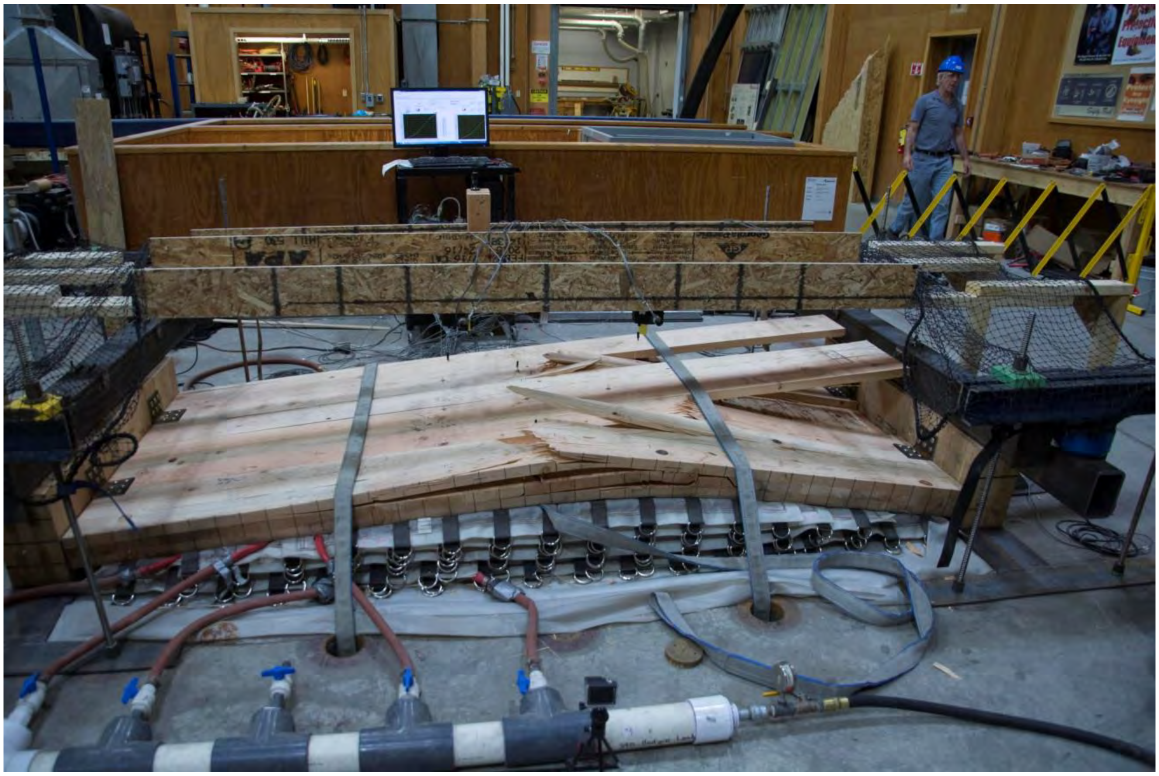
Figure A-15. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 1 (V1CB-1).

(a) Elevation view (looking west).



(b) Detail view – bracket deformation (looking west).

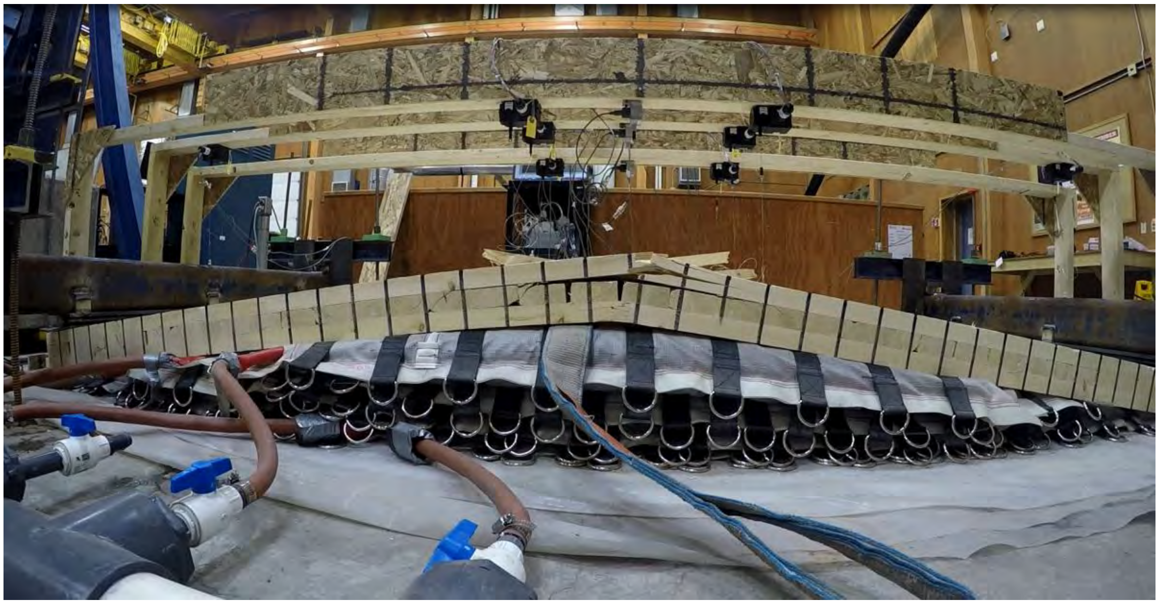
Figure A-16. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 2 (V1CB-2).



(a) Top view (looking west).

(b) Detail view.

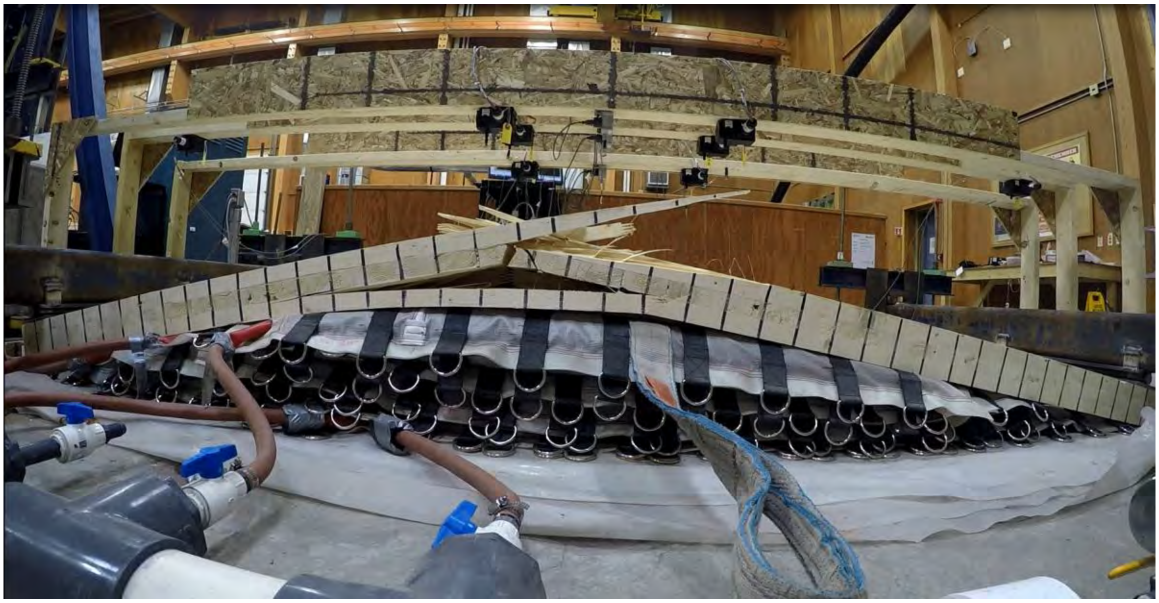
Figure A-17. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 3 (V1CB-3).



(a) Elevation view (looking west).

(b) Detail view (looking west).

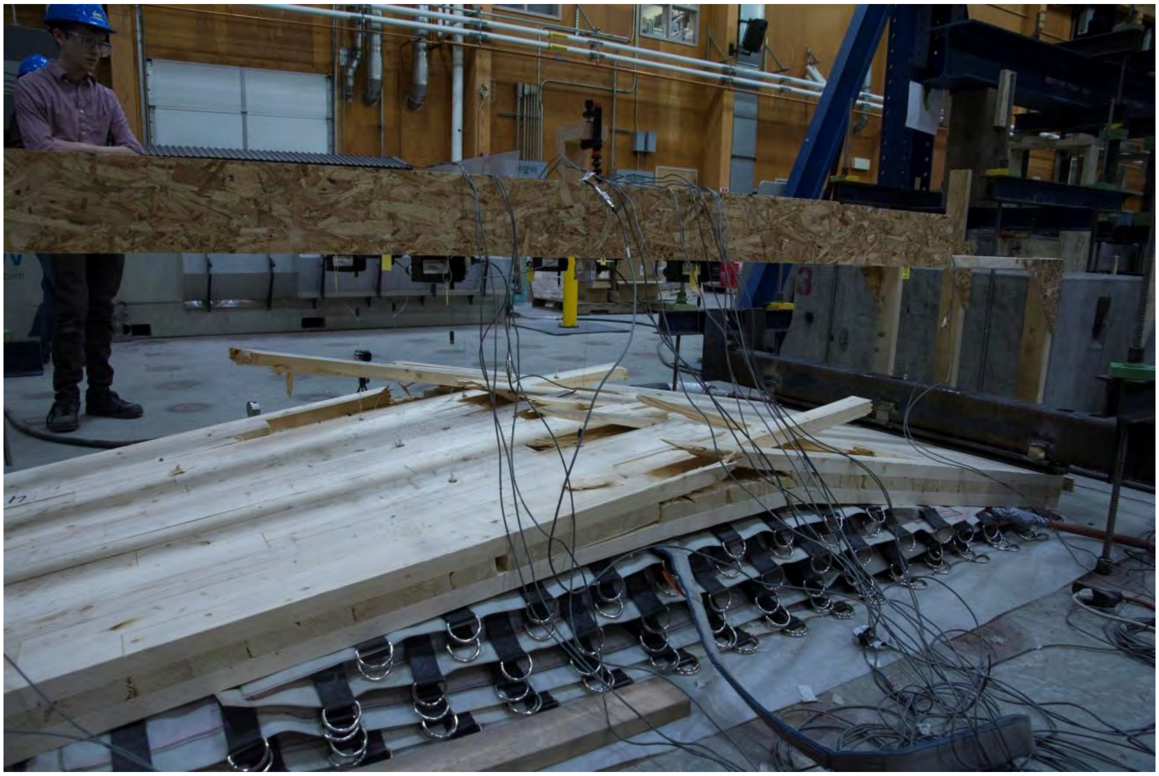
Figure A-18. 3-Ply Grade E1 CLT – Test 1 (E1-1).



(a) Elevation view (looking west).

(b) Detail view (looking west).

Figure A-19. 3-Ply Grade E1 CLT – Test 2 (E1-2).



(a) Overall view (looking west).

(b) Detail view (looking east).

Figure A-20. 3-Ply Grade E1 CLT – Test 3 (E1-3).

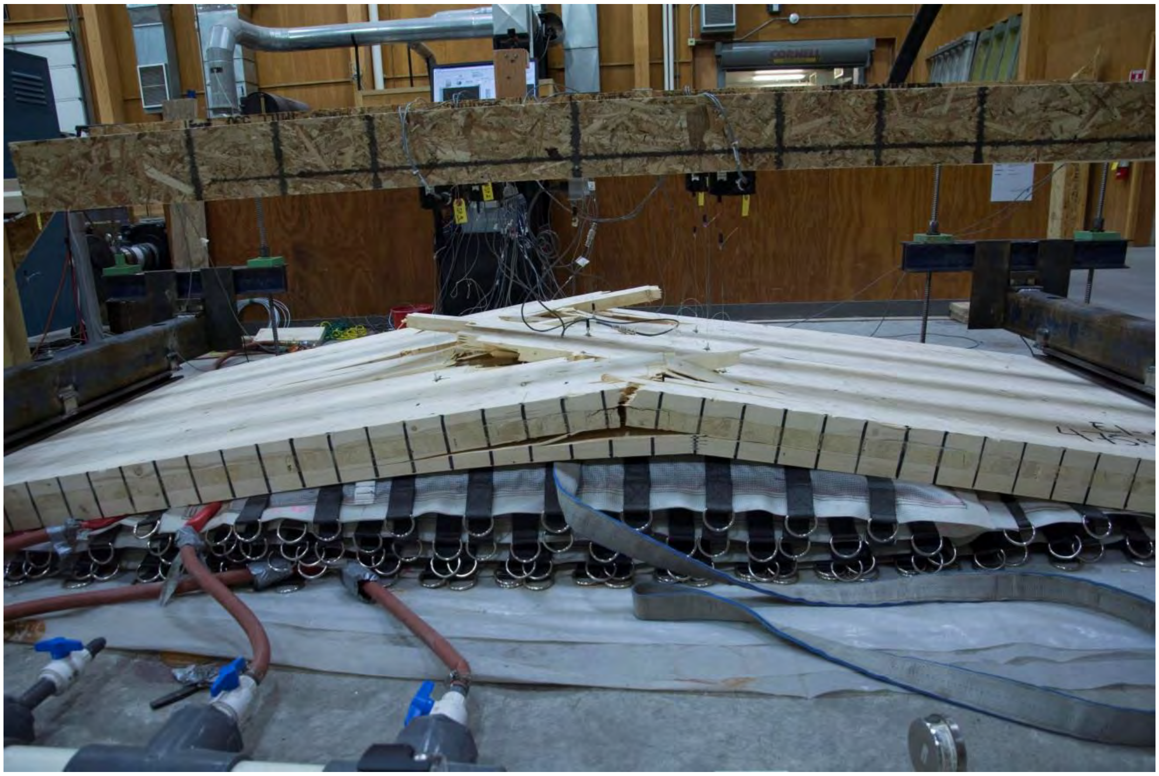


Figure A-21. 3-Ply Grade E1 CLT – Test 4 (E1-4).

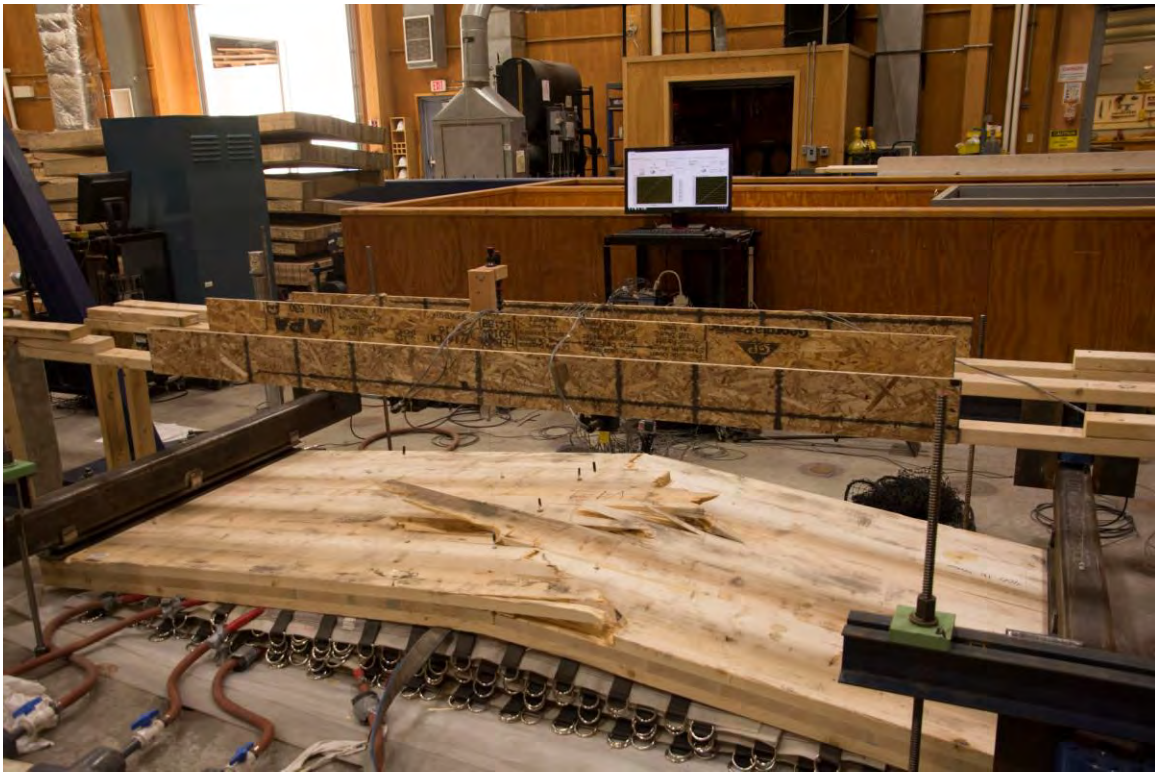


Figure A-22. 3-Ply Grade V4 CLT – Test 1 (V4-1).

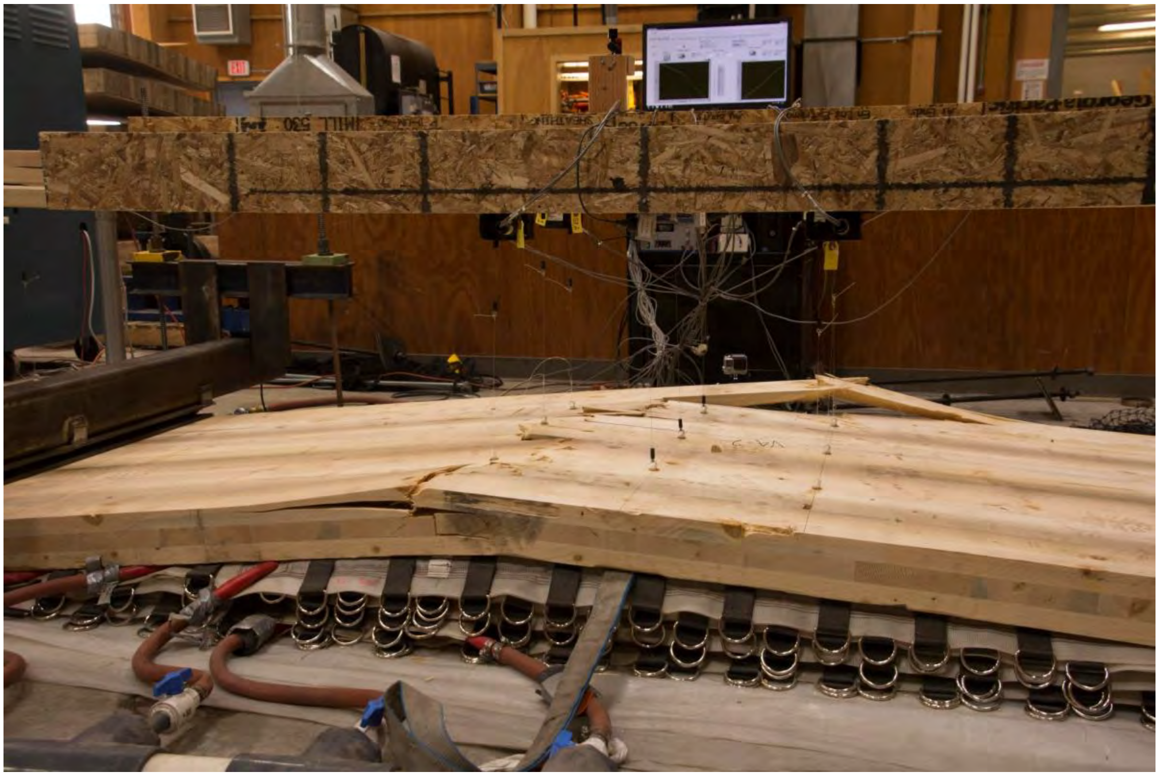


Figure A-23. 3-Ply Grade V4 CLT – Test 2 (V4-2).

(a) Initial failure.

(b) Subsequent failure.

Figure A-24. 3-Ply Grade V4 CLT – Test 3 (V4-3).

(a) Elevation view (looking west).

(b) Shear slip at end.

Figure A-25. 3-Ply Grade V4 CLT – Test 4 (V4-4).

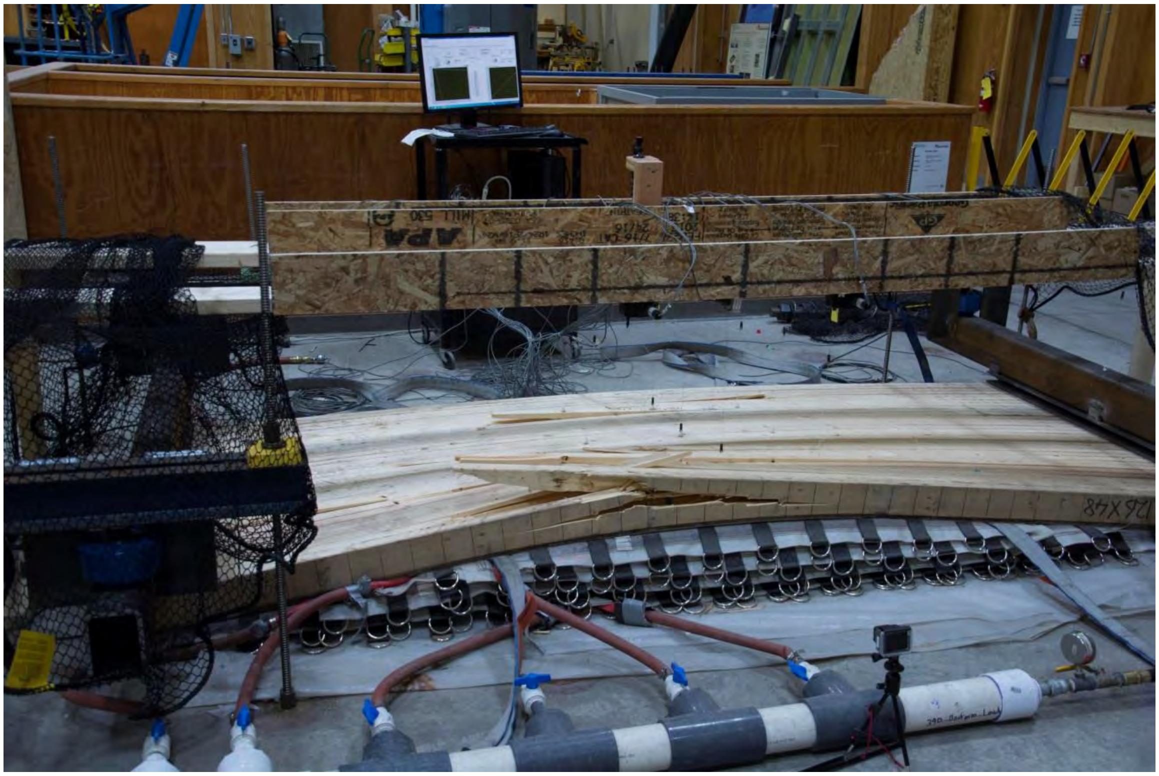


Figure A-26. 2x4 NLT – Test 1 (4NLT-1).

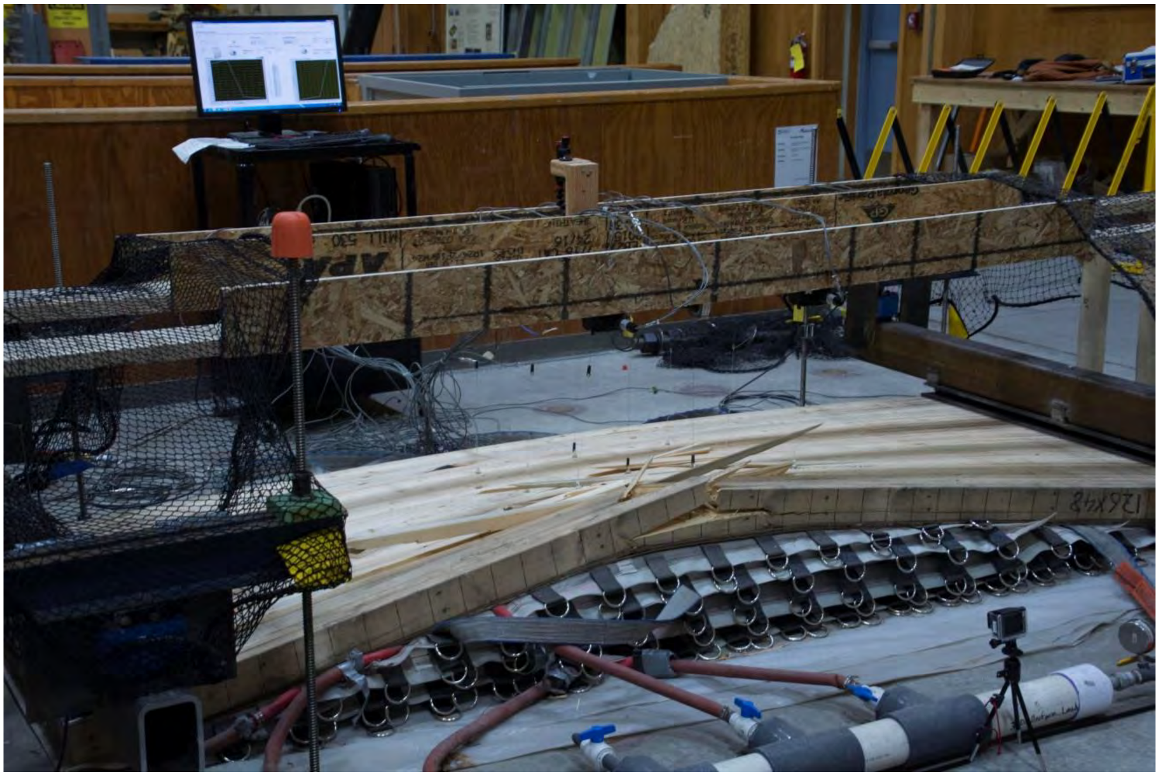
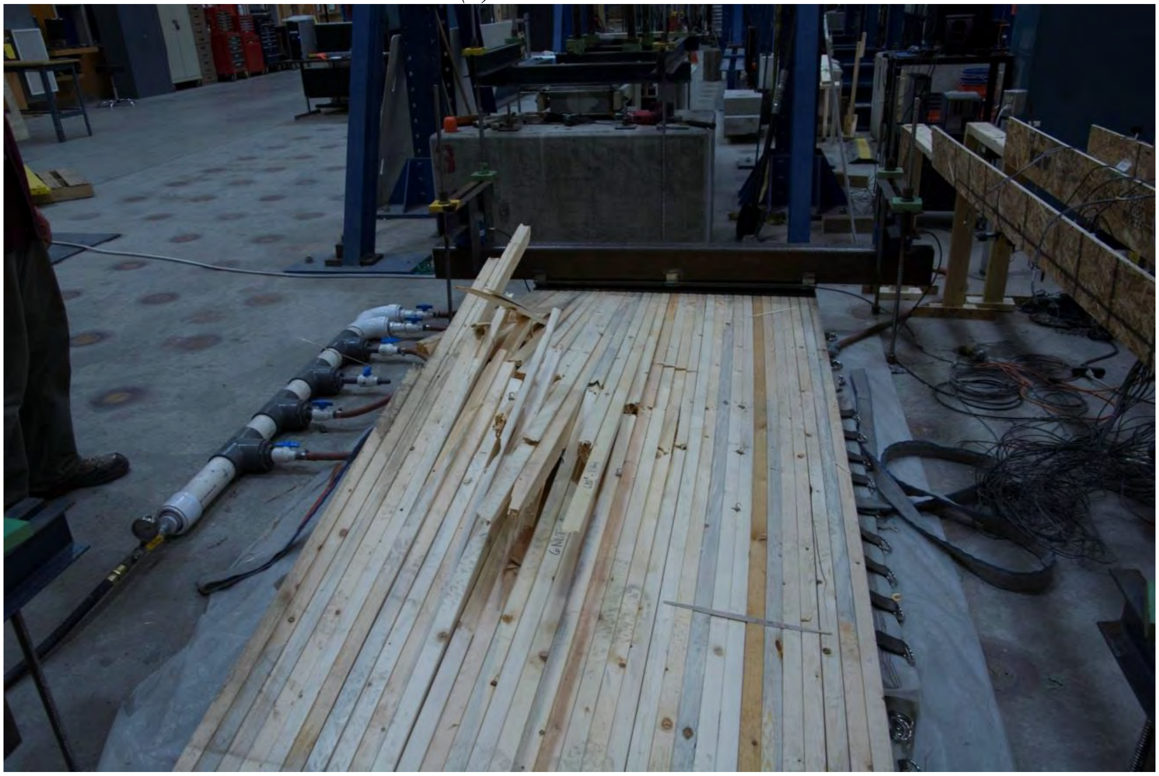


Figure A-27. 2x4 NLT – Test 2 (4NLT-2).

Figure A-28. 2x4 NLT – Test 3 (4NLT-3).

(a) Elevation view.



(b) Top view.

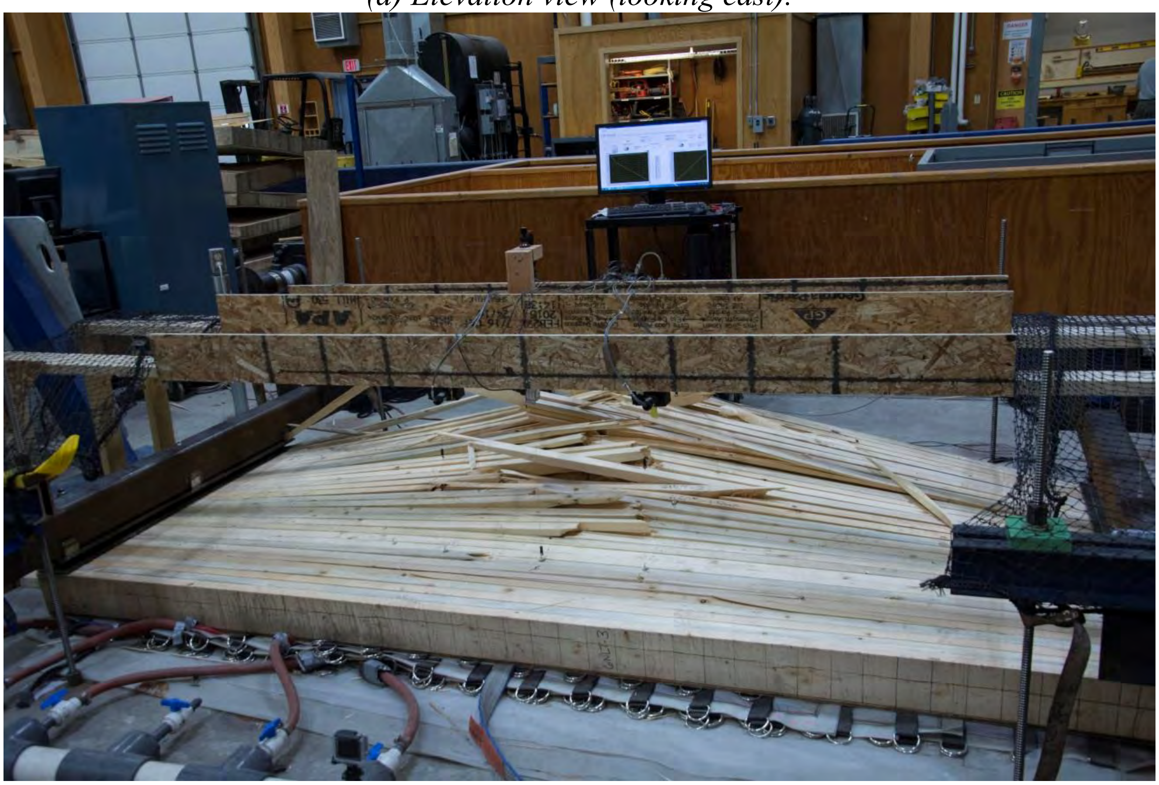
Figure A-29. 2x6 NLT – Test 1 (6NLT-1).

(a) Elevation view.

(b) Detail view.

Figure A-30. 2x6 NLT – Test 2 (6NLT-2).

(a) Elevation view (looking east).

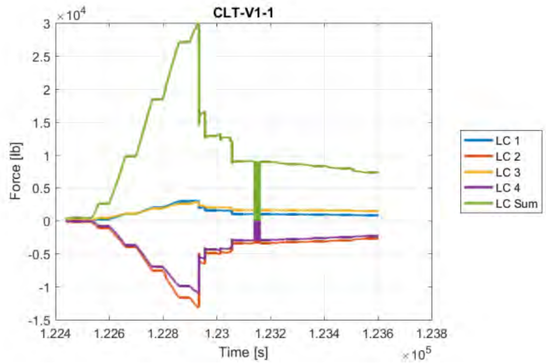


(b) Elevation view (looking west).

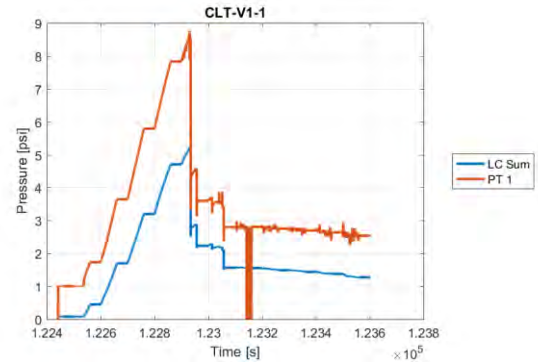
Figure A-31. 2x6 NLT – Test 3 (6NLT-3).

APPENDIX B

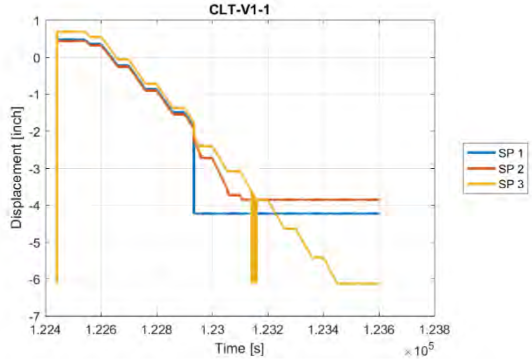
RECORDED DATA FROM EACH TEST



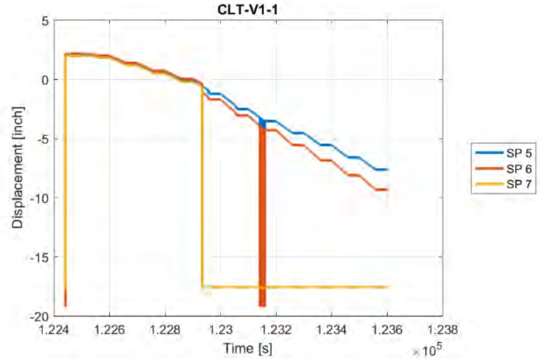
(a) LC.



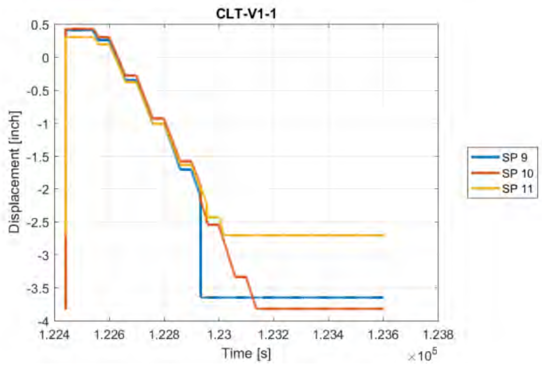
(b) LC vs. PT 1.



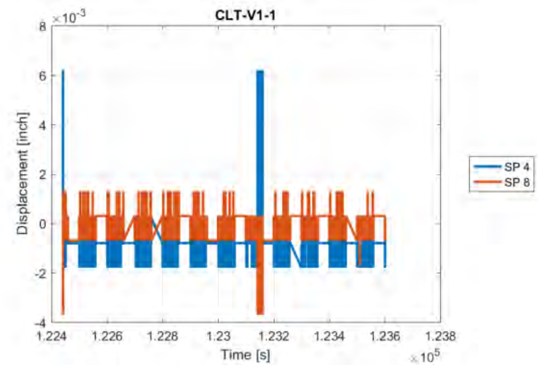
(c) SP 1 – SP 3.



(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

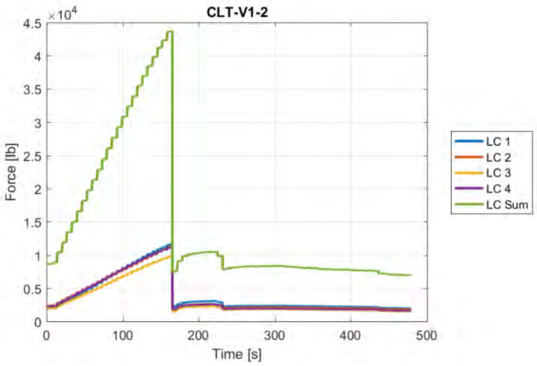


(f) SP 4 & SP 8.

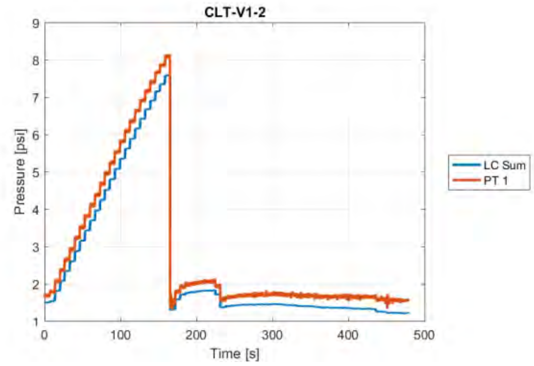
N/A

(g) Average SP.

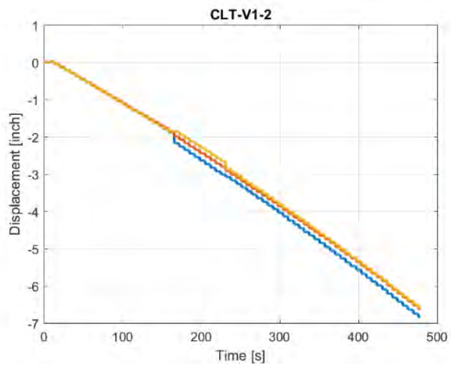
Figure B-1. 3-Ply Grade V1 CLT – Test 1 (V1-1).



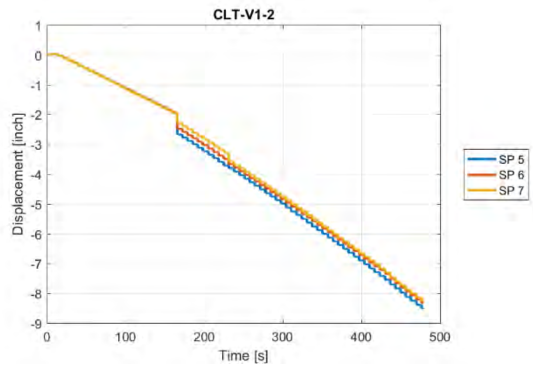
(a) LC.



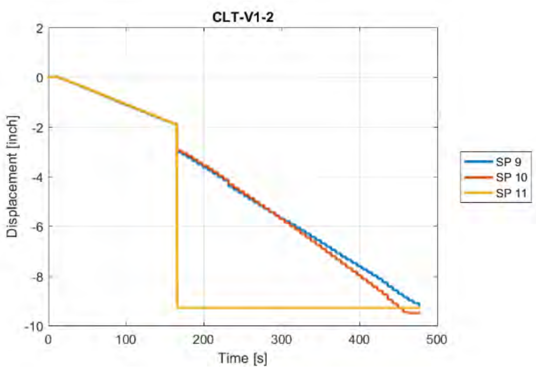
(b) LC vs. PT 1.



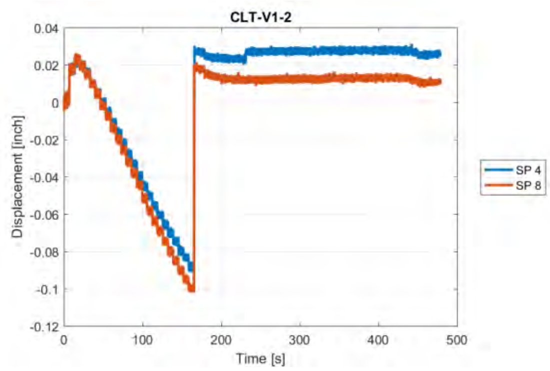
(c) SP 1 – SP 3.



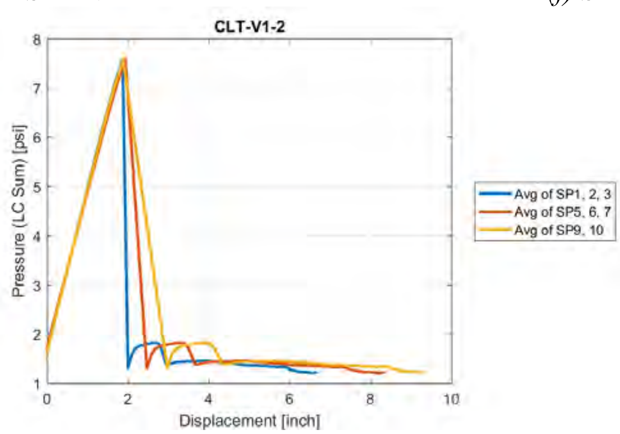
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

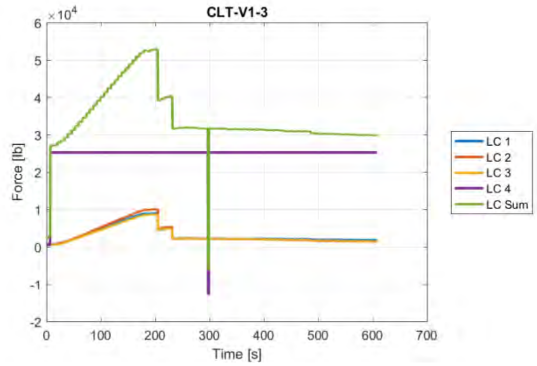


(f) SP 4 & SP 8.

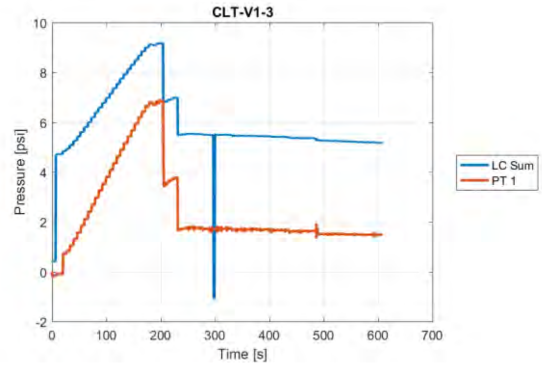


(g) Average SP.

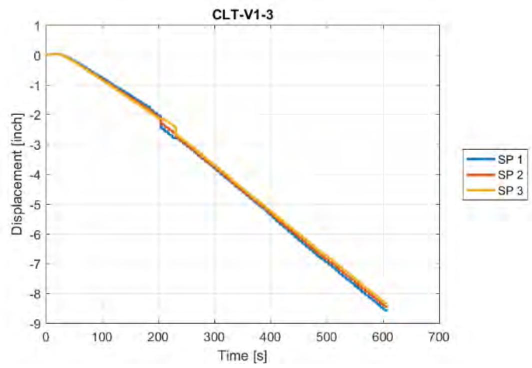
Figure B-2. 3-Ply Grade V1 CLT – Test 2 (V1-2).



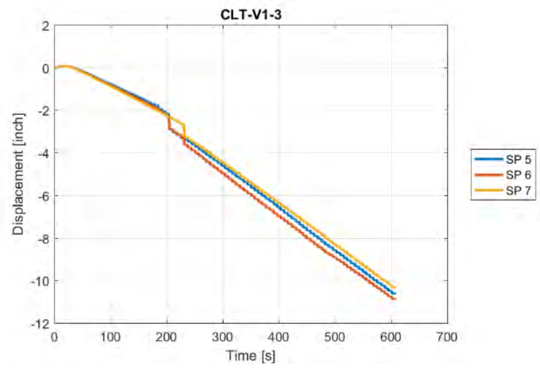
(a) LC.



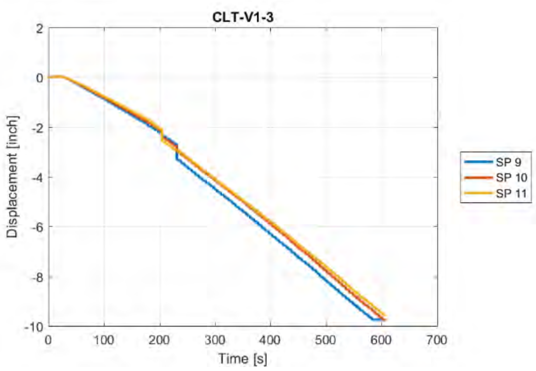
(b) LC vs. PT 1.



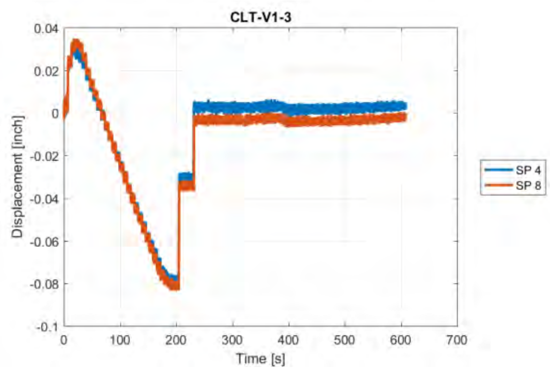
(c) SP 1 – SP 3.



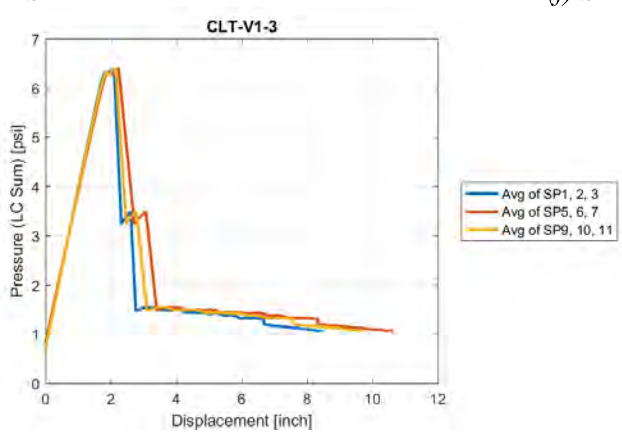
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

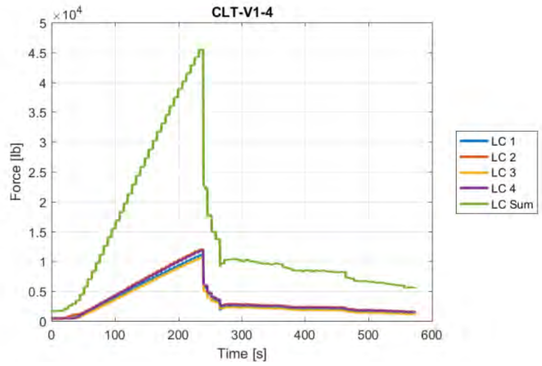


(f) SP 4 & SP 8.

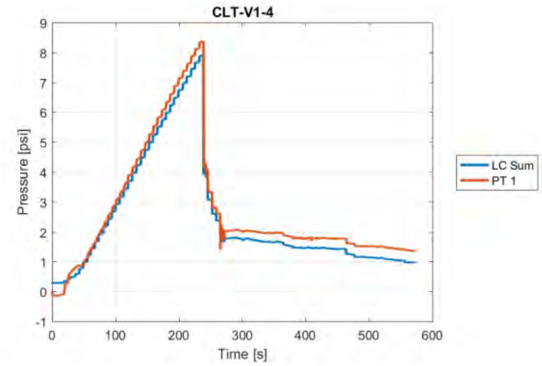


(g) Average SP.

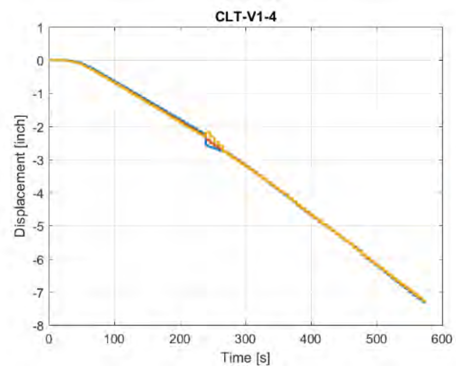
Figure B-3. 3-Ply Grade V1 CLT – Test 3 (V1-3).



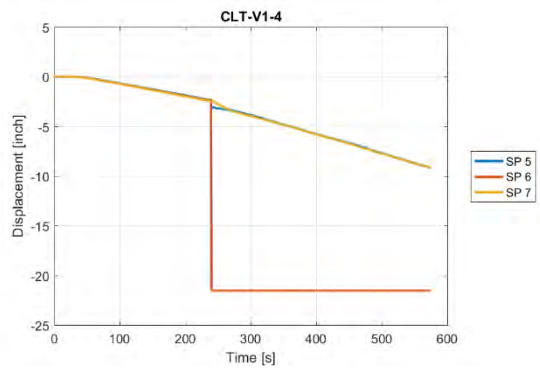
(a) LC.



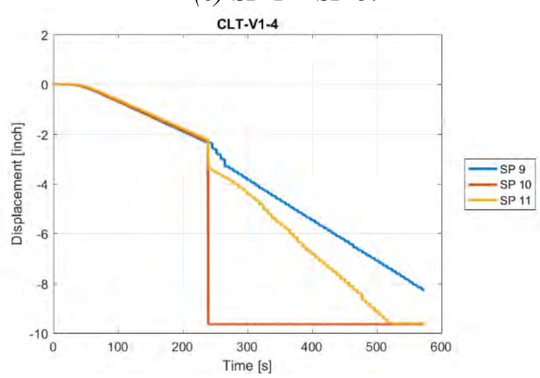
(b) LC vs. PT 1.



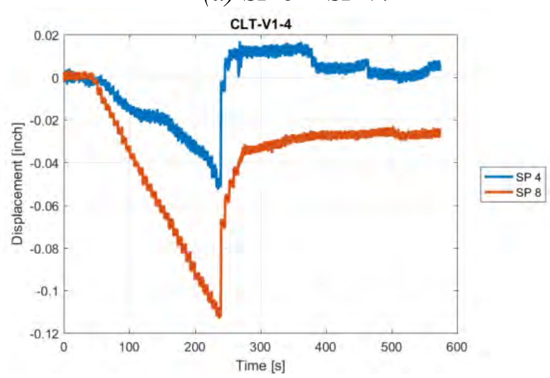
(c) SP 1 – SP 3.



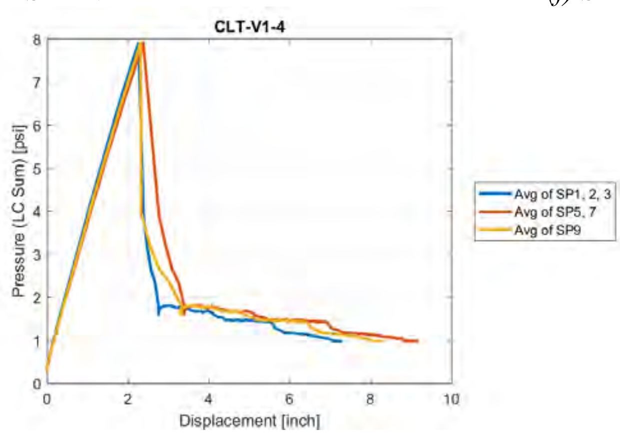
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

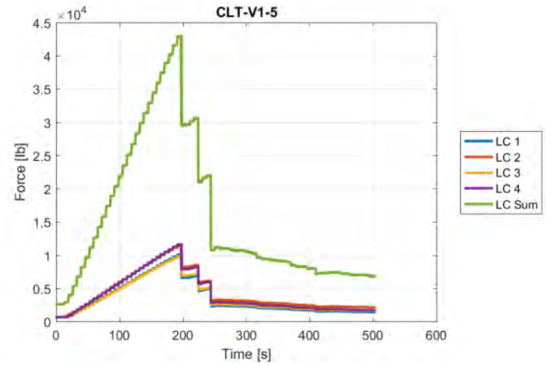


(f) SP 4 & SP 8.

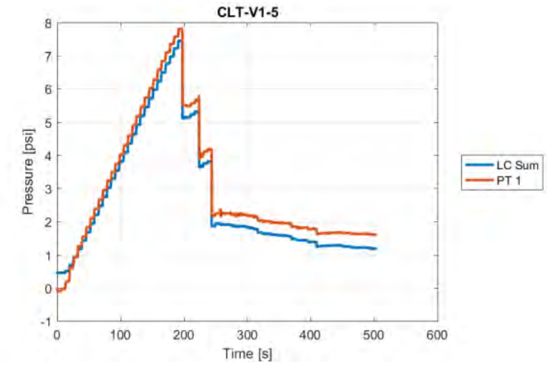


(g) Average SP.

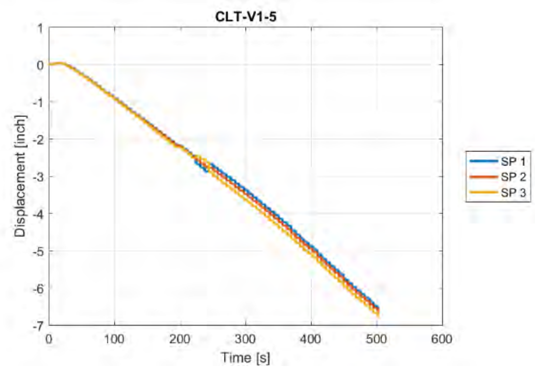
Figure B-4. 3-Ply Grade V1 CLT – Test 4 (V1-4).



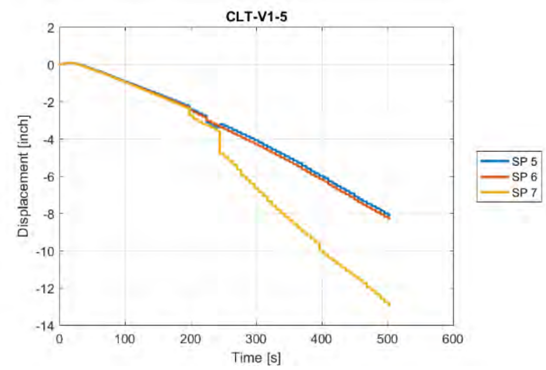
(a) LC.



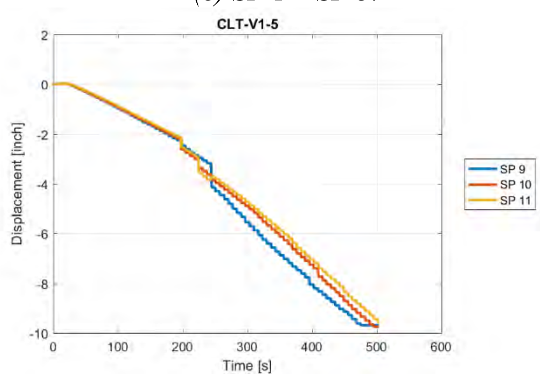
(b) LC vs. PT 1.



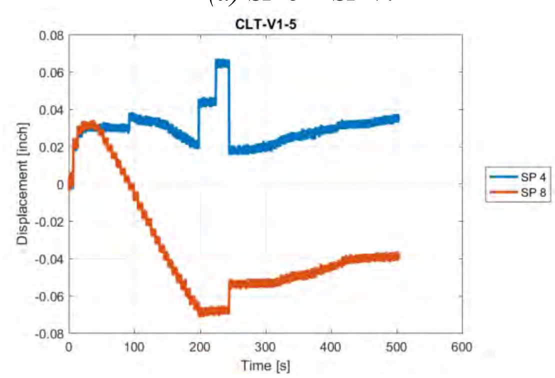
(c) SP 1 – SP 3.



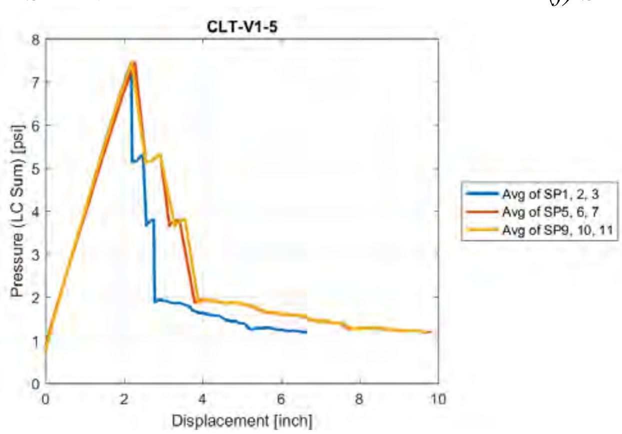
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

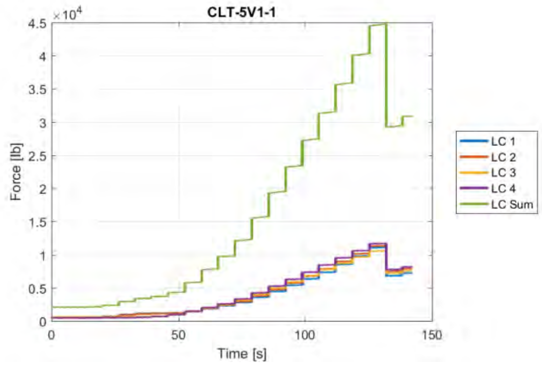


(f) SP 4 & SP 8.

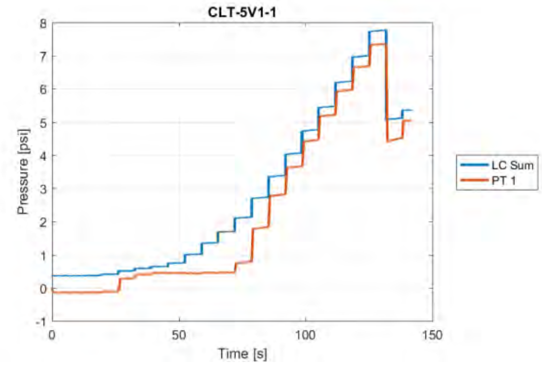


(g) Average SP.

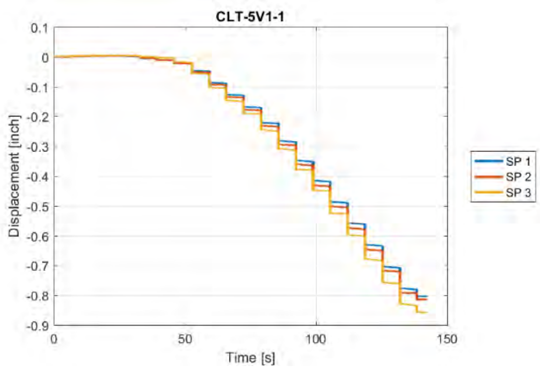
Figure B-5. 3-Ply Grade V1 CLT – Test 5 (V1-5).



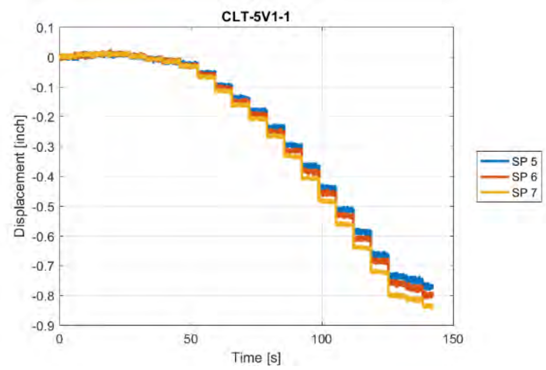
(a) LC.



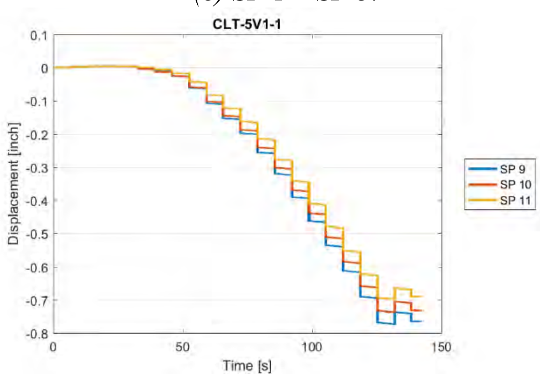
(b) LC vs. PT 1.



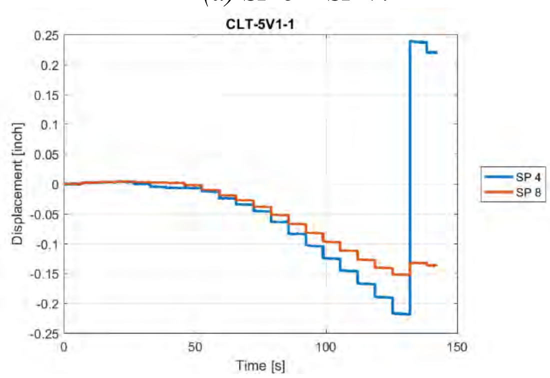
(c) SP 1 – SP 3.



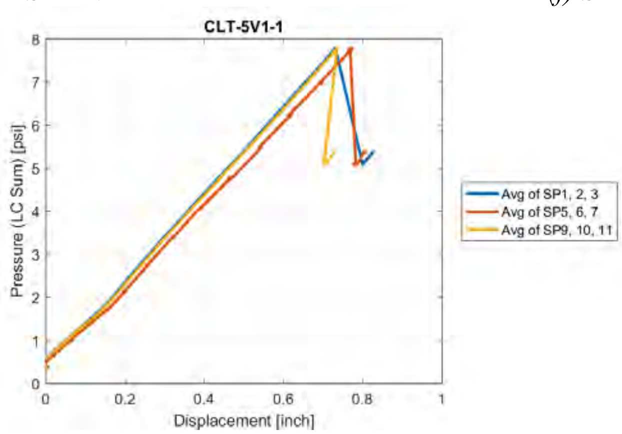
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.



(f) SP 4 & SP 8.



(g) Average SP.

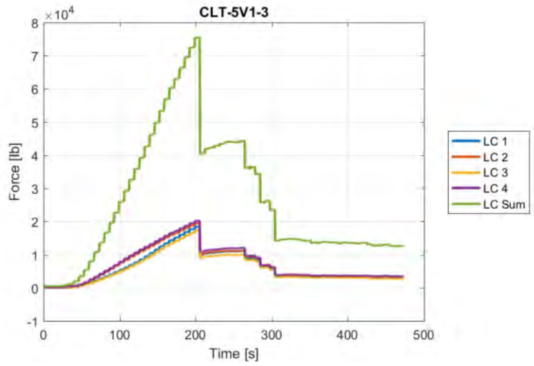
Figure B-6. 5-Ply Grade V1 CLT – Test 1 (5V1-1).

N/A	N/A
(a) LC.	*(b) LC vs. PT 1.*
N/A	N/A
(c) SP 1 – SP 3.	*(d) SP 5 – SP 7.*
N/A	N/A
(e) SP 9 – SP 11.	*(f) SP 4 & SP 8.*

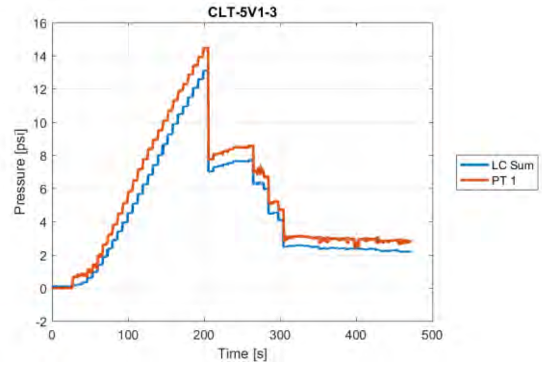
N/A

(g) Average SP.

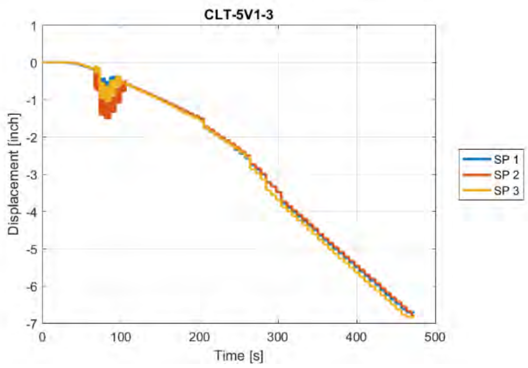
Figure B-7. 5-Ply Grade V1 CLT – Test 2 (5V1-2).



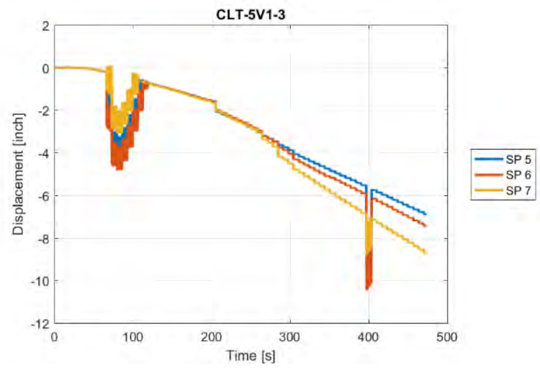
(a) LC.



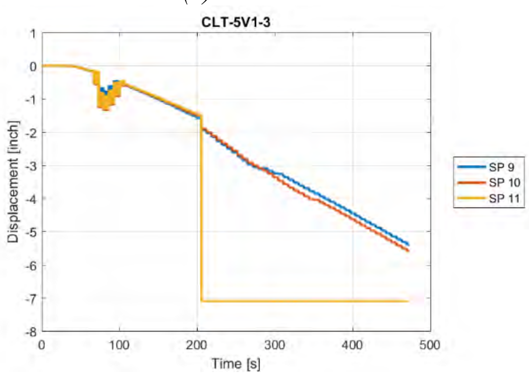
(b) LC vs. PT 1.



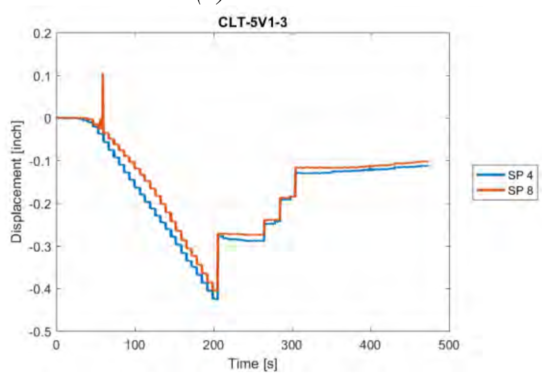
(c) SP 1 – SP 3.



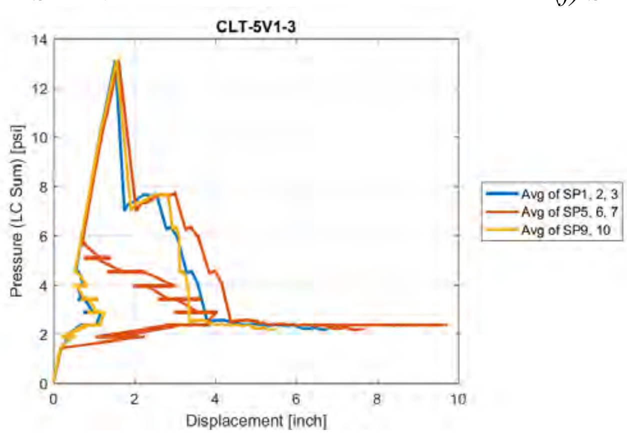
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

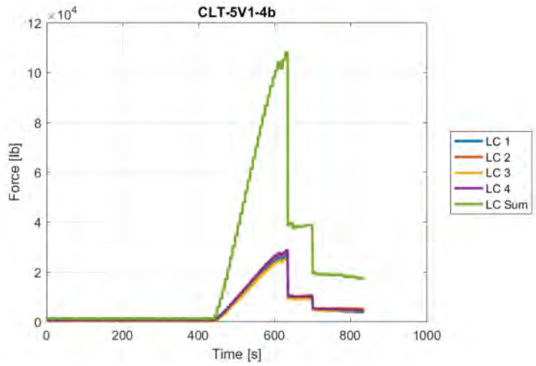


(f) SP 4 & SP 8.

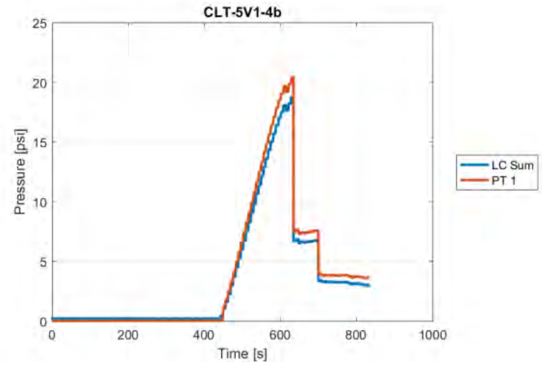


(g) Average SP.

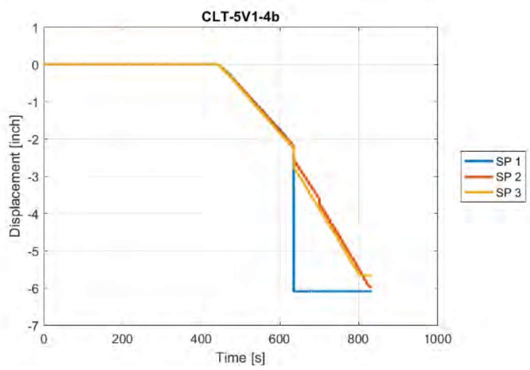
Figure B-8. 5-Ply Grade V1 CLT – Test 3 (5V1-3).



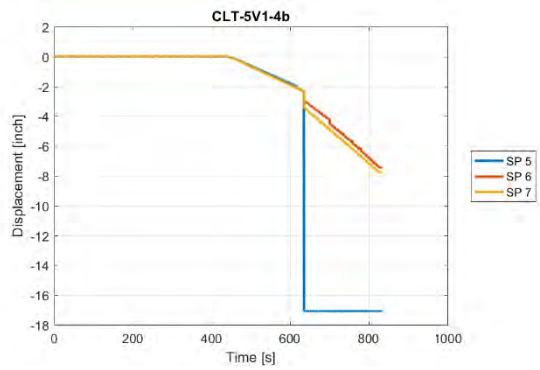
(a) LC.



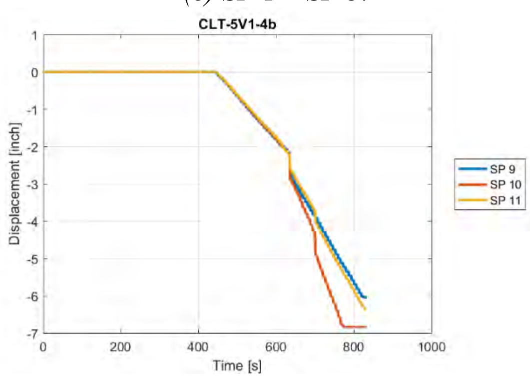
(b) LC vs. PT 1.



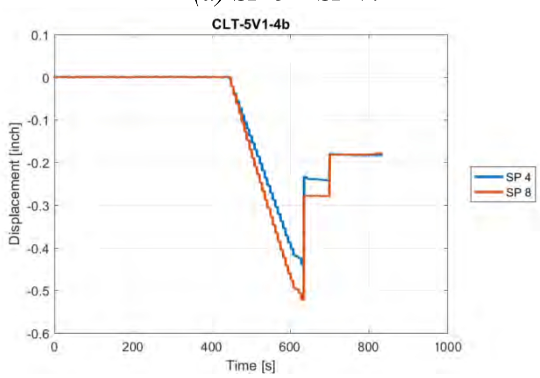
(c) SP 1 – SP 3.



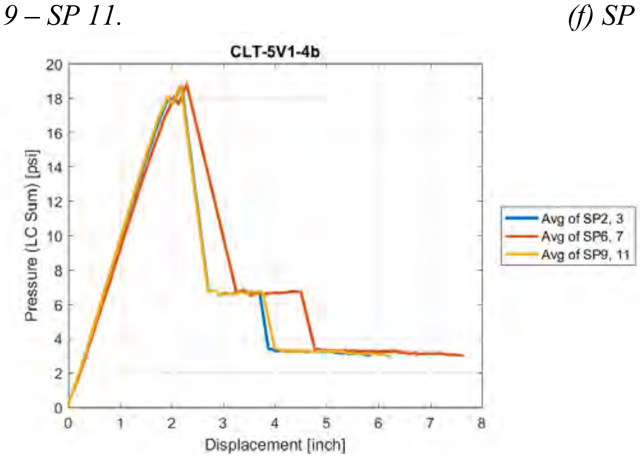
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

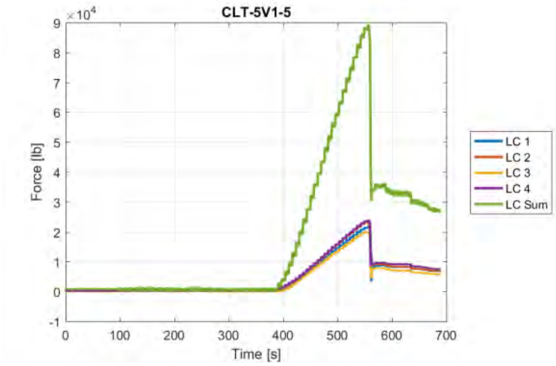


(f) SP 4 & SP 8.

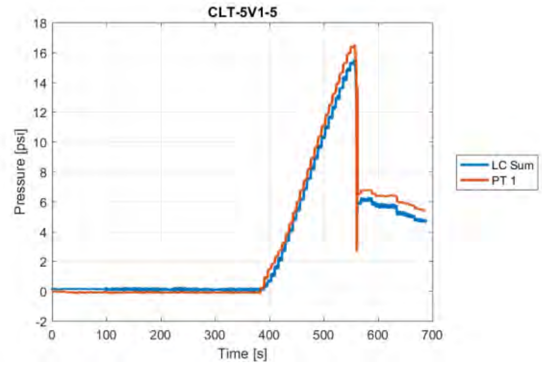


(g) Average SP.

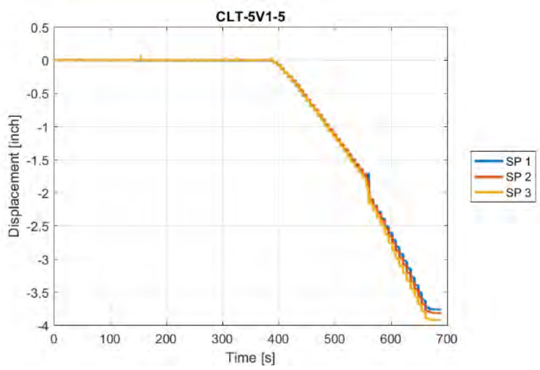
Figure B-9. 5-Ply Grade V1 CLT – Test 4 (5V1-4).



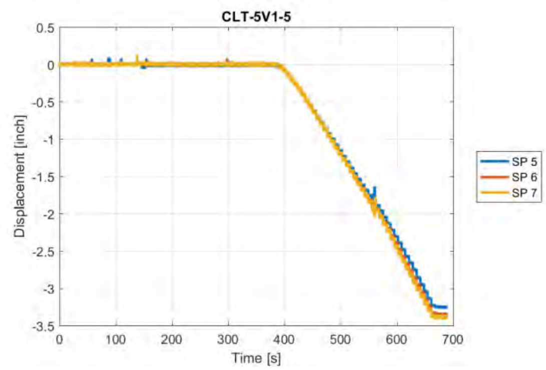
(a) LC.



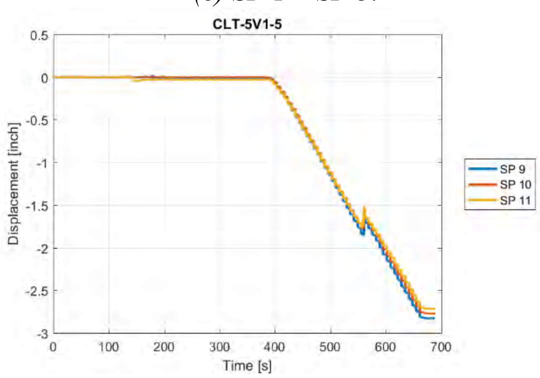
(b) LC vs. PT 1.



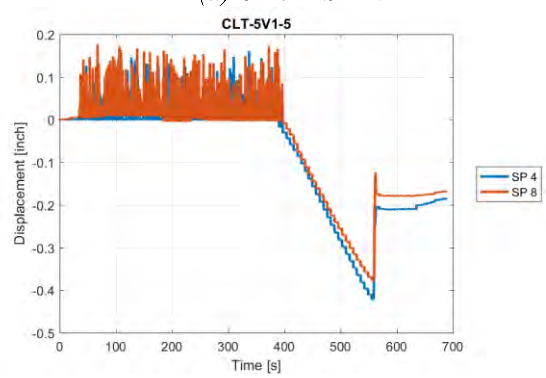
(c) SP 1 – SP 3.



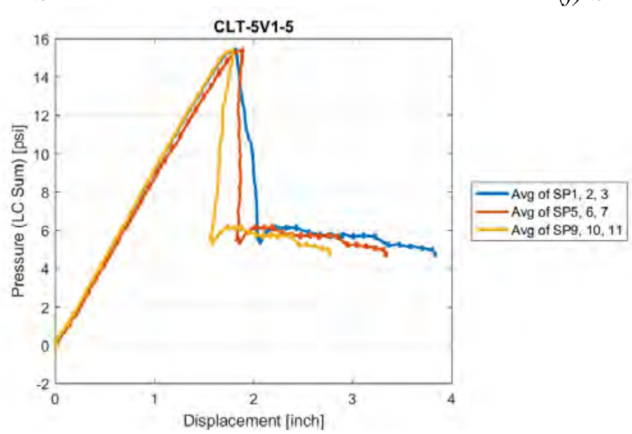
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.



(f) SP 4 & SP 8.



(g) Average SP.

Figure B-10. 5-Ply Grade V1 CLT – Test 5 (5V1-5).

N/A

(a) LC.

N/A

(b) LC vs. PT 1.

N/A

(c) SP 1 – SP 3.

N/A

(d) SP 5 – SP 7.

N/A

(e) SP 9 – SP 11.

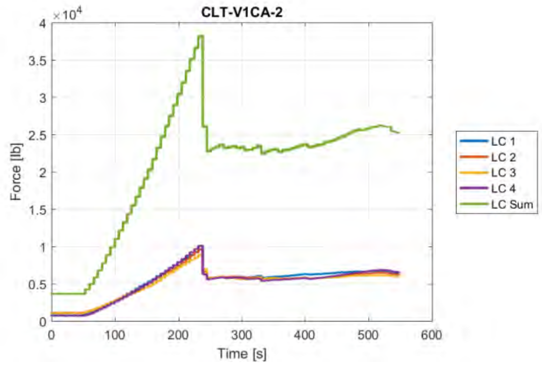
N/A

(f) SP 4 & SP 8.

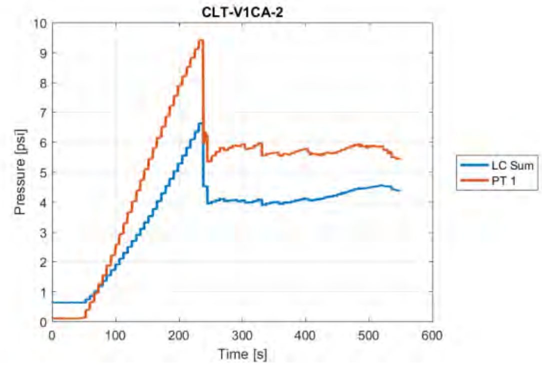
N/A

(g) Average SP.

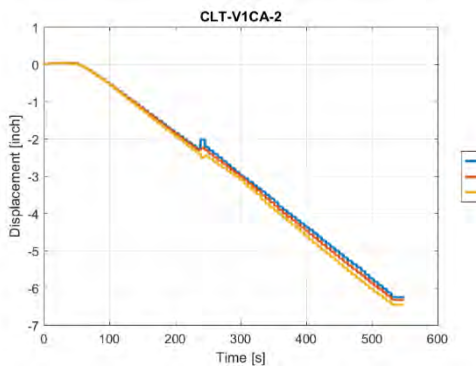
Figure B-11. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 1 (V1CA-1).



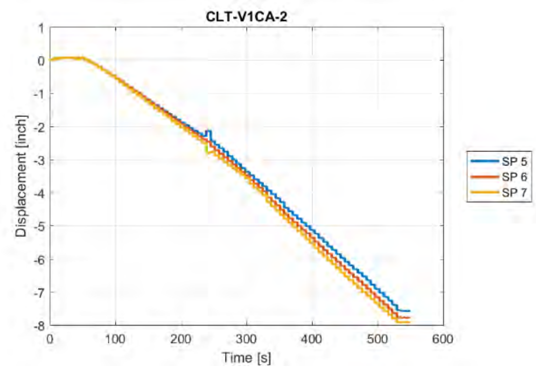
(a) LC.



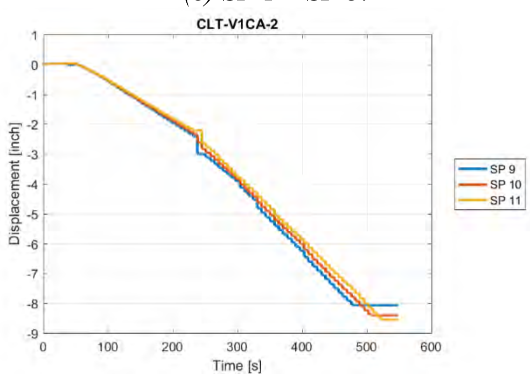
(b) LC vs. PT 1.



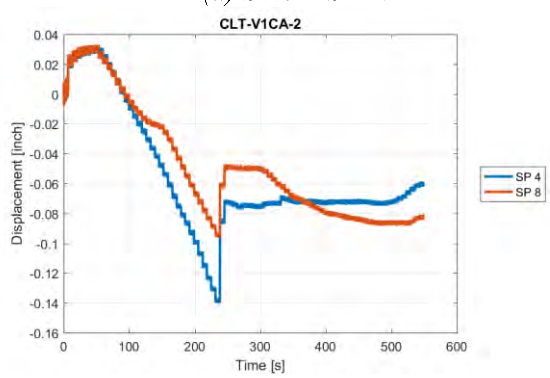
(c) SP 1 – SP 3.



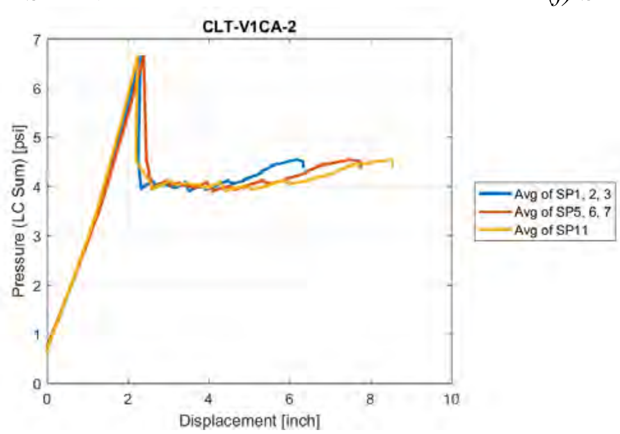
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

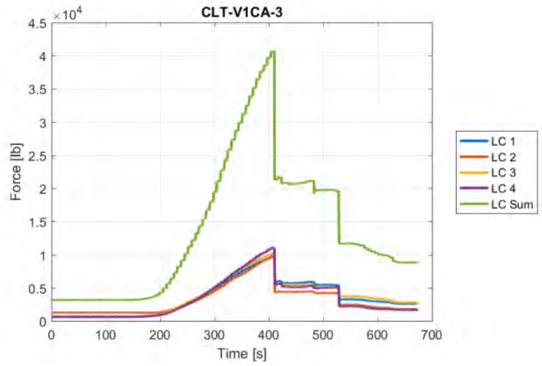


(f) SP 4 & SP 8.

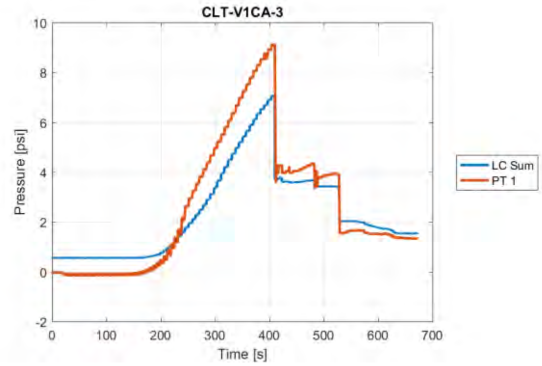


(g) Average SP.

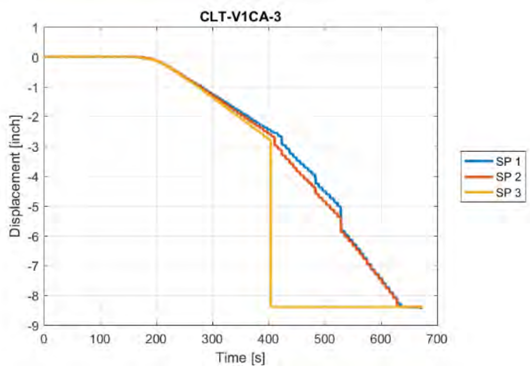
Figure B-12. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 2 (V1CA-2).



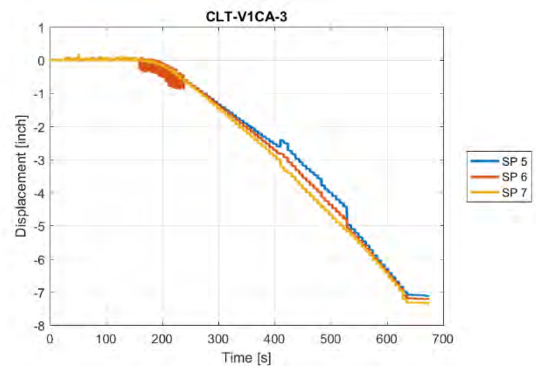
(a) LC.



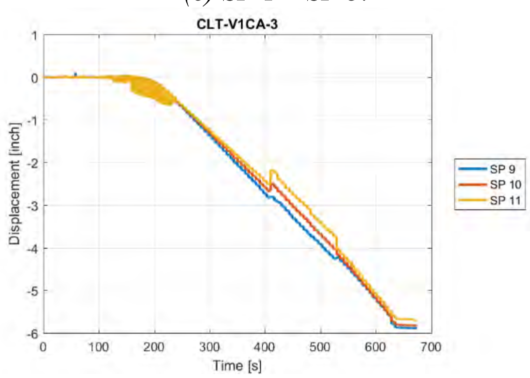
(b) LC vs. PT 1.



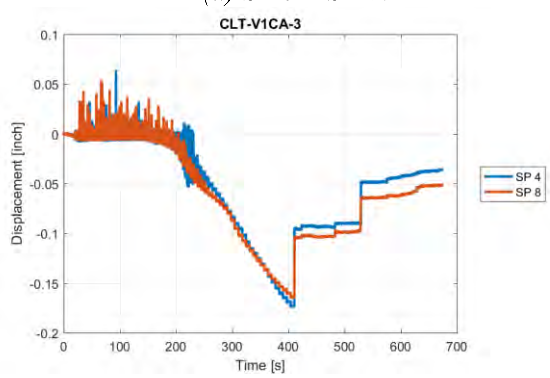
(c) SP 1 – SP 3.



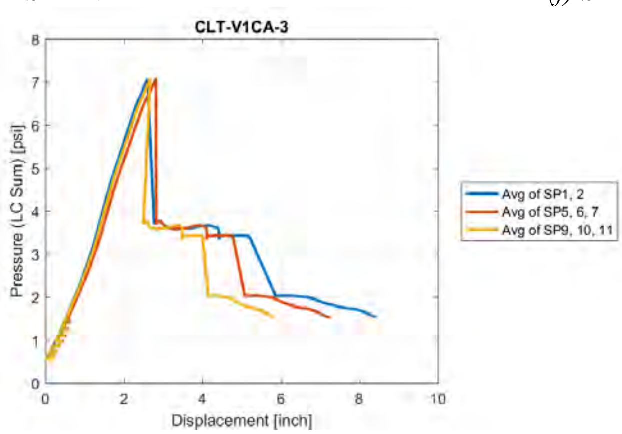
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

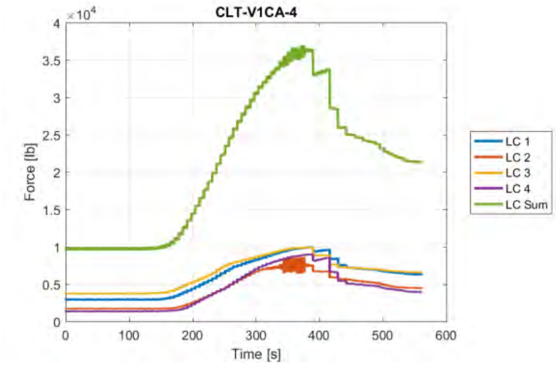


(f) SP 4 & SP 8.

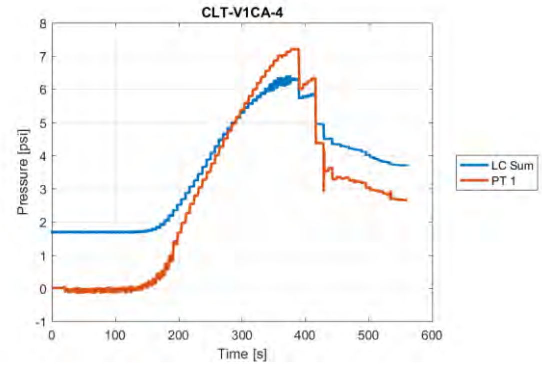


(g) Average SP.

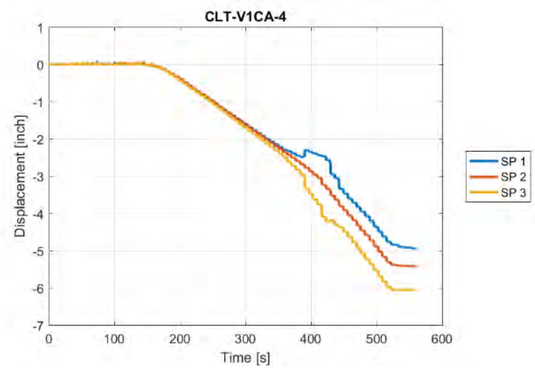
Figure B-13. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 3 (V1CA-3).



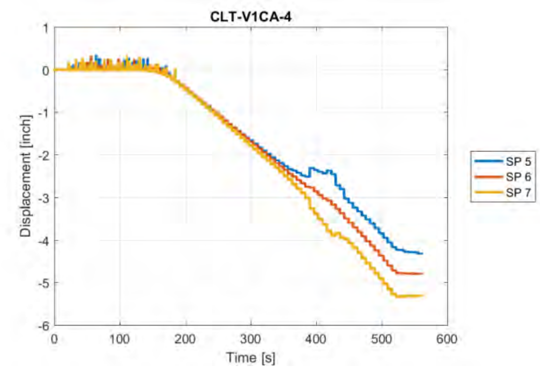
(a) LC.



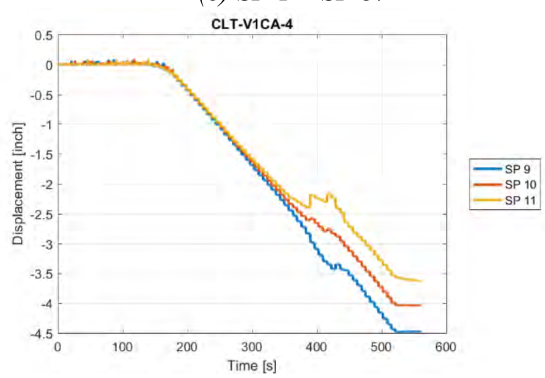
(b) LC vs. PT 1.



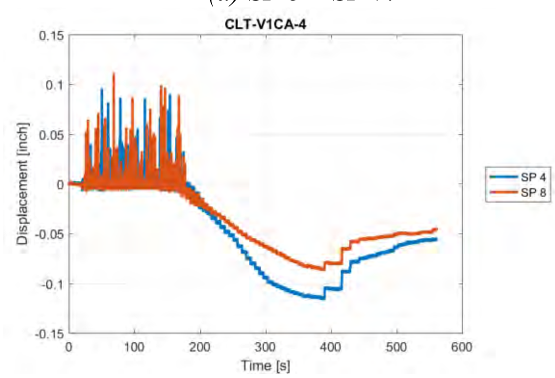
(c) SP 1 – SP 3.



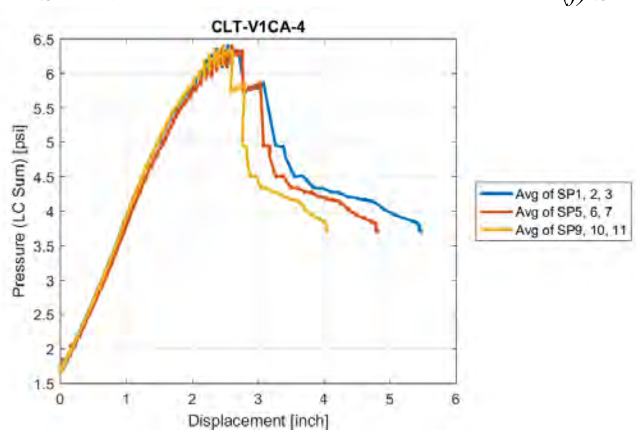
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

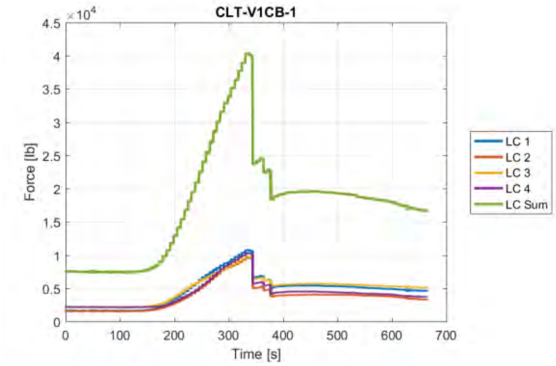


(f) SP 4 & SP 8.

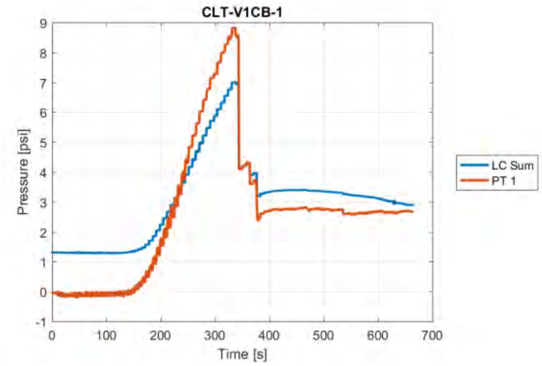


(g) Average SP.

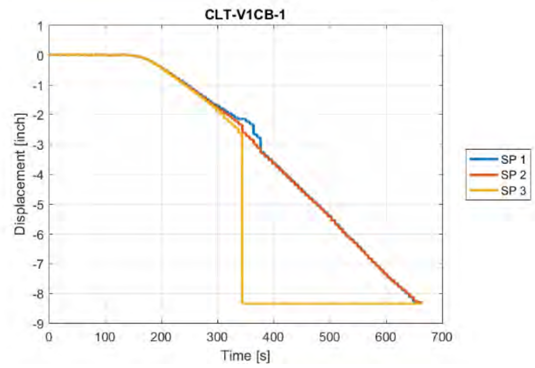
Figure B-14. 3-Ply Grade V1 CLT w/ SST Brackets at Each End – Test 4 (V1CA-4).



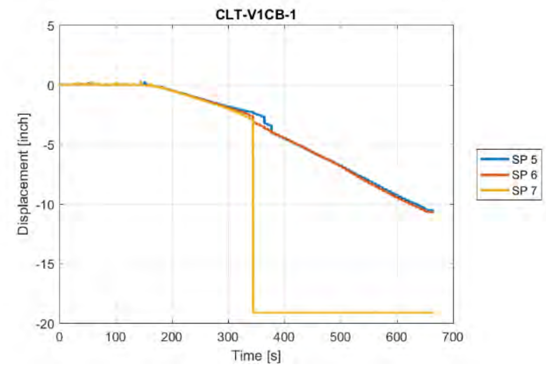
(a) LC.



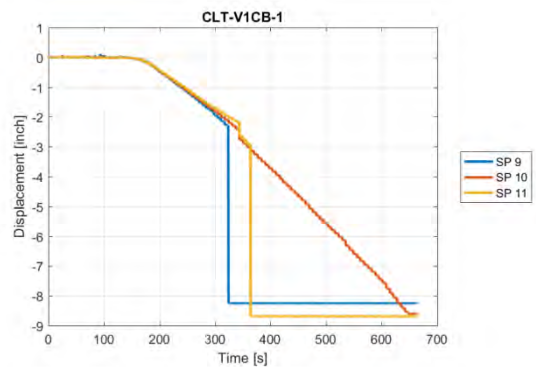
(b) LC vs. PT 1.



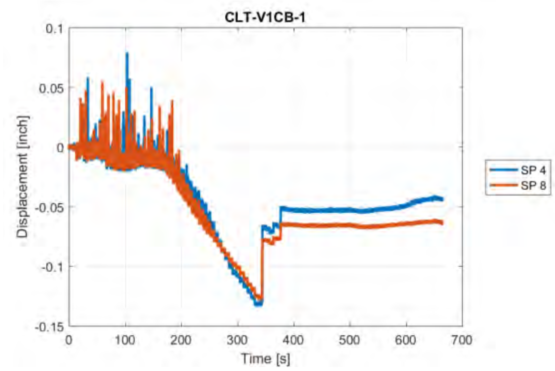
(c) SP 1 – SP 3.



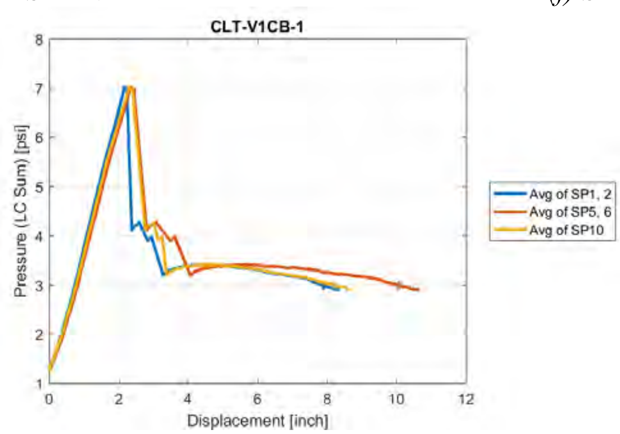
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

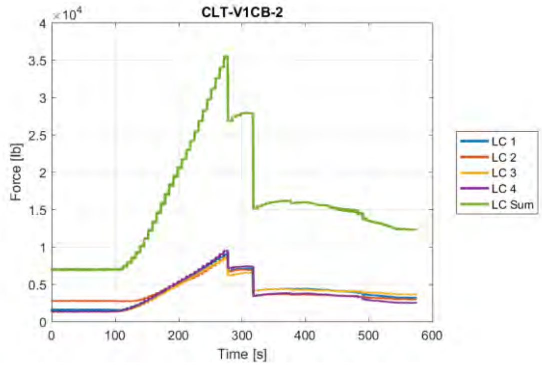


(f) SP 4 & SP 8.

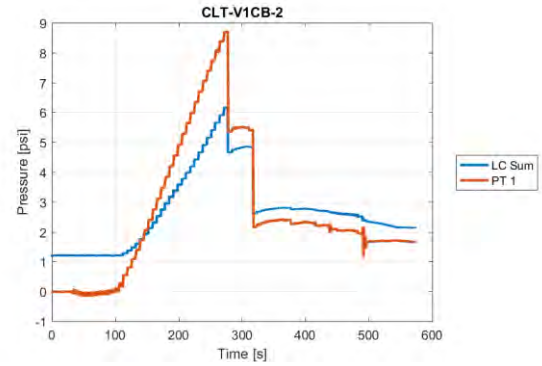


(g) Average SP.

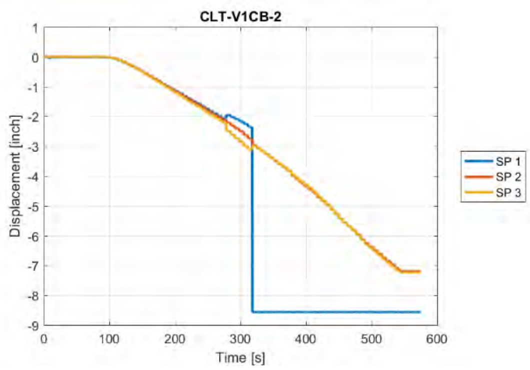
Figure B-15. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 1 (V1CB-1).



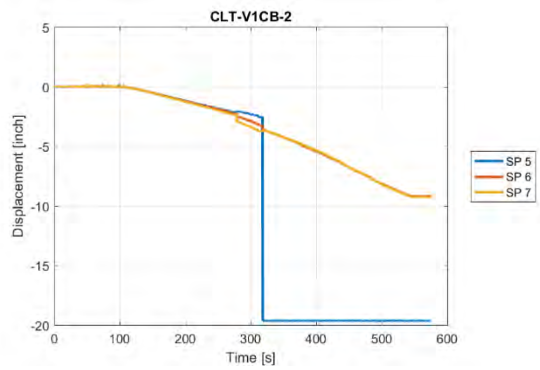
(a) LC.



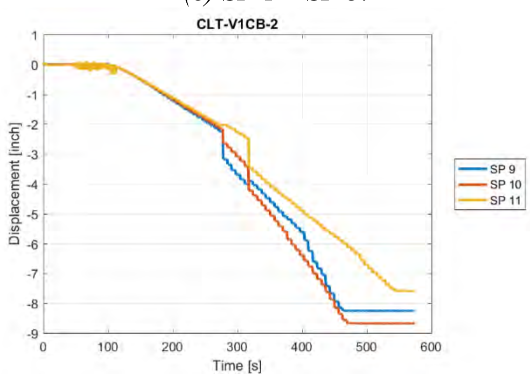
(b) LC vs. PT 1.



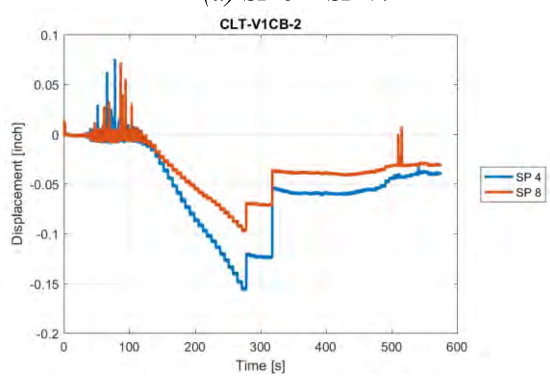
(c) SP 1 – SP 3.



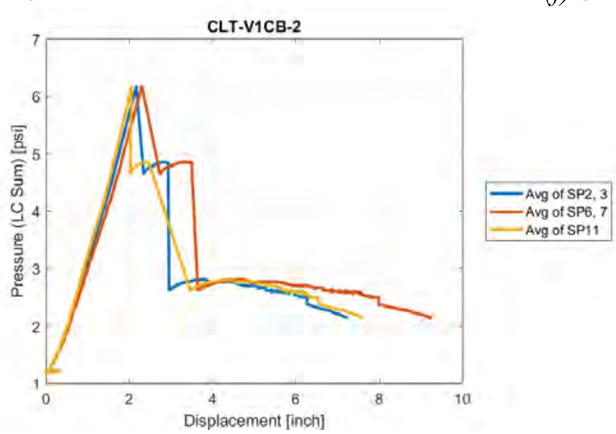
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

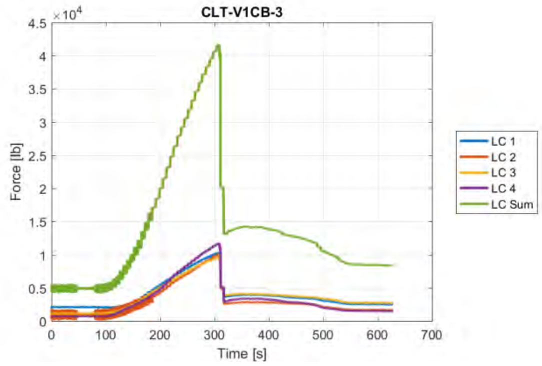


(f) SP 4 & SP 8.

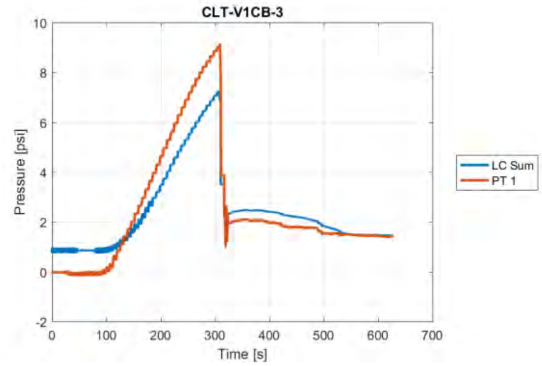


(g) Average SP.

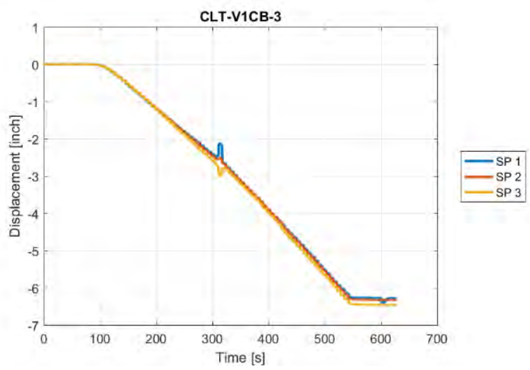
Figure B-16. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 2 (V1CB-2).



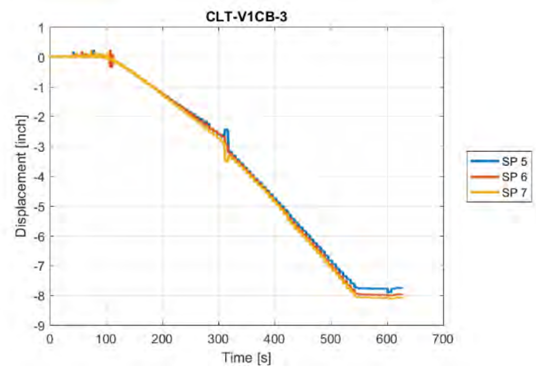
(a) LC.



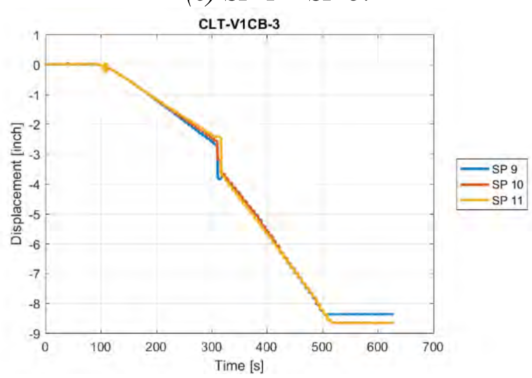
(b) LC vs. PT 1.



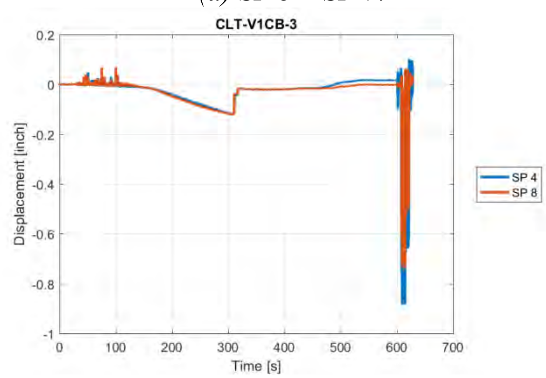
(c) SP 1 – SP 3.



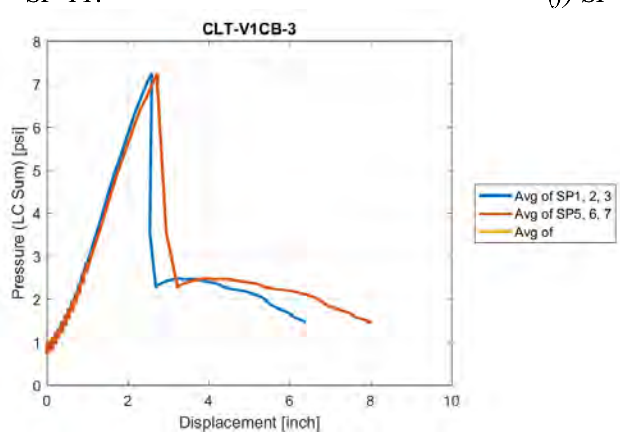
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

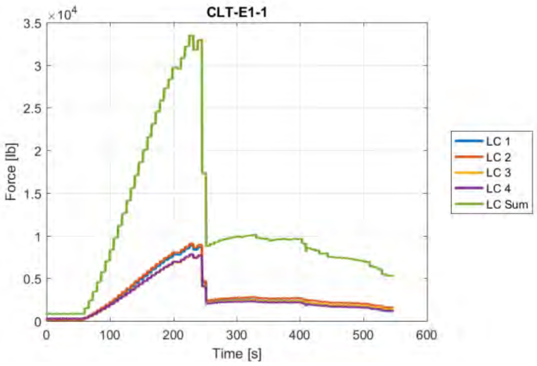


(f) SP 4 & SP 8.

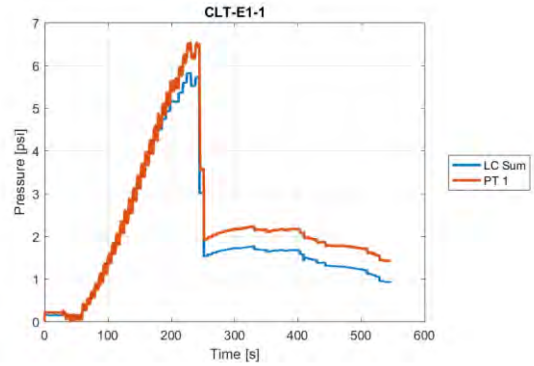


(g) Average SP.

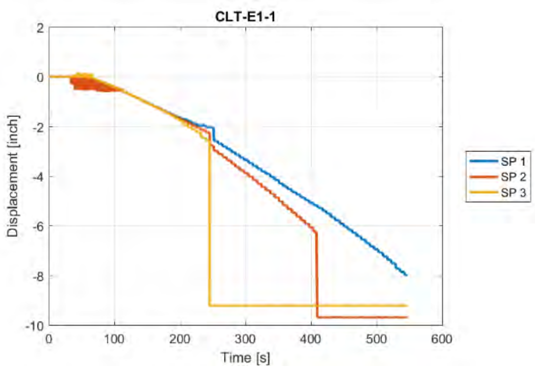
Figure B-17. 3-Ply Grade V1 CLT w/ L4x4x1/4 Brackets at Each End – Test 3 (V1CB-3).



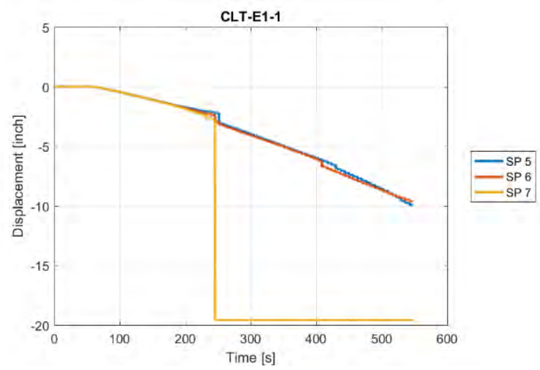
(a) LC.



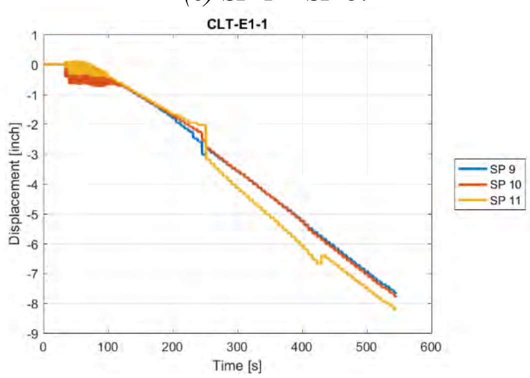
(b) LC vs. PT 1.



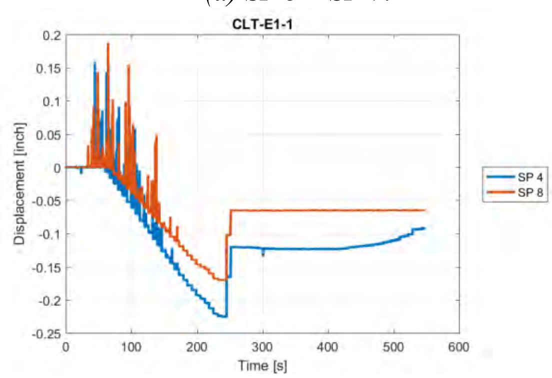
(c) SP 1 – SP 3.



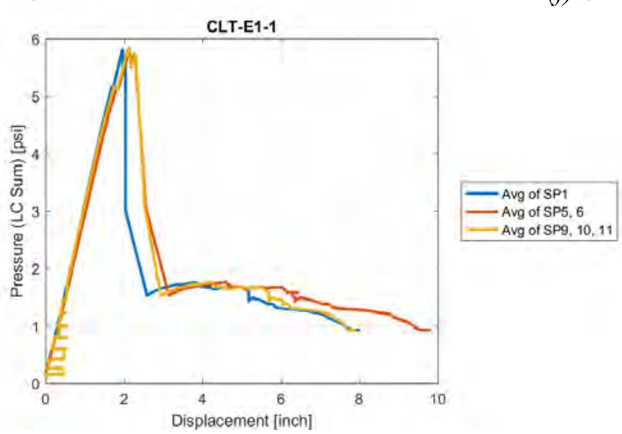
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

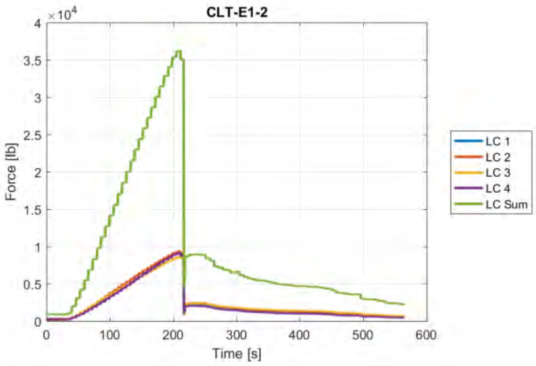


(f) SP 4 & SP 8.

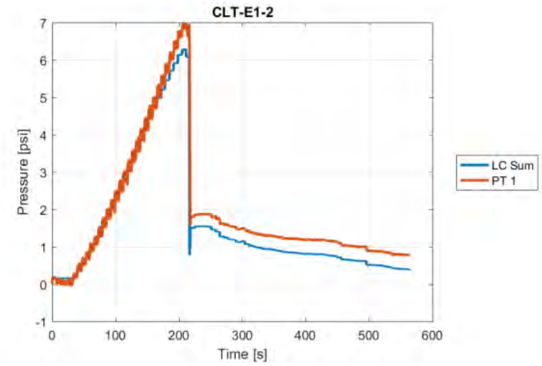


(g) Average SP.

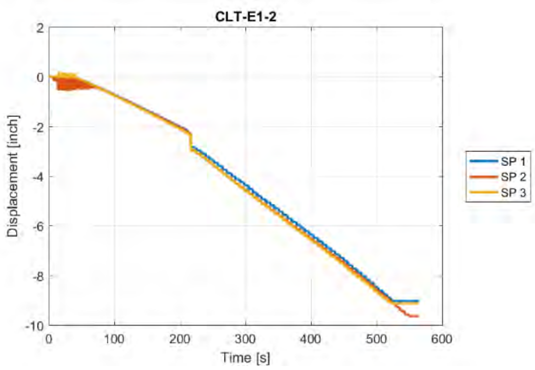
Figure B-18. 3-Ply Grade E1 CLT – Test 1 (E1-1).



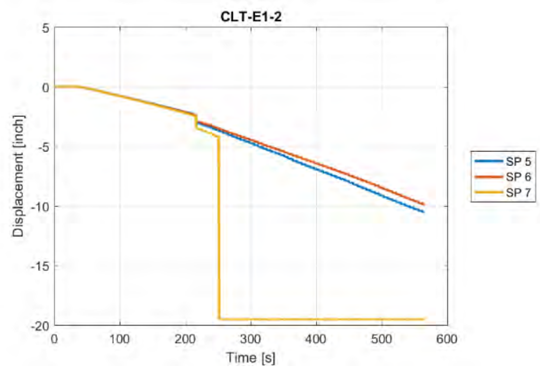
(a) LC.



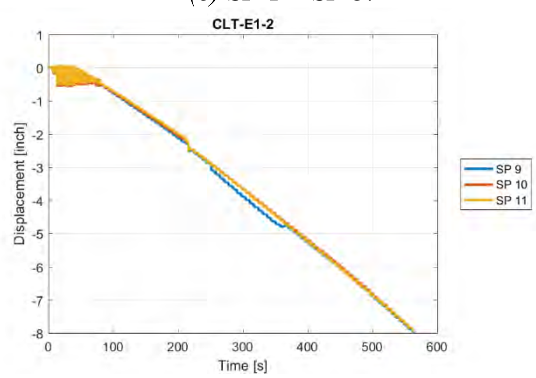
(b) LC vs. PT 1.



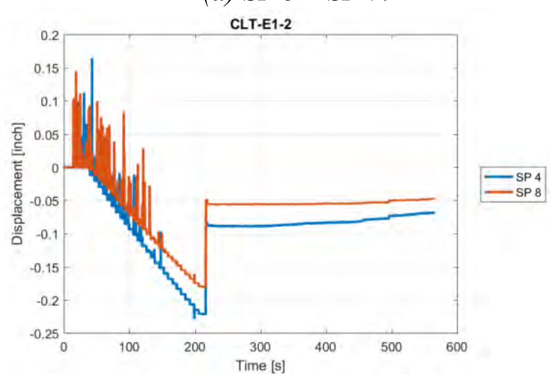
(c) SP 1 – SP 3.



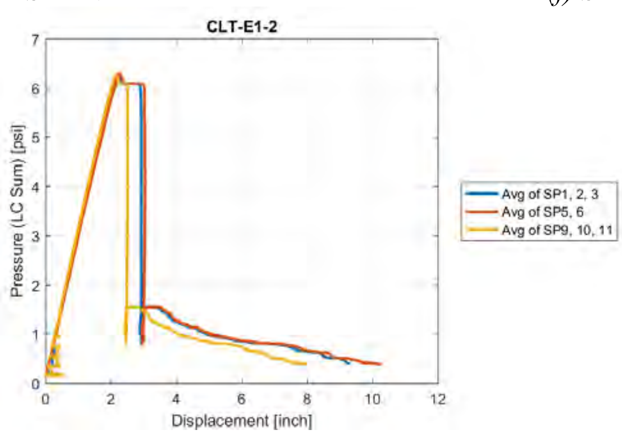
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

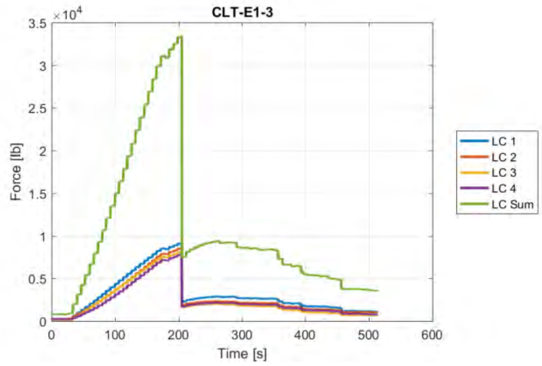


(f) SP 4 & SP 8.

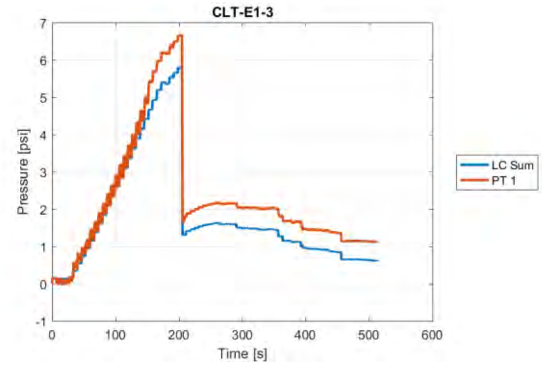


(g) Average SP.

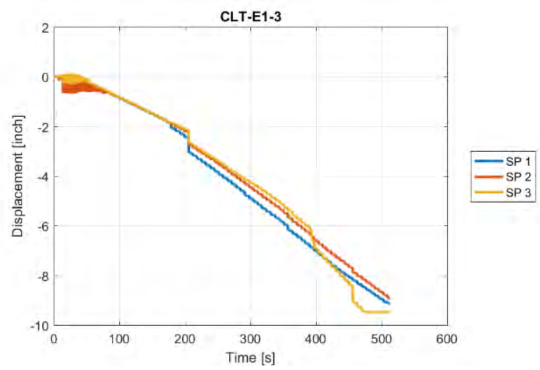
Figure B-19. 3-Ply Grade E1 CLT – Test 2 (E1-2).



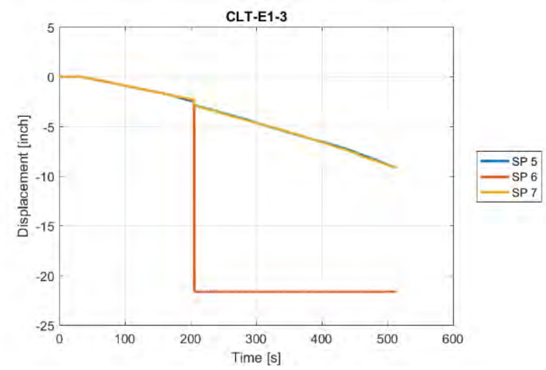
(a) LC.



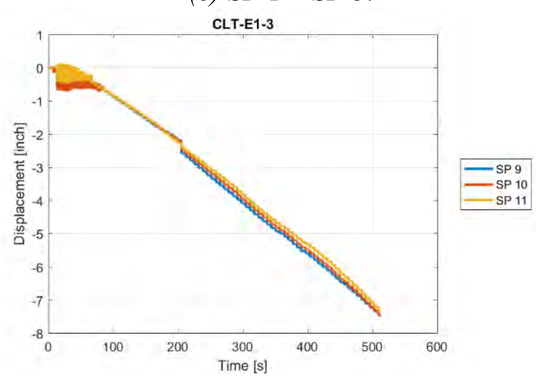
(b) LC vs. PT 1.



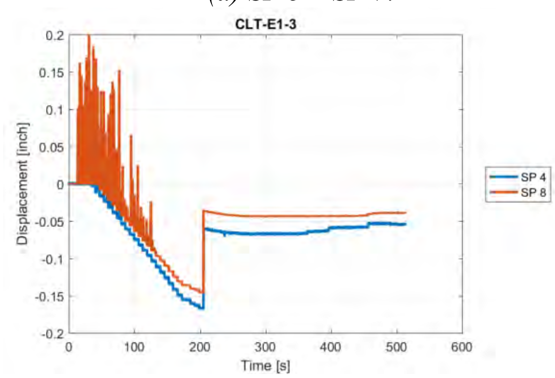
(c) SP 1 – SP 3.



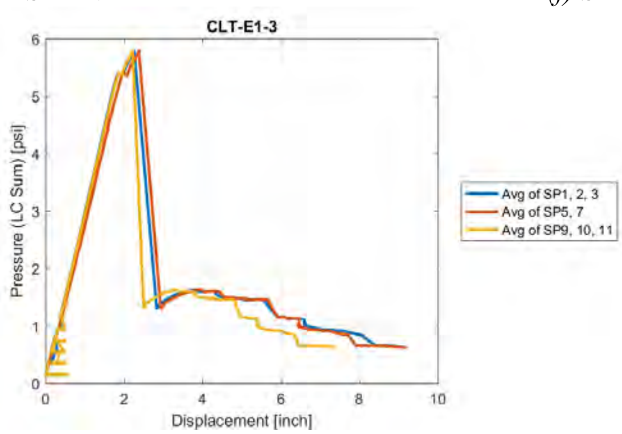
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

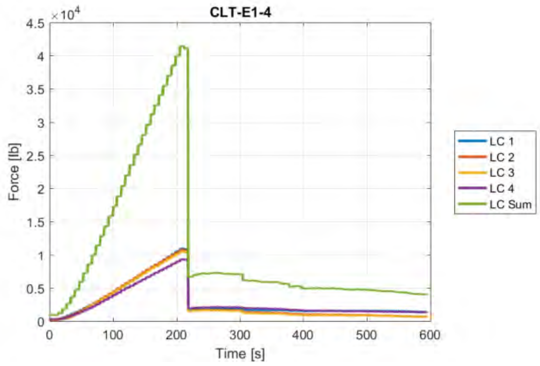


(f) SP 4 & SP 8.

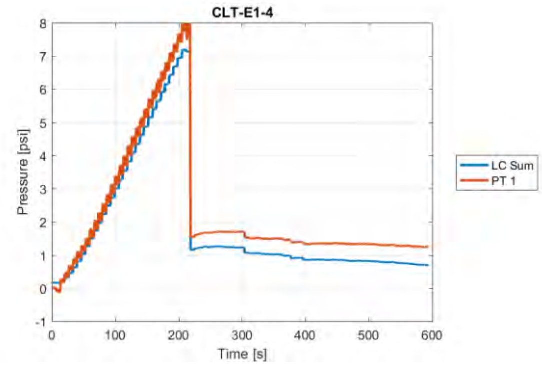


(g) Average SP.

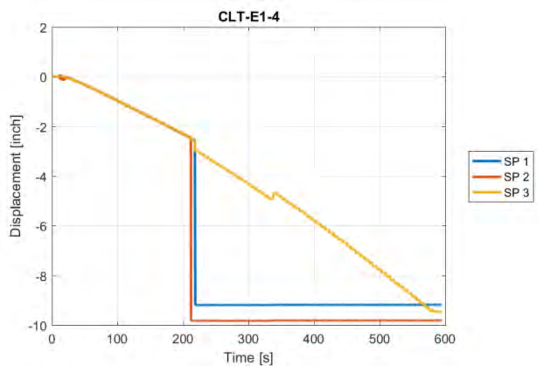
Figure B-20. 3-Ply Grade E1 CLT – Test 3 (E1-3).



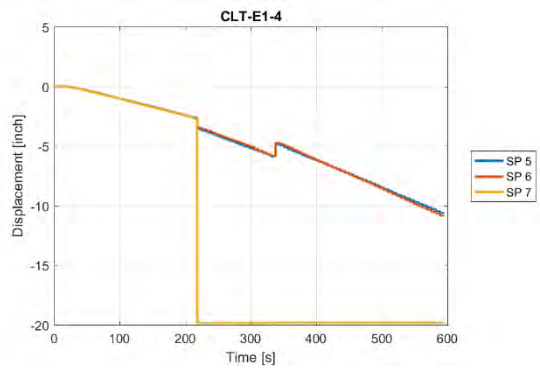
(a) LC.



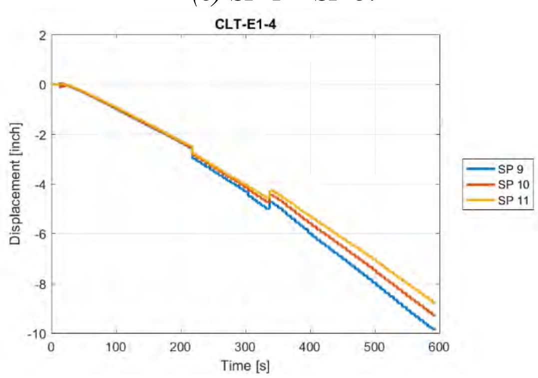
(b) LC vs. PT 1.



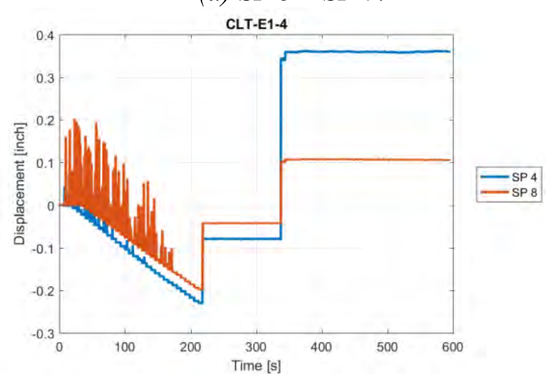
(c) SP 1 – SP 3.



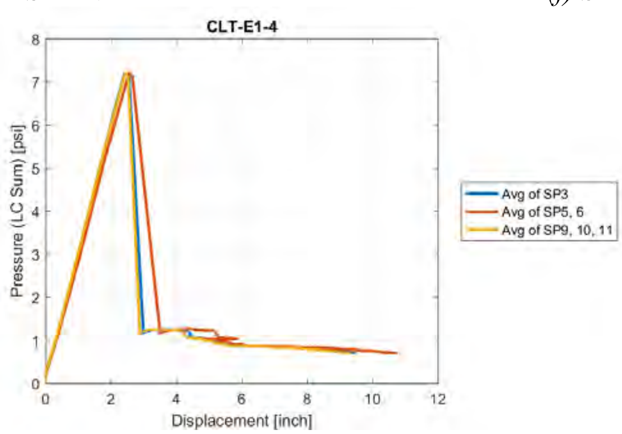
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

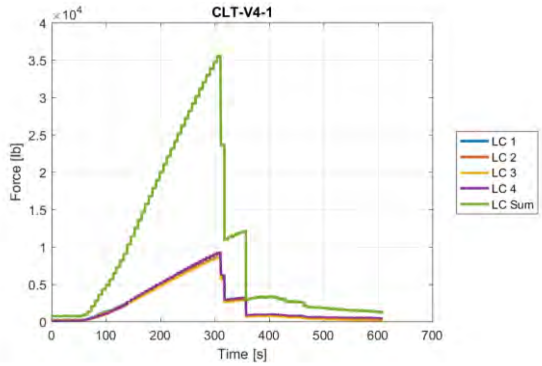


(f) SP 4 & SP 8.

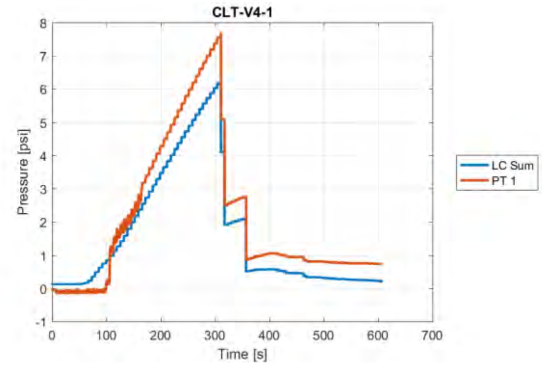


(g) Average SP.

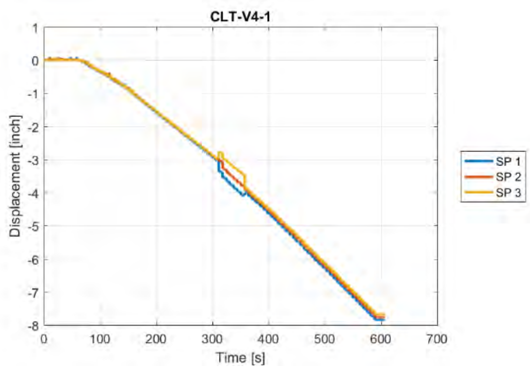
Figure B-21. 3-Ply Grade E1 CLT – Test 4 (E1-4).



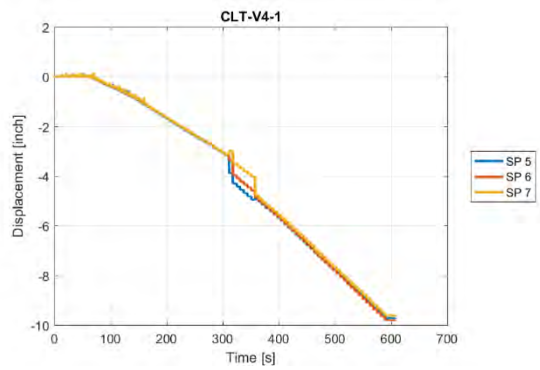
(a) LC.



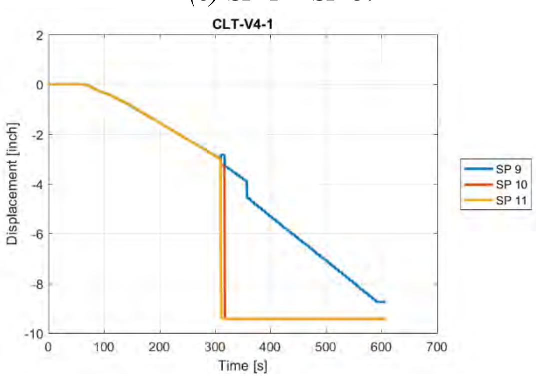
(b) LC vs. PT 1.



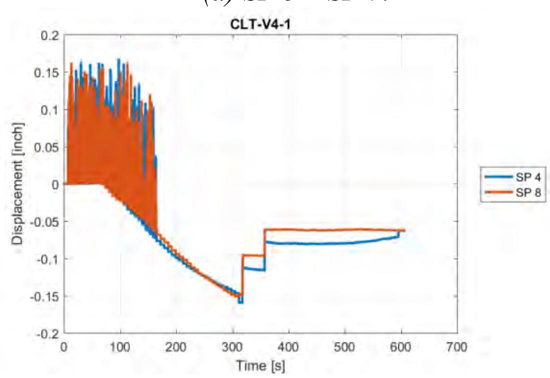
(c) SP 1 – SP 3.



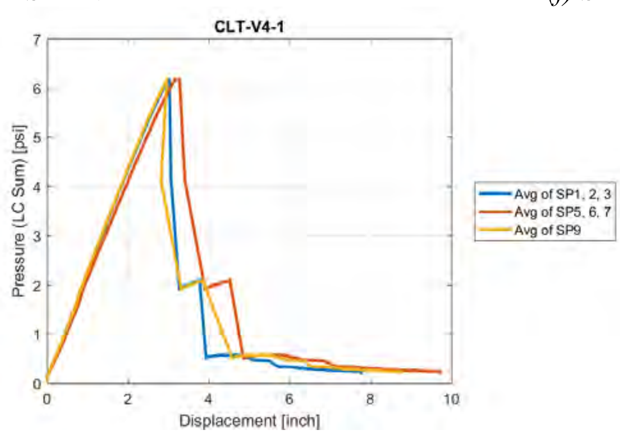
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

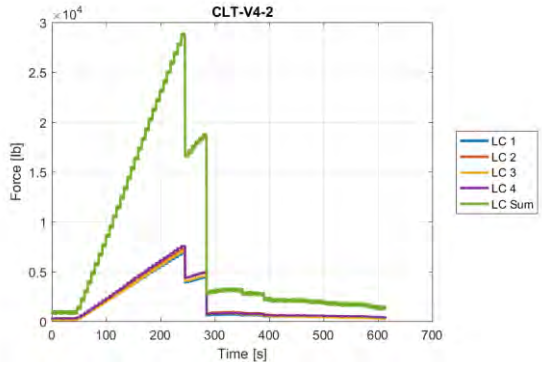


(f) SP 4 & SP 8.

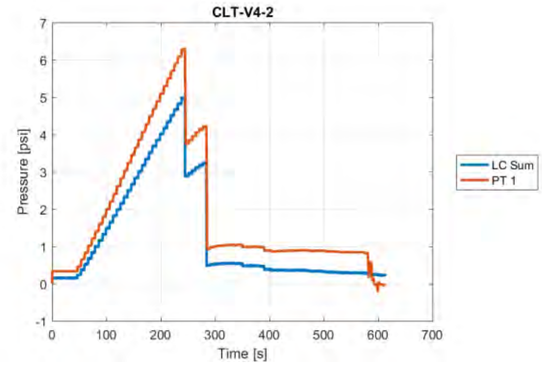


(g) Average SP.

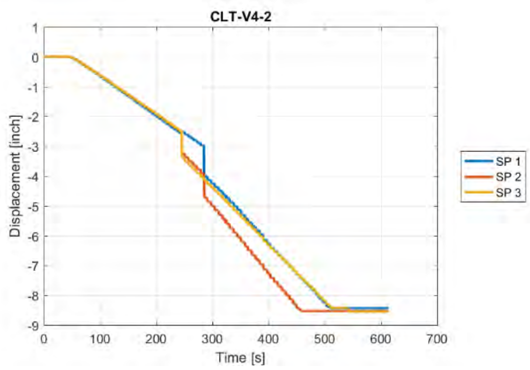
Figure B-22. 3-Ply Grade V4 CLT – Test 1 (V4-1).



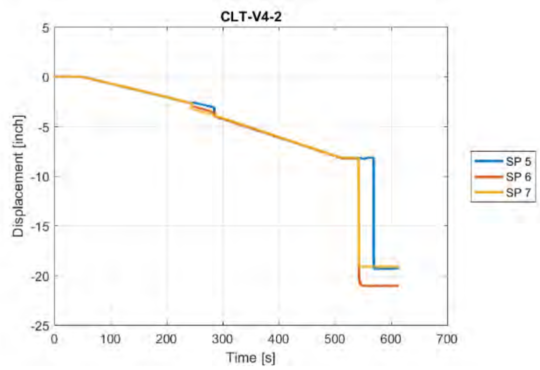
(a) LC.



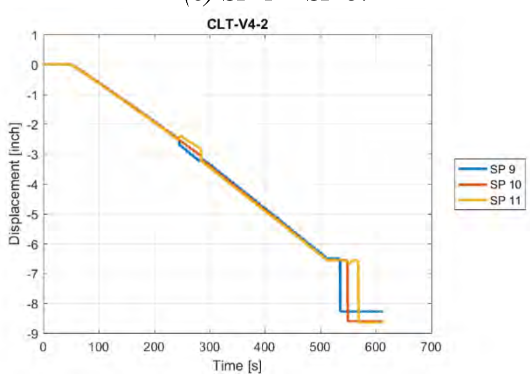
(b) LC vs. PT 1.



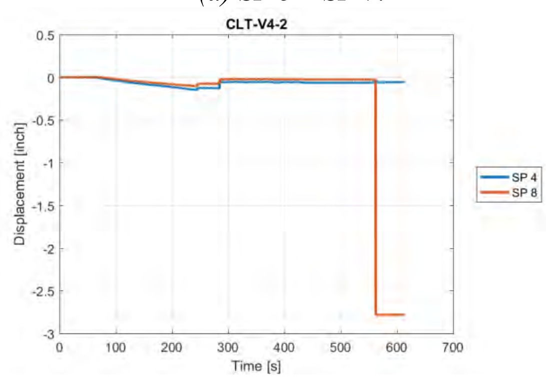
(c) SP 1 – SP 3.



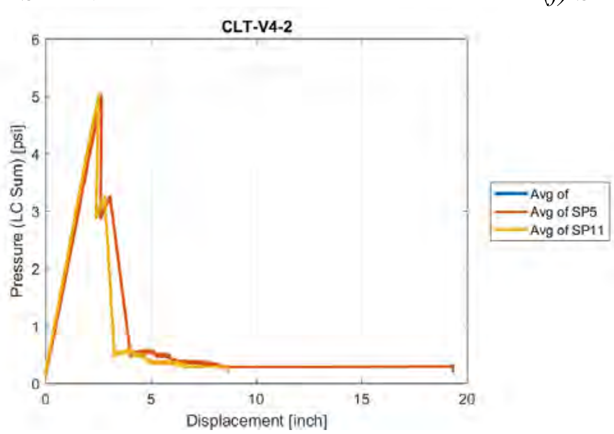
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

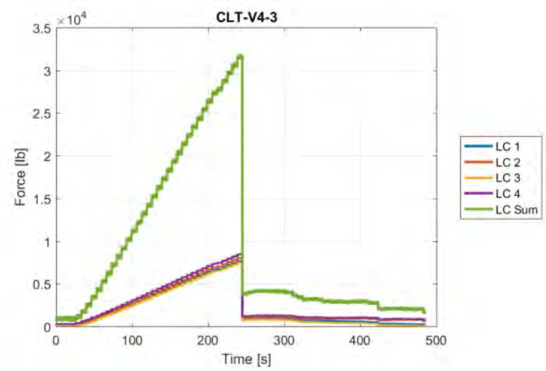


(f) SP 4 & SP 8.

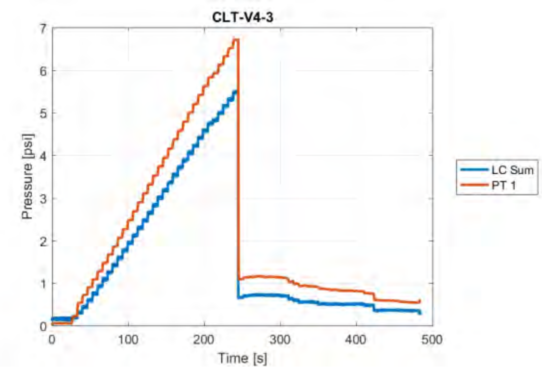


(g) Average SP.

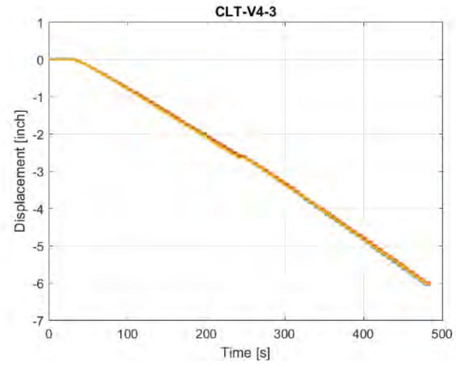
Figure B-23. 3-Ply Grade V4 CLT – Test 2 (V4-2).



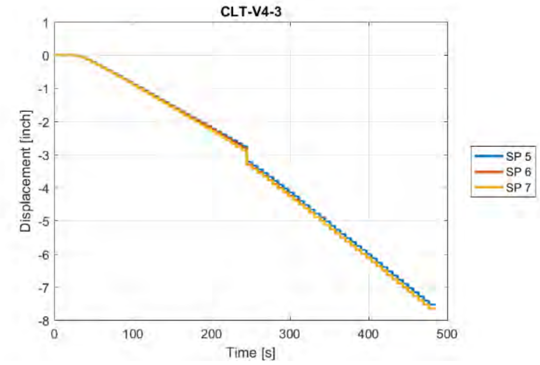
(a) LC.



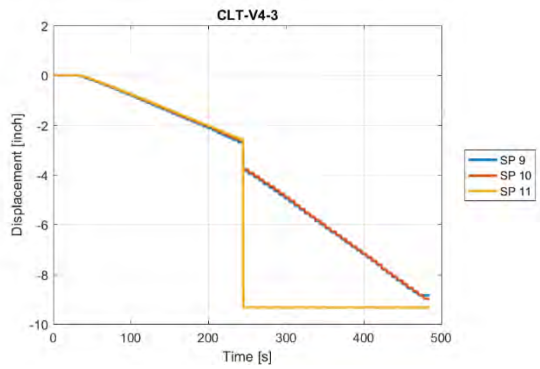
(b) LC vs. PT 1.



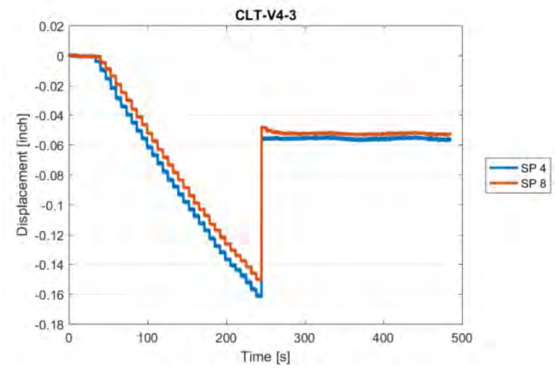
(c) SP 1 – SP 3.



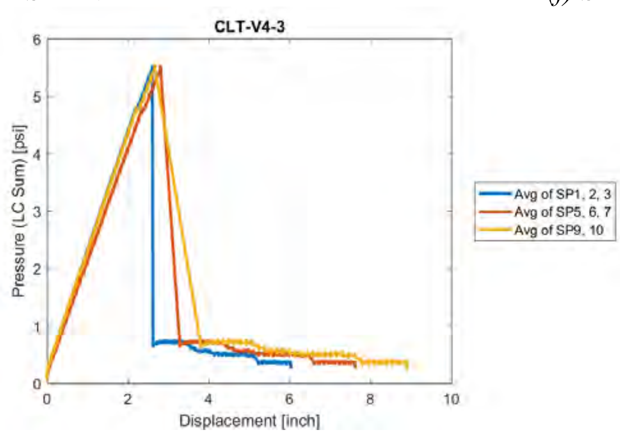
(d) SP 5 – SP 7.



(e) SP 9 – SP 11.

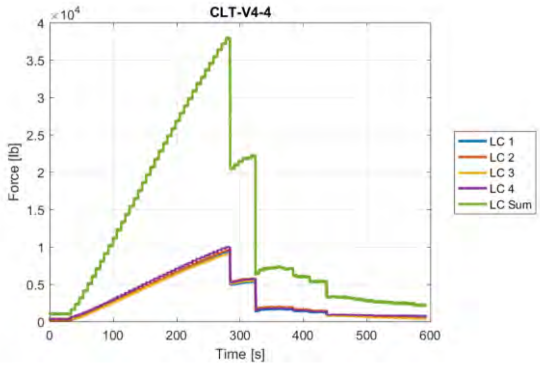


(f) SP 4 & SP 8.

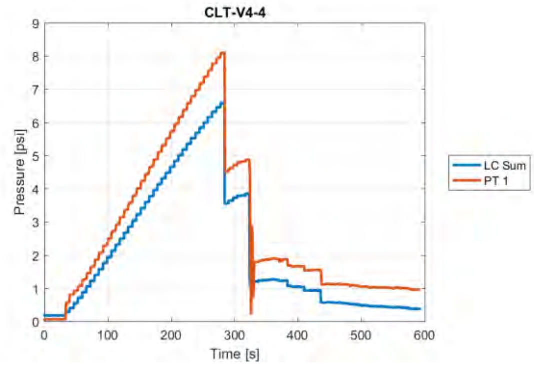


(g) Average SP.

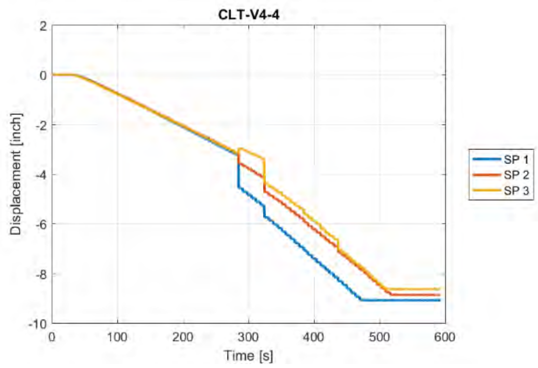
Figure B-24. 3-Ply Grade V4 CLT – Test 3 (V4-3).



(a) LC.



(b) LC vs. PT 1.



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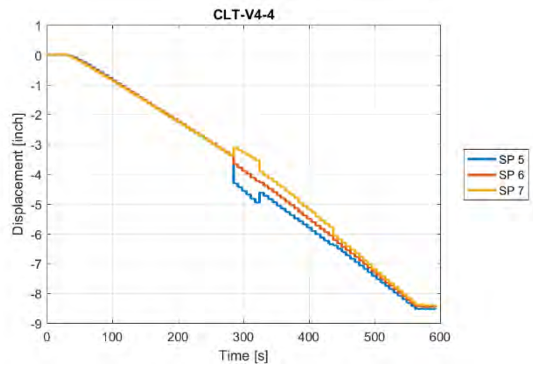

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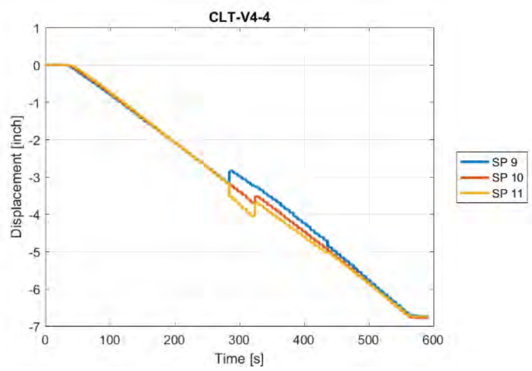

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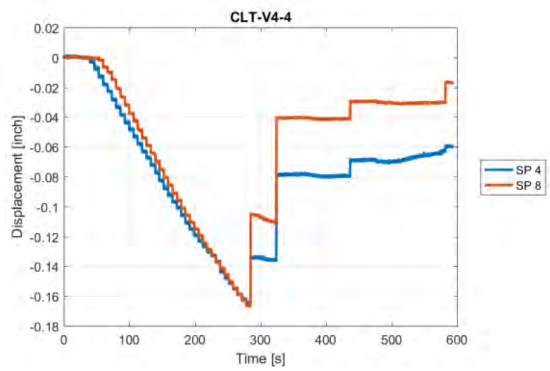
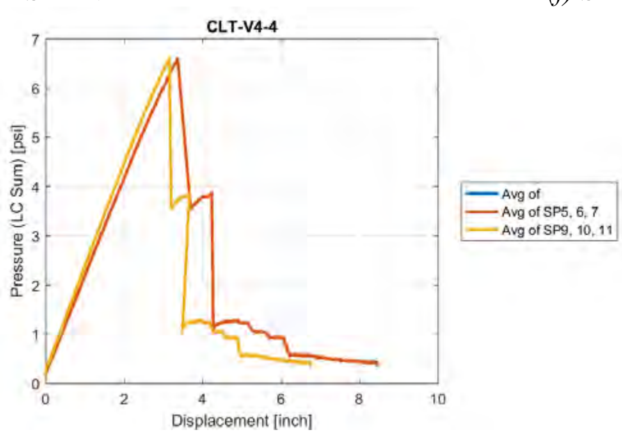
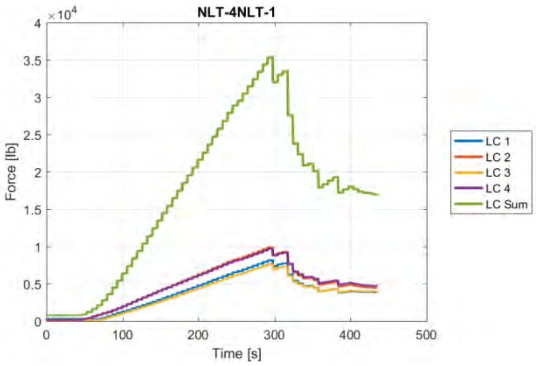


(f) SP 4 & SP 8.

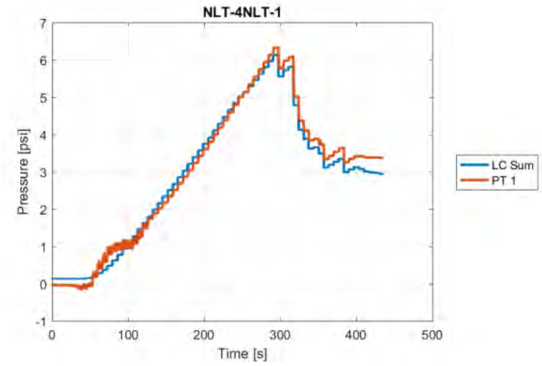


(g) Average SP.

Figure B-25. 3-Ply Grade V4 CLT – Test 4 (V4-4).



(a) LC.



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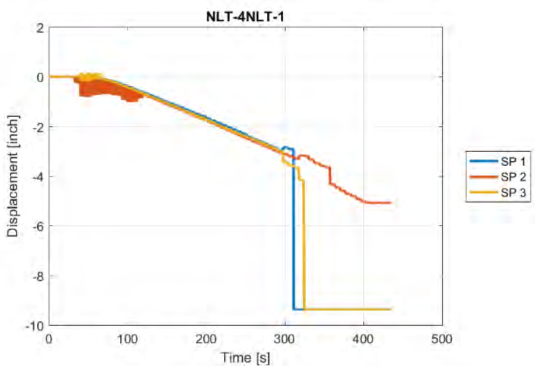

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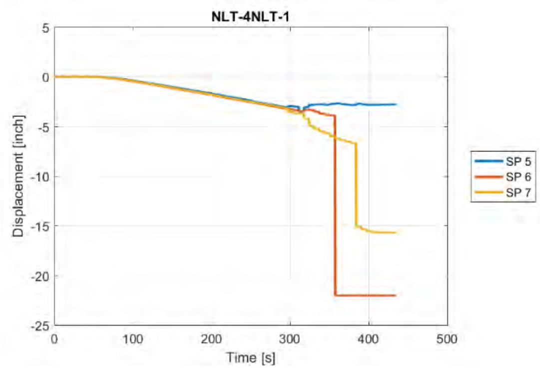

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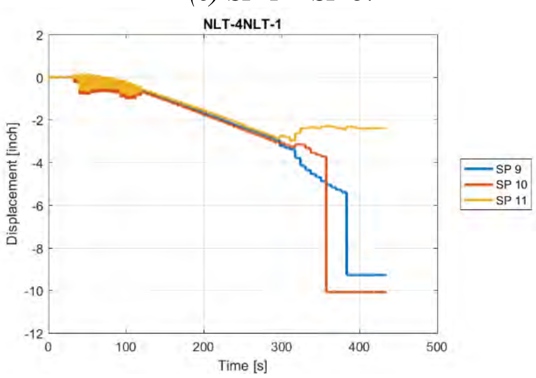

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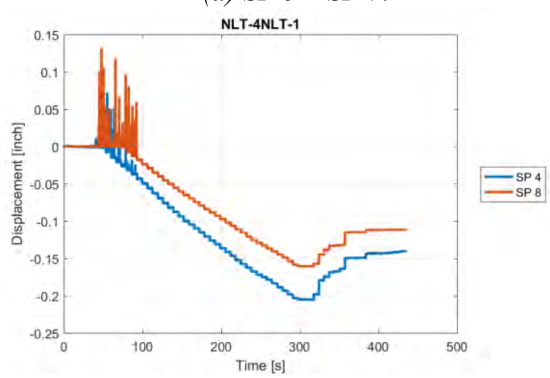
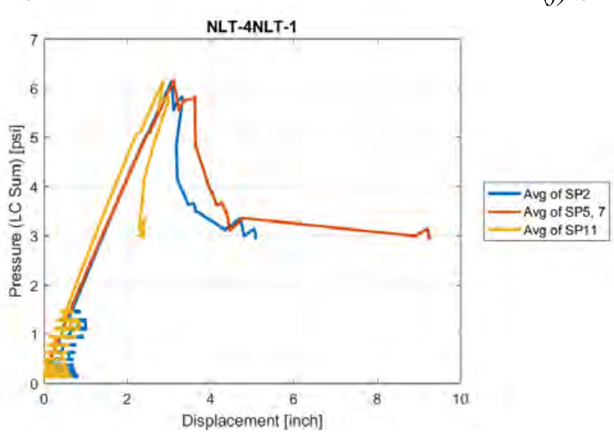
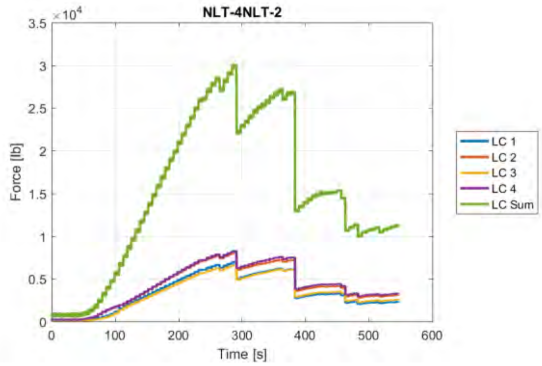


(f) SP 4 & SP 8.

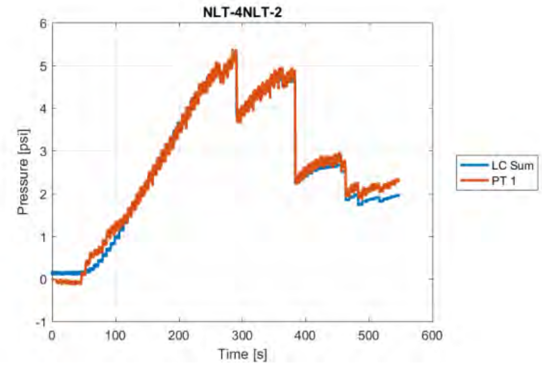


(g) Average SP.

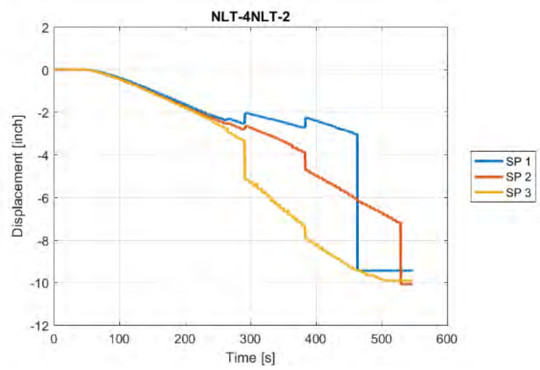
Figure B-26. 2x4 NLT – Test 1 (4NLT-1).



(a) LC.



(b) LC vs. PT 1.



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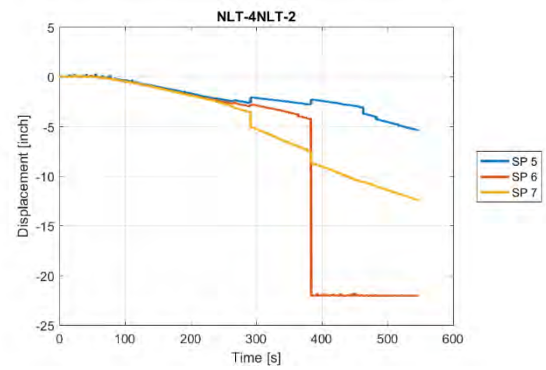

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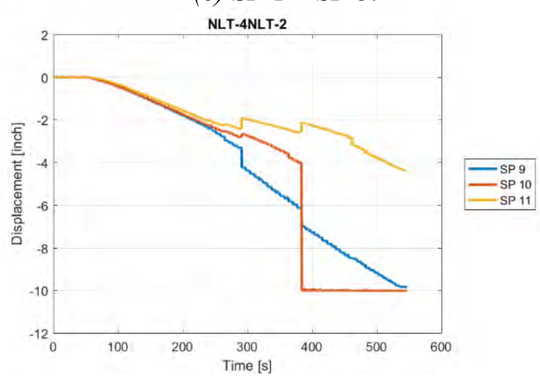
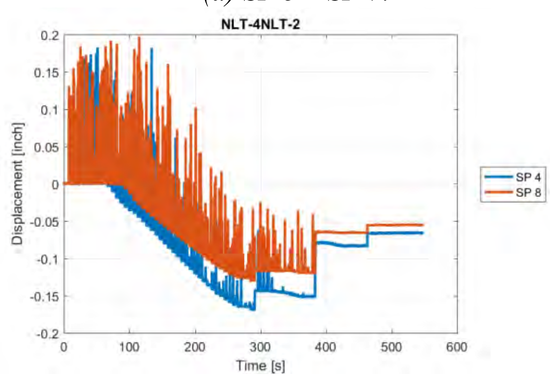
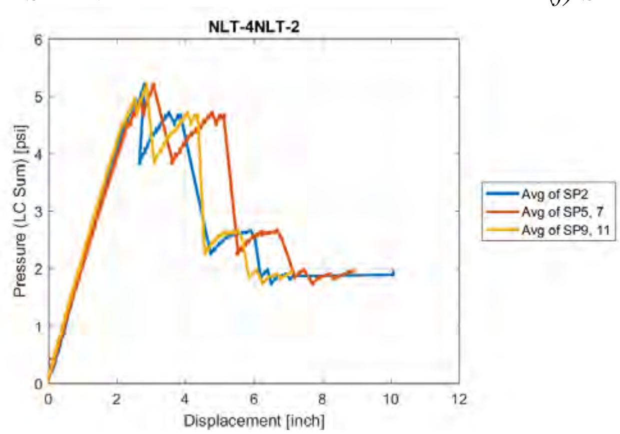


(e) SP 9 – SP 11.

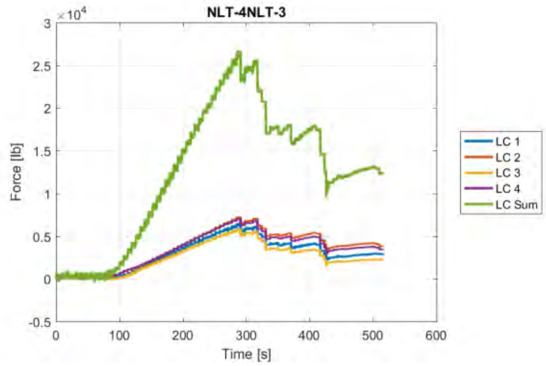


(f) SP 4 & SP 8.

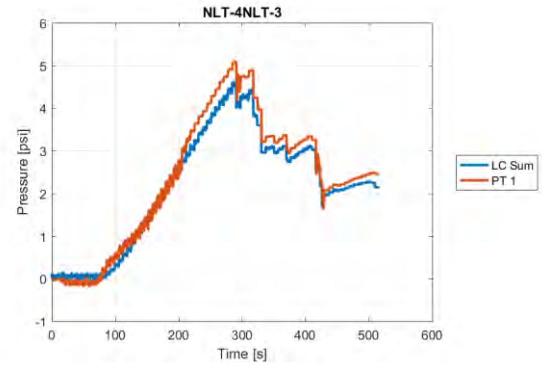


(g) Average SP.

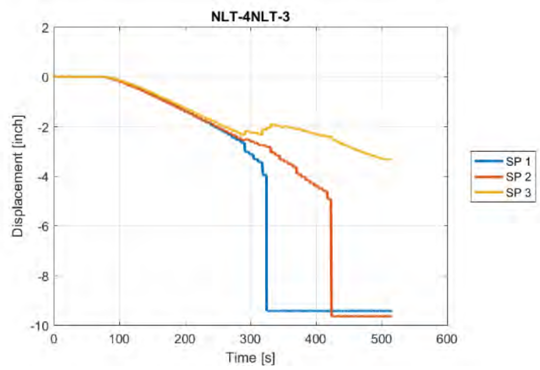
Figure B-27. 2x4 NLT – Test 2 (4NLT-2).



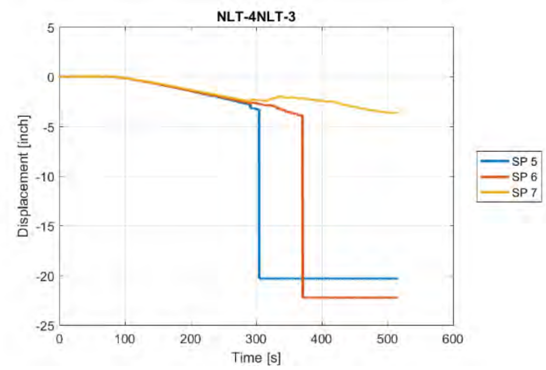
(a) LC.



(b) LC vs. PT 1.



(c) SP 1 – SP 3.



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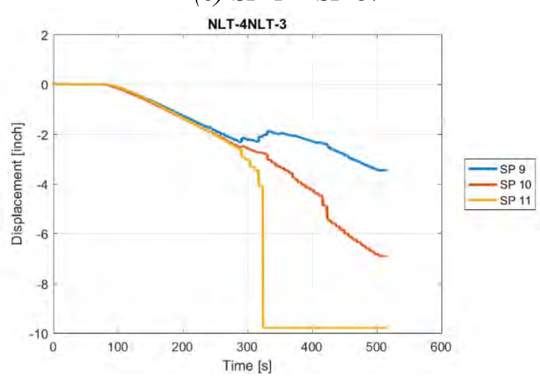

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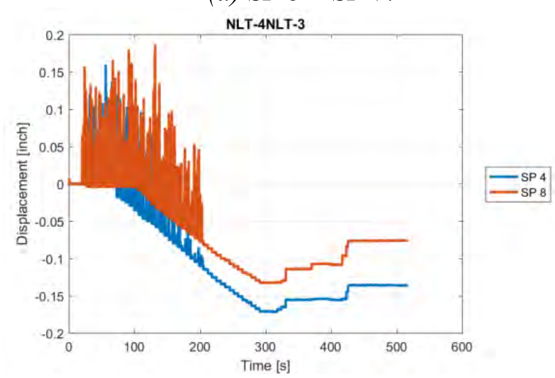

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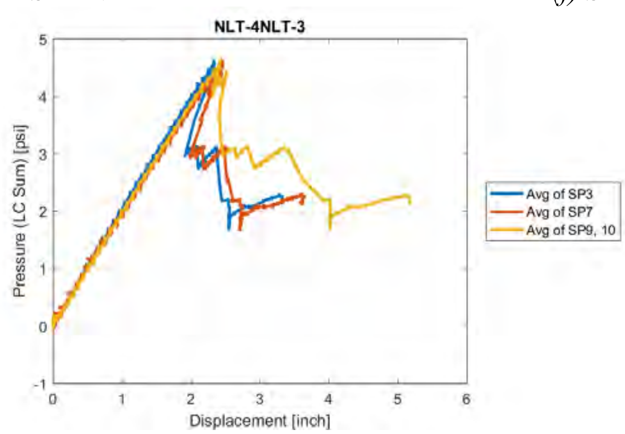
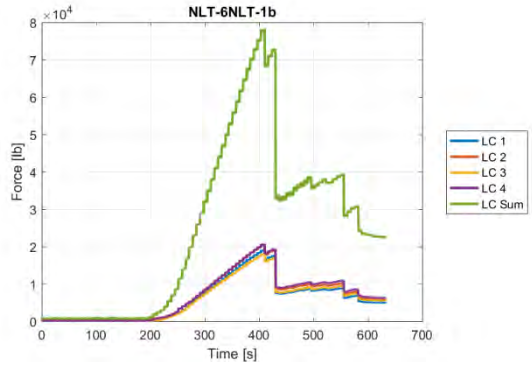
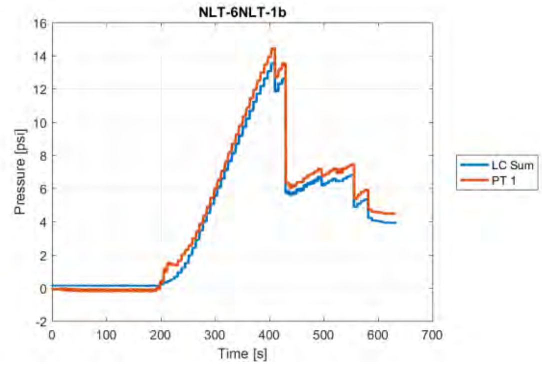


(g) Average SP.

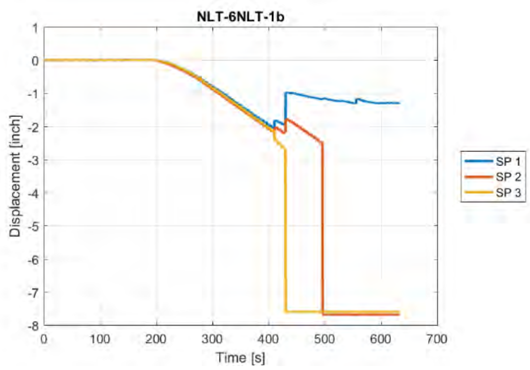
Figure B-28. 2x4 NLT – Test 3 (4NLT-3).



(a) LC.



(b) LC vs. PT 1.



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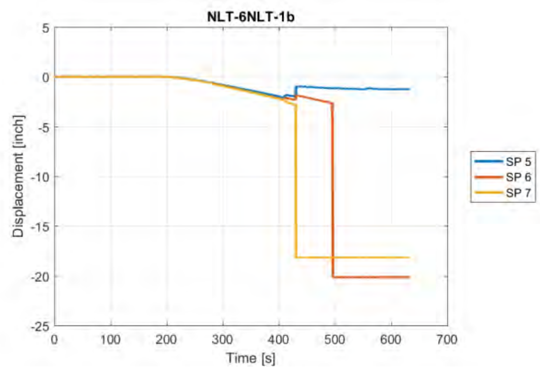

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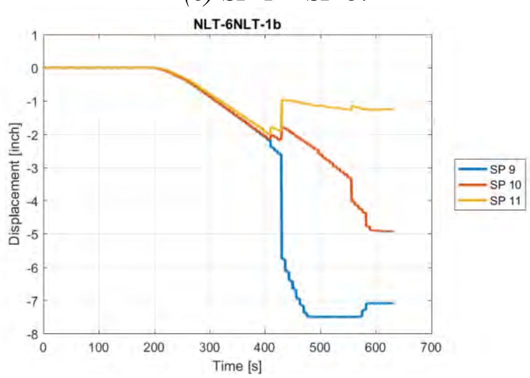

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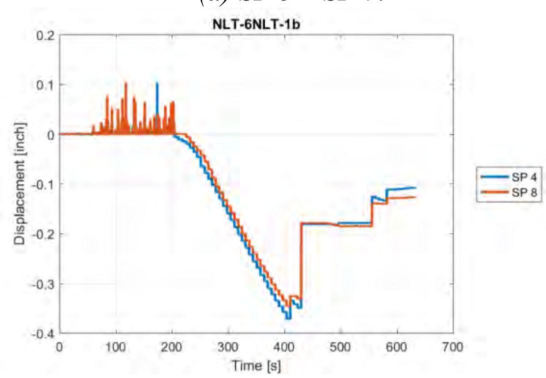
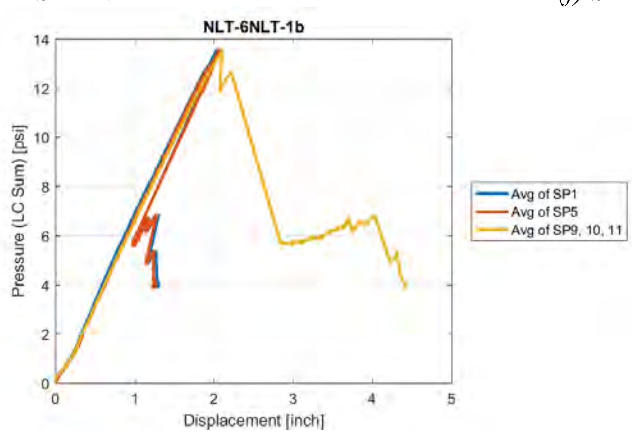
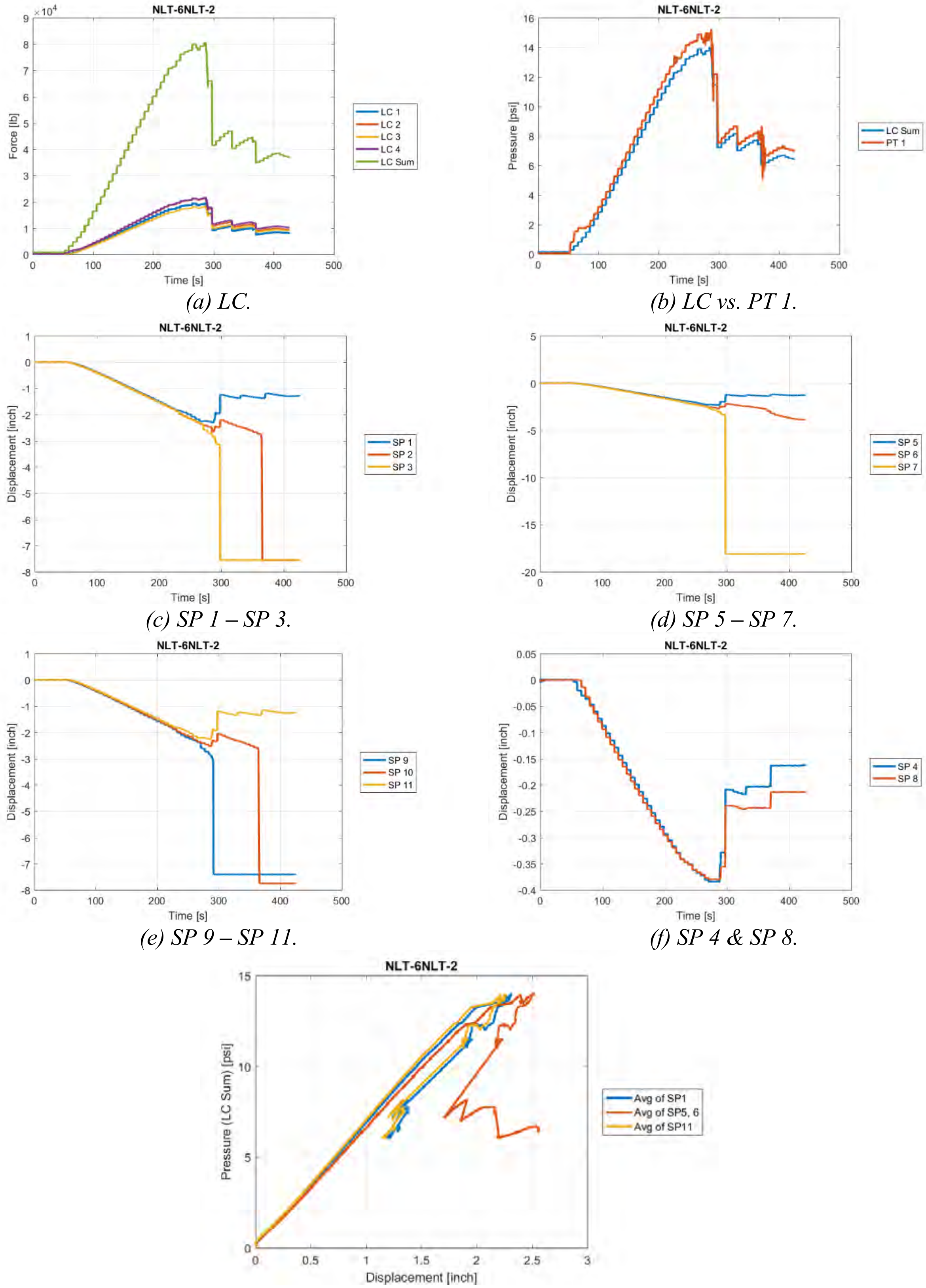


(f) SP 4 & SP 8.



(g) Average SP.

Figure B-29. 2x6 NLT – Test 1 (6NLT-1).



(a) LC.

(b) LC vs. PT 1.

(c) SP 1 – SP 3.

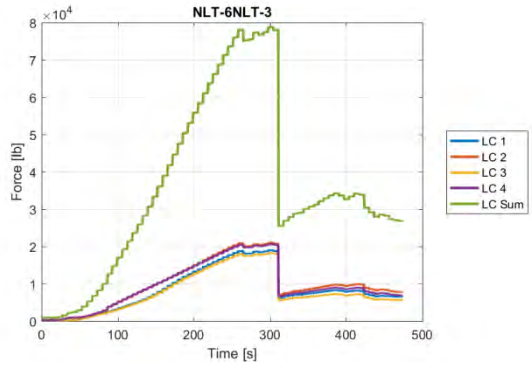
(d) SP 5 – SP 7.

(e) SP 9 – SP 11.

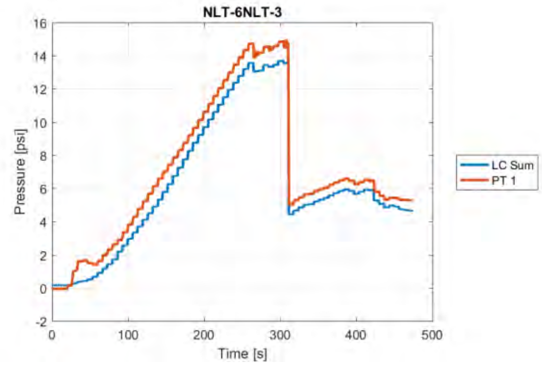
(f) SP 4 & SP 8.

(g) Average SP.

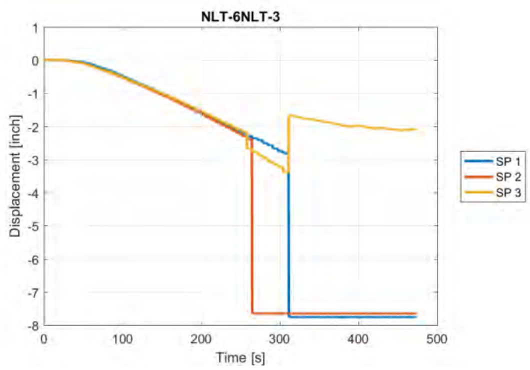
Figure B-30. 2x6 NLT – Test 2 (6NLT-2).



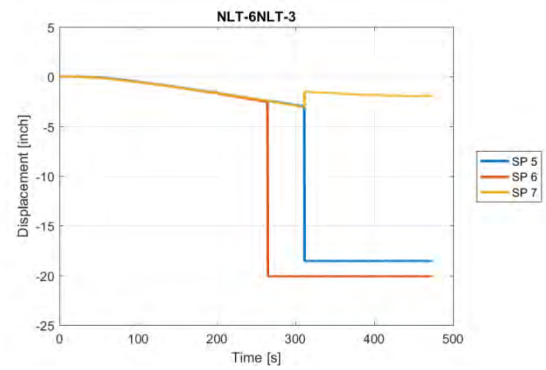
(a) LC.



(b) LC vs. PT 1.



(c) SP 1 – SP 3.



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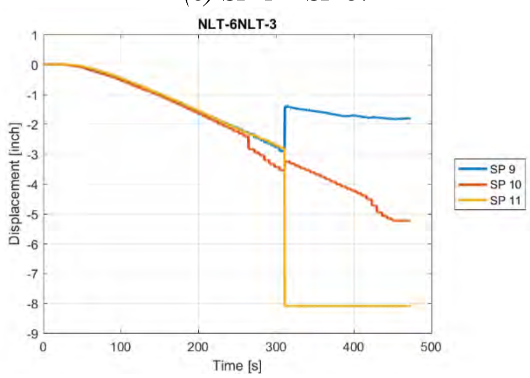

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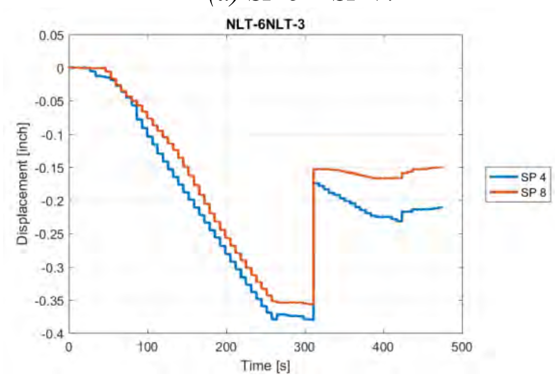
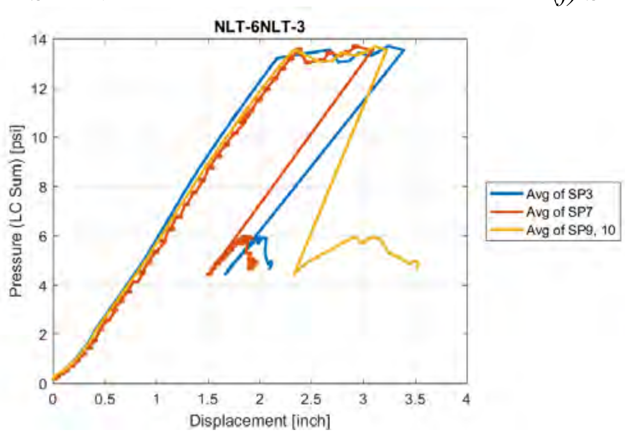


(f) SP 4 & SP 8.



(g) Average SP.

Figure B-31. 2x6 NLT – Test 3 (6NLT-3).

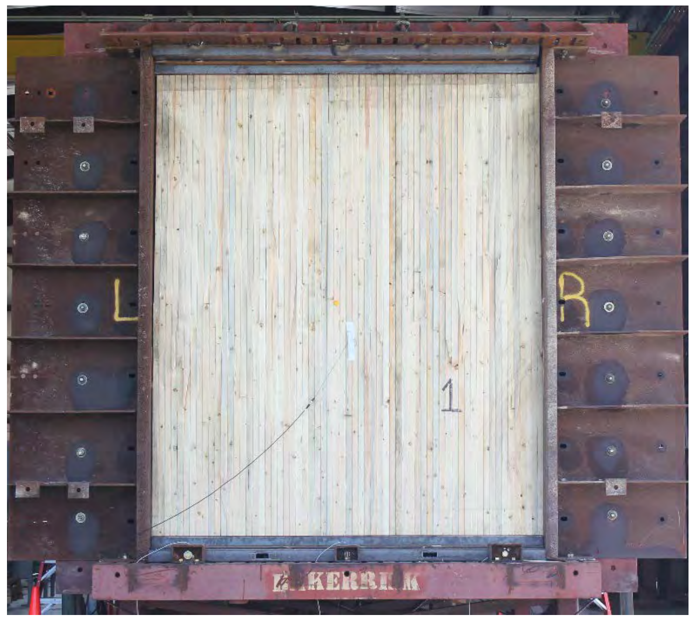
APPENDIX C

NLT SHOCK TUBE TESTING FINAL REPORT

Blast Testing of Nail Laminated Timber Panels

Final Report

Date Issued:
October 21, 2016



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01-05826-001-16

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1 INTRODUCTION

Baker Engineering and Risk Consultants (BakerRisk®) was contracted by Karagozian and Case Inc. (K&C) to perform shock tube testing of six Nail Laminated Timber panels. The test specimens have nominal thickness dimensions of 4″ and 6″. These tests were performed to support analytical models prepared by K&C. The data was used to validate and improve these analytical models. Test loads were chosen to cause low, medium and high damage response on the panels. This report summarizes the test objectives, the approach used, and the results obtained.

2 TEST SPECIMENS

Two types of specimens were provided to BakerRisk by K&C are nail laminated timber timber panels. The panels are 10′ tall and 8′ wide. The 4″ deep panels are constructed from standard 2×4 nominal (1.5″ × 3.5″ actual dimensions) members oriented in their strong-axis, clad with ½″ thick Douglas fir plywood on one side, for a total panel thickness of 4″. Each of the 2×4 members were attached adjacently with two 0.121″ × 3″ nails spaced at 16″ on center. The plywood is attached to the 2×4 members with 0.121″ × 3″ nails at approximately 6″ on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12″ centers horizontally and 16″ centers vertically. The 6″ panels were identical with but used 2×6 vertical members instead of 2×4 members. All the nail patterns were the same. Three of each panels were supplied for testing.

Six panels were provided for testing. A photo of a panel installed in the test frame is shown in Figure 1.

Figure 1. 4" Thick Nail Laminated Timber Panel

3 TEST APPROACH

3.1 Test Frame

All panels were secured in a near-rigid test frame designed and constructed by BakerRisk to resist the applied load and remain elastic. The frame consisted of a top and bottom horizontal slotted frame that were custom fabricated by BakerRisk and used in a number of earlier test programs. These pieces were bolted to the flange of the shock tube. Horizontal channel sections were attached to both the top and bottom slotted frames to shrink the clear vertical opening to 115″. The channel was welded to the top section to provide a rigid support. The channel on the bottom was bolted to the slotted frame using two 1″ diameter bolts only designed to prevent uplift of the channel. These bolts were placed in oversized holes which allowed the panel to slide in the direction of the load. The sliding channel on the bottom was supported by three rigid supports with 50,000 psi load cells located between the sliding channel and the rigid supports. The load cells were Futek LTH500 strain gauge type donut load cells. In order to provide a simple support condition, 0.5″ diameter round bar was used on the loaded and non-loaded supports. The use of round bar and a small support gap facilitates unrestricted rotation at the ends of the specimen.

A schematic sketch of the support condition is provided in Figure 2, and the support steel framing is shown in Figure 3.

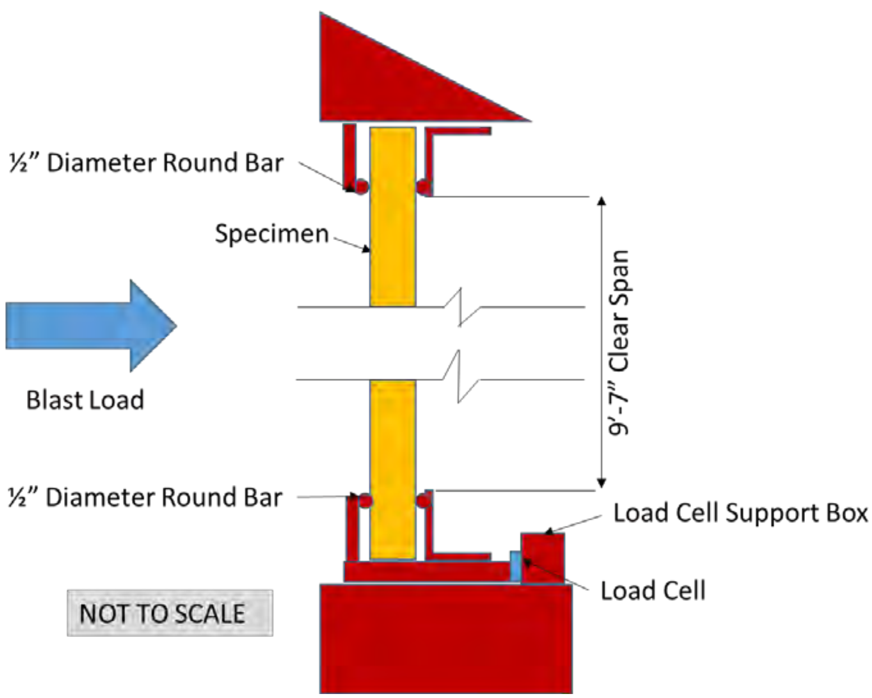


Figure 2. Schematic Vertical Section through Test Panel

Figure 3. Test 1 Pre Test Photo (Non-loaded Side)

3.2 Testing Apparatus

Dynamic testing of the prepared specimens was conducted in the BakerRisk shock tube facility. The shock tube is a test apparatus that consists of two major sections: a driver section and an expansion section. Blast pressures are generated when an aluminum rupture disk placed between the two sections fails due to pressure in the driver section. A shock wave then travels down the expansion section and loads the test specimen(s) mounted at the end of the expansion section. In this testing series, as is customary, the driver was baffled to reduce the effects of reloading by the reflections that exist in the shock tube. The shock tube is shown in Figure 4.

Figure 4. BakerRisk Shock Tube

The BakerRisk shock tube has an 8-foot square target area in its normal configuration, but it can be fitted with expansion sections to increase the target size to as large as 10 feet wide and 16 feet tall. In this test program the 10-foot square target configuration was utilized.

3.3 Active Instrumentation

For each test, the test load was measured using three dynamic pressure transducers located on each side wall and the floor of the shock tube just upstream of the test panels, close to the target end of the shock tube, in order to record the applied pressure data near the location of the test articles. Figure 5 shows the location of the pressure transducers. The load reported for each test is the average of the peak from each of the three gauges and represents the load applied to the test specimen(s). The pressure transducers used for all tests were model number 102M196, manufactured by PCB Piezotronics. An accelerometer (model 350B04, also by PCB) was placed at the center midspan of the test article, on the downstream face, to measure the specimens' dynamic displacement.

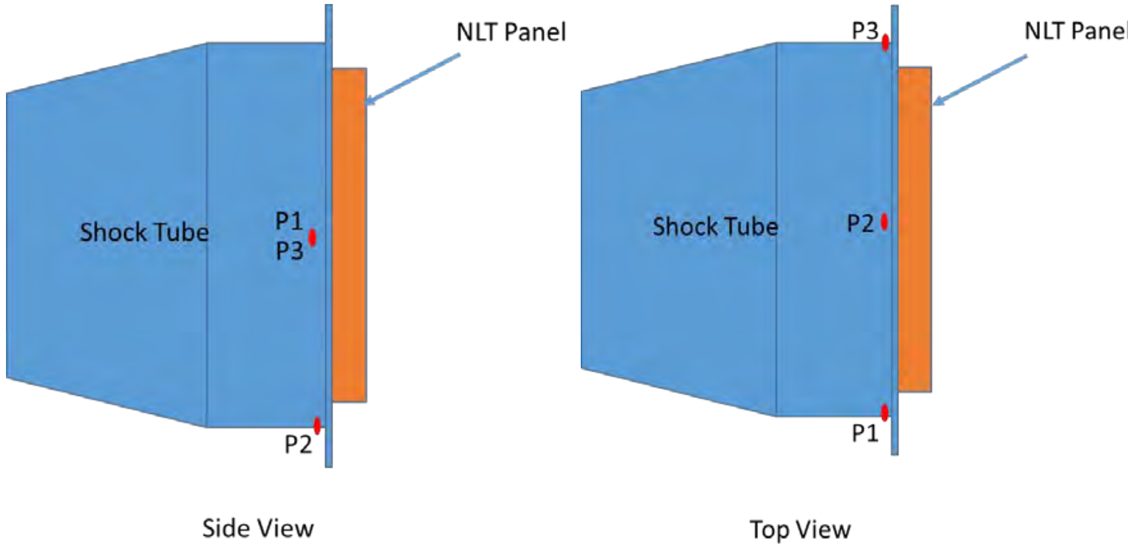


Figure 5. Pressure Transducer Location

The data from the pressure transducers and the accelerometer was recorded using a National Instruments PXIe–1071 system with 32 channels and having a 1.25 MHz sampling rate capability. The actual sampling rate used in the test was at 500 kHz for the pressure data and 50 kHz for the load cell data. Voltage signals from the transducers were conditioned using a PCB-481A02 multipurpose 16-channel amplifying power unit. All data was saved to computer disk for data plotting and interpretation. The load cell data was recorded using an NI 9237 4 channel card installed in a cDAQ-9188 8 slot chassis.

3.4 Other Documentation

In addition to the active instrumentation, each test was documented using two high-speed video cameras recording at 1000 frames per second. One camera was used to document the overall response of the specimen. The other camera was used at an oblique angle to view movement of the specimens in the support. Each test was also documented using a normal video camera and still photography before and after the test. All video and photographic data was captured and saved to computer disk.

4 TEST RESULTS

A total of 6 tests were conducted in this test program, one on each of the six panel specimens. Each panel was tested only one time. K&C provided the loading for each of the tests. The pressure was held constant with minor variations but the impulse was changed for each test. A summary of the recorded blast loads and observed panel responses are shown below in Table 1. Response descriptions are included with annotations of “Low,” “Medium,” and “High,” which correspond to blast resistant analysis and design industry standard qualitative response criteria.

Table 1. Summary of Test Results

Test	Total Panel Thickness (in)	Applied Peak Pressure		Applied Impulse		Peak Deflection (in)	Peak Support Rotation (deg)	Response Description
		(psi)	(kPa)	(psi-ms)	(kPa-ms)			
1	4	15.8	109	80.8	557	4.0	1.0	“Medium” - Significant cracking and wood spalling with debris throw; cracking through full cross-section of edge members.
2	4	13.0	89.6	60.2	415	3.3	0.82	“Medium” - Significant cracking with minimal debris throw; no cracking observed through full cross section of edge members.
3	4	15.0	103	94.5	652	> 20	> 5.0	“High” - Significant cracking and large deformation of both the 2×4 members and plywood; cracking observed through full cross section.
4	6	13.8	95.1	78.0	538	1.8	0.45	“Low” - No cracking observed, other than one shard ejected from non-loaded face. Near elastic response.
5	6	15.1	104	128	883	2.5	0.62	“Medium” – cracking evident during test, with cracks closing post-response. No cracking evident through edge members’ cross sections.
6	6	14.9	103	201	1390	2.7	0.67	“Medium” - Significant cracking and wood spalling with debris throw; cracking through full cross-section of edge members.

Following each test, the panels were removed from the supports and the edge members visually inspected. Results are fully detailed in Appendix A.

4.1 Test 1

The accelerometer gauge became detached from the test specimen due to specimen damage. The specimen was measured to have a peak deflection of 4″ at a time of 18 ms, as observed on the post-test high speed video displacement grid. The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range (Figure 6). The 2×4 wood members exhibited cracking at midspan through the majority of their cross-section as observed at the end members after removal of the specimen from the test frame.

Figure 6. Test 1 Debris Field (Left) and Damage View (Right)

4.2 Test 2

The test specimen was measured to have a peak deflection of 3.3″ at a time of 18 ms. The non-loaded face of the specimen was observed to have significant cracking with minimal wood fragment debris throw down range. Cracking was not observed through the cross-section of the 2×4 wood members after removal of the specimen from the test frame.

4.3 Test 3

The specimen was measured to have a peak deflection over 20″ at a time of approximately 106 ms, as observed on the post-test high speed video displacement grid. The accelerometer gauge became detached from the test specimen due to specimen damage.

The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range (Figure 7). The 2×4 wood members exhibited cracking at midspan through the majority of their cross-section as observed from both the non-loaded face and at the end members after removal of the specimen from the test frame. The loaded plywood face of the specimen also exhibited significant cracking and permanent deformation. The specimen exhibited significant shortening as the result of the high displacement and disengaged from the bottom support.

Figure 7. Test 3 Debris Field and Support Disengagement (Top) and Damage Views (Bottom)

4.4 Test 4

The specimen was measured to have a peak deflection of 1.8″ at a time of 13.5 ms and cracking was not observed, except for one shard of debris ejected from the non-loaded face. Cracking of the 2×4 wood member cross sections was not observed after removal of the specimen from the test frame.

4.5 Test 5

The test specimen was measured to have a peak deflection of 2.5″ at a time of 14.5 ms. The non-loaded face of the specimen was observed to exhibit cracking during the test with the wood members returning to their original position and cracks closing post-test. Cracking was observed through the cross-section of the 2×4 wood members after removal of the specimen from the test frame (Figure 8).

Figure 8. Test 5 Specimen Post-Test Cracking of Edge Member at Mid-Span

4.6 Test 6

The specimen was measured to have a peak deflection of 2.7″ at a time of 16.3 ms. The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range. The 2×4 wood members exhibited cracking at midspan through the majority of their cross-section as observed at the end members after removal of the specimen from the test frame (Figure 9).

Figure 9. Test 6 Specimen Post-Test Cracking of Edge Member at Mid-Span

5 CONCLUSION

All panels tested received some level of damage, though none were thrown from the supports due to excessive deflection. The threshold of "low" response appears to be close to 0.5 degrees of support rotation when examining the test results. The panels fail in a brittle fashion, and debris generation is significant, depending on load impulse, at support rotations above 0.5 degrees. It should be noted that support rotation threshold may change at different spans.

APPENDIX A. TEST DATA

Test 1

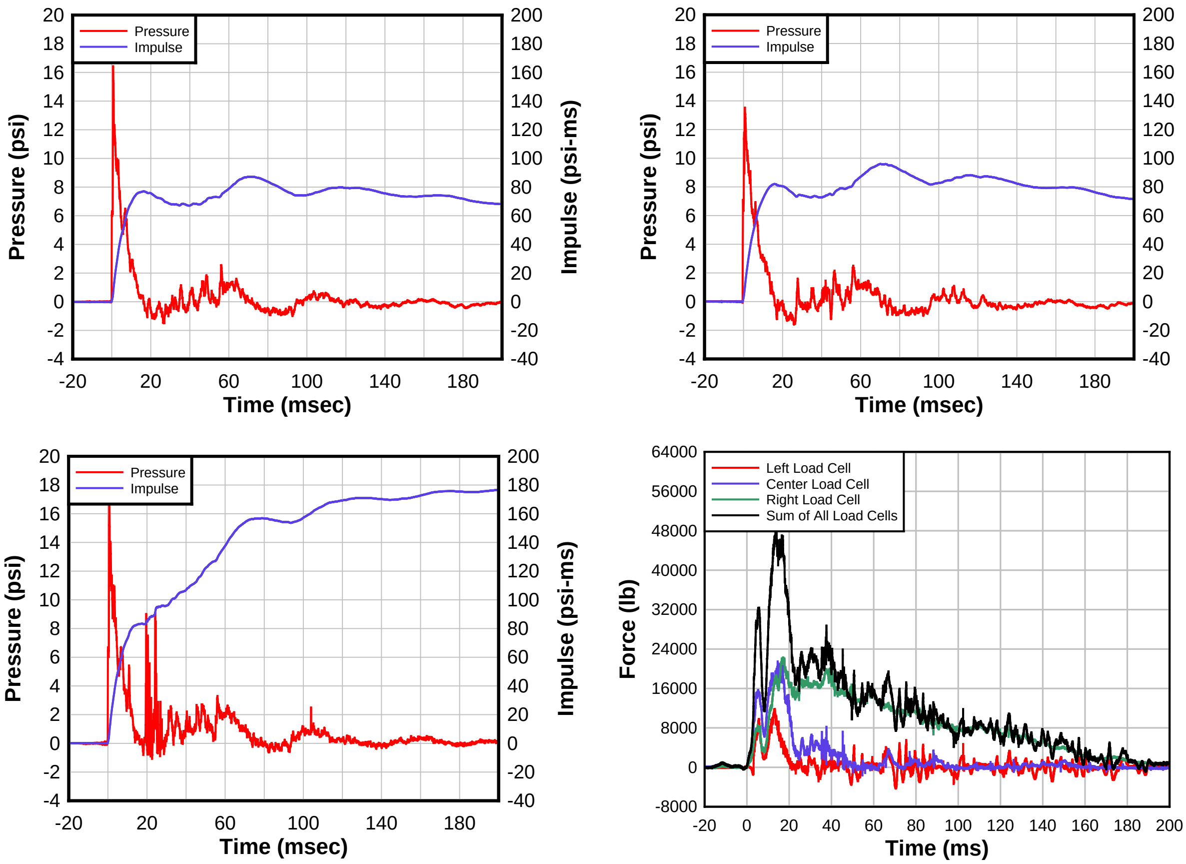
K&C Nail Laminated Wood Panels									
Test Name	**1**		**Test Date**	**4/13/2016**					
Driver Length		**45**	**Driver Pressure**		**125**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	16.4	77	16.2
2	102M196	25565	102.7	4	-1	1	13.5	82	15.9
3	102M196	25569	102.5	4	-1	1	17.4	83	17.5
				Average of Gauges 1-3			*15.8*	*80.8*	*16.5*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	N/A		N/A
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*11847.1*		*13.1*
6	FSH00586	637329	1.9894				*21480.0*		*14.6*
7	FSH00586	637328	1.9937				*22296.5*		*17.1*
				Sum of Gauges 5-7			*48052.6*		*13.6*

Test Specimen Description

The panels are 10’ tall and 8’ wide. The 4” deep panel are constructed from standard 2×4 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x4 members were attached adjacently with two 0.121” x 3” nails spaced at 16” on center. The plywood is attached to the 2×4 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12” centers horizontally and 16” centers vertically.

Response Description

The accelerometer gauge became detached from the test specimen due to specimen damage. The specimen was measured to have a peak deflection of 4 inches at a time of 18 ms, as observed on the post-test high speed video displacement grid. The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range. The 2x4 wood members exhibited cracking at midspan through the majority of their cross-section as observed at the end members after removal of the specimen from the test frame.



Test 1 Pressure Gauges and Load Cells

Test 2

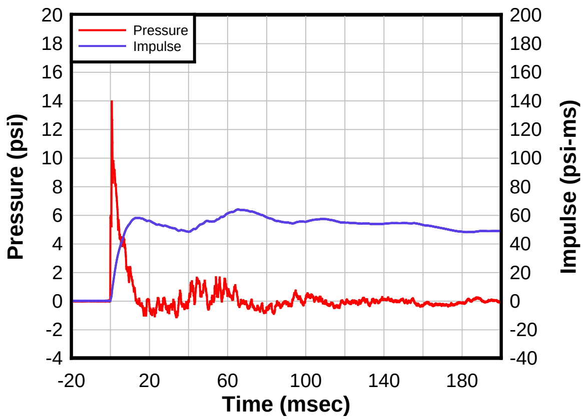
K&C Nail Laminated Wood Panels									
Test Name	**2**		**Test Date**	**4/13/2016**					
Driver Length		**37**	**Driver Pressure**		**120**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	14.0	58	13.4
2	102M196	25565	102.7	4	-1	1	10.9	62	14.8
3	102M196	25569	102.5	4	-1	1	14.1	61	15.5
				Average of Gauges 1-3			*13.0*	*60.2*	*14.6*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	3.3		17.9
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*14367.2*		*22.7*
6	FSH00586	637329	1.9894				*13812.6*		*14.3*
7	FSH00586	637328	1.9937				*13234.1*		*20.4*
				Sum of Gauges 5-7			*32528.8*		*14.3*

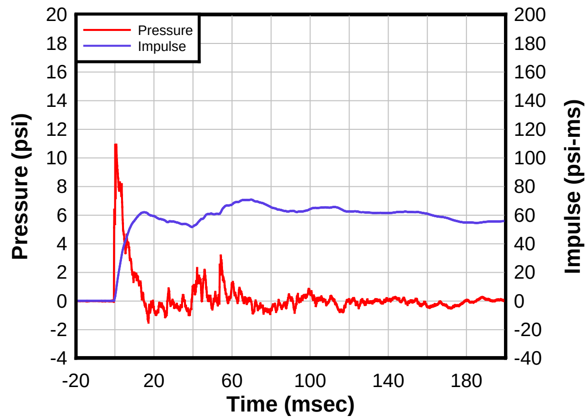
Test Specimen Description

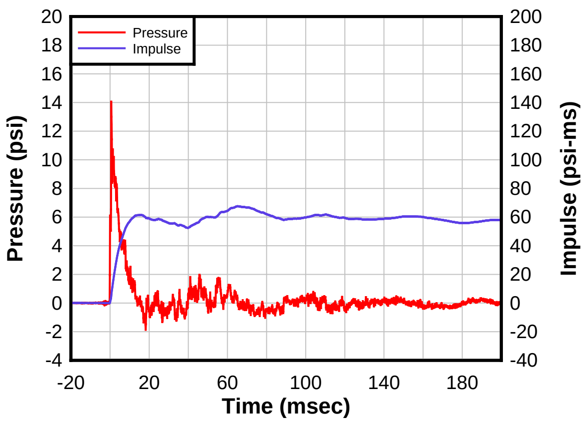
The panels are 10' tall and 8' wide. The 4" deep panel are constructed from standard 2×4 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x4 members were attached adjacently with two 0.121" x 3" nails spaced at 16" on center. The plywood is attached to the 2×4 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12" centers horizontally and 16" centers vertically.

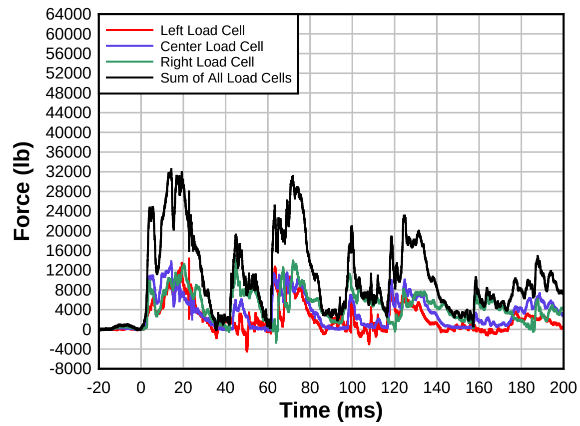
Response Description

The test specimen was measured to have a peak deflection of 3.3 inches at a time of 18 ms. The non-loaded face of the specimen was observed to have significant cracking with minimal wood fragment debris throw down range. Cracking was not observed through the cross-section of the 2x4 wood members after removal of the specimen from the test frame.

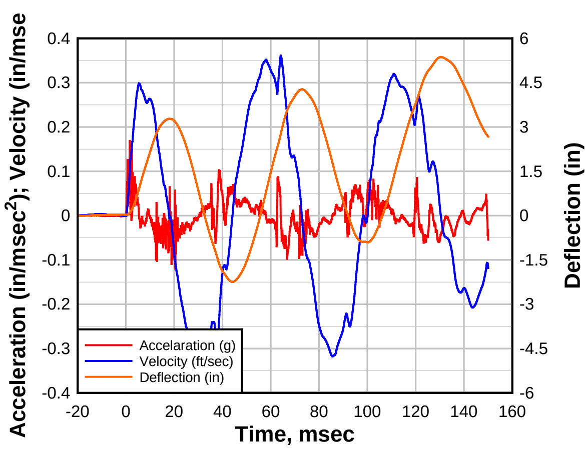








Test 2 Pressure Gauges and Load Cells



Test 2 Accelerometer

Test 3

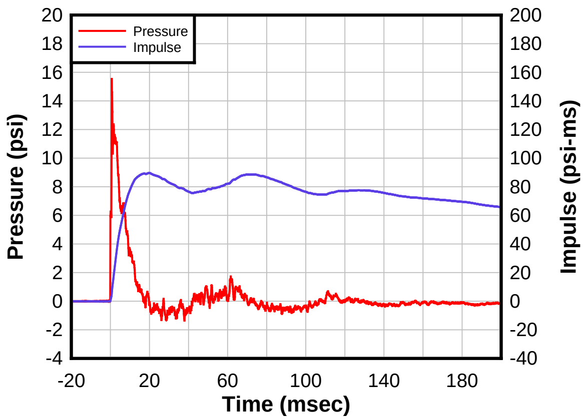
K&C Nail Laminated Wood Panels									
Test Name	**3**		**Test Date**	**4/13/2016**					
Driver Length		**57**	**Driver Pressure**		**123**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	15.5	90	19.7
2	102M196	25565	102.7	4	-1	1	13.5	96	19.7
3	102M196	25569	102.5	4	-1	1	16.0	98	28.4
				Average of Gauges 1-3			*15.0*	*94.5*	*22.6*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	N/A		N/A
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*16300.5*		*12.9*
6	FSH00586	637329	1.9894				*23079.0*		*14.9*
7	FSH00586	637328	1.9937				*16239.7*		*13.9*
				Sum of Gauges 5-7			*50317.4*		*14.1*

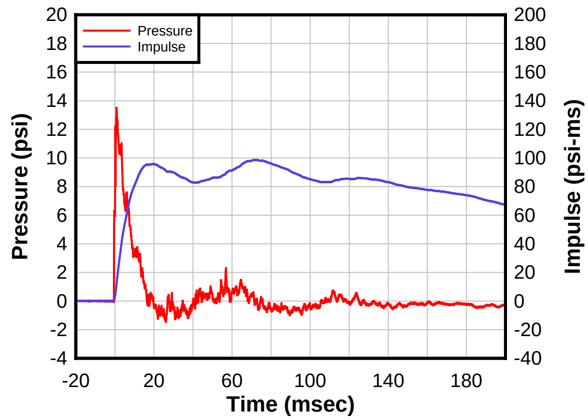
Test Specimen Description

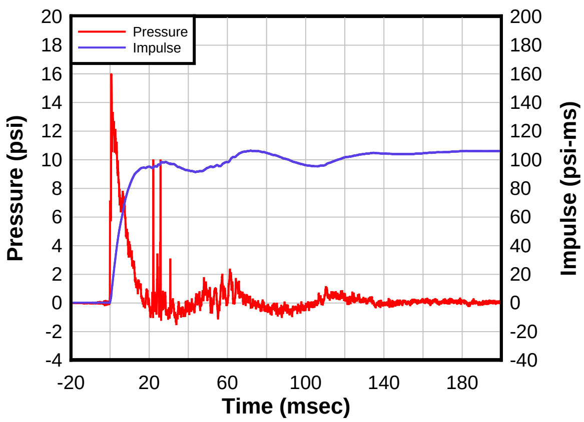
The panels are 10’ tall and 8’ wide. The 4” deep panel are constructed from standard 2×4 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x4 members were attached adjacently with two 0.121” x 3” nails spaced at 16” on center. The plywood is attached to the 2×4 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12” centers horizontally and 16” centers vertically.

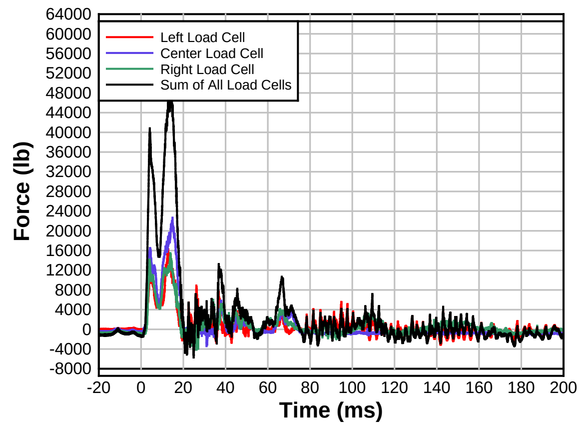
Response Description

The specimen was measured to have a peak deflection over 20 inches at a time of approximately 106 ms, as observed on the post-test high speed video displacement grid. The accelerometer gauge became detached from the test specimen due to specimen damage. The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range. The 2x4 wood members exhibited cracking at midspan through the majority of their cross-section as observed from both the non-loaded face and at the end members after removal of the specimen from the test frame. The loaded plywood face of the specimen also exhibited significant cracking and permanent deformation. The specimen exhibited significant shortening as the result of the high displacement and disengaged from the bottom support.









Test 3 Pressure Gauges and Load Cells

Test 4

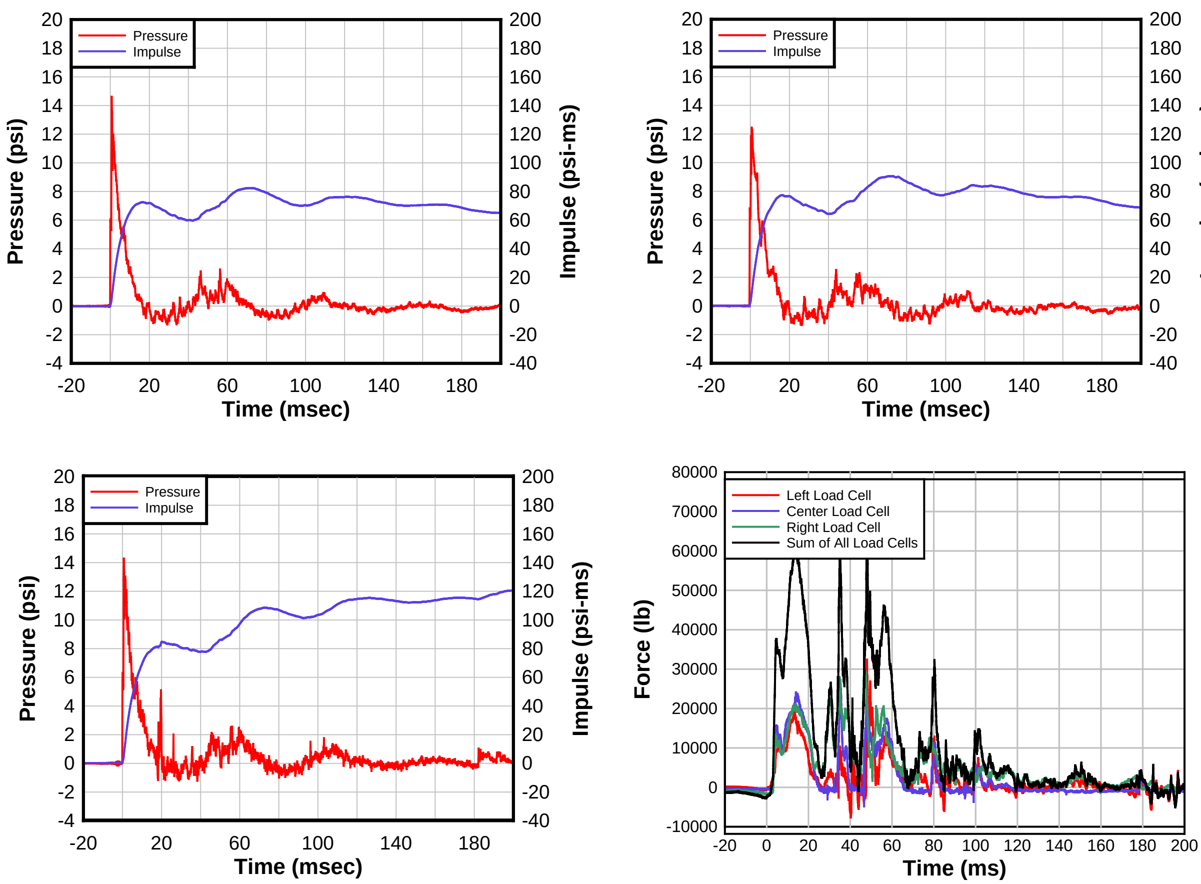
K&C Nail Laminated Wood Panels									
Test Name	**4**		**Test Date**	**4/13/2016**					
Driver Length		**42**	**Driver Pressure**		**123**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	14.6	73	16.5
2	102M196	25565	102.7	4	-1	1	12.4	77	16.1
3	102M196	25569	102.5	4	-1	1	14.3	84	20.2
				Average of Gauges 1-3			*13.8*	*78.0*	*17.6*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	1.8		13.5
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*19044.8*		*13.4*
6	FSH00586	637329	1.9894				*24632.2*		*14.0*
7	FSH00586	637328	1.9937				*22099.5*		*13.9*
				Sum of Gauges 5-7			*63647.2*		*14.0*

Test Specimen Description

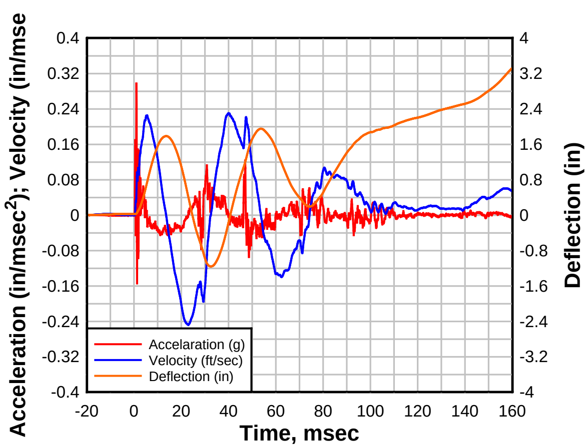
The panels are 10’ tall and 8’ wide. The 6” deep panel are constructed from standard 2×6 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x6 members were attached adjacently with two 0.121” x 3” nails spaced at 16” on center. The plywood is attached to the 2×6 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12” centers horizontally and 16” centers vertically.

Response Description

The specimen was measured to have a peak deflection of 1.8 inches at a time of 13.5 ms and cracking was not observed, except for one shard of debris ejected from the non-loaded face. Cracking of the 2x4 wood member cross sections was not observed after removal of the specimen from the test frame.



Test 4 Pressure Gauges and Load Cells



Test 4 Accelerometer

Test 5

K&C Nail Laminated Wood Panels

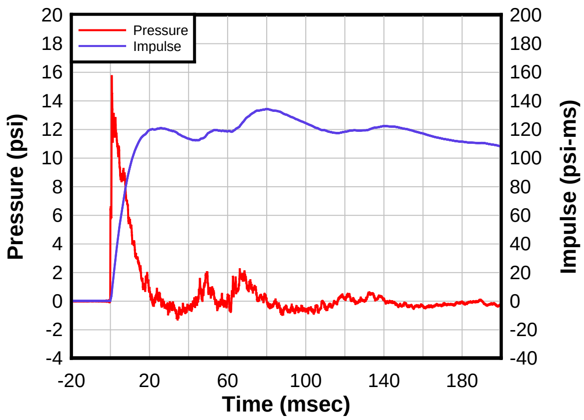
Test Name	**5**		**Test Date**	**4/13/2016**					
Driver Length		**72**	**Driver Pressure**		**123**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	15.7	121	25.9
2	102M196	25565	102.7	4	-1	1	13.7	128	28.5
3	102M196	25569	102.5	4	-1	1	15.7	135	15.5
				Average of Gauges 1-3			*15.1*	*128*	*23.3*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	2.5		14.5
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*29677.7*		*15.0*
6	FSH00586	637329	1.9894				*33189.5*		*14.7*
7	FSH00586	637328	1.9937				*29691.2*		*14.7*
				Sum of Gauges 5-7			*89546.6*		*14.7*

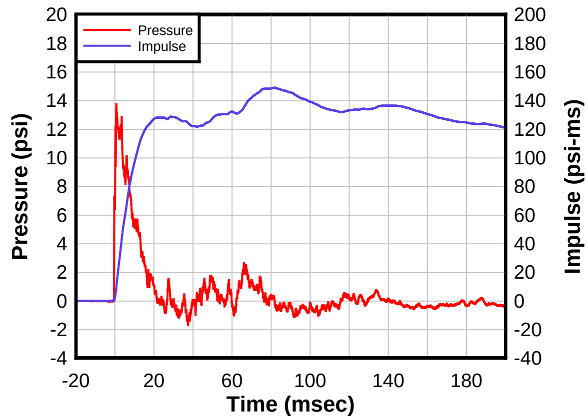
Test Specimen Description

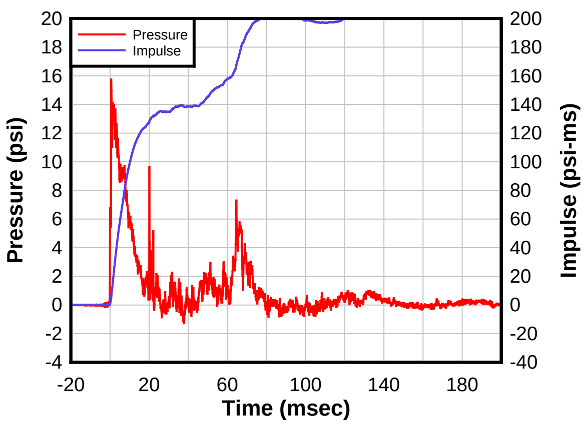
The panels are 10' tall and 8' wide. The 6" deep panel are constructed from standard 2×6 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x6 members were attached adjacently with two 0.121" x 3" nails spaced at 16" on center. The plywood is attached to the 2×6 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12" centers horizontally and 16" centers vertically.

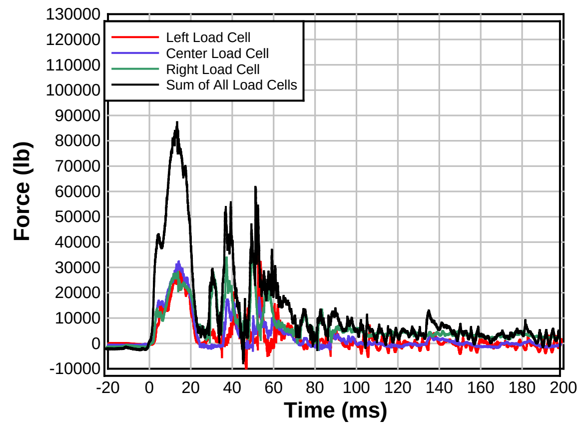
Response Description

The test specimen was measured to have a peak deflection of 2.5 inches at a time of 14.5 ms. The non-loaded face of the specimen was observed to exhibit cracking during the test with the wood members returning to their original position and cracks closing post-test. Cracking was observed through the cross-section of the 2x4 wood members after removal of the specimen from the test frame.

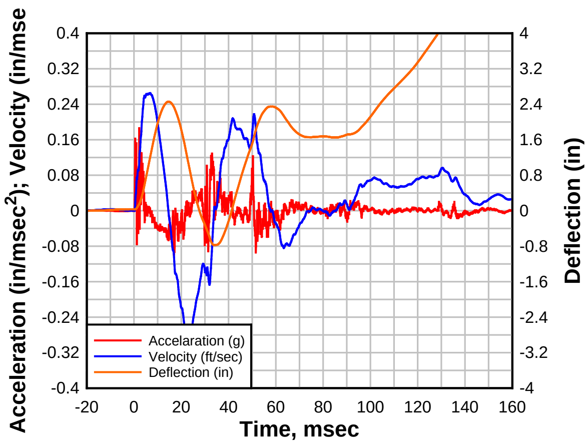








Test 5 Pressure Gauges and Load Cells



Test 5 Accelerometer

Test 6

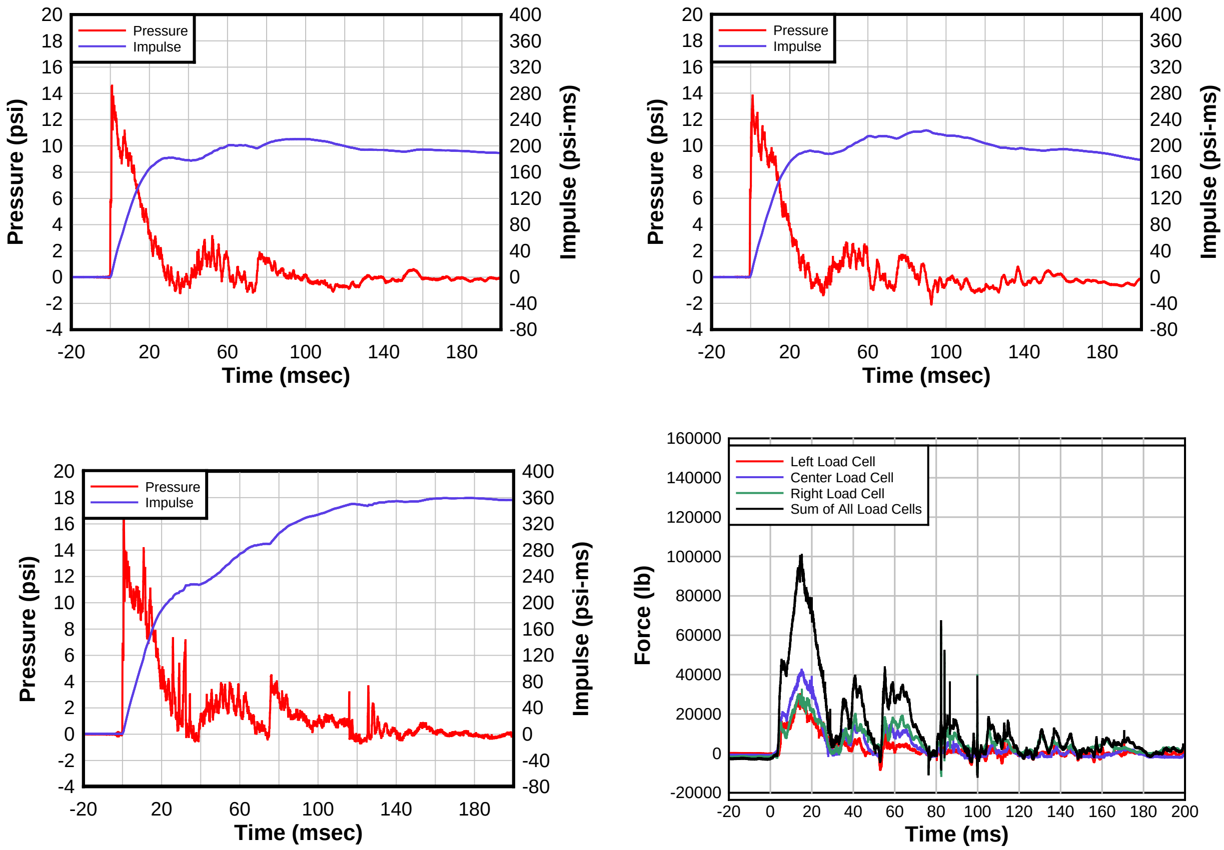
K&C Nail Laminated Wood Panels									
Test Name	**6**		**Test Date**	**4/13/2016**					
Driver Length		**116**	**Driver Pressure**		**123**	**psi**	**Air**		
Shock Tube Target		**10' square**			**Vents**	**Open**			
RWE	**24"**								
Gauge Number	**Gauge Type**	**Serial Number**	**Sensitivity**	**Full Scale Voltage (volts)**	**Negative Offset Voltage (volts)**	**Conditioner Gain**	**Peak Pressure (psi)**	**Maximum Impulse (psi-msec)**	**Duration (msec)**
Pressure Gauges in Shock Tube Walls									
1	102M196	25566	98.61	4	-1	1	14.6	182	31.7
2	102M196	25565	102.7	4	-1	1	13.8	192	30.4
3	102M196	25569	102.5	4	-1	1	16.2	228	36.5
				Average of Gauges 1-3			*14.9*	*201*	*32.9*
Accelerometer at Mid-Span							**Peak Deflection (in)**		**Time of Peak (ms)**
4	350B04	6188	4.78	10	-2	1	2.7		16.3
Load Cells at Bottom Support							**Peak Load (lbs)**		**Time of Peak Load (msec)**
5	FSH00586	637330	1.9813				*30261.7*		*14.4*
6	FSH00586	637329	1.9894				*43574.9*		*15.1*
7	FSH00586	637328	1.9937				*34316.5*		*15.2*
				Sum of Gauges 5-7			*103680.6*		*15.2*

Test Specimen Description

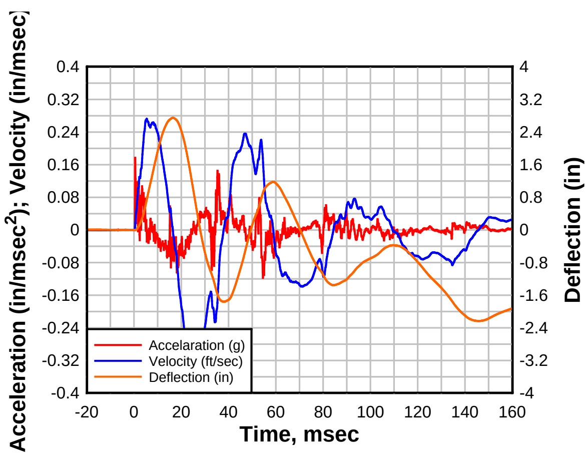
The panels are 10’ tall and 8’ wide. The 6” deep panel are constructed from standard 2×6 nominal (1.5" × 3.5" actual dimensions) members oriented in their strong-axis, clad with ½" thick Douglas fir plywood on one side, for a total panel thickness of 4". Each of the 2x6 members were attached adjacently with two 0.121” x 3” nails spaced at 16” on center. The plywood is attached to the 2×6 members with 0.121 × 3" nails at approximately 6" on center around the entire perimeter of each piece of plywood. There were additional nails that were spaced on the plywood at 12” centers horizontally and 16” centers vertically.

Response Description

The specimen was measured to have a peak deflection of 2.7 inches at a time of 16.3 ms. The non-loaded face of the specimen was observed to have significant cracking and wood spalling accompanied by wood fragment debris throw down range. The 2x4 wood members exhibited cracking at midspan through the majority of their cross-section as observed at the end members after removal of the specimen from the test frame.



Test 6 Pressure Gauges and Load Cells



Test 6 Accelerometer

APPENDIX D

SAVE BRIEFING FOR NLT SHOCK TUBE TESTS

Karagozian & Case

WoodWorks™ WOOD PRODUCTS COUNCIL

1865 THE UNIVERSITY OF MAINE

B-16-99 **D-2**

Karagozian & Case, Inc.
700 N. Brand Blvd.
Glendale, CA 91203
818-240-1919
www.kcse.com

Blast-Resistant Response of Nail-Laminated Timber (NLT)

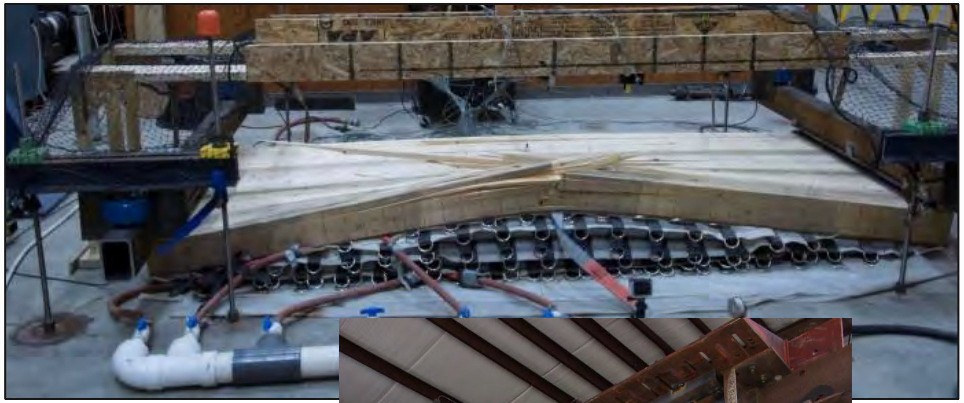
By:
Mark Weaver, S.E. (K&C)
Hossein Sadeghi (K&C)
Lisa Podesto, P.E. (WoodWorks)
Dr. Edwin Nagy, Ph.D., P.E., S.E. (UMaine)

Presented at:
Shock & Vibration Exchange
New Orleans, LA

October 18, 2016

Briefing Outline

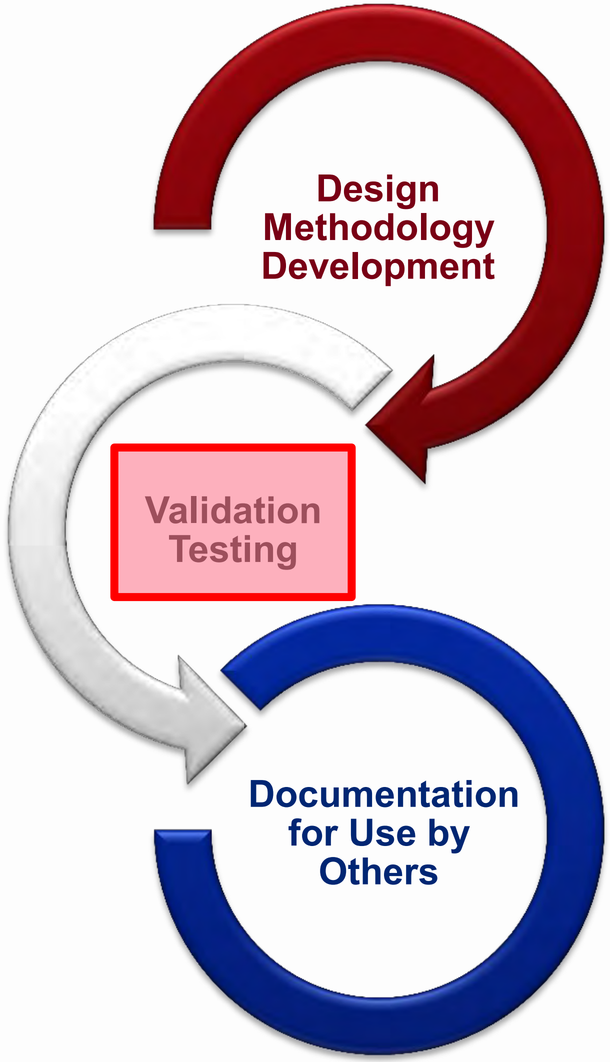
- Introduction
- Quasi-Static Testing
  - *Specimens & Test Setup*
  - *Results*
  - *Observations / Conclusions*
- Shock Tube Testing
  - *Specimens & Test Setup*
  - *Results*
  - *Observations / Conclusions*



Introduction

Objectives of Effort

- Propose design methodology to design NLT for blast loads.
- Conduct testing to investigate quasi-static & dynamic response of NLT as means to validate design methodology.
- Document design methodology & test data in form that will serve as reference for structural engineers interested in using NLT to resist blast loads.



Introduction
What is NLT?

- **Massive timber panel system.**
- **Use**
  - *In use for more than a century.*
  - *Used for floor, roof, & wall panels.*
- **Construction**
  - *Consists of dimensional lumber (typically 2x members) side-nailed together.*
  - *Plywood sheathing typically provided for structural diaphragm.*

Introduction
Overview of Objectives by Test Type

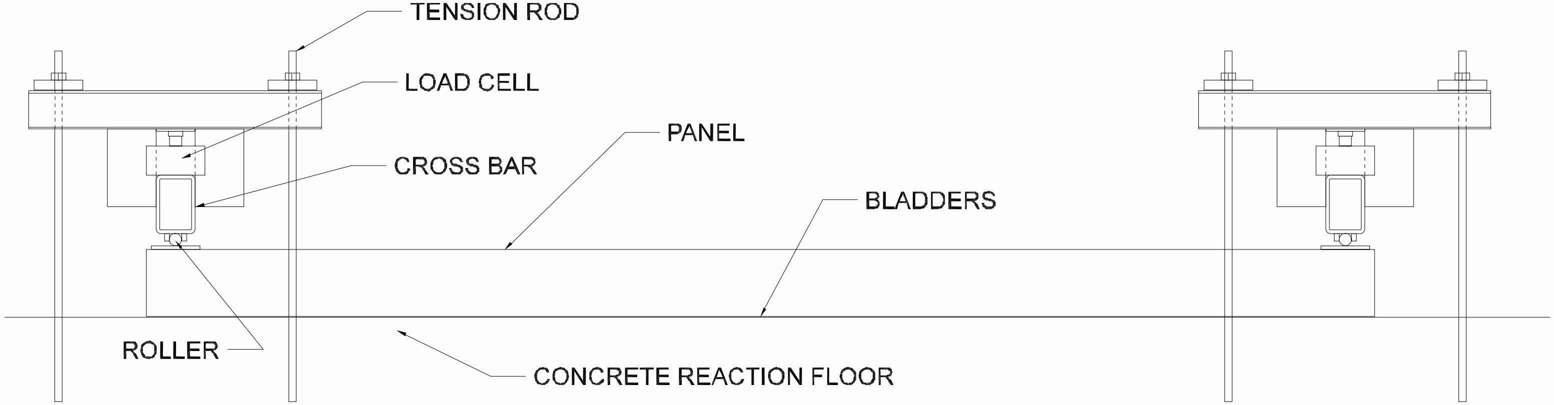
- **Quasi-Static (performed at University of Maine)**
  - ***Develop load-displacement response curve for SDOF dynamic analysis (i.e., resistance function).***
  - ***Investigate post-peak response.***
  - ***Propose SDOF response limits.***
- **Dynamic (Shock Tube) (performed at BakerRisk)**
  - ***Quantify dynamic increase factor (DIF).***
  - ***Test specimens with dynamic load similar to that expected from explosive event economically.***
  - ***Corroborate SDOF response limits.***

Quasi-Static Testing
Test Specimens

- All panels donated by StructureCraft.
- 4'(W) x 10'(L) panels w/ 2x members running in 10' direction.
- No. 2 or better Spruce Pine Fir 2x members.
- 1/2" plywood nailed to compression face of specimen.
  - *Edge nailing @ 12" O.C.*
  - *Interior nailing*
    - 16" O.C. in 10' direction.
    - 12" O.C. in 8' direction.

ID	2x Size	No. of Specimens
4NLT	2x4	3
6NLT	2x6	3

Quasi-Static Testing
Test Setup



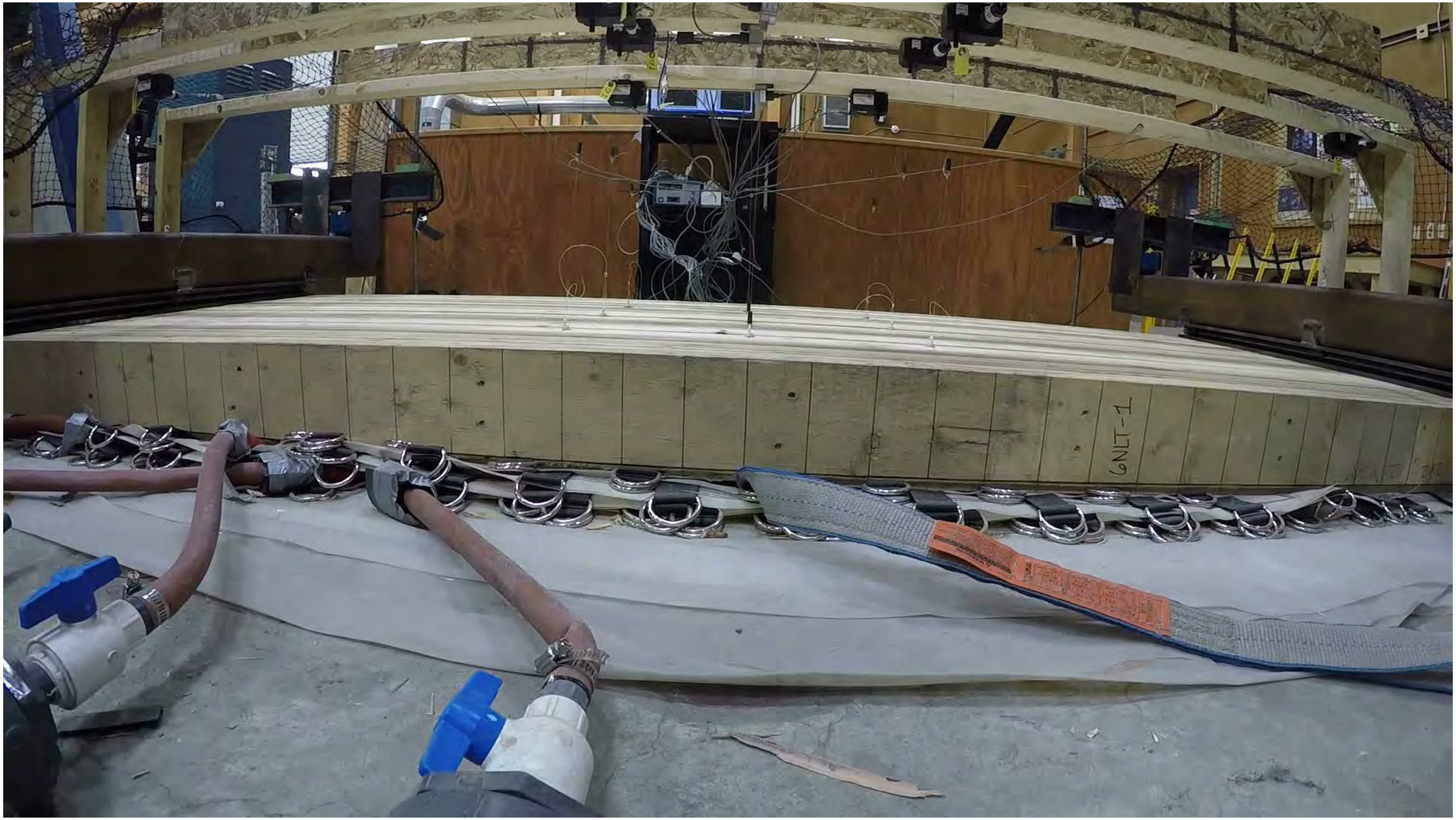
Vertical Lines to Track Shear Slippage @ Ply Interface

Crossbar

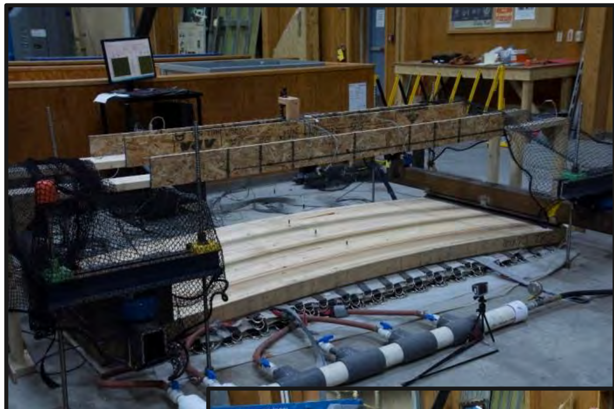
Load Cell

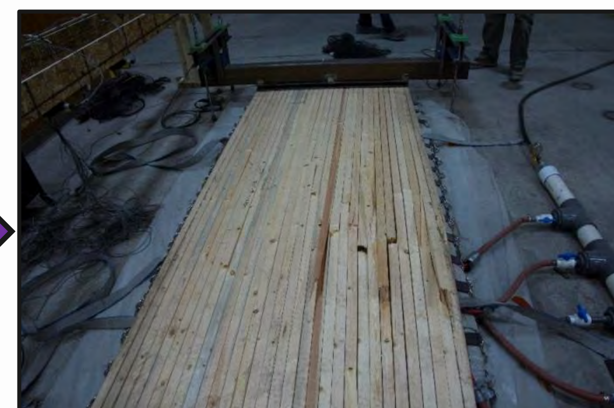
Bladders stacked used to apply uniform pressure

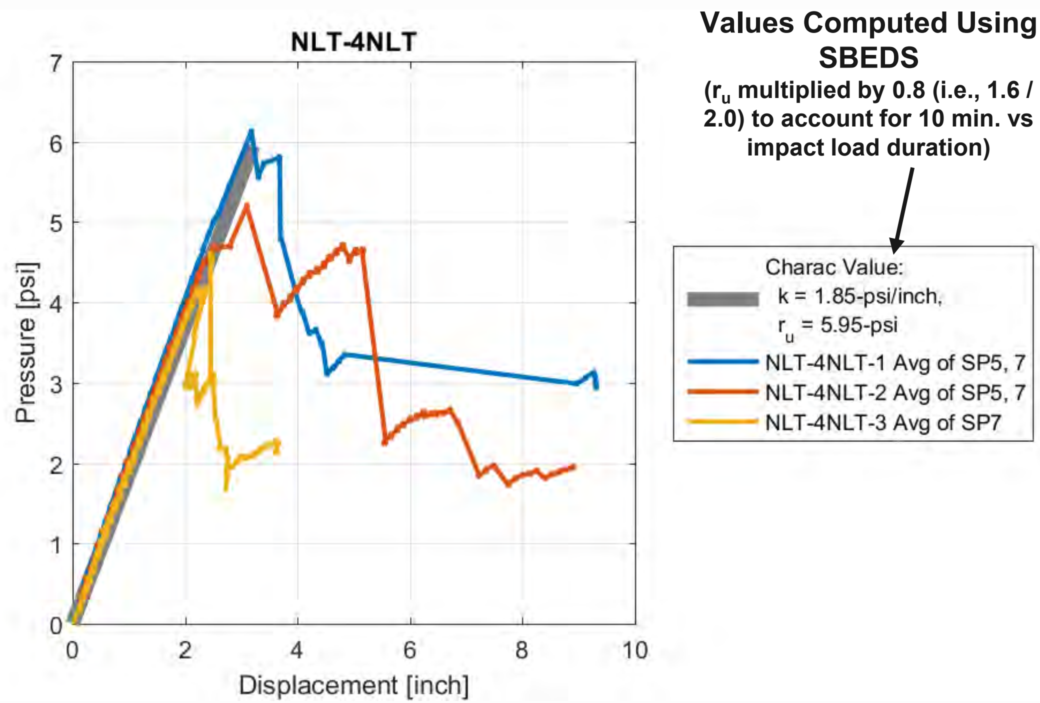
Quasi-Static Testing
Video



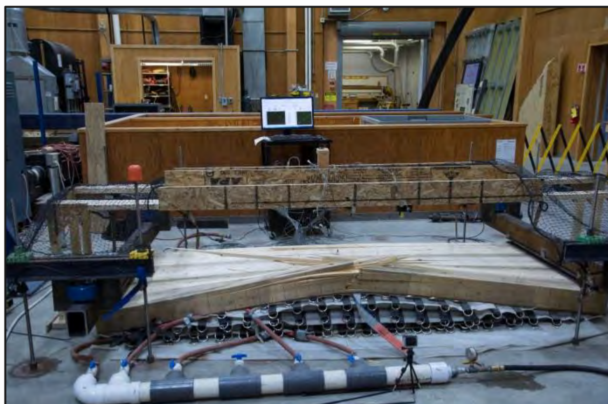
Quasi-Static Testing
2x4 Specimen Results

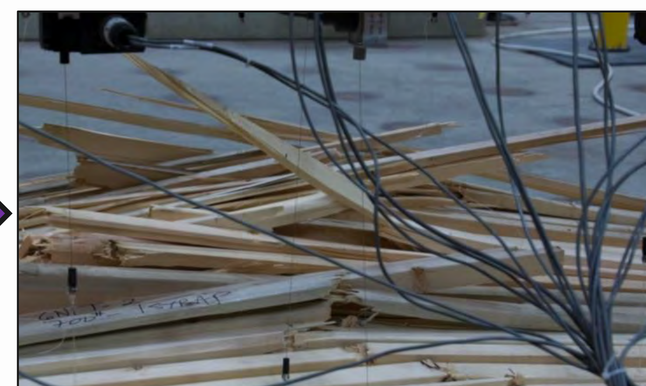


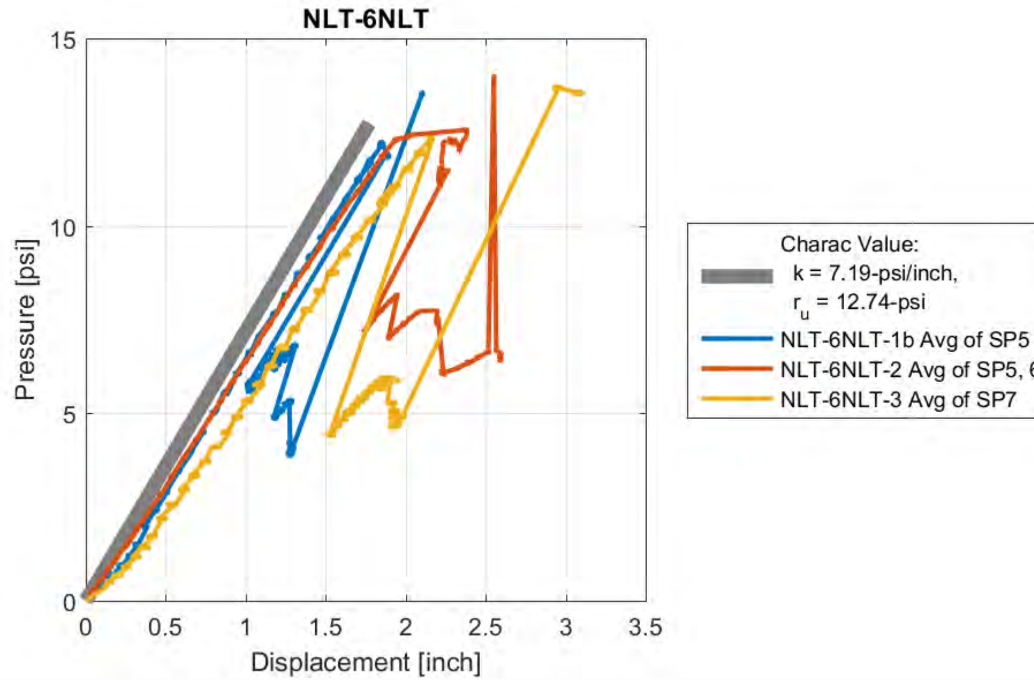




Quasi-Static Testing
2x6 Specimen Results





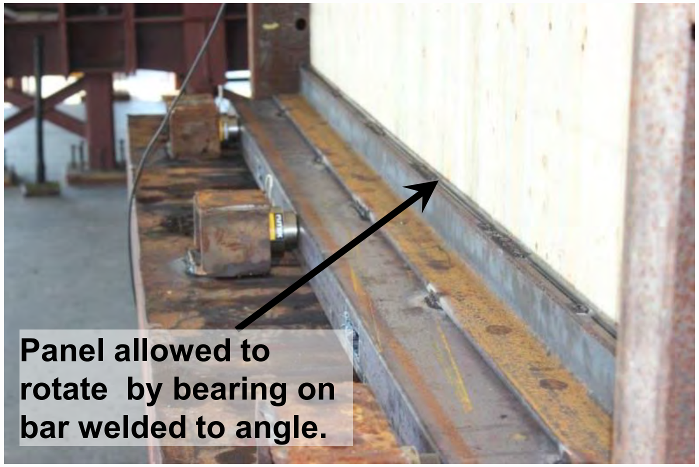


Quasi-Static Testing
Observations / Conclusions

- Typical failure
  - *Bending rupture near mid-span.*
  - *Rupture of boundary stud often initiated failure.*
  - *Failure did not propagate through entire panel.*
- Essentially elastic to failure.
- Post-peak response difficult to gauge considering number of displacement gauges that were lost at panel rupture.
- Significant variation in 2x4 NLT bending strength; not as much observed for 2x6 NLT bending strength.
- Comparison with SBEDS wood beam module.
  - *Stiffness: SBEDS strength generally consistent; however, under-predicts for 2x6 cases.*
  - *Strength: SBEDS strength generally consistent; however, over-predicts for several 2x4 cases.*

Shock Tube Testing
Overview

- Same type / number of test specimens as quasi-static tests except 8’ wide panels were used instead 4’ wide panels.
  - *No. 2 or better SPF.*
  - *9.5’ clear span.*
  - *Same nailing.*
- Blast load: Selected to achieve 0.75, 1.00, and 1.25 ductility ratio response based on SBEDS wood model.
  - *Pressure held constant while impulse varied.*



Shock Tube Testing
Video (2x4 NLT: $\mu = 1.0$ Target)

Shock Tube Testing

Video (2x6 NLT: $\mu = 1.0$ Target)

Shock Tube Testing

Results

Test ID	Specimen Description	P_r [psi]	I_r [psi-ms]	Δ_{test} [in]	Δ_{SBEDS} [in]
1	2x4 NLT	15.8	80.8	**4.0**	4.17
2	2x4 NLT	13.0	60.2	**3.3**	3.17
3	2x4 NLT	15.0	94.5	**20+**	4.30
4	2x6 NLT	13.8	78.0	**1.8**	1.25
5	2x6 NLT	15.1	128	**2.5**	1.83
6	2x6 NLT	14.9	201	**2.7**	2.18

- **Computed deflections are based on actual recorded pressure histories.**

Shock Tube Testing
Observations / Conclusions

- **Ductility response limit based on current SBEDS analytical model leads to significantly different qualitative responses for 2x4 and 2x6 NLT.**
  - *Including effects of sheathing may be necessary to obtain a viable SDOF model for NLT.*

- **2x4 NLT panel is blown out for ductility ratio close to 1.25.**
  - *Blow out happens at much lower ductility ratio than listed in PDC-TR 06-08.*
  - *Ultimate resistance calculation requires review.*

- **SBEDS model consistently under-predicts 2x6 NLT peak response.**
  - *Assumed panel stiffness is too high.*

APPENDIX E

PRE-TEST BRIEFING FOR CLT LIVE BLAST TESTS

K&C
Karagozian & Case

WoodWorks™
WOOD PRODUCTS COUNCIL

1865 THE UNIVERSITY OF MAINE

B-16-100 **E-2**

CLT Live Blast Pre-Test Briefing

Karagozian & Case, Inc.
700 N. Brand Blvd.
Glendale, CA 91203
818-240-1919
www.kcse.com

By:
Mark Weaver, S.E. (K&C)
Tim Brewer, CEng MICE (K&C)
Shengrui Lan, Ph.D. (K&C)
Lisa Podesto, P.E. (WoodWorks)
Dr. Edwin Nagy, Ph.D., P.E., S.E. (UMaine)

Presented at:
Tyndall AFB
Panama City, FL

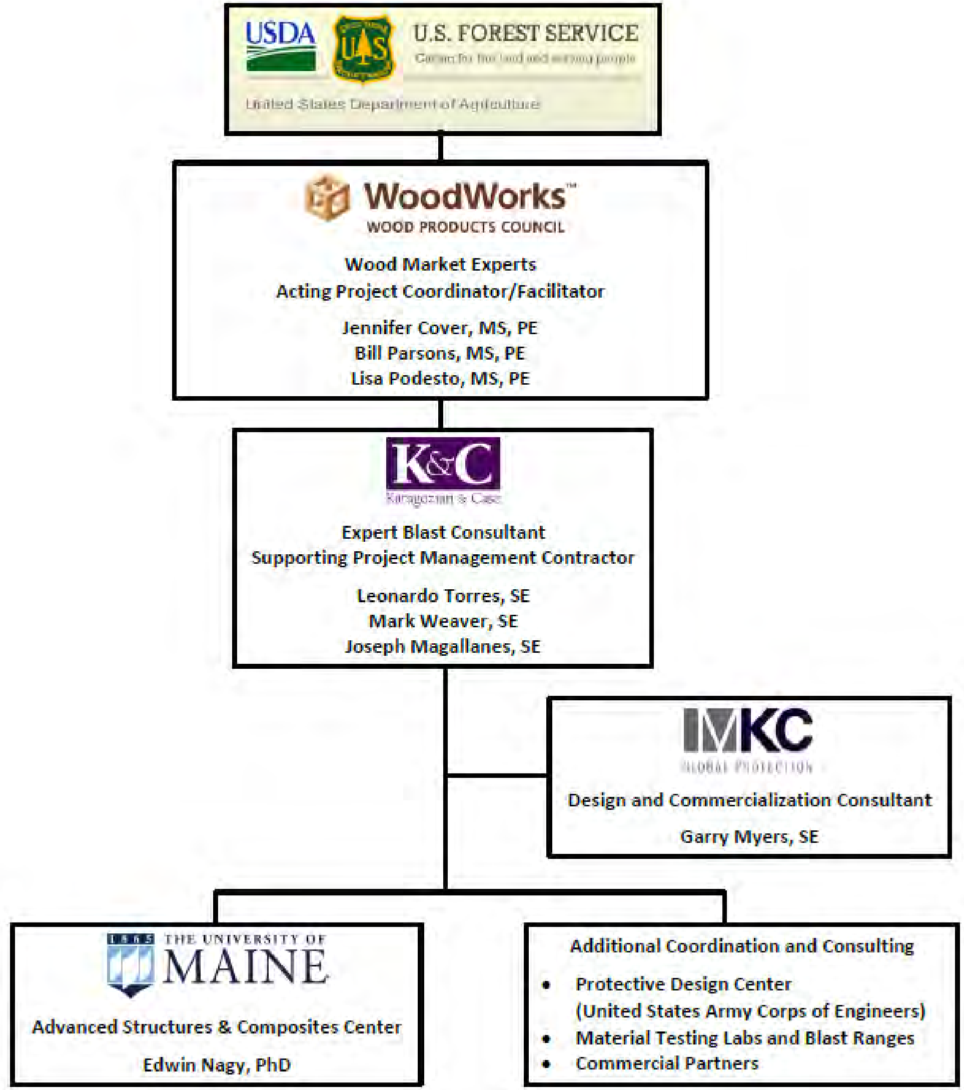
October 12, 2016

Briefing Outline

- CLT Testing
  - *UMaine quasi-static testing.*
  - *PDC shock tube testing.*
- Live Blast Test Plan
  - *Site Layout*
  - *Construction*
  - *Instrumentation*
  - *Participating Manufacturers*
  - *Design Methodology*
  - *FE Modeling*

Project Overview

- **Funding provided by domestic grant from USDA, Forest Service.**
- **Objectives**
  - ***Conduct testing to investigate dynamic response of massive timber & connections.***
  - ***Propose methodology to design massive timber for blast loads.***
  - ***Document design methodology and available test data in a form that ideally will serve as a basis / reference for structural engineers interested in using massive timber to resist blast loads.***

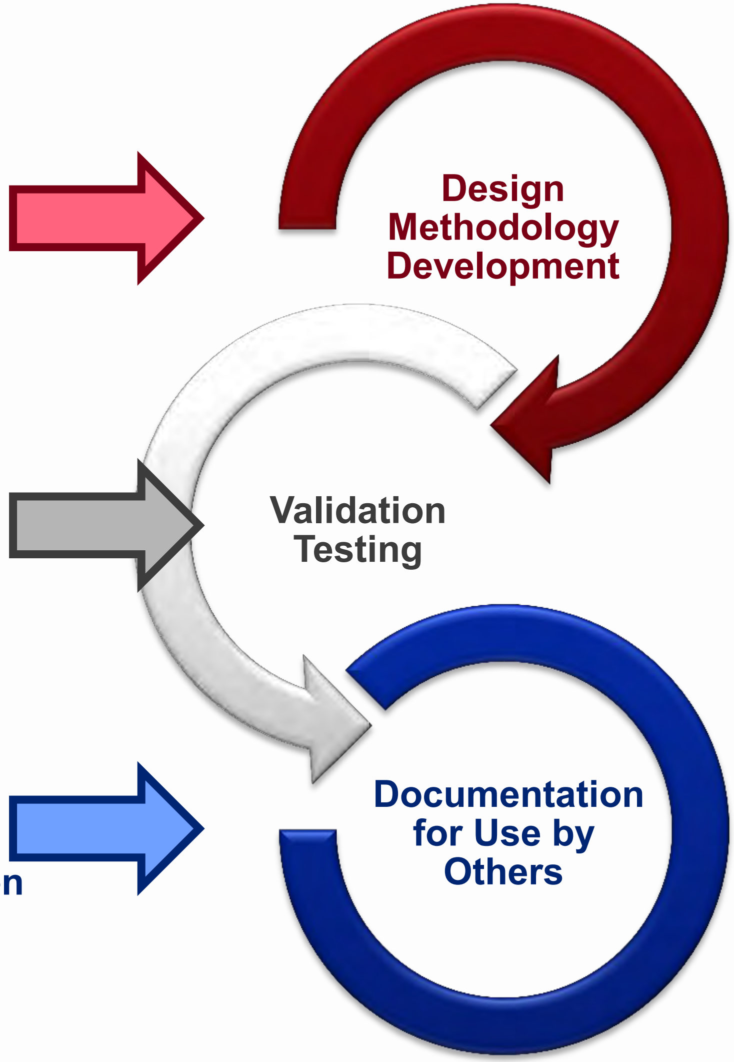


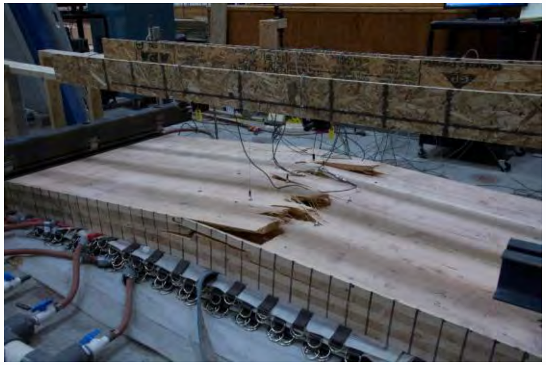
CLT Blast Design Methodology Development Overview

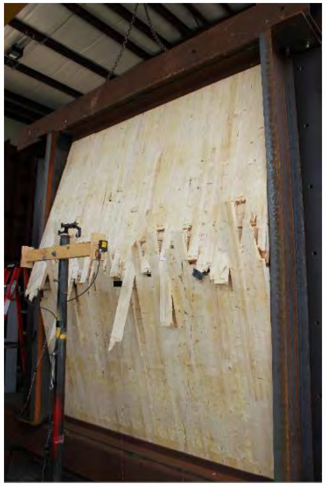
- ✓ Resistance Function
- ✓ Response Limits
- ✓ Dynamic Increase Factor
- ✓ FE Modeling Corroboration

- ✓ Quasi-Static
- ✓ Dynamic (Shock Tube)
- ✓ Dynamic (Live Blast)

- ✓ Test Reports
- ✓ Design Guidance
- ✓ Conventional Construction Parameters







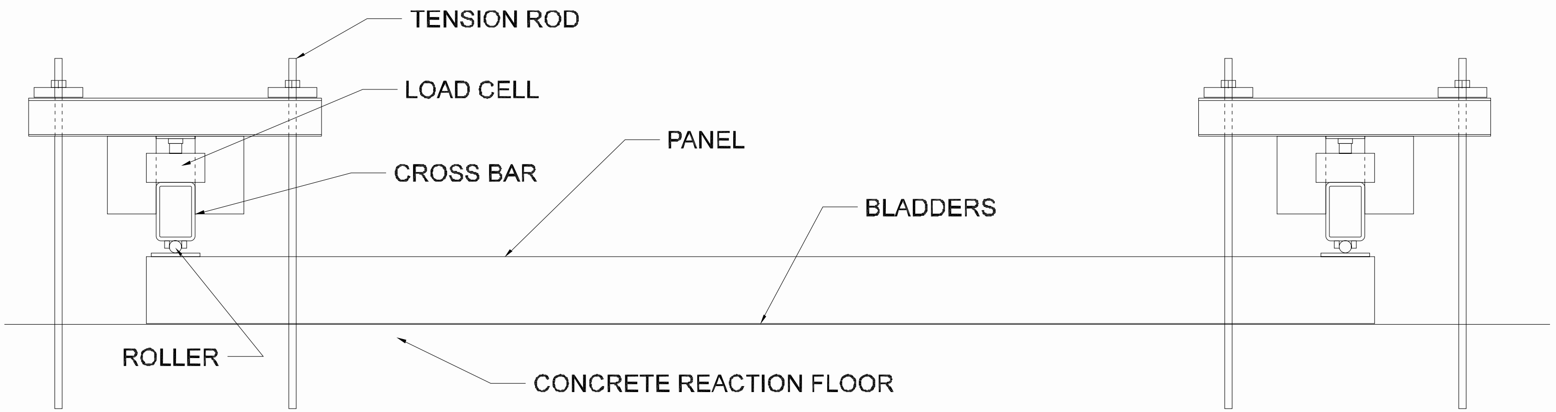
CLT Testing
Overview of Objectives

- **Quasi-Static**
  - *Develop dynamic analysis response curve for SDOF dynamic analysis (i.e., resistance function).*
  - *Quantify post-peak response.*
  - *Propose SDOF response limits.*
- **Dynamic (Shock Tube)**
  - *Quantify dynamic increase factor (DIF).*
  - *Test specimens with dynamic load similar to that expected from explosive event economically.*
  - *Corroborate SDOF response limits.*
- **Dynamic (Live Blast)**
  - *Demonstrate system level response for actual explosive event.*

Quasi-Static CLT Testing

Test Setup for Panel Tests

B-16-100
pg 6 **E-7**



Vertical Lines to Track Shear Slippage @ Ply Interface

Crossbar

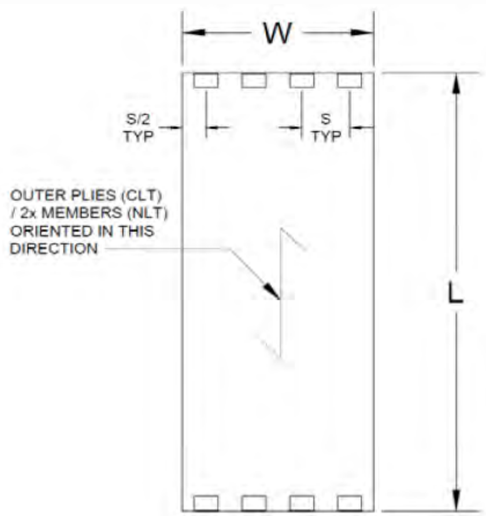
Load Cell

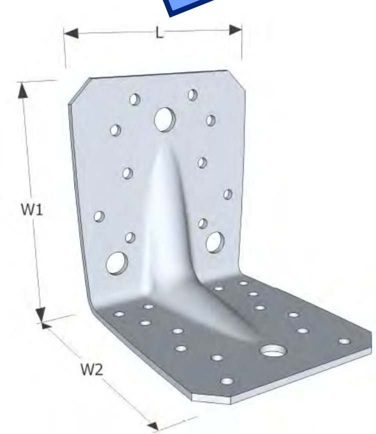
Bladders stacked used to apply uniform pressure

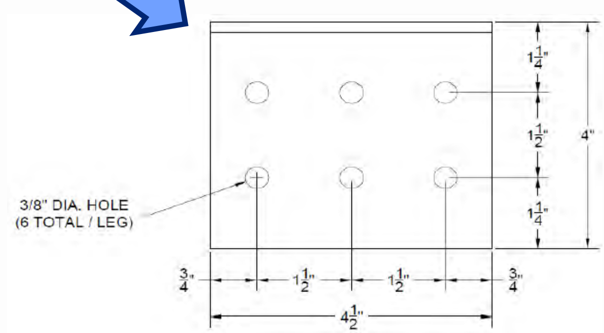
Quasi-Static CLT Testing

Specimen Schedule

ID	Description	Ply No.	L [in]	W [in]	QTY	Bracket Type	Horz. Leg Fasteners	Vert. Leg Fasteners
CLT-V1	Grade V1	3	126	48	4	No Connection		
CLT-E1	Grade E1	3	126	48	4	No Connection		
CLT-V4	Grade V4	3	126	48	4	No Connection		
CLT-5V1	Grade V1	5	126	48	3	No Connection		
CLT-V1CA	Grade V1	3	114	48	3	ABR105 (Simpson)	(14) SD10212 (Simpson)	(10) SD10212 (Simpson)
CLT-V1CB	Grade V1	3	114	48	3	4x4x1/4	(6) ASSY SK 5/16x4 (MyTiCon)	(6) ASSY SK 5/16x4 (MyTiCon)

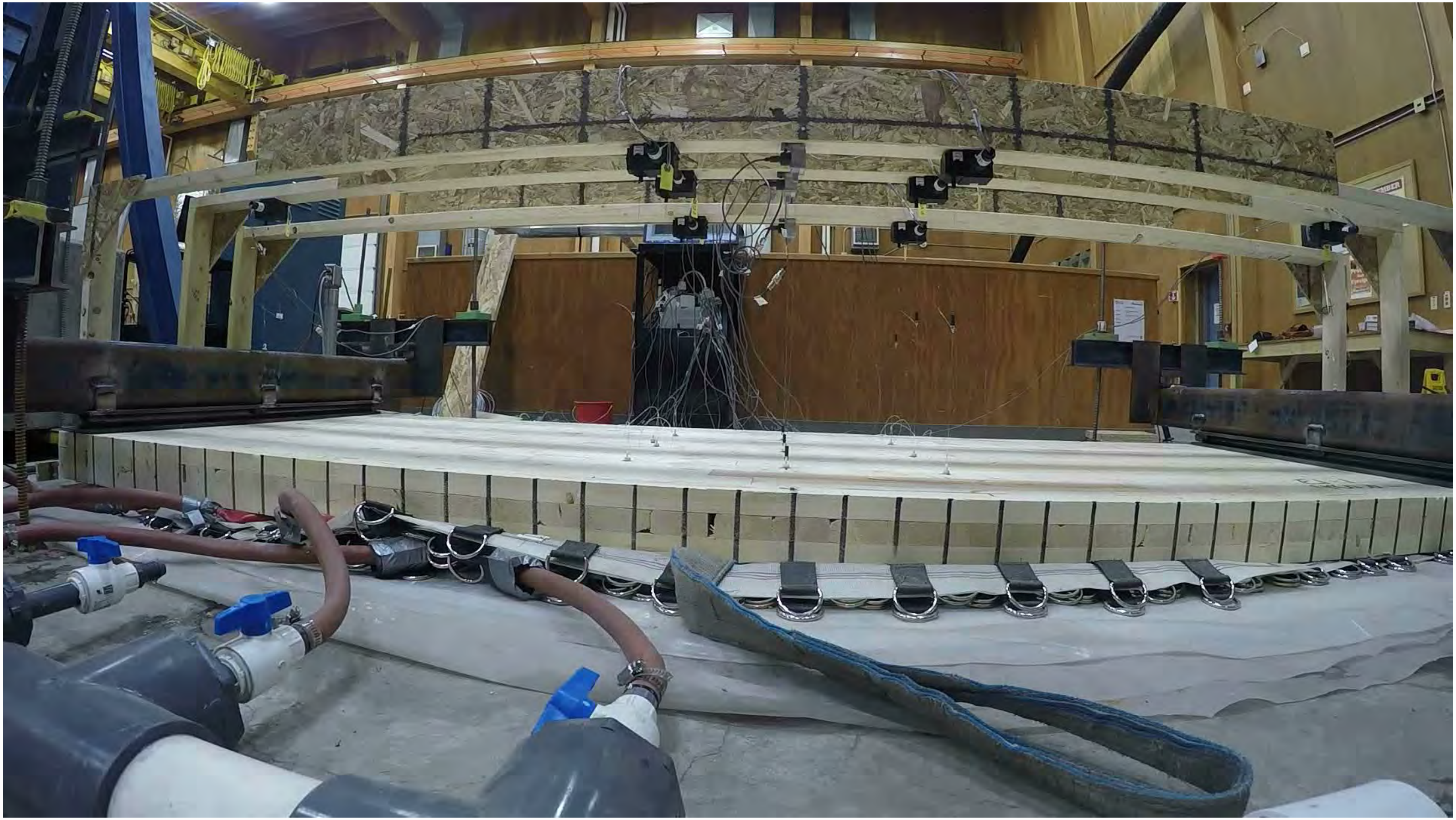




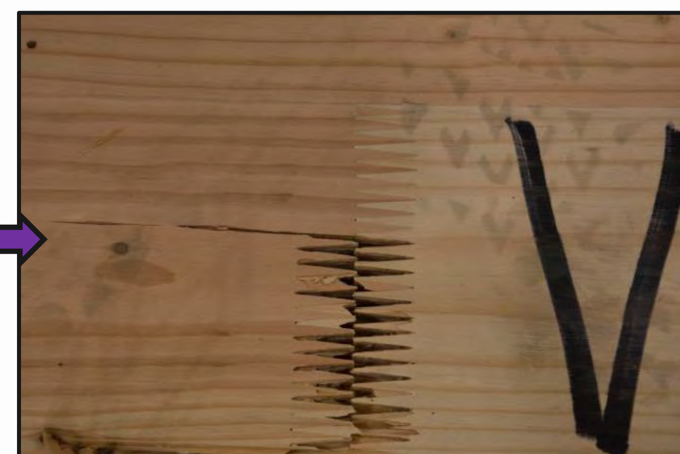


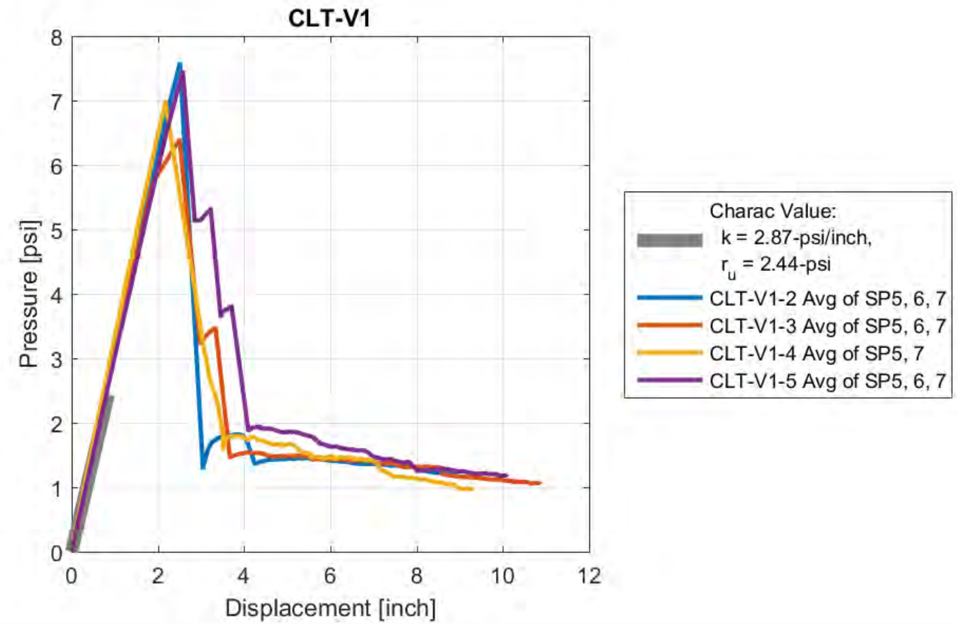
Quasi-Static Testing

Video

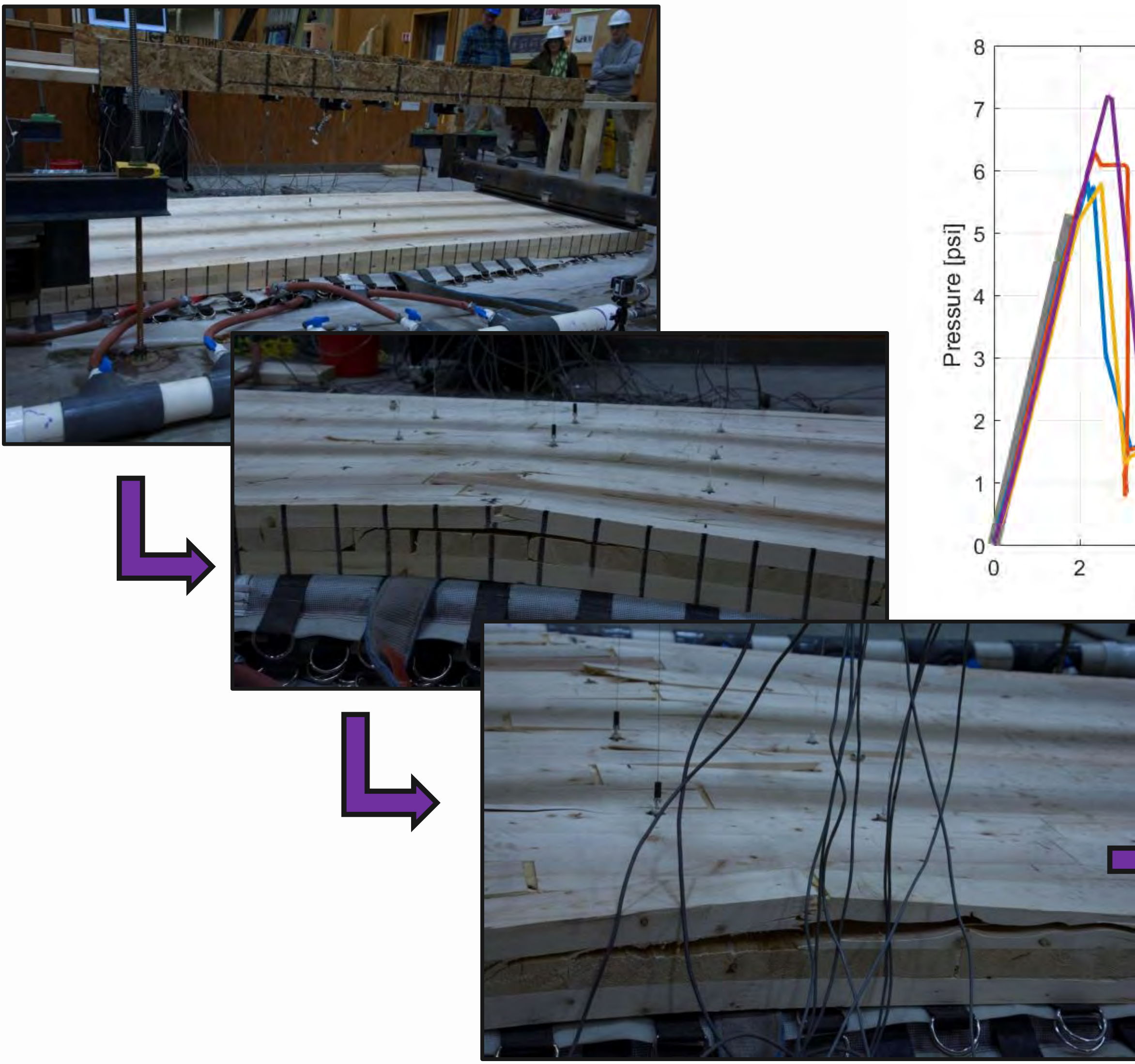


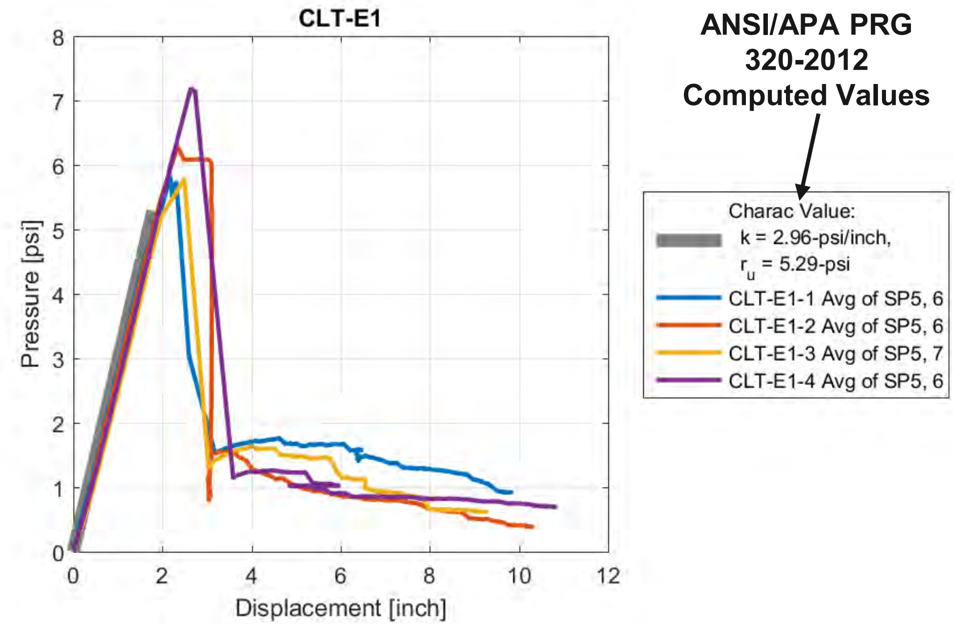
Quasi-Static Testing
3-Ply Grade V1



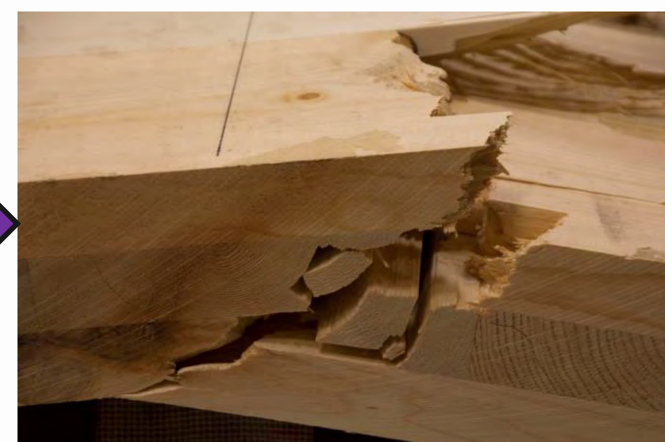


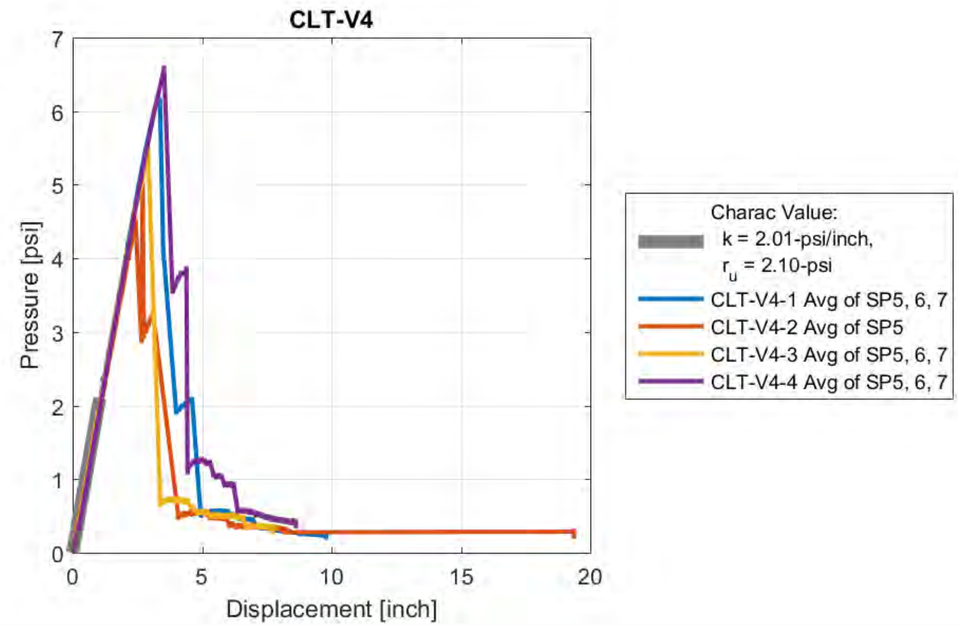
Quasi-Static Testing
3-Ply Grade E1





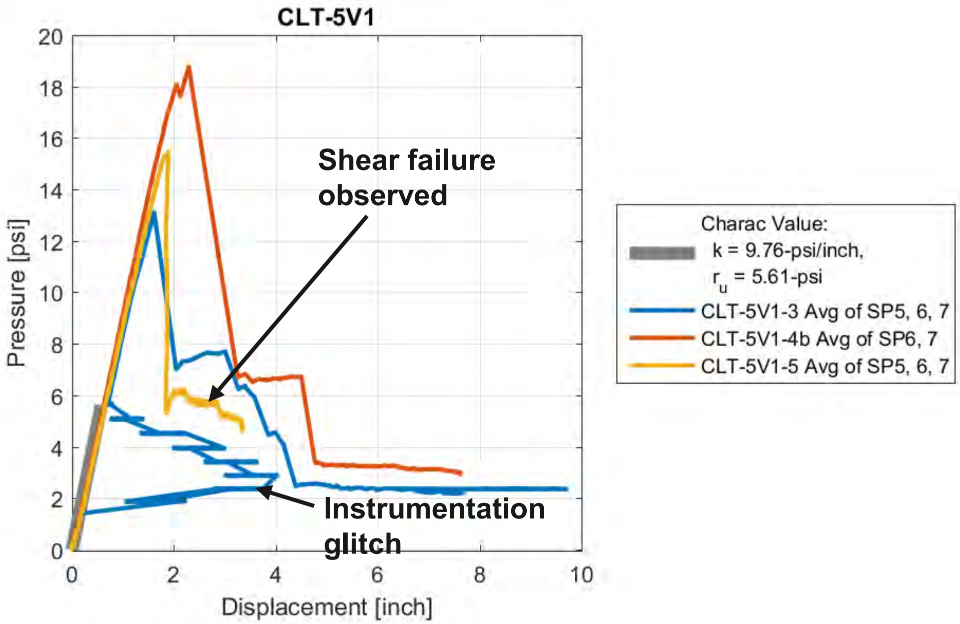
Quasi-Static Testing
3-Ply Grade V4

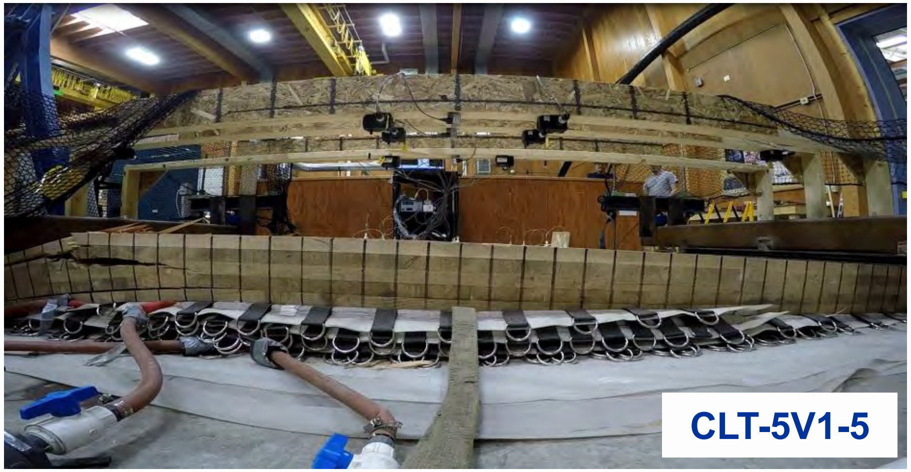




Quasi-Static Testing

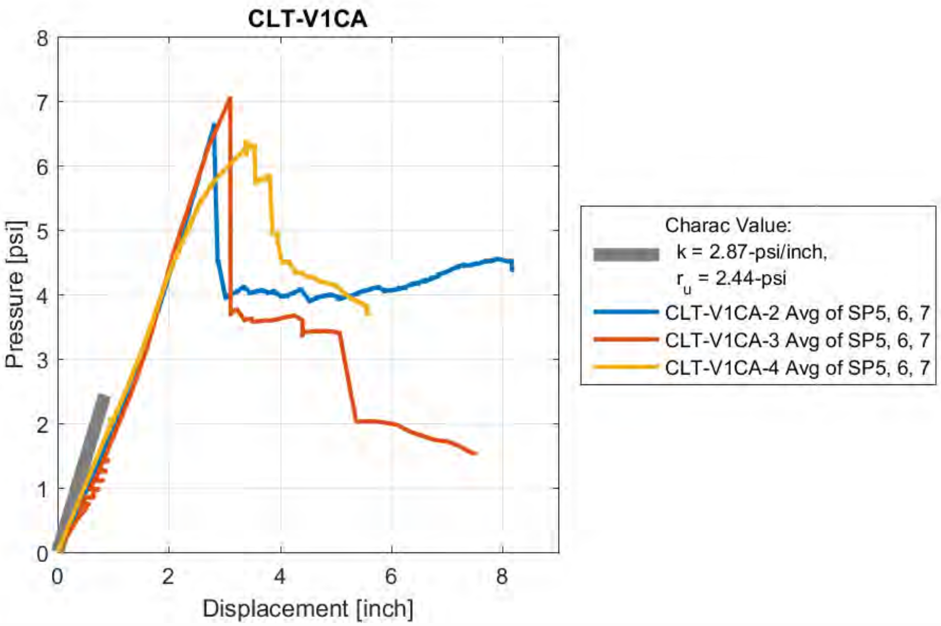
5-Ply Grade V1

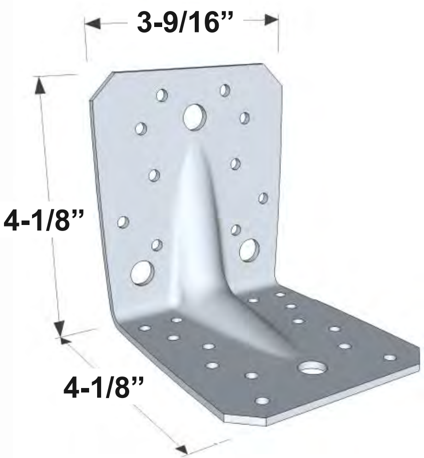




Quasi-Static Testing

Grade V1 Panel w/ ABR105 Bracket



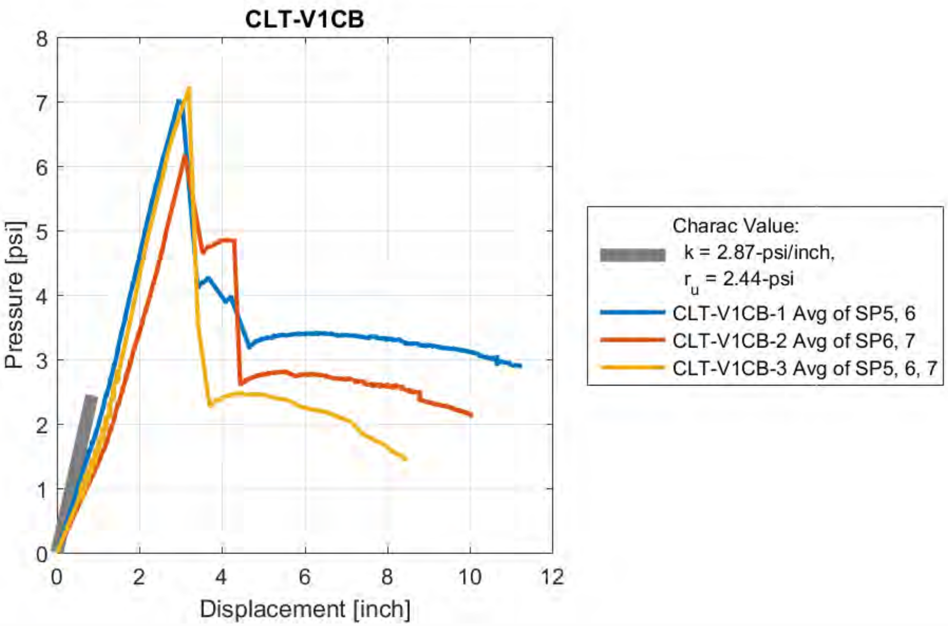


V1CA-2
4 Brackets

V1CA-3
3 Brackets

V1CA-4
2 Brackets

Quasi-Static Testing
Grade V1 Panel w/ L4x4x1/4 Bracket



V1CB-1
4 Brackets

V1CB-2
3 Brackets

V1CB-3
2 Brackets

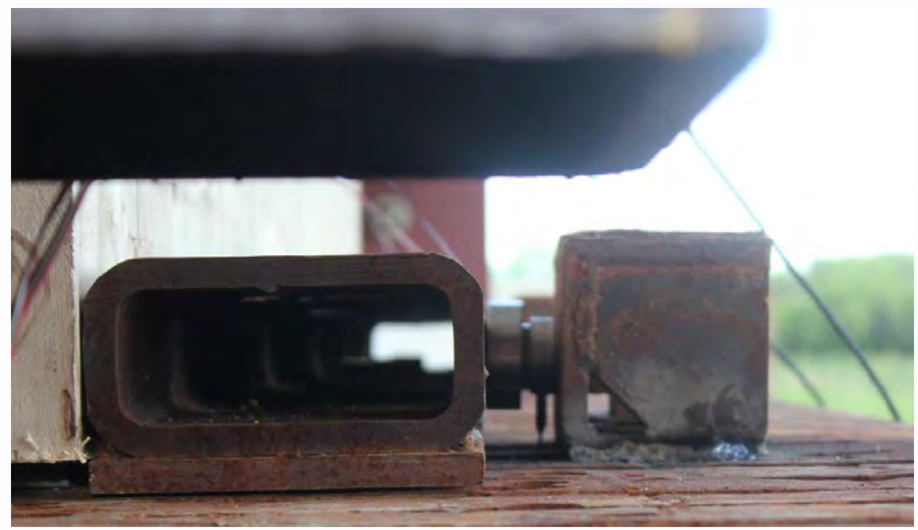
Quasi-Static Testing
Panel Testing Comments

- **Typical failure**
  - *Bending near the midspan.*
  - *Failures typically centered around knots, sloped grain, and finger joints.*
- **Essentially elastic to failure.**
- **Relatively consistent peak strength, residual strength, & elastic stiffness, particularly for 3-ply panels.**
- **Long unsupported distances at boundary can facilitate shear failure in panel.**

PDC Shock Tube Testing

Overview

- (3) 3-, 5-, and 7-ply Grade E1 specimens.
- Elastic material response.
- Specimens hit multiple times.



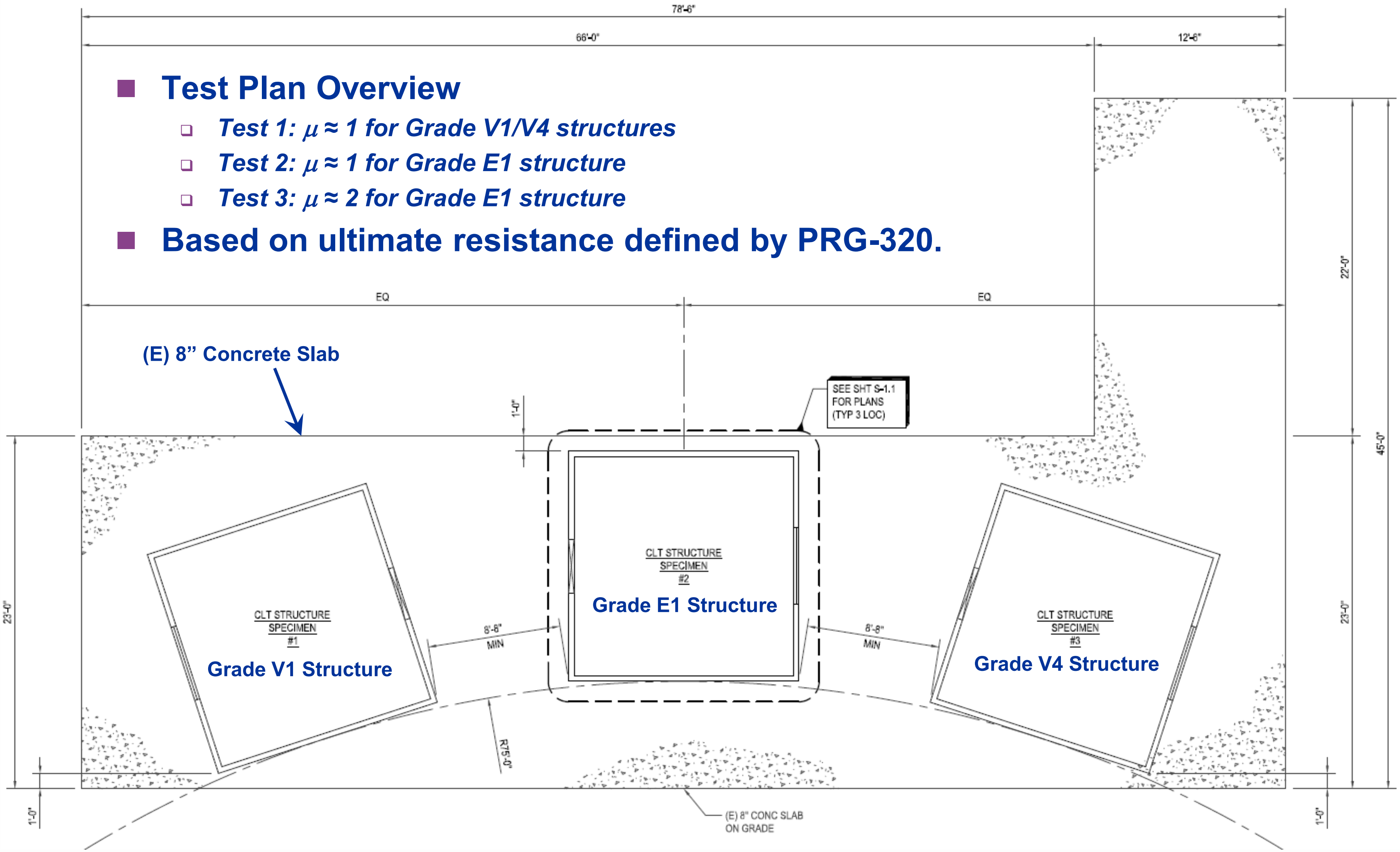
Shock Tube Testing
Displacement Comparison

Test No.	Sample No.	P_r [psi]	I_r [psi]	Δ_{test} [in]	Δ_{SDOF} [in]	% Diff.
1-1	3P-1	2.9	20	1.0	1.16	**5.5%**
1-2	3P-1	2.6	19	1.0	1.10	**10.0%**
1-3	3P-1	3.9	27	1.5	1.57	**4.5%**
1-4	3P-1	4.6	32	1.7	1.86	**9.4%**
1-5	3P-1	5.4	38	2.1	2.20	**4.8%**
1-6	3P-1	6.3	43	2.3	2.50	**8.7%**
1-7	3P-2	5.7	42	2.1	2.43	**15.7%**
1-8	3P-2	6.3	45	2.5	2.61	**4.4%**
1-9	3P-2	7.1	51	2.8	2.95	**5.4%**
2-1	3P-3	6.0	36	2.2	2.12	**-3.6%**
2-2	3P-3	7.6	47	2.9	2.76	**-4.8%**
2-3	5P-1	7.6	49	1.5	1.14	**-24.0%**
2-4	5P-1	8.6	56	1.7	1.30	**-23.5%**
2-5	5P-1	9.6	67	2.1	1.54	**-26.7%**
2-6	5P-1	10.8	77	2.4	1.76	**-26.7%**
2-7	5P-1	12.4	91	2.6	2.06	**-20.8%**
2-10	5P-3	14.4	112	2.4	2.51	**4.6%**

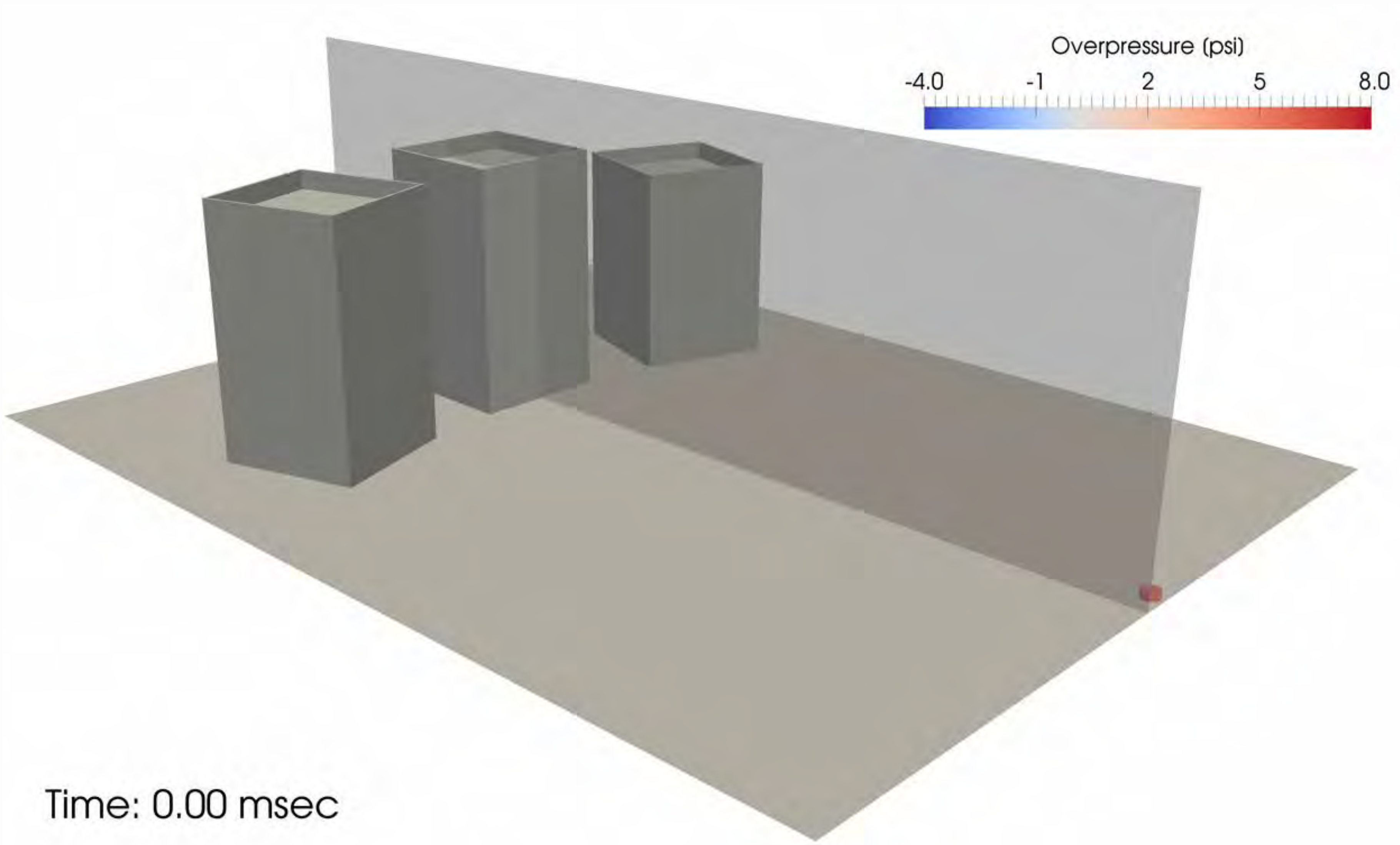
- **Minimal strength degradation when hit multiple times (provided no panel rupture).**
- **Shock tube test reports indicate a dynamic increase factor of between 1.2 and 1.35 for CLT.**

Live Blast Test
Site Layout

- **Test Plan Overview**
  - *Test 1: $\mu \approx 1$ for Grade V1/V4 structures*
  - *Test 2: $\mu \approx 1$ for Grade E1 structure*
  - *Test 3: $\mu \approx 2$ for Grade E1 structure*
- **Based on ultimate resistance defined by PRG-320.**



Live Blast Test
Site Layout (Cont'd)

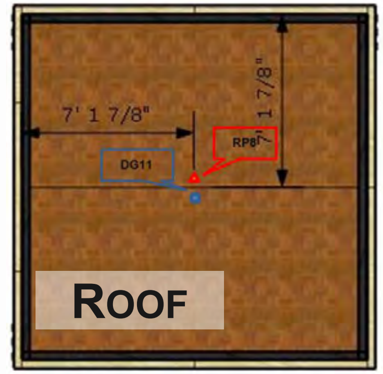


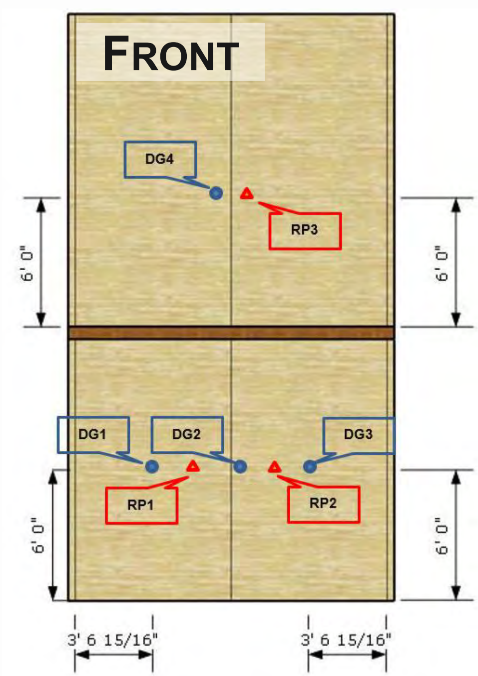
Live Blast Test
Construction

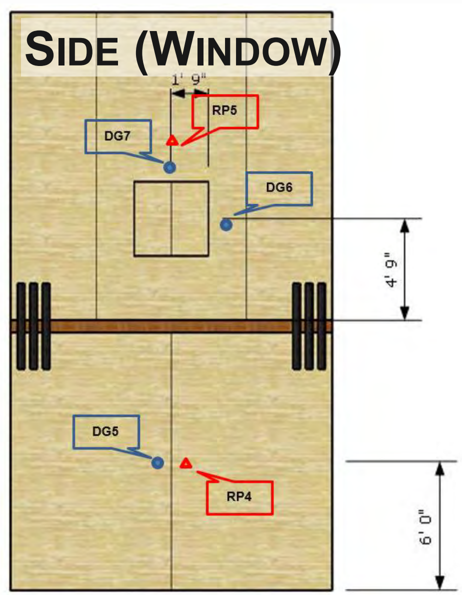
- **Total Construction Time = less than 8 days**
  - ***Including delays.***
  - ***2 days to set foundation anchorage.***

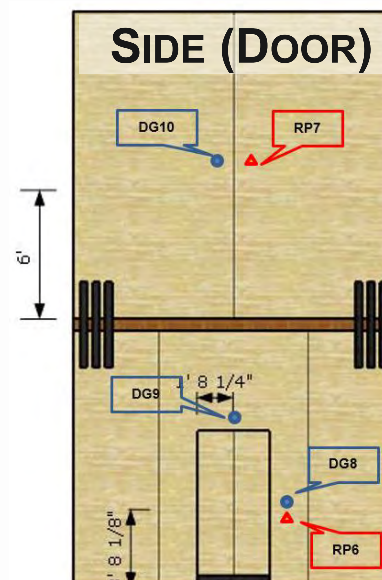
Live Blast Test
Instrumentation – Gauges

- 22 Gauges / BLDG (66 Total)
  - *8 pressure gauges per building (24 total)*
  - *11 deflection gauges per building (33 total)*
  - *1 interior incident pressure gauge (3 total)*
  - *2 free field incident pressure gauges*



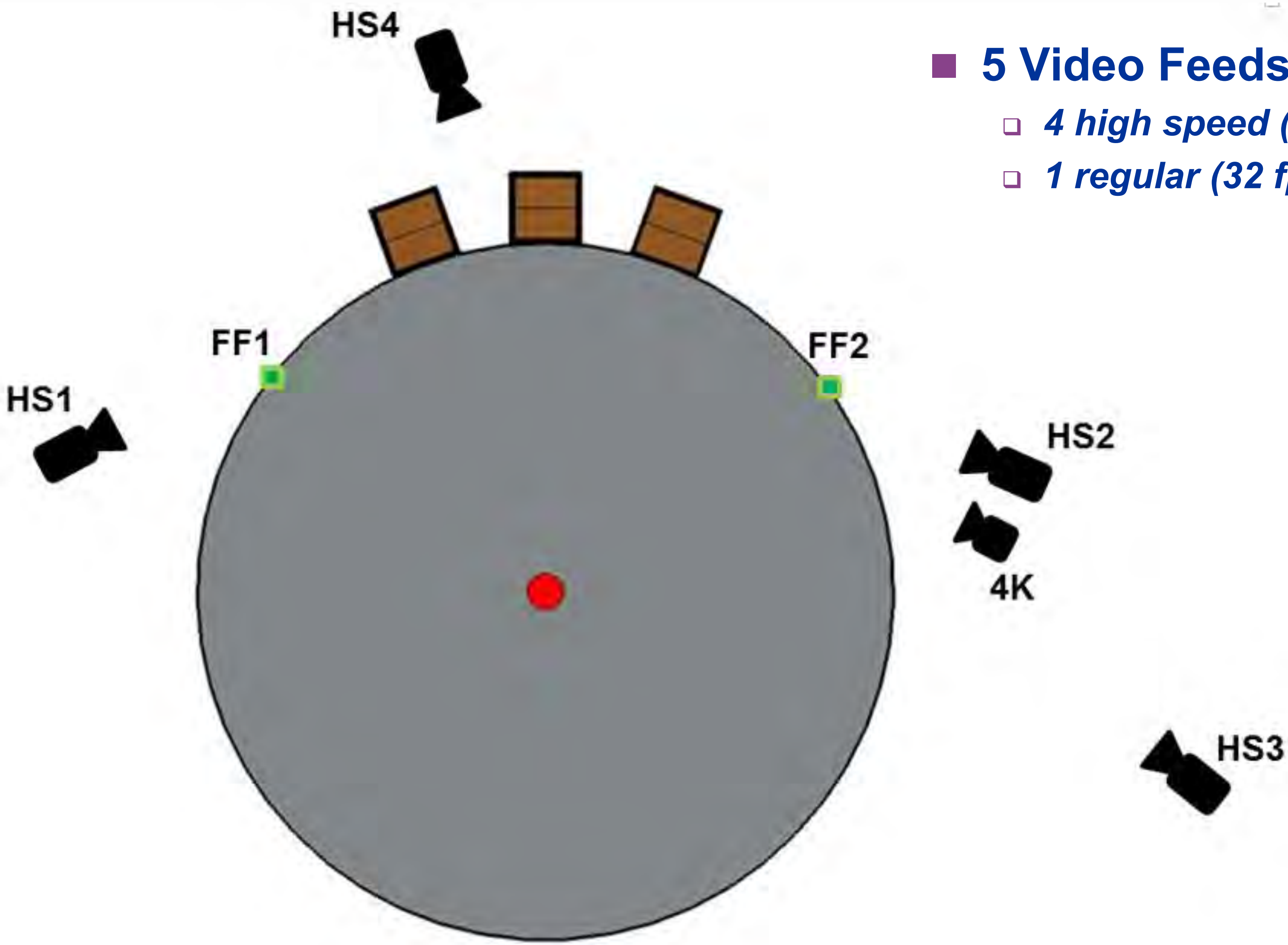






Live Blast Test
Instrumentation – Video

- **5 Video Feeds**
  - *4 high speed (3,270+ fps)*
  - *1 regular (32 fps)*



Participating Manufacturers
CLT Panel

TABLE A1.
ALLOWABLE DESIGN PROPERTIES[a,b,c] FOR PRG 320 CLT (for use in the U.S.)

	Major Strength Direction						Minor Strength Direction					
CLT Grades	$F_{b,0}$ (psi)	E_0 (10^6 psi)	$F_{t,0}$ (psi)	$F_{c,0}$ (psi)	$F_{v,0}$ (psi)	$F_{s,0}$ (psi)	$F_{b,90}$ (psi)	E_{90} (10^6 psi)	$F_{t,90}$ (psi)	$F_{c,90}$ (psi)	$F_{v,90}$ (psi)	$F_{s,90}$ (psi)
E1	1,950	1.7	1,375	1,800	135	45	500	1.2	250	650	135	45
E2	1,650	1.5	1,020	1,700	180	60	525	1.4	325	775	180	60
E3	1,200	1.2	600	1,400	110	35	350	0.9	150	475	110	35
E4	1,950	1.7	1,375	1,800	175	55	575	1.4	325	825	175	55
V1	900	1.6	575	1,350	180	60	525	1.4	325	775	180	60
V2	875	1.4	450	1,150	135	45	500	1.2	250	650	135	45
V3	975	1.6	550	1,450	175	55	575	1.4	325	825	175	55
V4	775	1.1	350	1,000	135	45	775	1.1	350	1,000	135	45

DR Johnson Lumber Co. Riddle OR

Grade V1
In Production
APA Certified

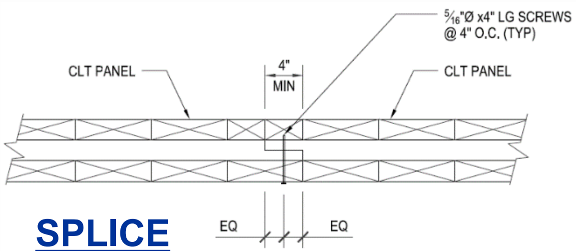
SMARTLAM

Grade V4
In Production
Not Yet APA Certified

NORDIC

Grade E1
In Production
APA Certified

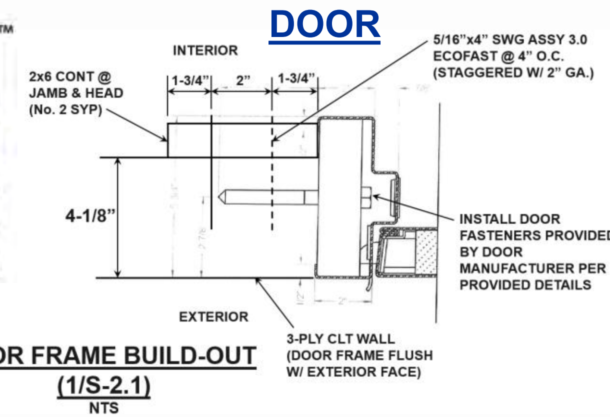
Participating Manufacturers
Connection Hardware



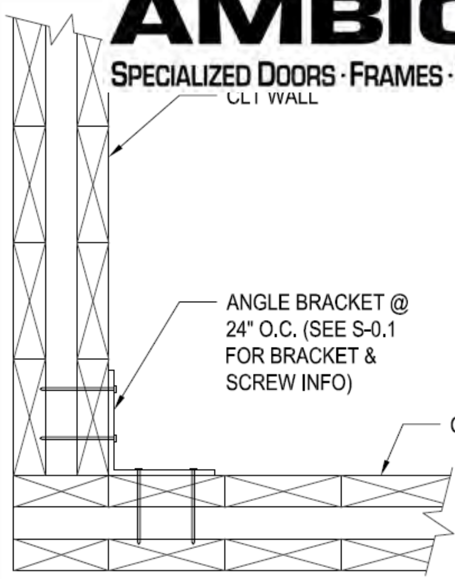
SPLICE

AMBICO
SPECIALIZED DOORS · FRAMES · WINDOWS

DOOR

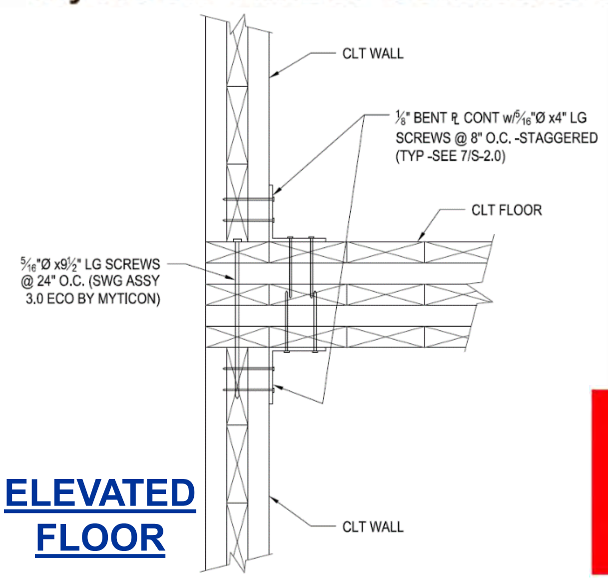


MTC
MyTiCon Timber Connectors



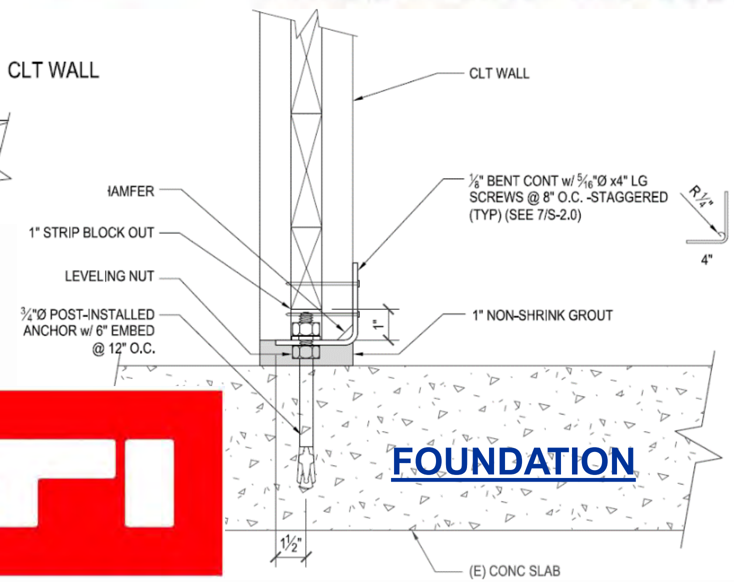
CORNER

AMERICANDIRECT
Innovation at the door



ELEVATED FLOOR

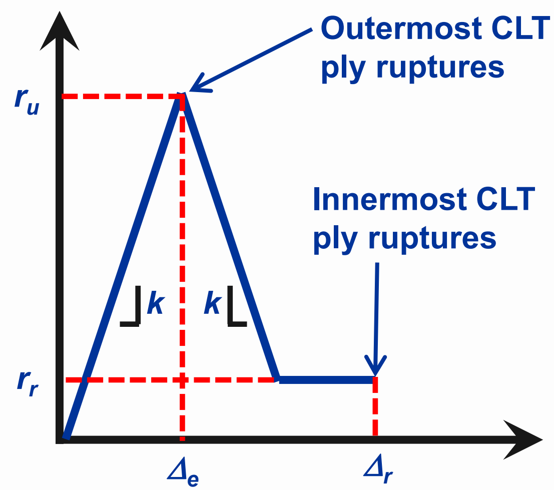
HILTI



FOUNDATION

Design Methodology
Panel

- Ultimate resistance, r_u, and equivalent stiffness, k, derived using "shear analogy" method (Kreuzinger, 1999).
  - *Compared well with quasi-static testing & dynamic testing.*
- Static increase factor (from ANSI/APA PRG 320-2012)
  - *1.3 for out-of-plane flexure.*
  - *2.0 for out-of-plane shear.*
- Dynamic increase factor set equal to C_D, (i.e., 2 for dynamic testing ; 1.6 for quasi-static testing (10 min duration)).
- Test response limit: Ductility, μ = 1 (i.e., elastic response only).
  - *Test data does not seem to support 4 different response limits.*



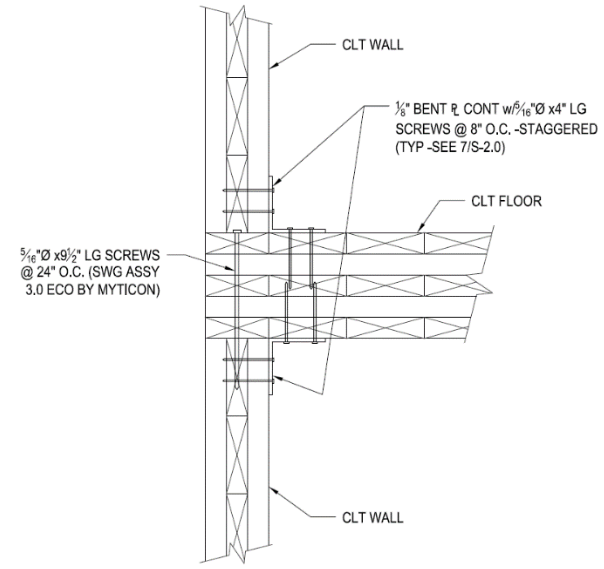
Schematic SDOF Flexural Resistance Function for 3-Ply CLT

Design Methodology
Conventional Construction Example

Wall Type	Sections	Span	Min. Static Material Strength	EWI Standoff Distance	EWII Standoff Distance
Reinforced Concrete	≥ 6”	12’ – 20’	3,000 psi	66	**16**
Reinforced Masonry	8” – 12”	10’ – 14’	1,500 psi	86	**30**
CLT – EIFS	3-ply	10’ – 12’	Grade E1	90	**40**
CLT – EIFS	3-ply	10’ – 12’	Grade V1	180	**75**
CLT – EIFS	3-ply	10’ – 12’	Grade V4	200	**80**
Wood Studs – EIFS	2x4 & 2x6	8’ – 10’	875 psi	207	**86**
Steel Studs – EIFS	600S162-43; 600S162-54; 600S162-68	8’ – 12’	50,000 psi	361	**151**

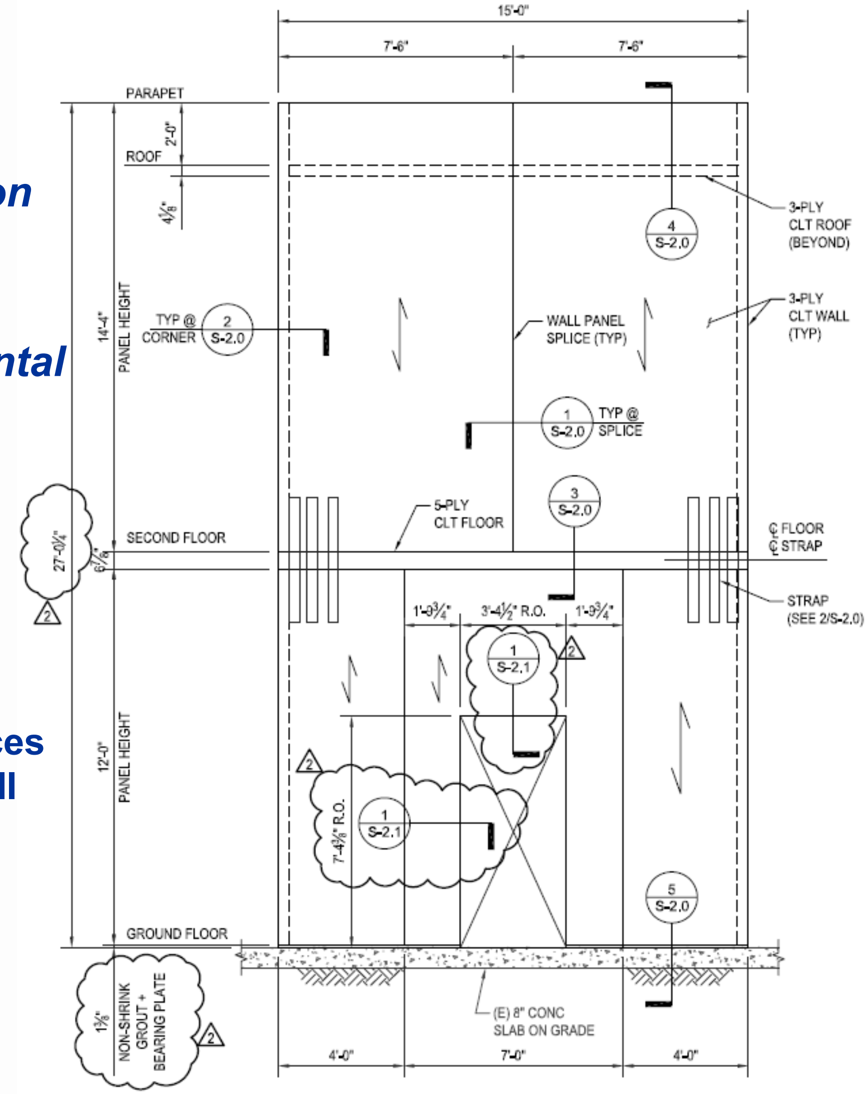
Design Methodology
Connection

- Designed to have ductile limit state control. No strength reduction factor used for these limit states.
  - *Flexural yielding of steel bent plate.*
  - *Shear yielding of self-drilling screws.*
- Promote failure in panel, rather than sudden loss of capacity due to connection rupture.
- Continuous rather than discrete support where outer ply direction is perpendicular to connection.



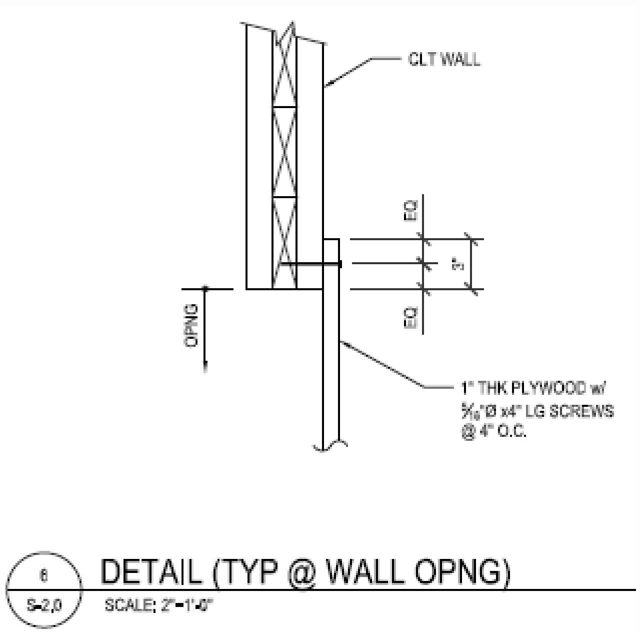
Design Methodology
System

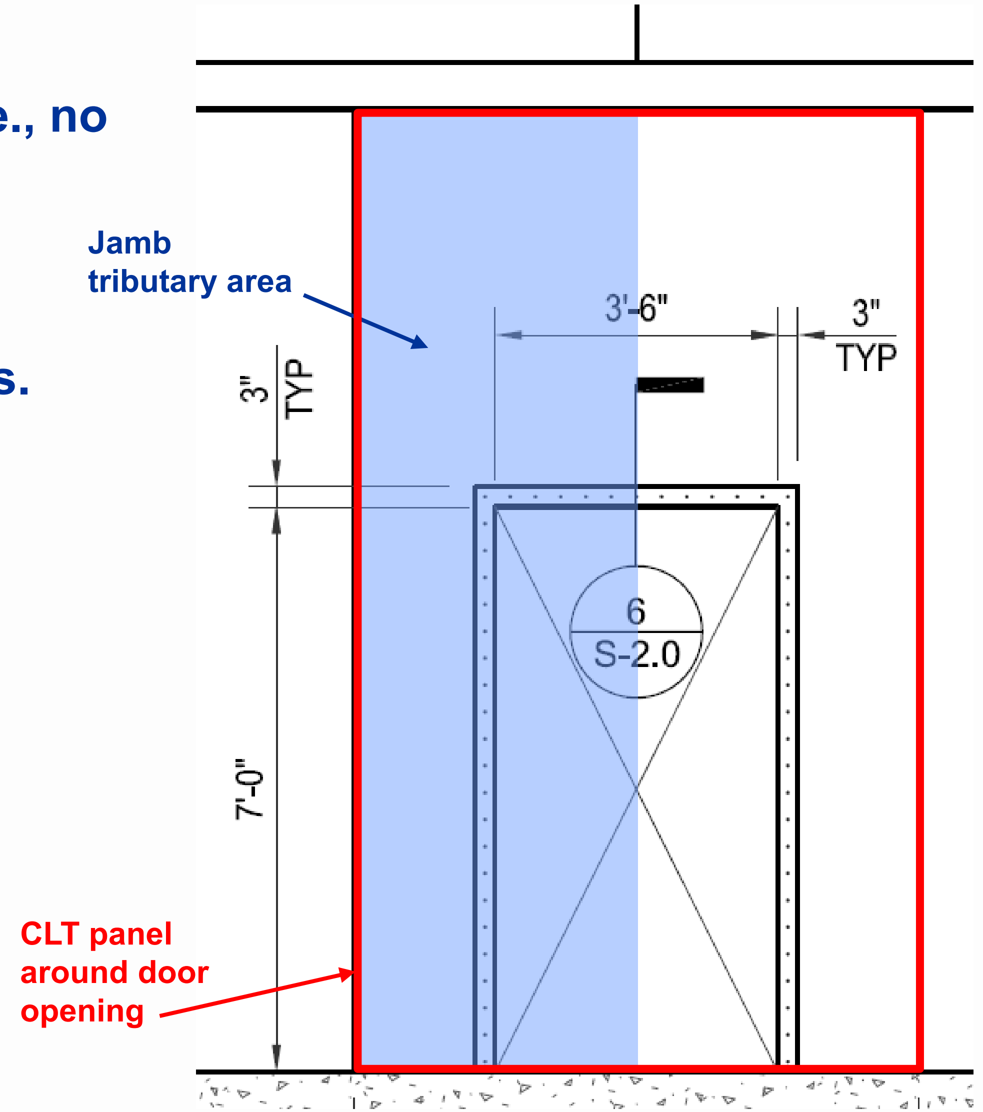
- **Diaphragm**
  - *Input force consists of dynamic reaction from wall panels.*
  - *Assume diaphragm pieces act independently in flexure unless horizontal shear for composite section can be resisted by connection at interface.*
  - *Chord displacement ductility = 1.*
- **Shear Wall**
  - *Collector connection*
    - Based on diaphragm dynamic shear forces + rebound dynamic reaction of shear wall acting concurrently.
  - *Cantilever SDOF*
    - Tributary mass assumed uniform.
    - Use to check overturning anchorage requirement.



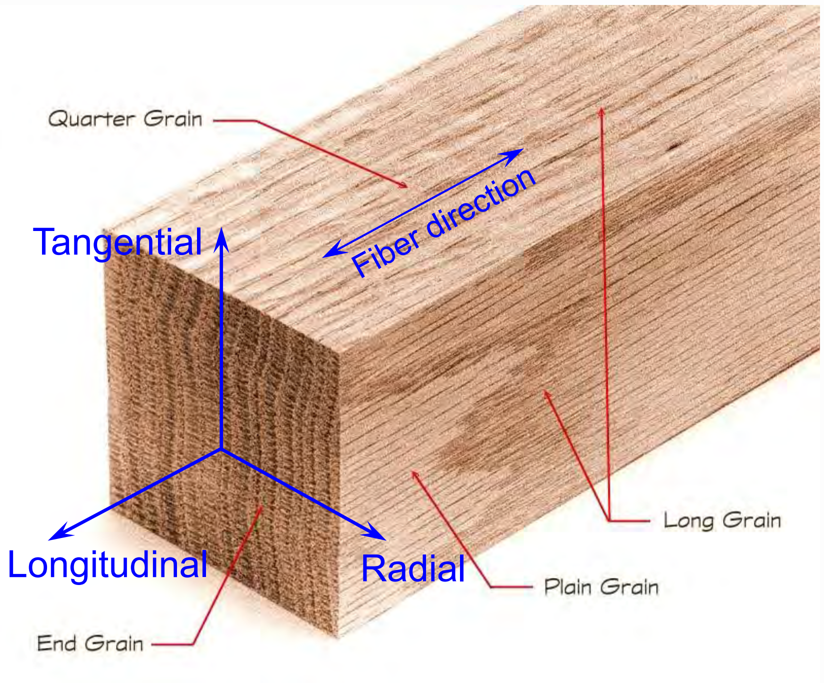
Design Methodology
Door & Window Openings

- Solid panel around openings (i.e., no seams).
- Minor axis panel bending for head/sill.
- Tributary area for opening jambs.





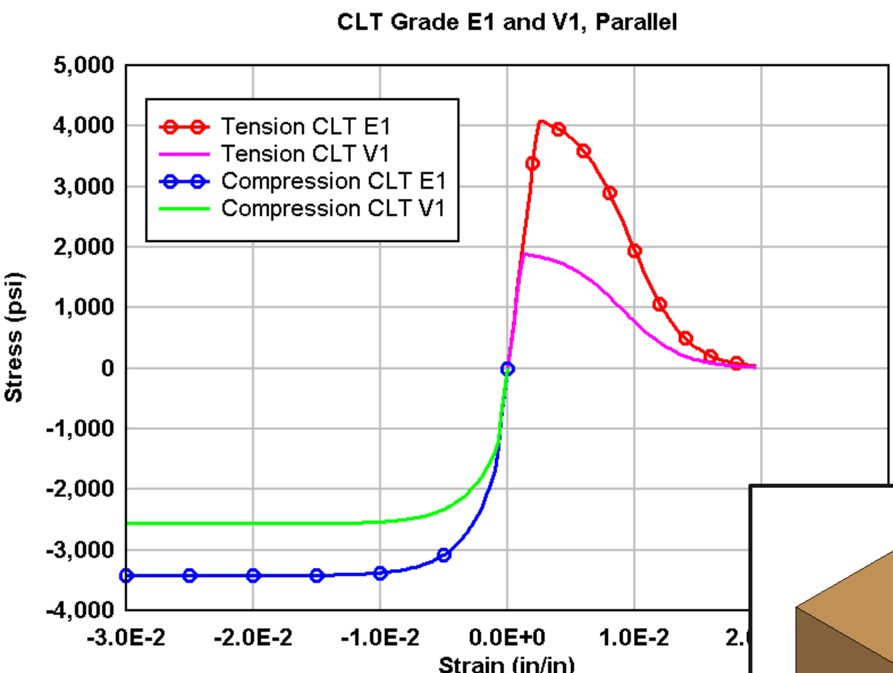
FE Modeling
Introduction – Wood Materials



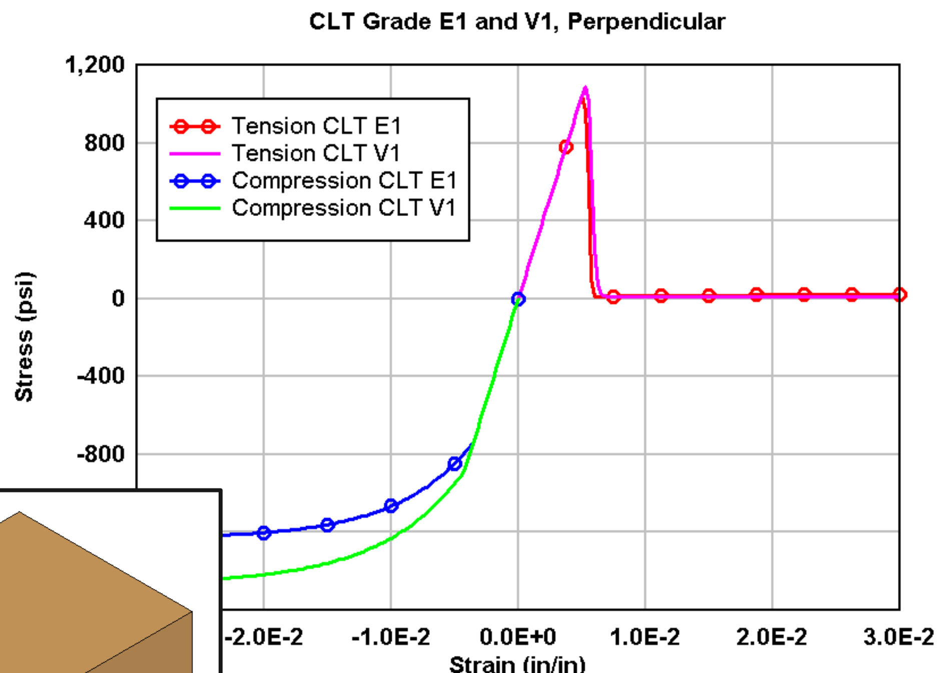
- Orthotropic
  - *Longitudinal*
  - *Tangential*
  - *Redial*
- Transversely Isotropic
  - *Parallel (longitudinal)*
  - *Perpendicular (tangential & radial)*
- Key factors affecting stress strain relationship
  - *Loading direction*
  - *Moisture*
  - *Temperature*

FE Modeling
Single Element Test

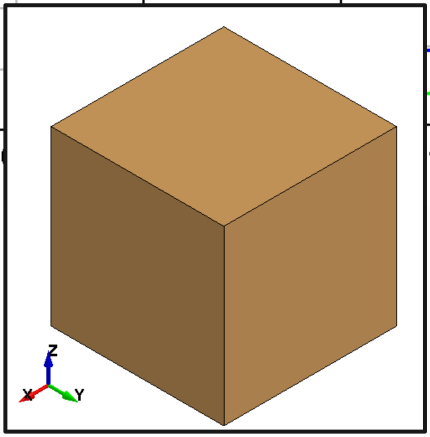
- *Material: CLT Grade E1 and Grade V1.*
- *Strength values from single element test agree with Inputs.*



Parallel

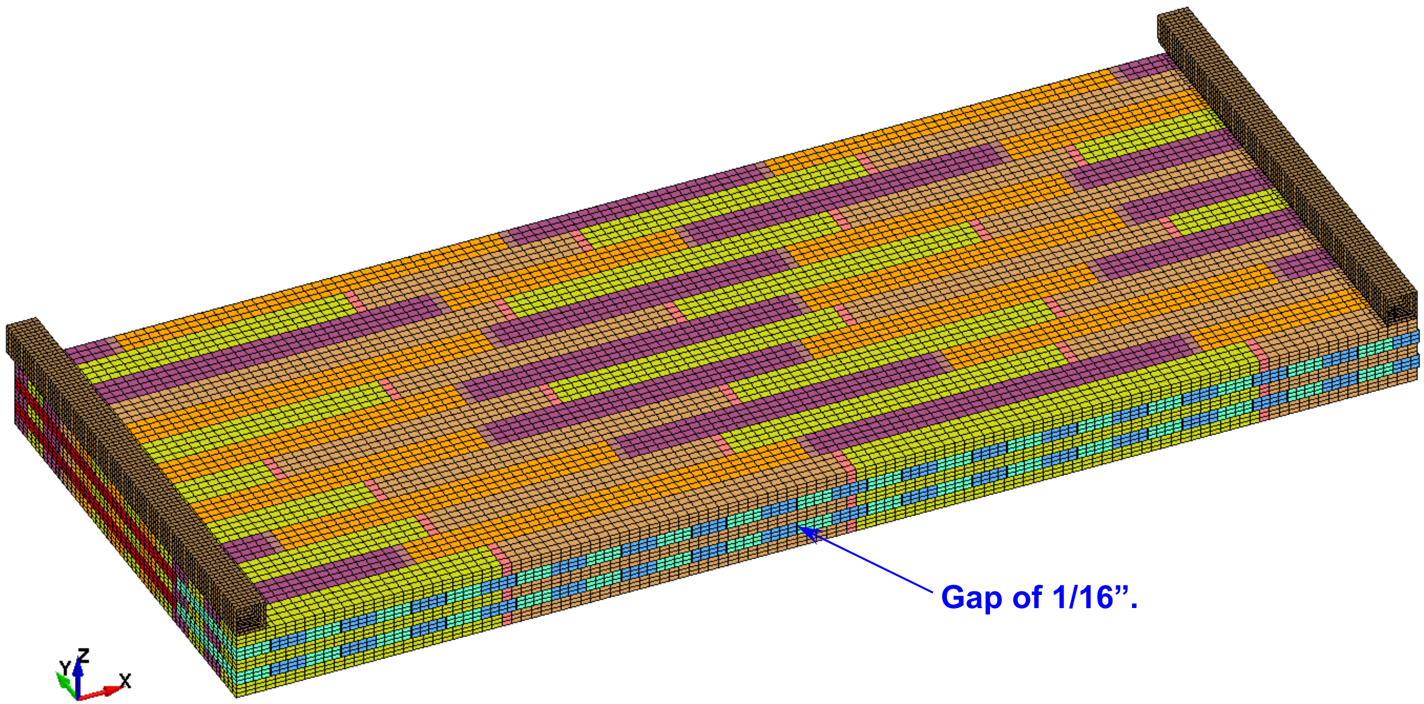


Perpendicular



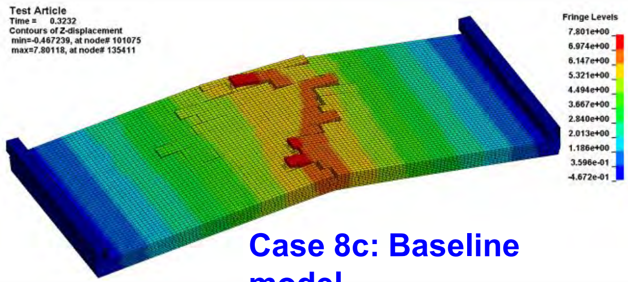
2”x2”x2” solid element

FE Modeling
Quasi-Static Test

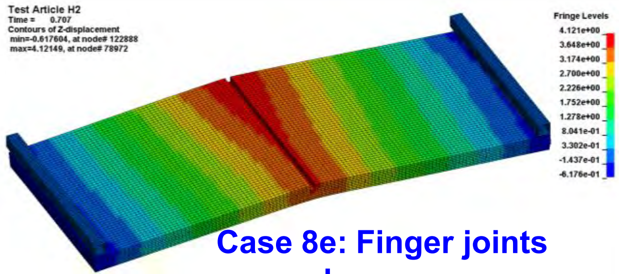


- Model was built by lumber boards, each board being 1.375" thick by 3.5" wide and length is varying with a maximum of 36" in this model.
- Glue is applied on between layers (in-plane) only
- Vertical interface between boards are modeled as contact interfaces.
- Finger joints are modeled and arrange "randomly" as contact interfaces.
- A gap of 1/16" between boards in perpendicular layers is modeled.

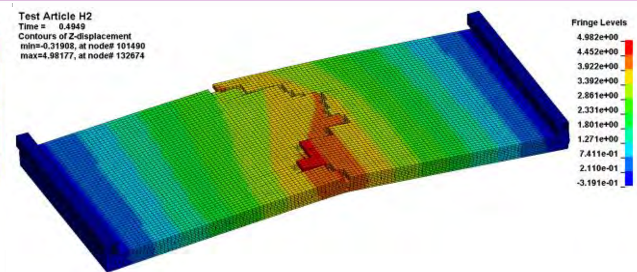
FE Modeling
3-ply Panel Results



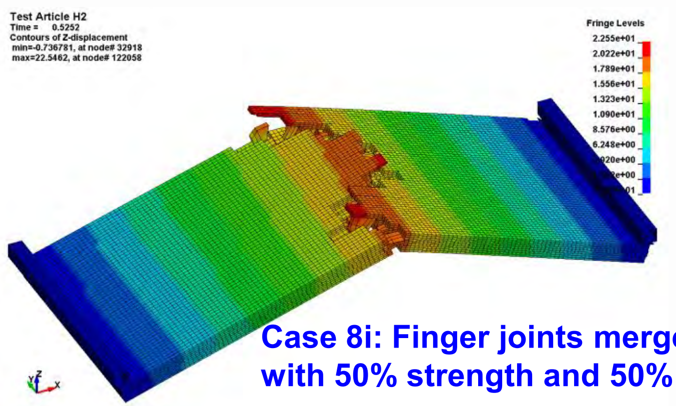
Case 8c: Baseline model.



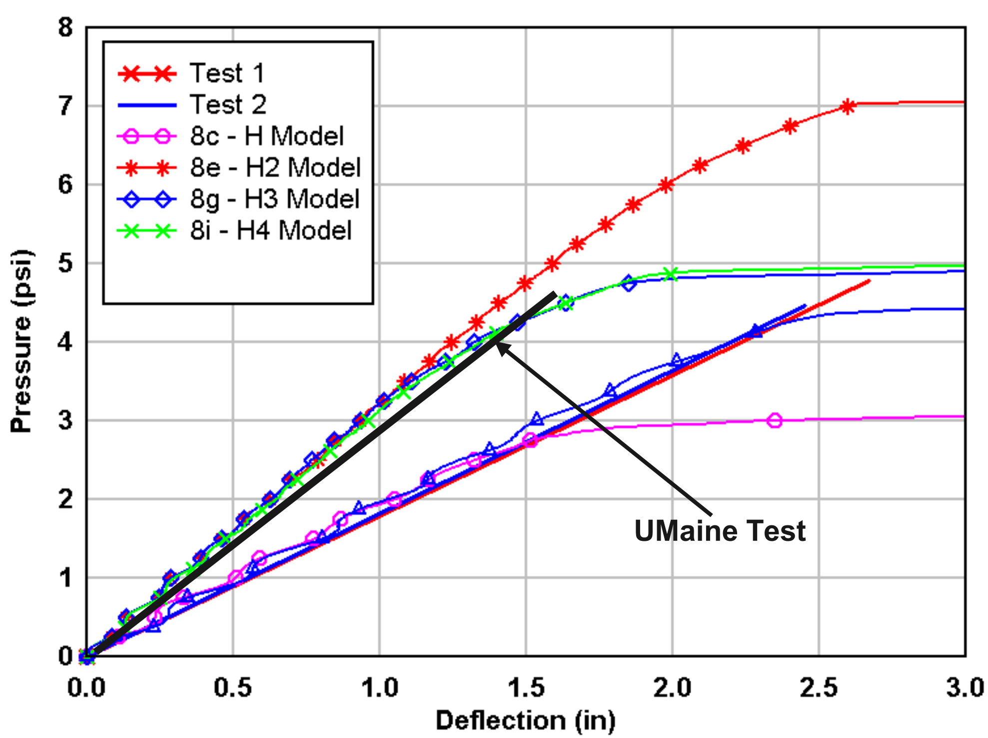
Case 8e: Finger joints merged.



Case 8g: Finger joints merged but with 50% strength.

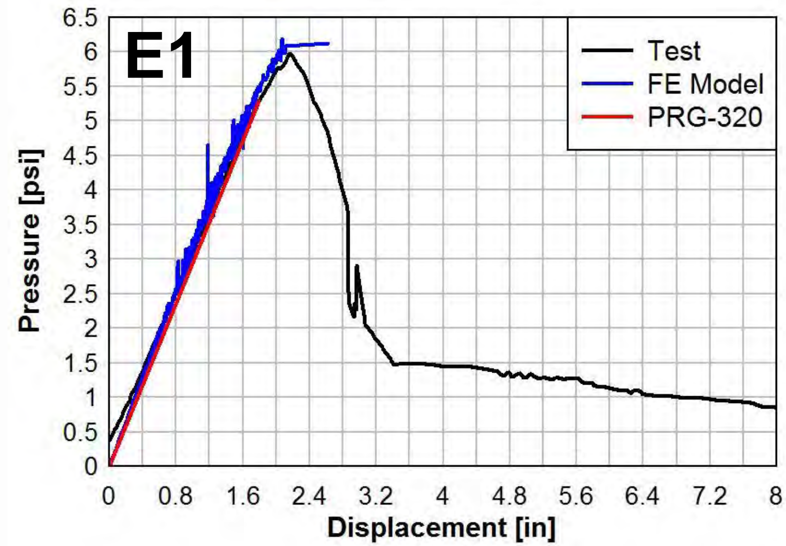


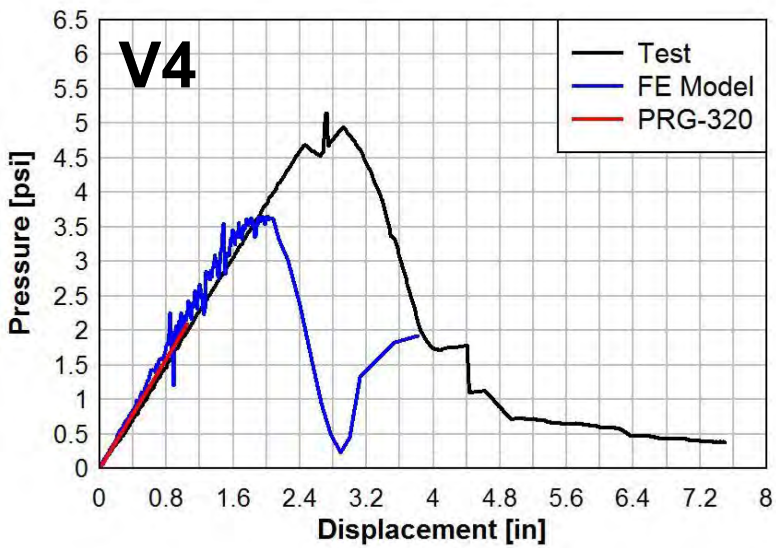
Case 8i: Finger joints merged but with 50% strength and 50% stiffness.

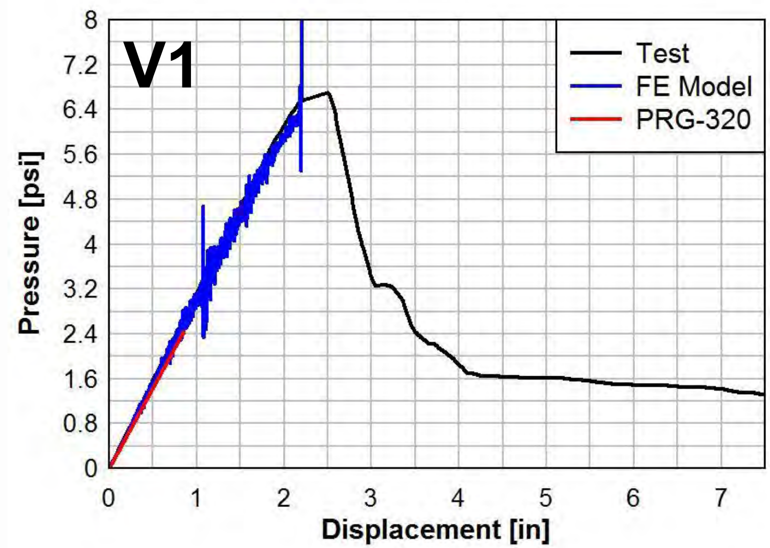


FE Modeling
3-ply Panel Results (Cont'd)

Grade	$r_{PRG\text{-}320}$ [psi]	r_{TEST} [psi]	SIF
E1	5.29	6.00	1.13
V1	2.44	6.67	2.73
V4	2.10	5.13	2.44





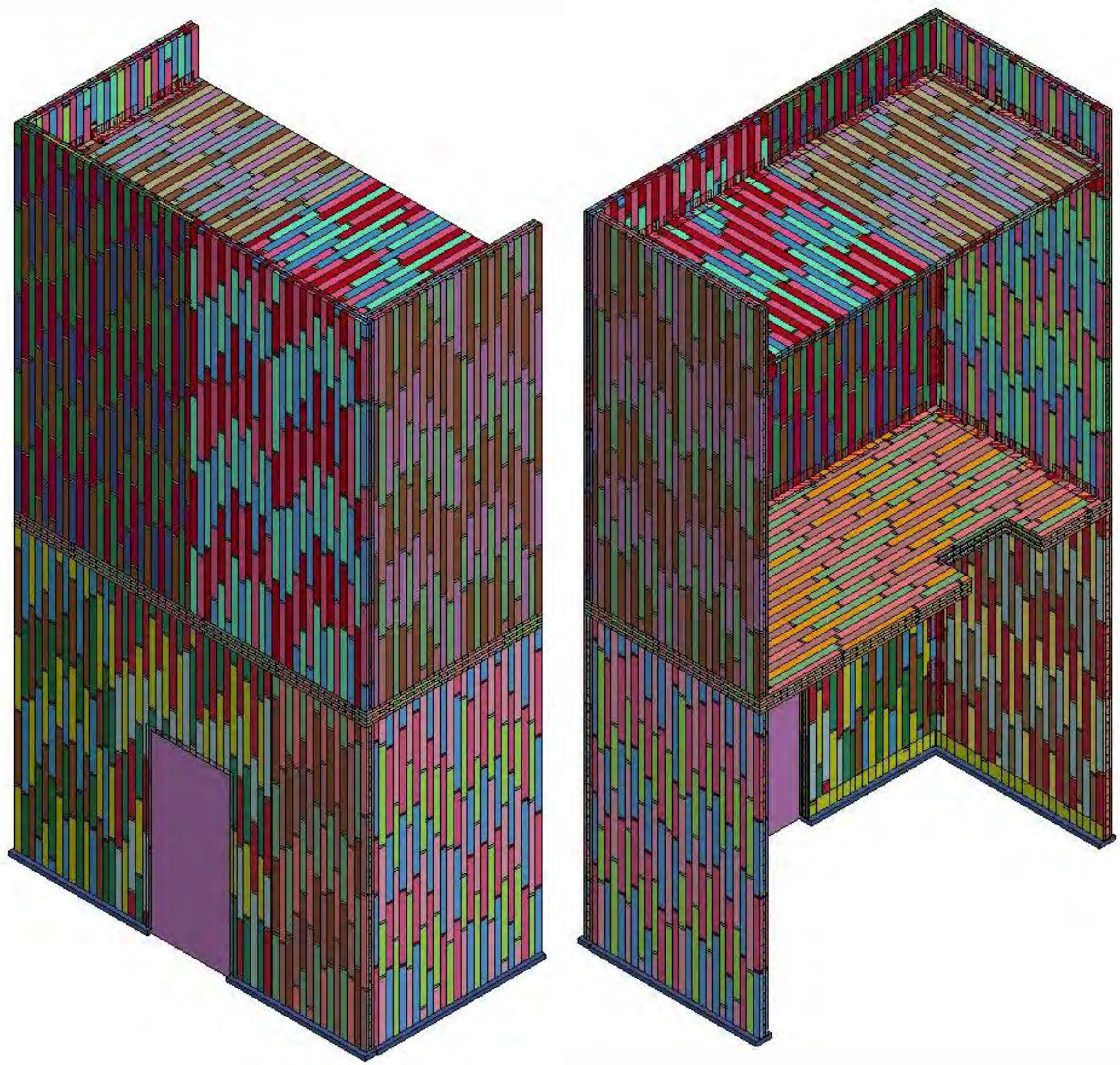


FE Modeling
Observations from Quasi-Static Test Modeling

- **Important to model as much detail of CLT panel as possible:**
  - ***Finger joints in parallel boards.***
  - ***Finger joints with distinct properties.***
  - ***Weaker material in perpendicular boards.***
  - ***Gap between perpendicular boards.***
  - ***Size of boards and spacing of finger joints.***
- **Similar failure model can be predicted when the finger joints are "randomly" arranged in the model with reference to test panel picture.**
- **Post-peak response still needs work.**

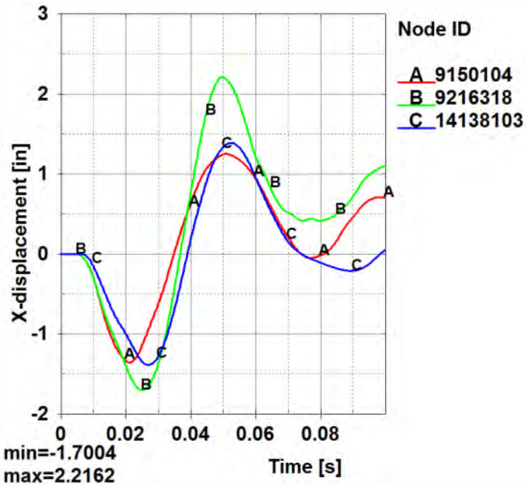
FE Modeling
Test Structure FE Model

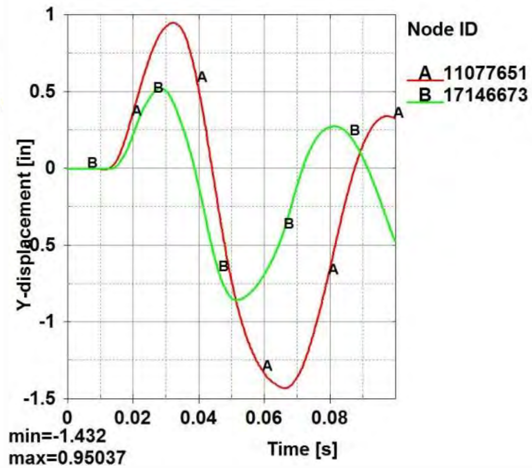
- **Half-symmetry employed.**
  - *Approximation to limit run time.*
- **Two Cases**
  - *Shot 2*
  - *Shot 3*
- **Not explicitly modeled.**
  - *Self-drilling screws at angle connections or laps.*
  - *Overturning straps.*
  - *Door.*



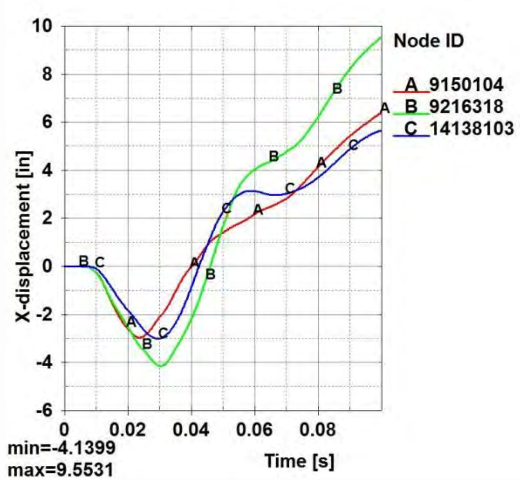
FE Modeling
Peak Displacement

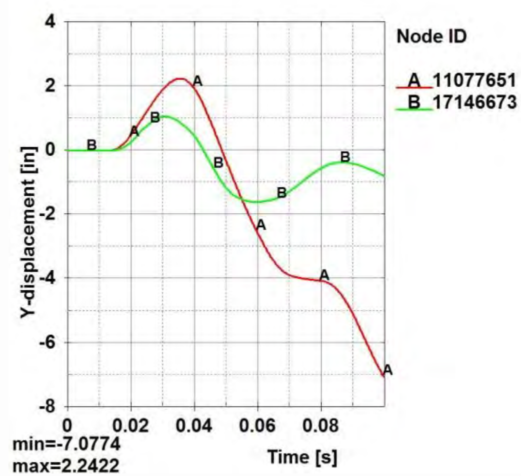
S H O T 2

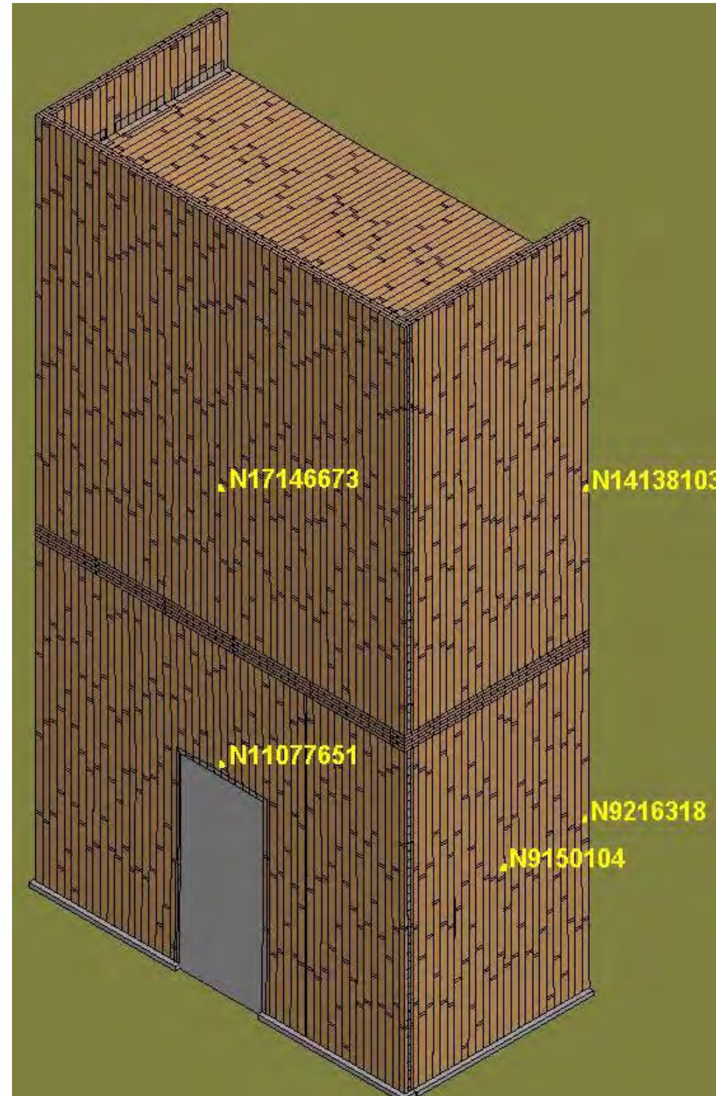




S H O T 3







Peak Displacement Summary

CLT Grade	Shot	Δ_{SDOF} (PRG-320) [in]	Δ_{SDOF} (UMaine SIF) [in]	Δ_{FE} [in]
E1	1	1.85 (R)	1.85 (R)	N/A
	2	2.64 (R)	2.64 (R)	2.22 (R)
	3	6.80 (I)	5.55 (I)	10+ (R)
V1	1	1.25 (R)	1.86 (R)	N/A
	2	3.09 (I)	2.68 (R)	N/A
	3	10+ (I)	4.66 (I)	N/A
V4	1	1.93 (R)	1.67 (R)	N/A
	2	2.53 (I)	2.22 (R)	N/A
	3	10+ (I)	3.93 (I)	N/A

Displacement ductility less than 1.
Displacement ductility between 1 and 2.
Displacement ductility greater than 2.

APPENDIX F

CLT LIVE BLAST TESTING FINAL REPORT

Karagozian & Case

TR-17-17.1

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RESULTS FROM BLAST TESTS OF FULL-SCALE CROSS-LAMINATED TIMBER STRUCTURES

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[2] RTL Networks Inc. – Tyndall AFB, FL
[3] Jacobs Technology SLG – Fort Walton Beach, FL
[4] WoodWorks – Washington, DC

September 27, 2017

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EXECUTIVE SUMMARY

A series of blast tests was performed on three two-story, single-bay cross-laminated timber (CLT) structures at Tyndall Air Force Base. The structures, including anchorage to an existing concrete slab, were constructed in full over a period of eight days. Each structure was constructed using a different grade of CLT (i.e., grade designations V1, E1, and V4) and included window and door openings consistent with an actual building. Self-tapping screws and adhesive anchors were utilized in concert with steel angles to connect the constituent panels of each structure to each other and the foundation.

Three shots were performed to demonstrate the effectiveness of CLT over a spectrum of airblast loads. The first two shots were designed to stress the CLT structures within their respective elastic limits. The third shot was designed to push the structures beyond their elastic limits such that post-peak response could be observed. Reflected pressure and peak displacements were recorded at front, side, and roof faces using a total of sixty-two gages to thoroughly measure the response of the structure.

For the first two tests, peak recorded displacements were consistent with pre-test predictions indicating the efficacy of the design assumptions and methodology in predicting elastic response of CLT to dynamic loads. Furthermore, results from the third test indicated a controlled response in which localized panel rupture was observed but connection integrity and load carrying ability were not compromised for each of the three structures tested.

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LIST OF ACRONYMS

AFB	Air Force Base
AFCEC	Air Force Civil Engineering Center
CLT	Cross-Laminated Timber
DoD	Department of Defense
EOR	Engineer of Record
FPS	Frames Per Second
K&C	Karagozian & Case, Inc.
K-B	Kingery-Bulmash
MDOF	Multi-Degree-of-Freedom
MSR	Machine Stress Rated
SDOF	Single-Degree-of-Freedom
SST	Simpson Strong-Tie
STS	Self-Tapping Screw
UMaine	University of Maine
WW	WoodWorks

CHAPTER 1

INTRODUCTION

As part of a Wood Innovation Grant funded by the U.S. Forest Service and the Softwood Lumber Board, WoodWorks (WW), Karagozian and Case, Inc. (K&C), and the Air Force Civil Engineer Center (AFCEC) partnered via a Cooperative Research and Development Agreement to investigate the capability of cross-laminated timber (CLT) construction to resist airblast loads. Towards this end, three two-story, single-bay CLT structures were constructed at Tyndall Air Force Base (AFB) and subjected to three explosive loadings of increasing magnitude. This report documents the technical approach, test setup, results obtained, and conclusions generated from these three tests.

1.1 BACKGROUND

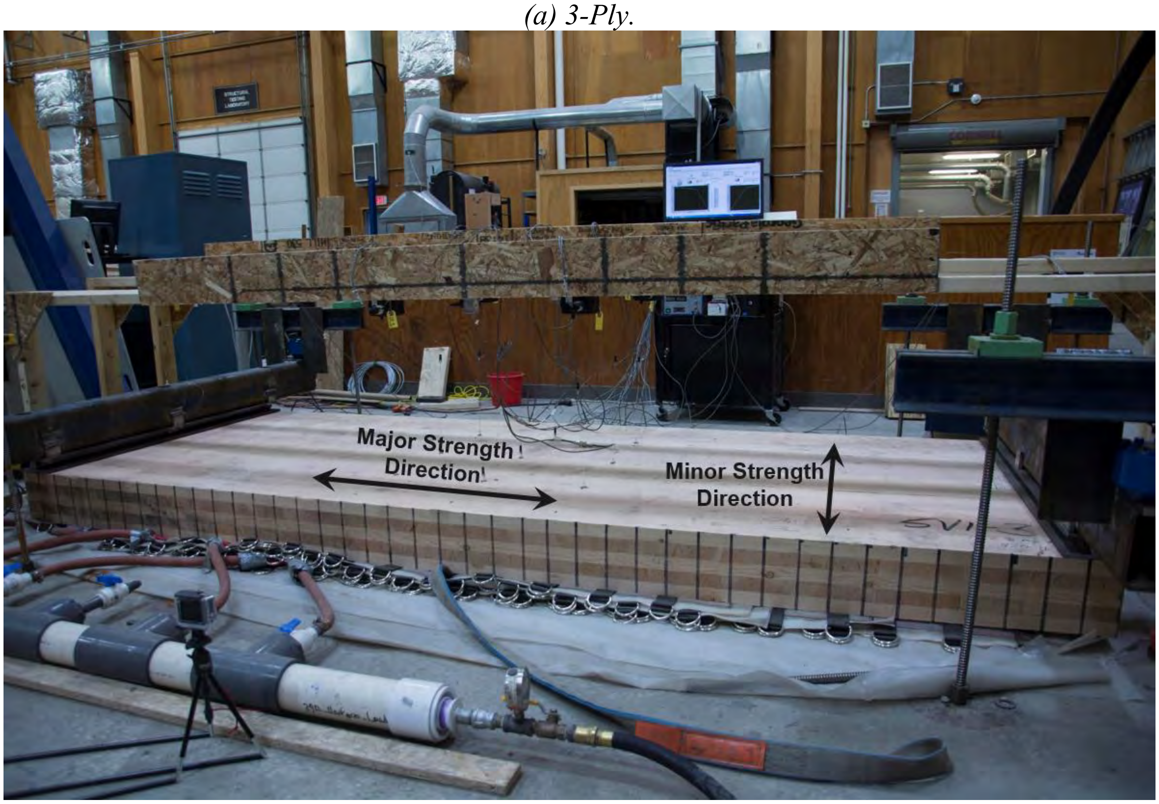
1.1.1 CLT Panel Description

CLT is an engineered wood panel that consists of several layers of dimensional lumber boards stacked in alternating directions that are bonded with structural adhesives and pressed. CLT is typically manufactured in 3-ply, 5-ply, and 7-ply thicknesses. Photographs showing 3-ply and 5- ply CLT panels are included as Figure 1-1.

The alternating orientation of individual panel plies allows CLT to be an intrinsically two-way spanning material. The direction of the outermost plies in a CLT panel is commonly referred to as the panel's "major strength direction", while the direction of those plies offset 90 degrees from the outermost plies is referred to as the "minor strength direction". CLT panel strength and stiffness often differ significantly in the major and minor strength directions.

Two major grade classifications exist for CLT: (1) "E" or engineered (i.e., panel contains machine stress rated (MSR) lumber in its layup) and (2) "V" or visually-graded (i.e., panel utilizes only visually-graded lumber in its layup). Annex A of ANSI/APA PRG 320-2012 [1], defines four "E" and three "V" grade panel layups and includes allowable design properties for each in the major and minor strength directions. Custom grades not listed in Annex A are possible as well. Although not listed in Annex A, Grade V4 CLT (i.e., No. 2 Spruce-Pine-Fir (South) lumber in both the major and minor strength directions) meets the custom CLT grade requirements specified in Section 7.2.1 of PRG 320.

(a) 3-Ply.



(b) 5-Ply.

Figure 1-1. CLT Panels.

1.1.2 UFC 4-010-01 Analysis Requirement

The motivation for the testing described herein derives from the antiterrorism requirements set forth in UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings* [2] for inhabited Department of Defense (DoD) buildings. UFC 4-010-01 contains prescriptive analysis assumptions (i.e., Table 2-3 of UFC 4-010-01) and "conventional construction" standoff distances (i.e., Table B-2 of UFC 4-010-01) for several types of construction that, if adhered to, release the engineer of record (EOR) from having to analyze individual exterior wall or roof structural components for airblast loads.

One type of construction that is not explicitly addressed by UFC 4-010-01 is mass timber construction such as CLT. As such, CLT must be analyzed for airblast loads if an EOR intends to use it as part of the exterior wall or roof structural system in an inhabited DoD building. This requirement, coupled with the lack of test data documenting the response of CLT panels exposed to airblast loads, limits the usage of CLT in inhabited DoD buildings.

1.2 OBJECTIVES

The overarching objective of the testing documented herein was to demonstrate the ability of CLT construction to resist airblast loads generated by high explosives. Specific objectives included:

- To investigate the system-level response of CLT structures to airblast loads generated by high explosives.

- To document the response of CLT panels to airblast loads generated by high explosives and compare this response with those predicted by single-degree-of-freedom (SDOF) analysis methods.

- To document the response of CLT panels around openings (e.g., door, window) to airblast loads generated by high explosives and compare this response with those predicted by SDOF analysis methods.

- To document the responses of various connection configurations commonly used in CLT construction to airblast loads generated by high explosives.

1.3 REPORT OUTLINE

The remainder of this report is divided into five chapters:

- Chapter 2 describes the technical approach that was used to plan the testing effort.

- Chapter 3 provides details concerning test setup involving the CLT test structures, explosive charges, and instrumentation employed.

- Chapter 4 documents the results obtained from each of the three blast tests, which include visual observations and gage data recorded for each test.

- Chapter 5 compares the obtained gage data with results obtained using SDOF analysis methods.
- Chapter 6 presents general conclusions made as a result of this testing effort.

References, construction drawings for the CLT test structures, as-built drawings for the doors used in the CLT test structures, and the quick look report generated by AFCEC are included as Appendices A, B, C, and D, respectively.

CHAPTER 2

TECHNICAL APPROACH

Two testing efforts were helpful in planning and preparing for the blast testing described herein:

- A series of laboratory tests that investigated the out-of-plane bending response of CLT panels in the post-peak realm to a quasi-static uniformly-applied load.
- A series of shock tube tests that investigated the dynamic out-of-plane bending response of CLT panels.

This chapter provides a brief overview of each testing effort and identifies how their respective observations and conclusions were useful in planning for the blast testing described herein.

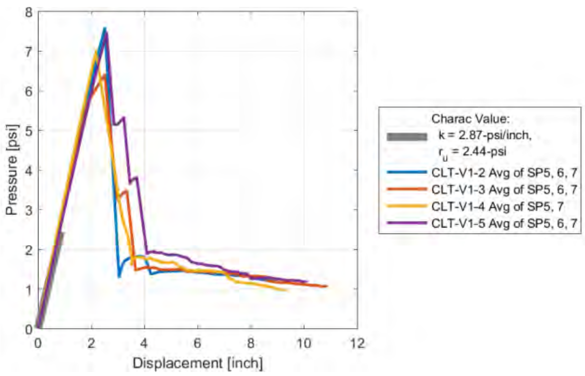
2.1 QUASI-STATIC LABORATORY TESTING

2.1.1 Overview

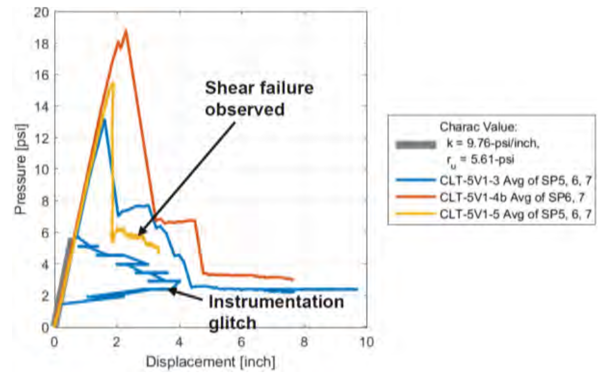
The University of Maine (UMaine) in conjunction with WW and K&C performed a testing program aimed at investigating the bending response of Grade V1 (3-ply and 5-ply), Grade E1, and Grade V4 CLT panels in their major strength direction under a uniformly-applied quasi-static load [3]. The apparatus utilized for the testing was developed by UMaine and consisted of a series of rubber bladders filled with water capable of applying a uniform quasi-static pressure in a controlled fashion. This apparatus is shown with a CLT panel at the end of a test in Figure 2-1.

Figure 2-1. UMaine Test Apparatus with CLT Panel at Conclusion of Test.

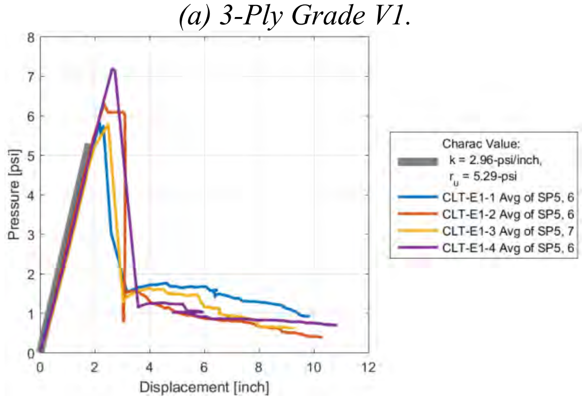
Applied pressure, out-of-plane displacement, and total resisted load were measured and recorded as panels were displaced well beyond the displacement associated with peak panel strength. Load-displacement plots for each CLT grade and ply configuration tested are shown in Figure 2-2.



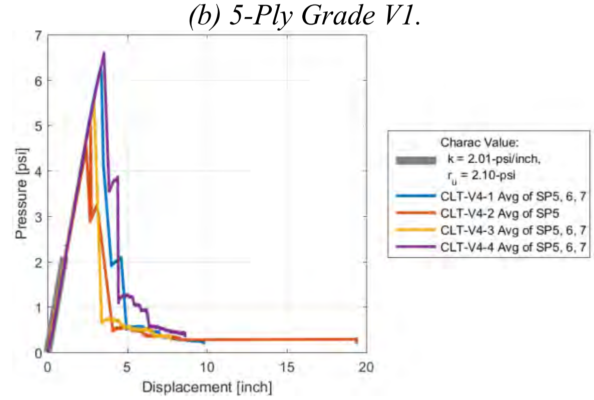
(a) 3-Ply Grade V1.



(b) 5-Ply Grade V1.



(c) 3-Ply Grade E1.

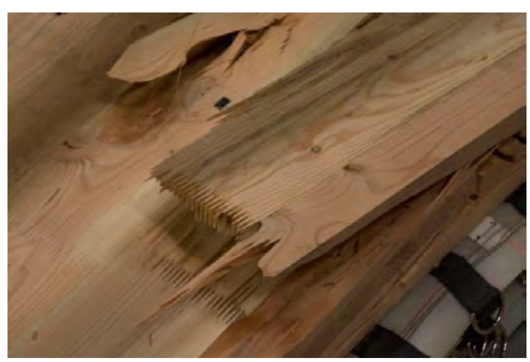


(d) 3-Ply Grade V4.

Figure 2-2. Quasi-Static Testing Load-Displacement Plot Results.

Typical failure pressures for 3-ply CLT panels were between 5 and 8 psi, corresponding to a total load of between 28,000 and 46,000 pounds of applied load. The 5-ply CLT panels failed with a pressure of around 15 psi or approximately 86,000 pounds of applied load. With one exception, all CLT panels failed near panel mid-span, presumably due to flexural stress. The location of panel rupture typically centered on knots, sloped grain, and finger joints (Figure 2-3). No shear slip between panel plies away from the location of panel rupture was observed.

(a) Sloped Grain.



(b) Finger Joint.

Figure 2-3. Quasi-Static Testing Typical Panel Failure Locations.

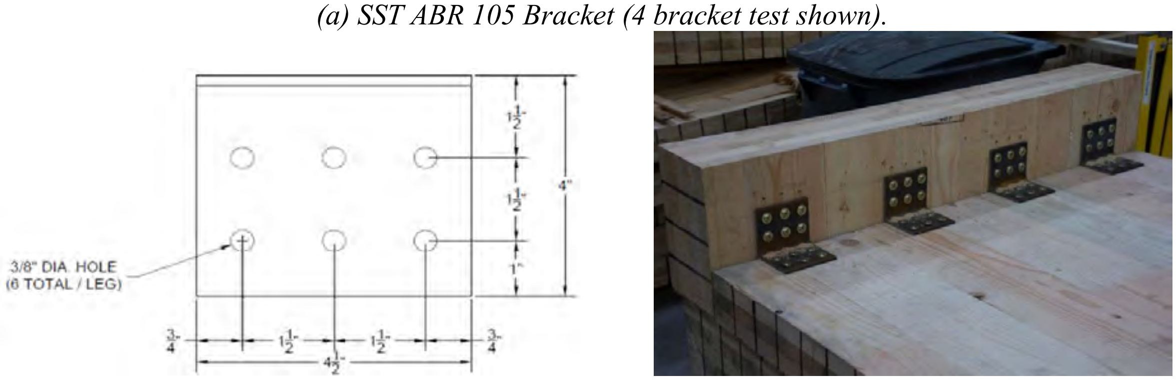
While most of the panels were tested with end conditions that did not restrain panel rotation, six 3-ply Grade V1 CLT panels were tested with connections meant to represent those that might be used to attach a wall to a floor and ceiling in a building designed to resist significant out-of-plane wall loading. Two types of angle brackets were used:

- An 11-gauge Simpson Strong-Tie (SST) ABR105 bracket (Figure 2-4a). The SST brackets were secured using SD10212 (i.e., #10 x 2-1/2”) self-tapping screws manufactured by SST.

- A 4.5-inch length of pre-drilled ASTM A36 L4×4×1/4 angle (Figure 2-4b). The L4x4 brackets were secured using SWG ASSY® SK 5/16x4 self-tapping screws manufactured by MyTiCon.

The number of angle brackets was varied between two and four between tests.

(a) SST ABR 105 Bracket (4 bracket test shown).

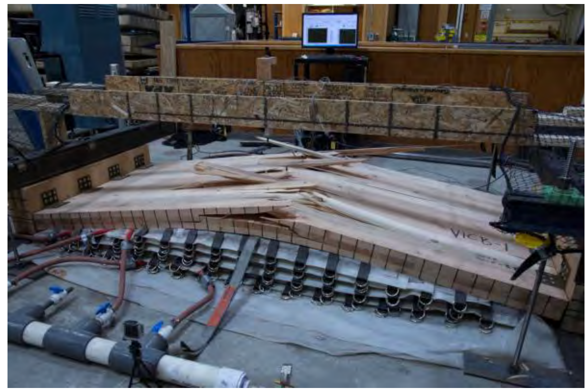


(b) L4x4x1/4 Bracket (4 bracket test shown).

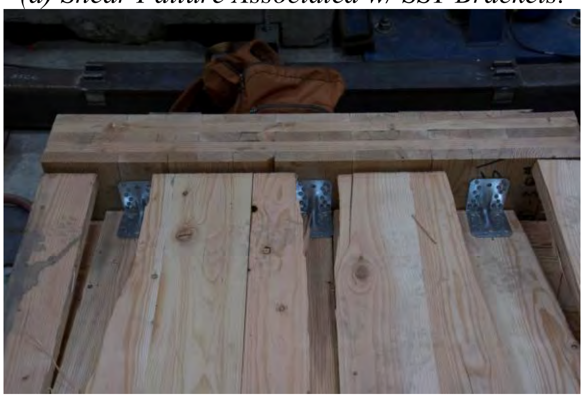
Figure 2-4. Quasi-Static Testing Connection Types.

The panels with SST brackets typically exhibited shear failures near one end of the panel (Figure 2-5a) while the panels with the L4x4 brackets typically exhibited a flexural failure near mid-span (i.e., similar to panel-without-connection tests) (Figure 2-5b). For both brackets, top boards not directly supported by angle brackets pulled away from those that were (Figure 2-5c). In general, both brackets were capable of deforming significantly while still being able to support their respective loads (Figure 2-5d). The measured peak strength of the CLT panel was independent of the number of angle brackets.

(a) Shear Failure Associated w/ SST Brackets.



(b) Flexural Failure Associated w/ L4x4 Brackets.



(c) Top Board Disengagement.

(d) SST Bracket Deformation (L4x4 Similar).

Figure 2-5. Quasi-Static Testing Connection Test Failure Patterns.

2.1.2 Technical Approach Relevance

The quasi-static laboratory testing generated the following observations and conclusions that were used for test planning:

- When CLT panels ruptured due to flexure, negligible shear slip between panel plies away from the location of panel rupture was observed (i.e., see black lines on side of panel in Figure 2-1). This observation lends credence to a fully-composite panel, at the core of the shear analogy model [4].

- The shear analogy model can be employed with the characteristic, or mean, modulus of elasticity values shown in Table 1 of ANSI/APA PRG 320-2012 to faithfully reproduce the observed elastic bending stiffness for the panels tested. Figure 2-2 shows this computed stiffness as a dark gray line. As can be observed from Figure 2-2, CLT panel response was essentially linear elastic prior to panel rupture.

- The shear analogy model can be used with the characteristic, or 5-percent exclusion, bending strength values shown in Table 1 of PRG 320 and the 0.85 conservatism reduction factor specified in Annex A of PRG 320 to generate major strength direction bending capacities that are lower-bound values for the panels tested.

- The mean tested bending strength for the Grade E1 CLT panels was much nearer to its characteristic, or 5-percent exclusion, bending strength than the mean tested bending strengths for the Grade V1 and Grade V4 CLT panels were to their respective characteristic bending strengths. For the Grade E1 CLT panels, the characteristic and mean tested bending strengths were within roughly 20-percent of each other. On the other hand, the Grade V1 and Grade V4 CLT panels had mean tested bending strengths of almost three times that of their corresponding characteristic bending strengths. Figure 2-6 illustrates this phenomenon by plotting the relative frequency of the outermost ply's bending strength assuming a normal distribution. These distributions were constructed by setting the 5-percent exclusion value to that defined in Table 1 of PRG 320 and mean value to the mean tested bending strength.

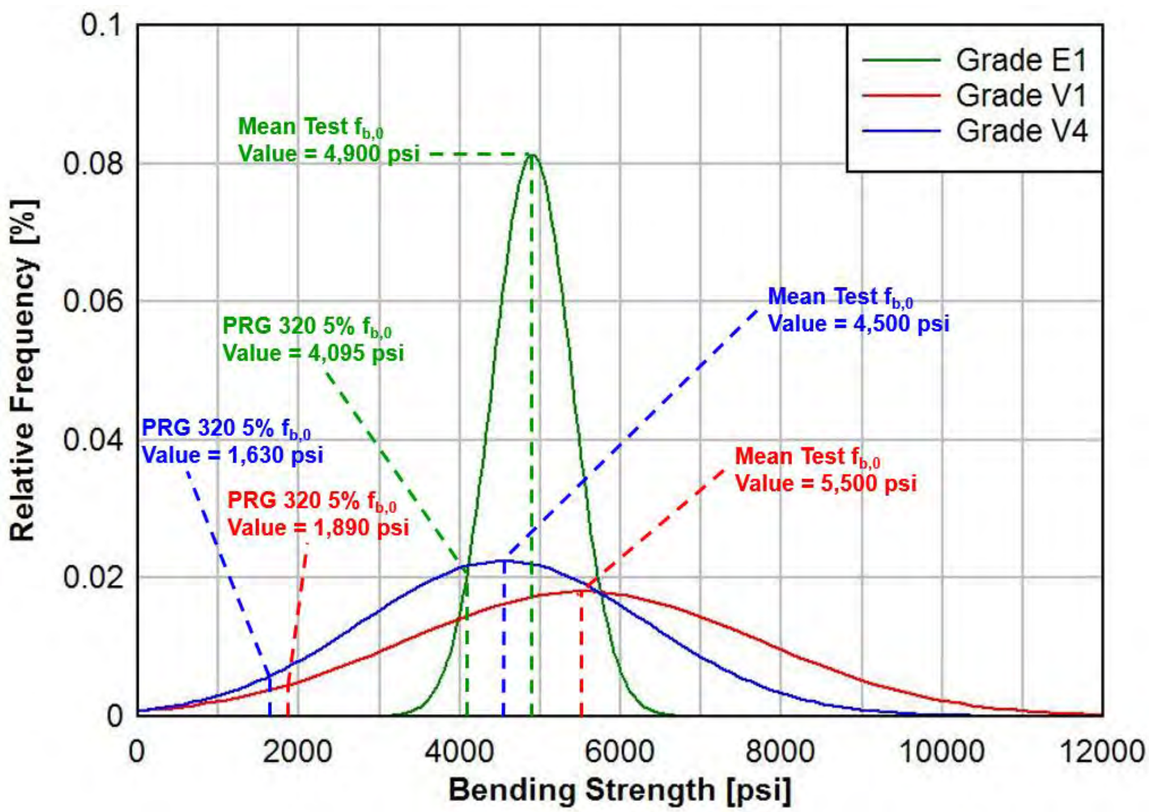


Figure 2-6. Bending Strength Normal Distributions by CLT Grade.

- Upon panel rupture, there was a relatively sudden drop in panel strength to a residual panel strength plateau. The value of this residual strength plateau always exceeded the strength computed using the shear analogy model and ignoring the ruptured ply. For example, the residual strength plateau value of a 5-ply panel was greater than the characteristic bending strength of a 3-ply panel in all circumstances (Figure 2-2).

- Fastener length and the corresponding number of plies that are engaged can impact the ultimate failure mode observed. Although more testing would be needed to corroborate this conclusion, it appears where the fasteners were long enough to engage all panel plies, the fasteners served to act as shear reinforcement and resist the augmented shearing forces associated with discrete support points (Figure 2-5a and b).

- Panels that are not continuously supported are susceptible to top board disengagement at high deformations (Figure 2-5c).

2.2 SHOCK TUBE TESTING

2.2.1 Overview

A series of shock tube tests were performed on 3-ply, 5-ply, and 7-ply Grade E1 CLT panels. Panel response was limited to the elastic range and each panel was hit multiple times with progressively increasing loads. The observations and results obtained via these tests are documented in two reports [5][6].

2.2.2 Technical Approach Relevance

The shock tube testing confirmed many of the observations gleaned from the quasi-static laboratory tests and provided insight into the elastic dynamic response of CLT panels. Specific conclusions included:

- The stiffness and strengths computed using the shear analogy model could be used to approximate panel displacement response in the elastic range to a uniformly-applied transient load.

- The load duration factor, C_D, used by the *National Design Specification for Wood Construction* [7], was applicable to the panels tested. Because the C_D for impact loading is 2.0 and the 10-minute duration of 1.6 is used to determine PRG 320 design values, an effective increase factor of 1.25 (2.0/1.6) can be used to convert published CLT design values to load factored design values for impact.

- Provided the panel remained in its elastic range, striking the panel multiple times (e.g., one panel was hit six times) did not appear to alter panel strength or stiffness on subsequent tests for the panels tested.

CHAPTER 3

TEST SETUP

The setup for the blast testing of CLT construction is described in this chapter. Section 3.1 describes CLT test structure details such as site layout, panel sizes, connection details, opening details, and construction notes. Section 3.2 then documents details concerning the explosive charges used. Finally, Section 3.3 describes details about the instrumentation employed for each test.

3.1 TEST STRUCTURES

Three single-bay, two-story CLT structures were constructed at Tyndall AFB. Two of the structures had roughly 12-feet story heights and one structure had roughly 10-feet story heights. The two structures with the same story height were identical except that one was constructed using Grade E1 CLT panels and the other was constructed using Grade V1 CLT panels. The 10-foot story height structure was constructed using Grade V4 CLT panels. Construction drawings showing each of the structures are included in Appendix B. Figure 3-1 is a photograph of the three CLT test structures prior to the first test.

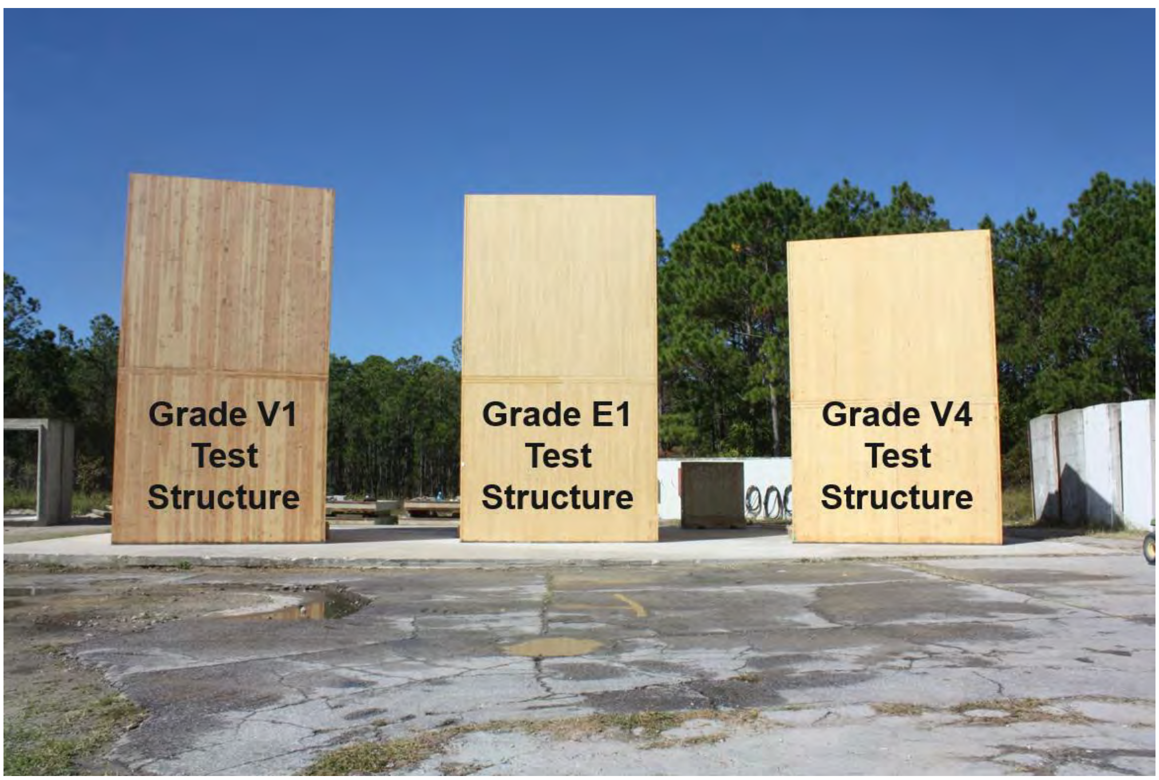


Figure 3-1. Pre-Test Photograph of All Test Structures.

3.1.1 Site Layout

The structures were constructed so that their front face was situated 75 feet from the center of the explosive charge. The test structures were spaced far enough apart to limit shockwave reflections between adjacent structures. The test structure constructed using an E-grade CLT (i.e., E1) was centered and flanked by test structures constructed using V-grade CLT (i.e., V1 and V4). Figure 3-2 shows the orientation of the test structures in plan.

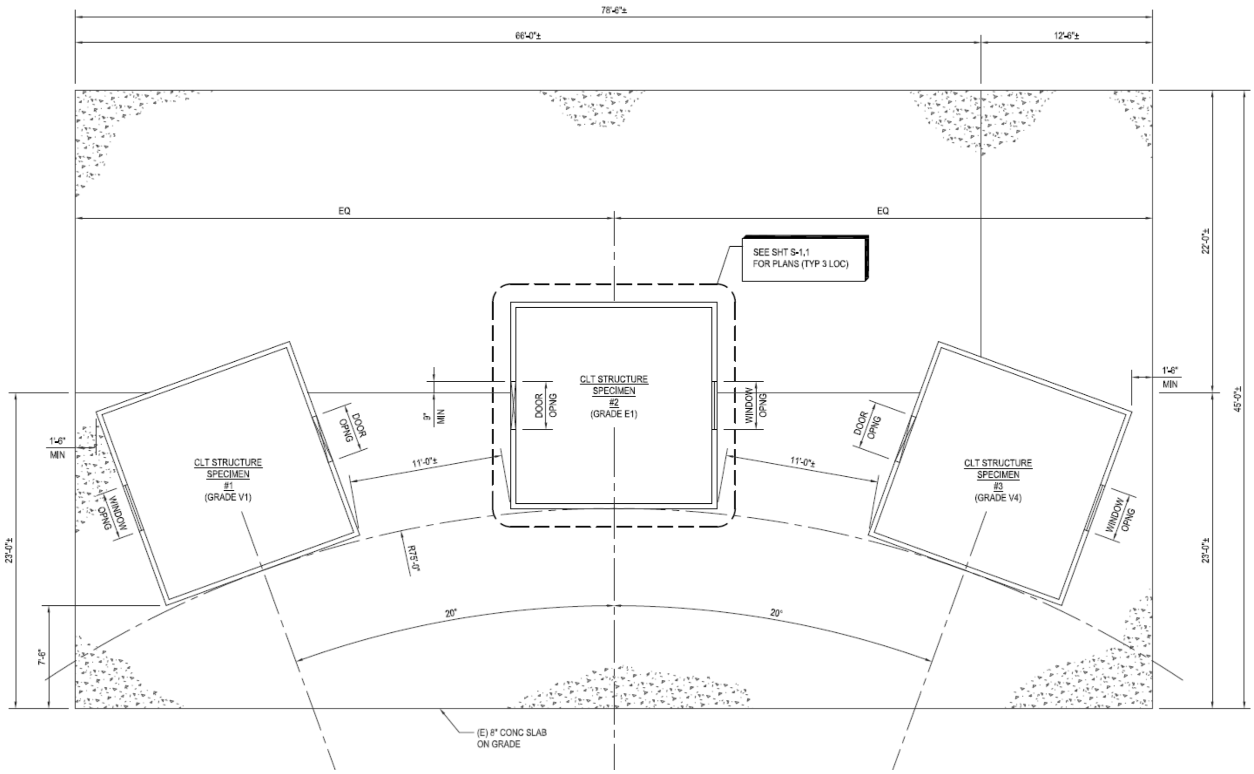


Figure 3-2. Site Plan.

3.1.2 Panels

Panels were provided by three different CLT manufacturers and all panels and plants were third party certified to PRG 320 standards. Grade E1 panels were provided by Nordic Structures, Grade V1 panels were provided by DR Johnson, and Grade V4 panels were provided by SmartLam. Wall and roof panels were 3-ply panels (i.e., $4^{1}/_{8}$ inches thick) and the elevated floor panel at the second floor was a 5-ply panel (i.e., $6^{7}/_{8}$ inches thick). The width of the individual lamella used to construct the CLT panels varied between grades; 7 inches, $3^{1}/_{4}$ inches, and 7 inches wide for the Grade V1, Grade E1, and Grade V4, respectively. The average board lengths and finger jointing used in each lamination also varied by grade. Lamella characteristics of each grade are consistent with those tested at UMaine [3].

Two different types of CLT construction were included in the buildings. The first floor was constructed using platform framing and the second floor was constructed using balloon framing with a parapet. The utilization of different framing types enabled many of the typical connection configurations found in a CLT building to be tested. These connection configurations are described in the following section.

3.1.3 Connections

Connections were made to emulate typical CLT connection configurations. Five basic types of connections were employed: (1) panel-to-foundation, (2) panel-to-panel splice, (3) wall-to- floor panel (platform framing), (4) wall-to-roof panel (balloon framing), and (5) wall panel at corner.

Most connection configurations utilized $^{5}/_{16}$-inch diameter SWG ASSY® self-tapping screws (STSs) of various lengths manufactured by MyTiCon to secure adjacent panels to one another. Based on the results of the connection tests performed at UMaine, STS length was selected to allow the screw to engage all plies of a given panel where practical. Where screw withdrawal was a potential limit state, the SK (i.e., washer head) screw was utilized (i.e., the bottom screw in Figure 3-3). Otherwise, the ECO (i.e., counter-sunk head) screw was used (i.e., the top two screws in Figure 3-3).

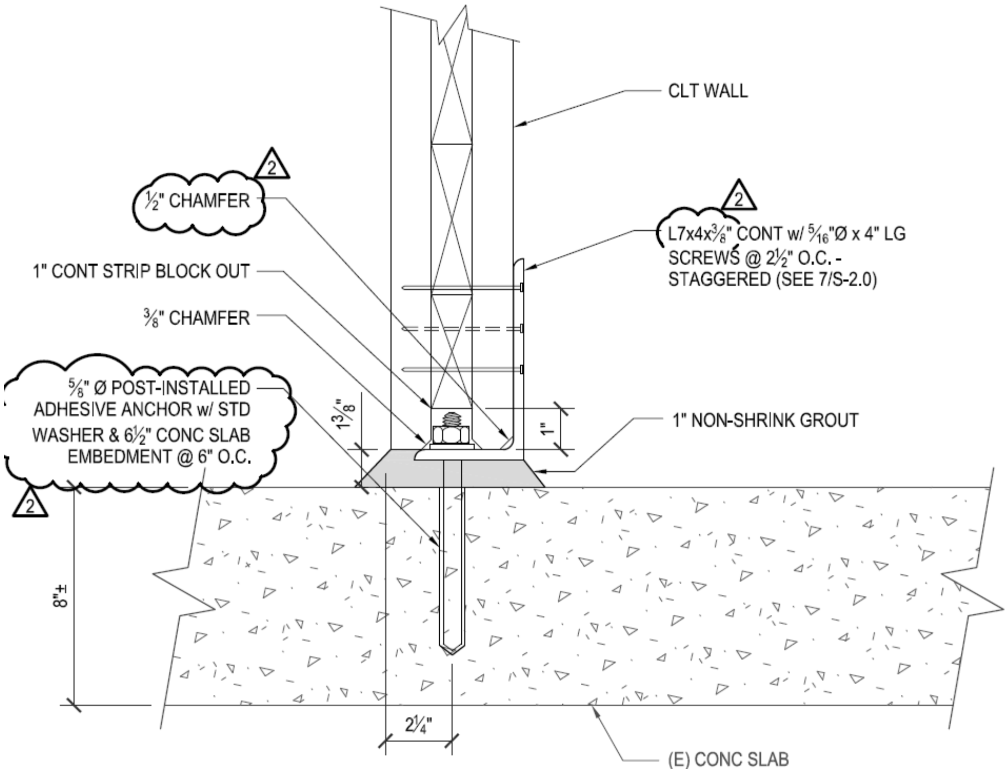
Figure 3-3. Self-Tapping Screws Used in Test Structure Connections.

3.1.3.1 PANEL-TO-FOUNDATION CONNECTION

The panel-to-foundation connection is shown in Figure 3-4. This connection aims to limit the visibility of the connection elements while still allowing for a robust connection capable of resisting panel inbound and rebound forces (i.e., deriving from airblast loads applied in the out-of-plane direction) and global structure overturning forces simultaneously.

The connection is constructed using continuous L7x4x3/8 angle and $^{5}/_{16}$-inch diameter by 4-inch long STSs. The length was chosen to ensure the screw penetrated all three plies of the wall panel. The angle was secured to the existing 8-inch thick reinforced concrete slab with $^{5}/_{8}$-inch diameter ASTM A193 B7 threaded rod and HIT-HY 200 adhesive manufactured by Hilti with

$6^{1}/_{2}$-inch embedment. The angle was originally scheduled to be constructed using $^{3}/_{16}$-inch thick bent-plate but was changed to a standard angle shape to reduce cost.



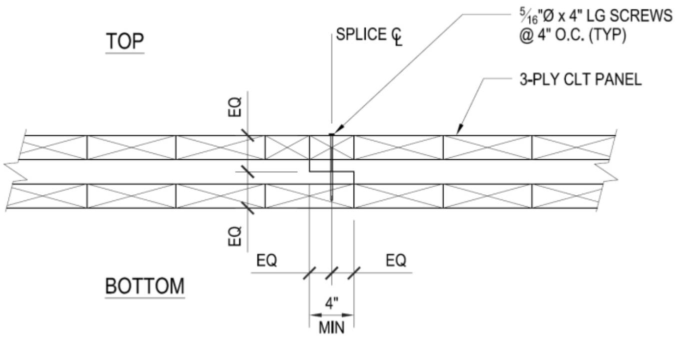
(a) Detail.

(b) Angle Placement Prior to Panel Install.

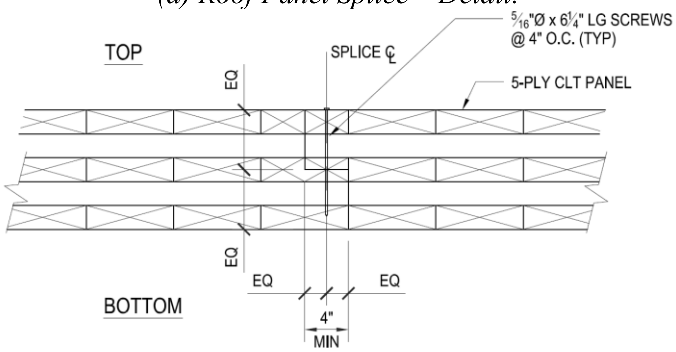
Figure 3-4. Panel-to-Foundation Connection.

3.1.3.2 *PANEL-TO-PANEL SPLICE CONNECTION*

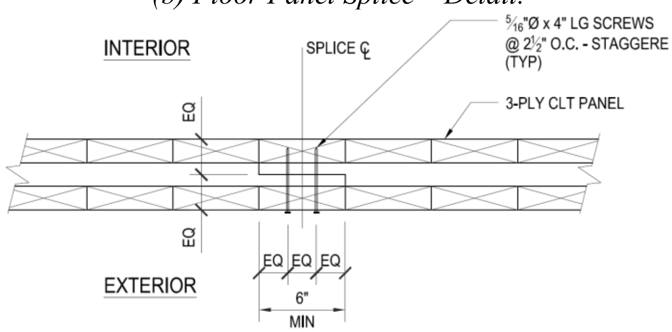
The panel splice connections are shown in Figure 3-5. Half-lapped joints were used to cause adjacent diaphragm and shear wall panels to act together. Self-tapping screw spacing was computed to resist the in-plane shear forces associated with Test 2. In all cases, screw length was sized to engage all plies of the respective CLT panel.



(a) Roof Panel Splice – Detail.



(b) Floor Panel Splice – Detail.



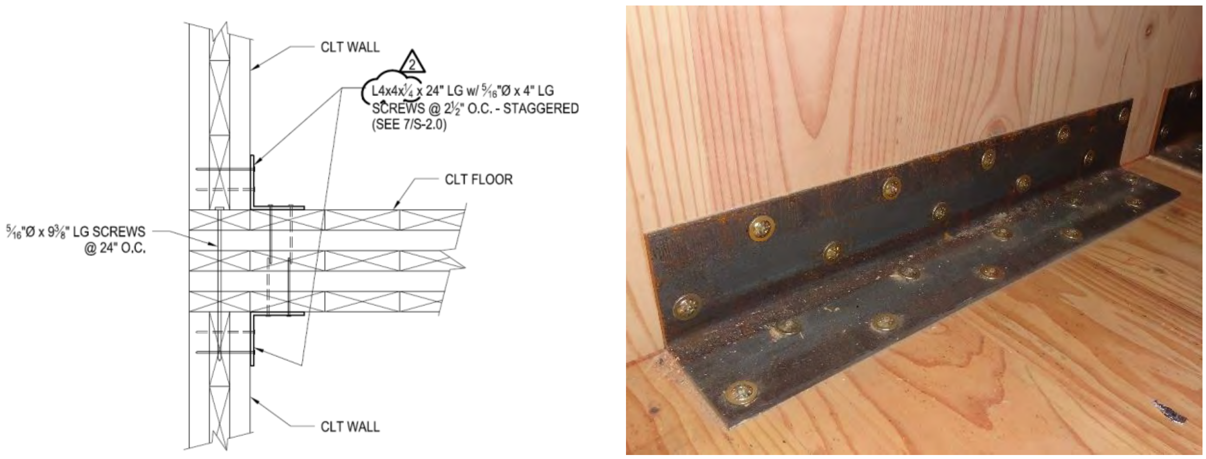
(c) Wall Panel Splice – Detail.

(d) Wall Panel Splice – As Installed.

Figure 3-5. Panel Splice Connection.

3.1.3.3 *WALL-TO-FLOOR PANEL (PLATFORM FRAMING) CONNECTION*

The wall-to-floor panel connection for the platform framing condition is shown in Figure 3-6. This connection is designed to resist the out-of-plane shear forces delivered by the first and second floor wall panels. Inward panel response is resisted by angle bearing and screw shear limit states while rebound panel response is resisted by screw withdrawal, screw head pull-through, and screw shear limit states. Self-tapping screw spacing was computed to resist the out-of-plane shear forces associated with Test 2.

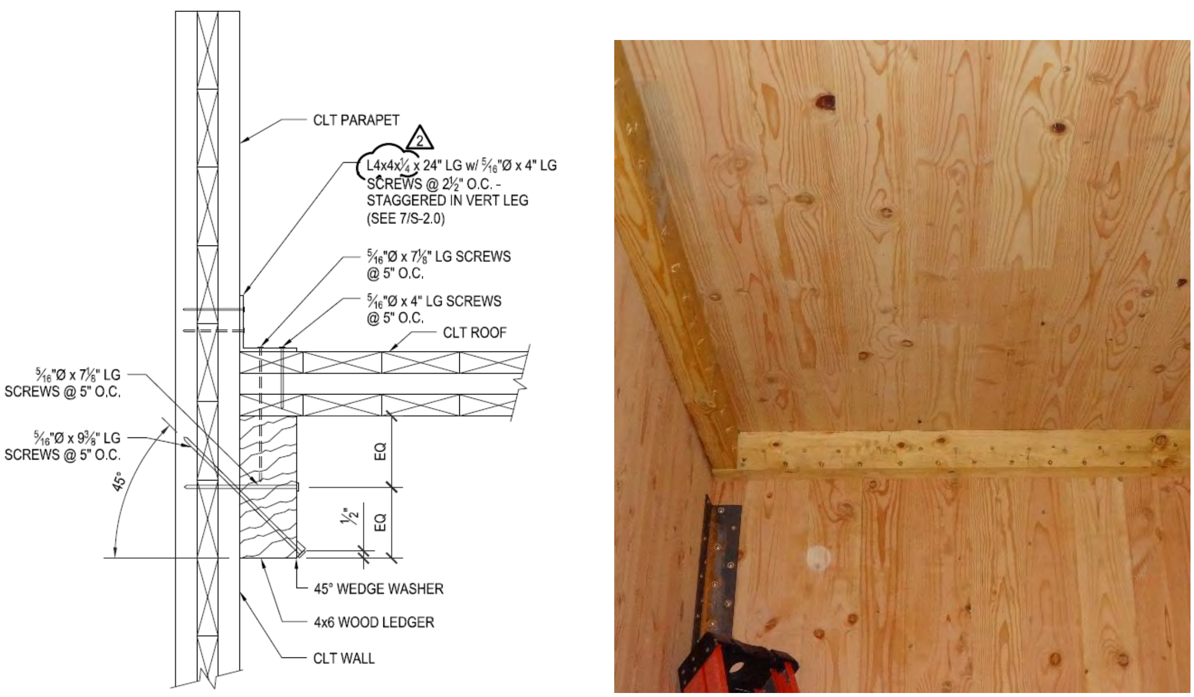


/(a) Detail. (b) As Installed.

Figure 3-6. Wall-to-Floor Panel (Platform Framing) Connection.

3.1.3.4 *WALL-TO-FLOOR PANEL (BALLOON FRAMING) CONNECTION*

The wall-to-floor panel connection for the balloon framing condition is shown in Figure 3-7. This connection is designed to resist the out-of-plane shear forces delivered by the second-floor wall panels and roof. Inward panel response is resisted by angle bearing and screw shear limit states while rebound panel response is resisted by screw withdrawal, screw head pull-through, and screw shear limit states. Self-tapping screw spacing was computed to resist the out-of-plane shear forces associated with Test 2.

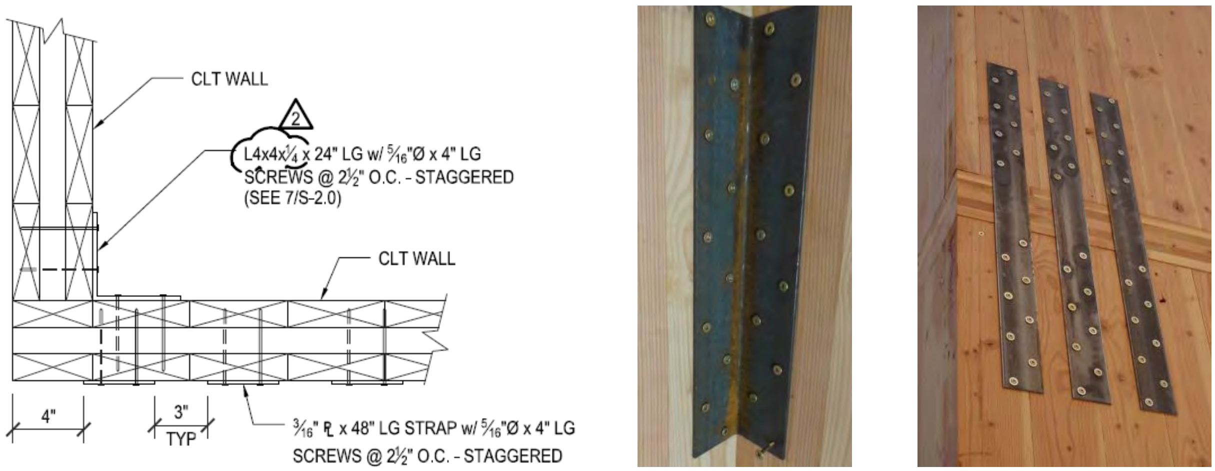


(a) Detail. (b) As Installed.

Figure 3-7. Wall-to-Floor Panel (Balloon Framing) Connection.

3.1.3.5 *WALL PANEL AT CORNER CONNECTION*

The wall panel at corner connection is shown in Figure 3-8. This connection ties wall panels so they can act together in transferring transfer overturning forces to the foundation anchorage. The connection consists of two parts: (1) internal 24-inch lengths of L4x4x1/4 angle (Figure 3-8b) and (2) three external straps (Figure 3-8c). Self-tapping screw number was computed to resist the boundary member tension forces associated with Test 2.



(a) Detail. *(b) As Installed (Interior).* *(c) As Installed (Exterior).*

Figure 3-8. Wall Panel at Corner Connection.

3.1.4 Openings

Typical window (i.e., 3'-6" square rough opening) and pedestrian door (i.e., 3'-4 $^{1}/_{2}$" wide by 7'-4 $^{3}/_{8}$" high rough opening) openings were included in each structure.

The window opening detail and as-installed condition are shown in Figure 3-9. The window opening was cut out of a solid CLT panel and was covered with two $^{3}/_{4}$-inch pieces of plywood to allow airblast loads applied at the opening to be transferred to the opening's head, sill, and jambs. The plywood was designed to remain elastic under the airblast loads imparted by Test 2.

(a) Detail. *(b) As Installed.*

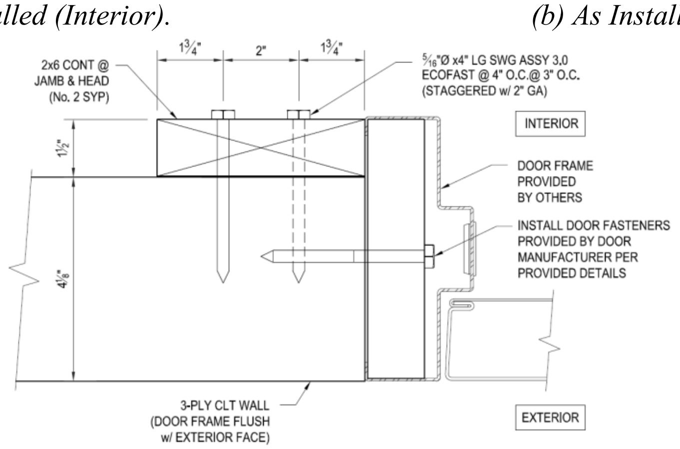
Figure 3-9. Window Opening Connection.

Actual $1^3/_4$-inch thick by 36-inch wide by 86-inch high pedestrian doors manufactured using 14 gage galvannealed steel were provided by American Direct and manufactured by Ambico. The door shop drawings provided by American Direct are included as Appendix C. As-installed photographs of the door are included as Figure 3-10a and b. Doors were designed to exhibit a low level of protection (i.e., as defined in UFC 4-010-01) for Explosive Weight II (i.e., as defined in UFC 4-010-02 [8]) with 105-feet of standoff distance.

The door openings were built out using dimensional lumber to accommodate the $5^3/_4$-inch wide frame in the $4^1/_8$-inch thick 3-ply CLT wall panels. The detail for this door framing detail is shown in Figure 3-10c.

(a) As Installed (Interior).

(b) As Installed (Exterior).



(c) Door Frame Detail.

Figure 3-10. Door Opening Figures.

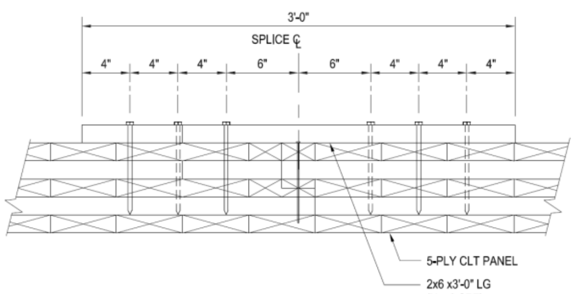
Two types of fasteners were used to secure the door frame to the CLT test structures: (1) ten $^{1}/_{2}$-inch diameter by 5-inch long lag screws and (used at the Grade E1 and V1 structures) and (2) twenty-eight $^{5}/_{16}$-inch diameter by $5^{1}/_{2}$-inch long SWG ASSY® Kombi STS manufactured by MyTiCon (used at the Grade V4 structure). Fasteners were uniformly spaced along the three supported sides of the door frame as shown in Appendix C.

No locking hardware was employed to lock the door during the blast tests to limit the possibility that the door would jam shut due to the applied airblast load. Additionally, no hinges were provided for the Grade V1 or Grade E1 test structures. (Three stainless steel heavy weight bearing hinges (i.e., T4A3386 NRP $4^{1}/_{2}$"x$4^{1}/_{2}$") manufactured by McKinney were used to secure the door panel to the door frame in the Grade V4 test structure.) Instead, sand bags and dimensional lumber were used to keep the door closed at the beginning of the test for all test structures as shown in Figure 3-10b.

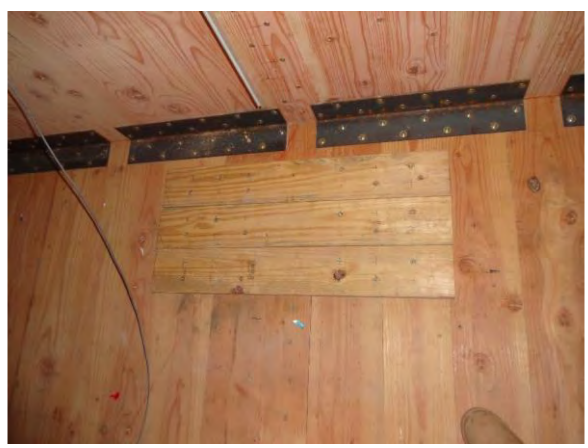
3.1.5 Construction

Lend Lease constructed the three CLT test structures over a period of eight days. Construction activities included post-installed anchor installation, panel erection, STS installation, and non-shrink grout installation.

During construction, the second-floor panels were mistakenly rotated 90 degrees from what was originally specified. As such, the second-floor diaphragm required a retrofit detail to adequately transfer chord forces associated with Test 2. Dimensional lumber was used to transfer these chord forces and allow for a continuous diaphragm. This retrofit detail is shown in Figure 3-11 and recorded in the construction drawings included as Appendix B.



(a) Detail.



(b) As Installed.

Figure 3-11. Diaphragm Chord Retrofit Connection.

3.2 EXPLOSIVE CHARGE

3.2.1 Charge Description

Characteristics of the charges utilized for the three tests are listed in Table 3-1. Charges were created using flake TNT (ρ = 0.0287 lb/in^3) and formed using Sonotubes® of various diameters and lengths. The method of detonation consisted of replacing 1 pound of flake TNT with a 1-pound cast block of TNT that was tied into a detonator. The TNT block with its detonator was placed in the top-center of the charge. In all cases, the bottom of the charge was elevated 18 inches off the ground. The ground below the charge was compacted soil.

Table 3-1. Charge Characteristics by Test.

Test	Diameter (D) [in]	Height (H) [in]	H/D	Weight [lb]
1	14	7.24	0.52	32
2	18	9.17	0.51	67
3	24	15.3	0.64	199

3.2.2 Standoff Distance

A standoff distance of 75 feet was used for all tests. This standoff distance was measured from the center of the charge to the front face of the CLT test structures.

3.2.3 Charge Weight Selection

Charge weights were selected to cause the first-floor front panels on the CLT test structures to respond in accordance with target response objectives. The target response objectives for each test were as follows:

- **Test 1**: To displace the first-floor front panels of the Grade V1 and Grade V4 CLT test structures to their respective elastic limit displacements.
- **Test 2**: To displace the first-floor front panels of the Grade E1 test structure to its elastic limit displacement.
- **Test 3**: To displace the first-floor front panels of the Grade E1 test structure to 1.5 times its elastic limit displacement.

Elastic limit displacements, x_E, were set equal to the panel's ultimate resistance, r, divided by its elastic stiffness, k. The shear analogy model and the characteristic values listed in Table 1 of ANSI/APA PRG 320-2012 were used to compute r and k. Simple-simple boundary conditions were assumed. (It should be noted that the 0.85 conservatism reduction factor specified in Annex A of PRG 320 for bending strength was not included when computing r.) Table 3-2 lists x_E, r, and k for the first-floor front panel for each CLT test structure.

Using the parameters listed in Table 3-2, SDOF dynamic analyses were performed to determine the charge weight that would accomplish the target response objectives. An elasto-plastic resistance function (Figure 5-3) was utilized with the assumptions documented in Section 5.2 of this report to perform these analyses. The resulting charge weights are recorded in Table 3-1 and the computed displacement ductility for each CLT test structure is recorded in Table 3-3.

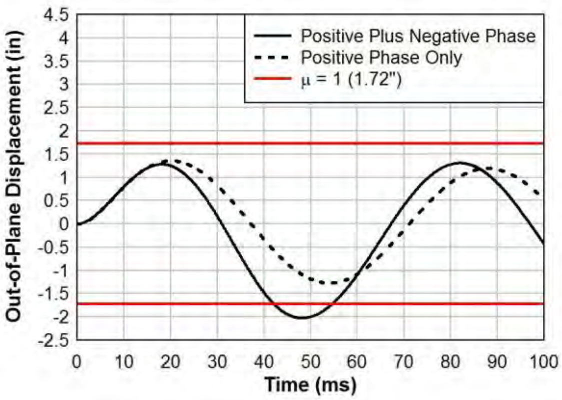
Both positive-phase-only and positive-plus-negative-phase airblast load cases were considered in the SDOF analyses. In all cases, the inclusion of the negative phase led to maximum displacement response; this result is illustrated in Figure 3-12.

Table 3-2. SDOF Dynamic Analysis Parameters.

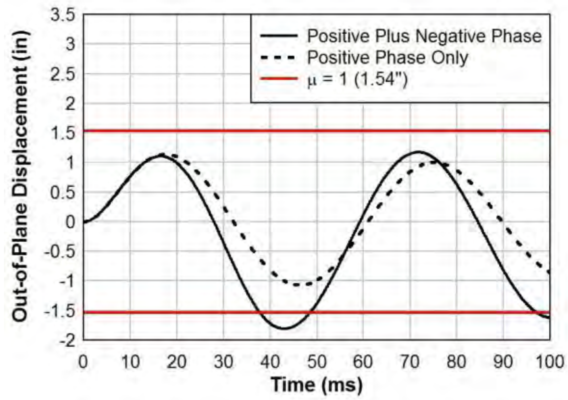
Structure Grade	L [ft]	m [psi-ms^2/in]	k [psi/in]	r [psi]	x_E [in]
V1	12	216.2	1.45	2.49	1.72
V4	10	216.2	2.01	3.09	1.54
E1	12	216.2	1.50	5.39	3.58

Table 3-3. Pre-Test Displacement Ductility by Test and Test Structure Grade.

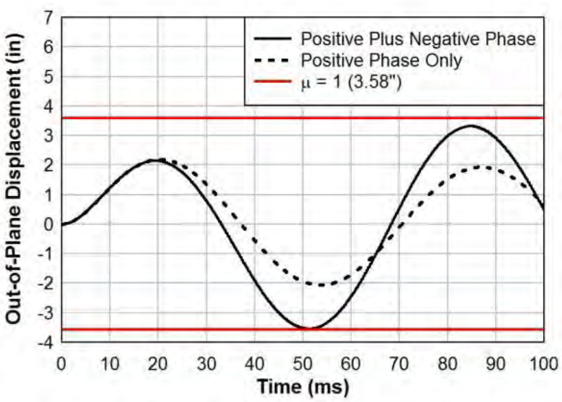
Structure Grade	Test 1		Test 2		Test 3	
	Target	*Computed*	*Target*	*Computed*	*Target*	*Computed*
V1	1.00	1.18	-	-	-	-
V4	1.00	1.18	-	-	-	-
E1	-	-	1.00	0.99	1.50	1.51



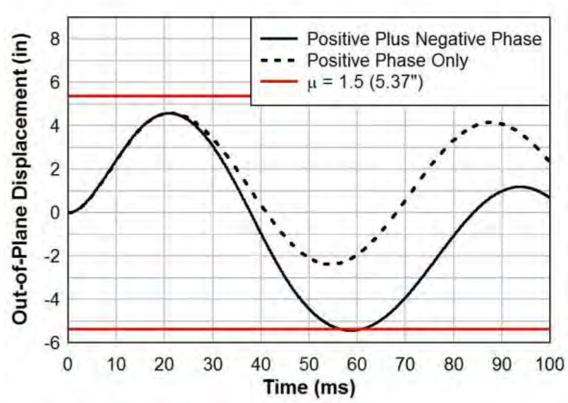
(a) Test 1 – Grade V1.



(a) Test 1 – Grade V4.



(c) Test 2 – Grade E1.



(d) Test 3 – Grade E1.

Figure 3-12. Pre-Test Target vs. Computed Displacement Plots (Front Panel / 1st Floor).

3.3 INSTRUMENTATION

The instrumentation for each test structure included pressure gages, displacement gages, and video cameras as described below.

3.3.1 Pressure

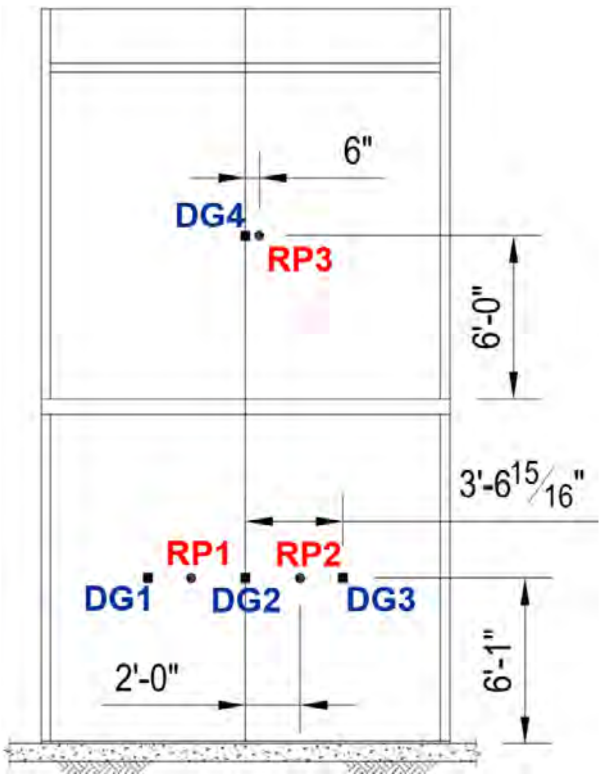
Table 3-4 provides a summary of the twenty-nine Kulite XT-190 pressure gages that were used for each test:

- Twenty-four gages were mounted to the exterior surface of the three test structures (i.e., eight per structure) to measure reflected pressure.
- Three gages were mounted on stands located inside each test structure on the first floor (i.e., one per structure) to measure internal pressure.
- Two gages were mounted to a wood block resting on the ground to measure incident overpressure seventy-five feet away from the explosive charge.

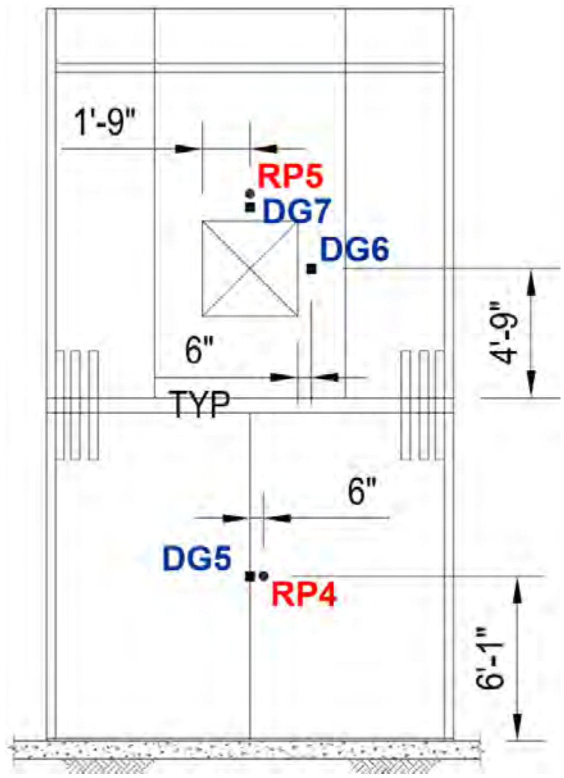
The locations of the reflected pressure gages (i.e., labeled RP1 to RP24) are shown schematically in Figure 3-13 through Figure 3-15. Figure 3-16 shows photographs of the pressure gages used.

Table 3-4. Pressure Gage Summary.

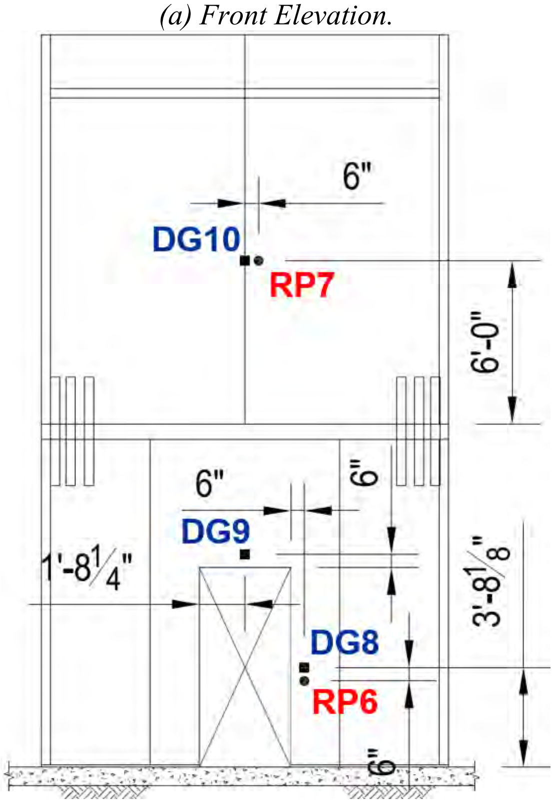
ID	Structure Grade	Measurement	Location	Range
RP1 – RP3	V1	Reflected Pressure	Flush w/ wall (Figure 3-13)	± 25 psi
RP4 – RP8	V1	Reflected Pressure	Flush w/ wall (Figure 3-13)	± 5 psi
RP9 – RP11	E1	Reflected Pressure	Flush w/ wall (Figure 3-14)	± 25 psi
RP12 – RP16	E1	Reflected Pressure	Flush w/ wall (Figure 3-14)	± 5 psi
RP17 – RP19	V4	Reflected Pressure	Flush w/ wall (Figure 3-15)	± 25 psi
RP20 – RP24	V4	Reflected Pressure	Flush w/ wall (Figure 3-15)	± 5 psi
IP1	V1	Internal Pressure	Inside test structure at 1st floor	± 5 psi
IP2	E1	Internal Pressure	Inside test structure at 1st floor	± 5 psi
IP3	V4	Internal Pressure	Inside test structure at 1st floor	± 5 psi
FF1 – FF2	N/A	Incident Overpressure	75 feet from charge (Figure 3-19)	± 10 psi



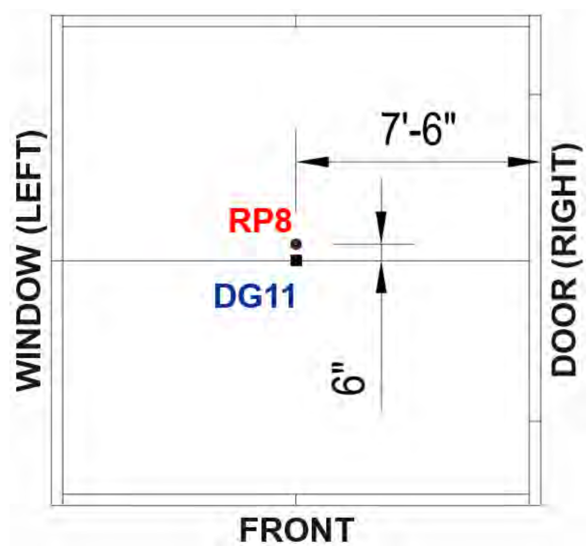
(a) Front Elevation.



(b) Window (Left) Elevation.

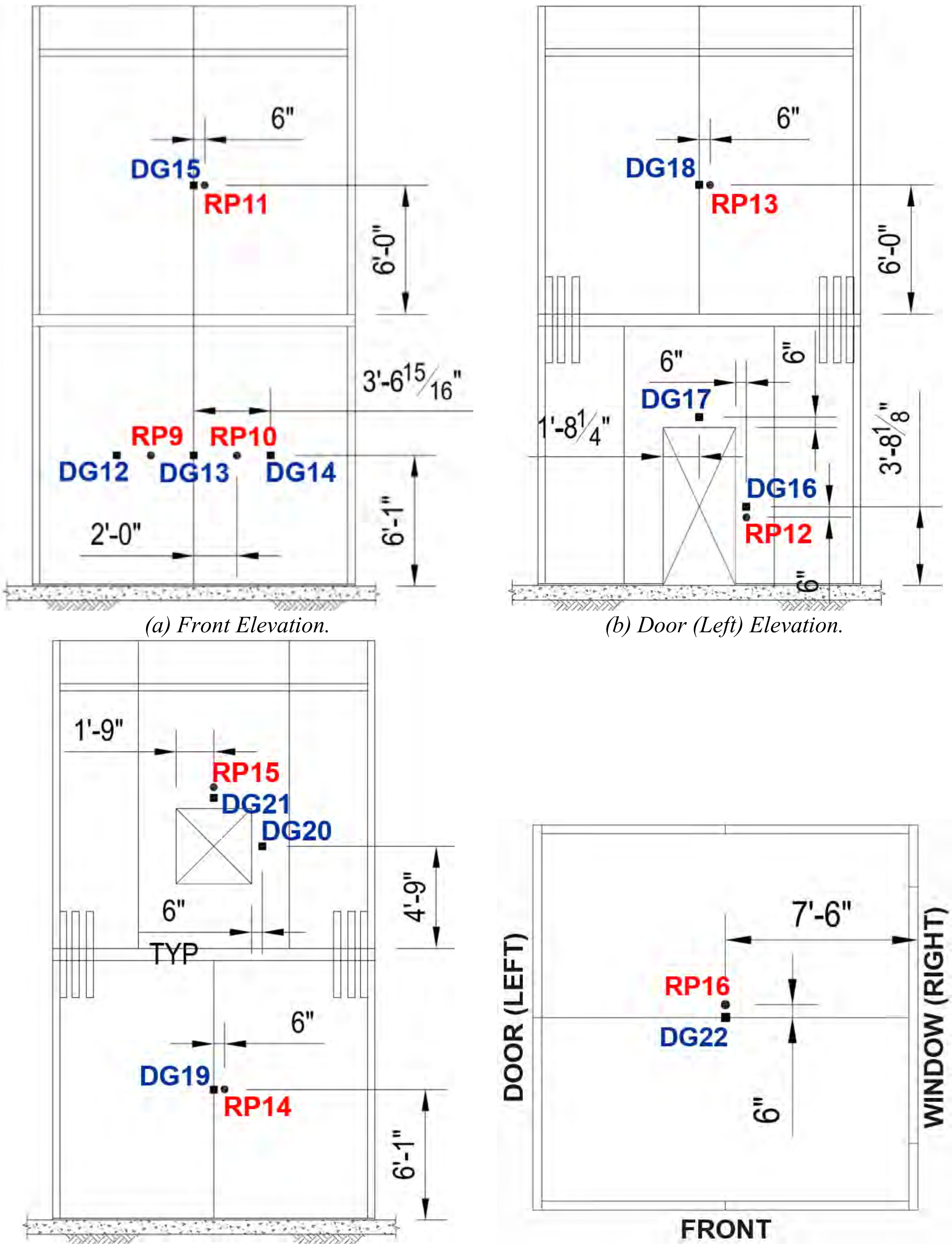


(c) Door (Right) Elevation.



(d) Roof.

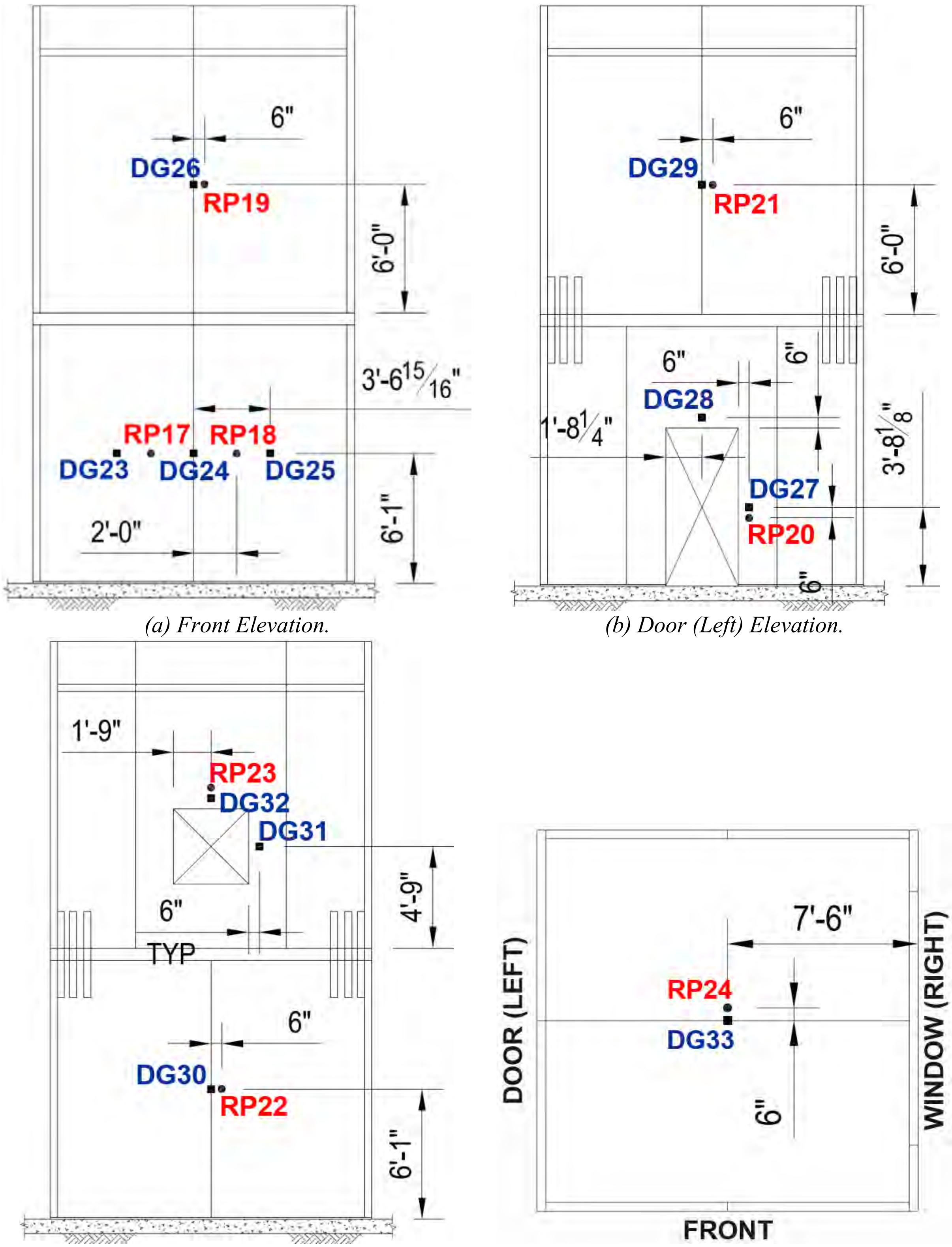
Figure 3-13. Grade V1 Structure Reflected Pressure Gage Key Plan.



(a) Front Elevation. *(b) Door (Left) Elevation.*

(c) Window (Right) Elevation. *(d) Roof.*

Figure 3-14. Grade E1 Structure Reflected Pressure Gage Key Plan.



(a) Front Elevation. *(b) Door (Left) Elevation.*

(c) Window (Right) Elevation. *(d) Roof.*

Figure 3-15. Grade V4 Structure Reflected Pressure Gage Key Plan.

(a) Reflected Pressure. *(b) Internal Pressure.* *(c) External Incident Overpressure.*

Figure 3-16. Pressure Gages Used in Testing.

Following Test 1, it was observed that several reflected pressure gages popped out of their flush mount (Figure 3-17a), presumably due to negative phase pressure and/or panel rebound. As such, a single self-tapping screw was used to secure the reflected pressure gages for the remaining two shots (Figure 3-17b).

(a) Gage Pop Out. *(b) With Self Drilling Screw.*

Figure 3-17. Attachment Problem Observed for Reflected Pressure Gages.

3.3.2 Displacement

Table 3-5 provides details concerning the thirty-three gages (i.e., eleven per test structure) used to measure displacement for each test. The displacement gage used was a rack and wheel potentiometer and was supported by stands manufactured out of steel tubes and angles (Figure 3-18). The locations of the displacement gages are shown schematically in Figure 3-13 through Figure 3-15.

Table 3-5. Displacement Gage Summary.

ID	Structure Grade	Measurement	Location	Range
DG1 – DG11	V1	Out-of-Plane Displacement	Flush w/ wall (Figure 3-13)	36” (in) 12” (out)
DG12 – DG22	E1	Out-of-Plane Displacement	Flush w/ wall (Figure 3-14)	36” (in) 12” (out)
DG23 – DG33	V4	Out-of-Plane Displacement	Flush w/ wall (Figure 3-15)	36” (in) 12” (out)

Figure 3-18. Rack and Wheel Displacement Gages with Support Stands.

3.3.3 Video

Five video cameras were used to record each test from different angles. Details concerning the video cameras are included in Table 3-6. Four of the five cameras were high-speed cameras and were capable of recording at least 3,270 frames per second (fps). Figure 3-19 provides a schematic representation of how the high-speed video cameras were positioned.

Table 3-6. Video Camera Summary.

ID	Camera	View	Resolution / Speed
HS1	Miro 320S Phantom	Side view of Grade V1 structure	1280x720 @ 3270 fps
HS2	Miro 320S Phantom	Side view of Grade V4 structure	1280x720 @ 3270 fps
HS3	Miro 320S Phantom	Between Grades V1 & E1 structures from behind	1280x720 @ 3270 fps
HS4	V12 Phantom	Overall view	1280x720 @ 6960 fps
4K	Sony 4K Ultra-HD	Overall view	32 fps

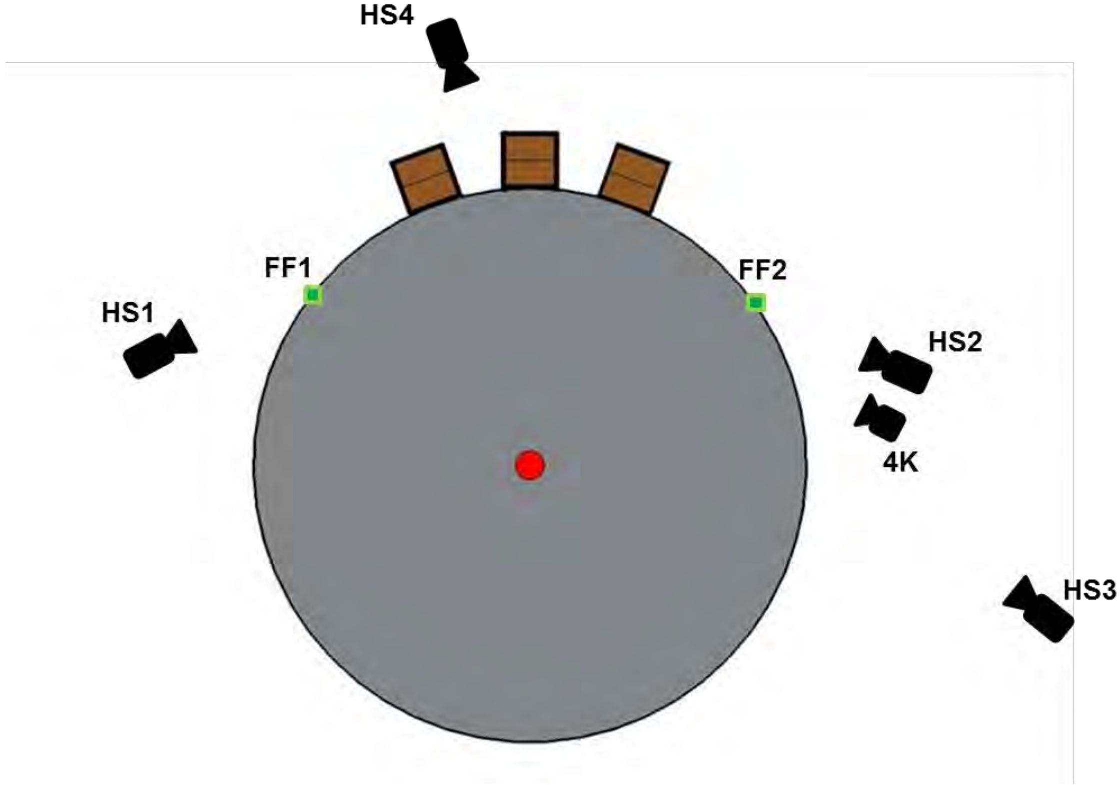


Figure 3-19. Video Camera and Free-Field Pressure Gage Key Plan.

CHAPTER 4

TEST RESULTS

The results of the three blast tests are described in this chapter. The chapter opens with a description of visual observations made following each test. Then the pressure and displacement data recorded for each test are presented.

4.1 OBSERVATIONS

4.1.1 Test 1

Test 1 was performed on the morning of October 12, 2016. Figure 4-1 shows the elevations of the three test structures directly facing the explosive charge following Test 1.

Outside of a few knots popping out of exposed CLT panel plies (Figure 4-2), no signs of damage to or permanent deformation in the constituent panels of the test structures were observed following Test 1. While no damage was observed on the CLT panels themselves, the grout placed under the foundation angle cracked and broke up in isolated cases (Figure 4-3).

Photographs of the post-test condition of the first-floor panel directly facing the charge are included as Figure 4-4.

Figure 4-1. Test 1 Post-Test Photograph of All Test Structures.

Figure 4-2. Knot Pop Out on Exposed Face of Grade V1 Test Structure Following Test 1.

Figure 4-3. Test 1 Post-Test Photograph of Grout Breakup.

(a) Grade V1 – Exterior.

(b) Grade V1 – Interior.

Figure 4-4. Test 1 Post-Test Photographs of First-Floor Front Panel.

(c) Grade E1 – Exterior.

(d) Grade E1 – Interior.

Figure 4-4. Test 1 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

(e) Grade V4 – Exterior.

(f) Grade V4 – Interior.

Figure 4-4. Test 1 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

4.1.2 Test 2

Test 2 was performed on the afternoon of October 12, 2016. Figure 4-5 show the elevations of the three test structures directly facing the explosive charge following Test 2.

Besides a few more knots popping out of exposed CLT panel plies, no signs of damage to or permanent deformation in the constituent panels of the CLT test structures were observed following Test 2. Further cracking and breaking up of the grout placed under the foundation angle was visible both from inside and outside of the test structures following Test 2 (Figure 4-6). Additionally, the sand bags retaining the door in its frame overturned as a result of door rebound during Test 2 (Figure 4-7).

Photographs of the post-test condition of the first-floor panel directly facing the charge are included as Figure 4-8.

Figure 4-5. Test 2 Post-Test Photograph of All Test Structures.

(a) Exterior.

(b) Interior.

Figure 4-6. Test 2 Post-Test Photograph of Grout Breakup.

Figure 4-7. Test 2 Post-Test Photograph of Sand Bag Overturning.

(a) Grade V1 – Exterior.

(b) Grade V1 – Interior.

Figure 4-8. Test 2 Post-Test Photographs of First-Floor Front Panel.

(c) Grade E1 – Exterior.

(d) Grade E1 – Interior.

Figure 4-8. Test 2 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

(e) Grade V4 – Exterior.

(f) Grade V4 – Interior.

Figure 4-8. Test 2 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

4.1.3 Test 3

Test 3 was performed on the morning of October 13, 2016. Figure 4-9 show the elevations of the three test structures directly facing the charge following Test 3.

Damage to both interior and exterior faces was observed in all three test structures following Test 3. Observable damage was primarily concentrated in the front panel facing the explosive charge.

Photographs of the post-test condition of the first-floor panel directly facing the charge from the exterior and interior are included as Figure 4-10. For the Grade V1 and Grade E1 test structures, noticeable damage was observed near mid-height and mid-width of the first-floor front panel on both the interior and exterior faces. On the other hand, most of the observable damage for the Grade V4 test structure was located on the interior face of the first-floor front panel, although there was minor damage observed on exterior face of this structure (Figure 4-11).

Figure 4-9. Test 3 Post-Test Photograph of All Test Structures.

(a) Grade V1 – Exterior.

(b) Grade V1 – Interior.

Figure 4-10. Test 3 Post-Test Photographs of First-Floor Front Panel.

(c) Grade E1 – Exterior.

(d) Grade E1 – Interior.

Figure 4-10. Test 3 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

(e) Grade V4 – Exterior.

(f) Grade V4 – Interior.

Figure 4-10. Test 3 Post-Test Photographs of First-Floor Front Panel. (Cont'd)

(a) Crack in Board.

(b) Finger Joint Crack.

Figure 4-11. Test 3 Post-Test Photographs of Grade V4 Test Structure Damage.

Although most of the damage to the CLT panels was concentrated in the first-floor front panel, localized damage was observed at various points throughout the rest of the structure. These areas are identified in photographs included as Figure 4-12.

(a) Grade V4 Test Structure Near Instrumentation Hole (back wall panel).

(b) Grade V4 Test Structure Near Door Frame.

(c) Grade E1 Test Structure Near Door Frame.

Figure 4-12. Test 3 Post-Test Photograph of Localized Damage Away from Front Panel.

Also, small pieces of debris were found on the inside of the Grade V4 test structure at the first floor following Test 3. Similar debris was not observed for the Grade V1 or Grade E1 test structures. Examples of this debris are shown in Figure 4-13.

(a) Grade V4 Test Structure – Many Small Pieces of CLT Panel Debris.

(b) Grade V4 Test Structure – Board Delamination.

(c) Grade V1 Test Structure (Grade E1 similar).

Figure 4-13. Test 3 Post-Test Photograph of Internal Debris.

All doors opened (in rebound) due to Test 3. Visible damage in the form of inelastic deformation of the door frame (Figure 4-14a) and rupture of the dimensional lumber restraints securing the door was observed in the test structures (Figure 4-14b).

(a) Door Frame Inelastic Deformation.

(b) 2x Restraint Rupture.

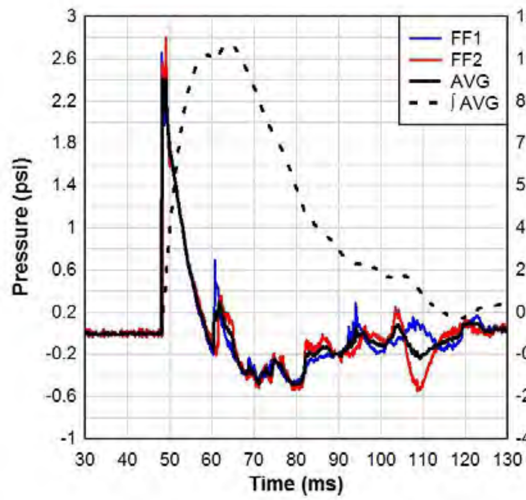
Figure 4-14. Test 3 Post-Test Photograph of Damage Near Door Frame.

4.2 RECORDED DATA

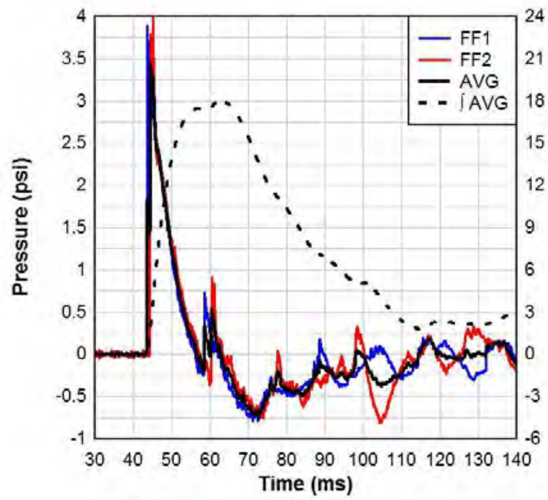
Pressure and panel displacement data was recorded using the instrumentation described in Chapter 3. All raw unfiltered pressure and displacement data recorded during the three tests is included in a Quick Look Report in Appendix D.

4.2.1 Pressure

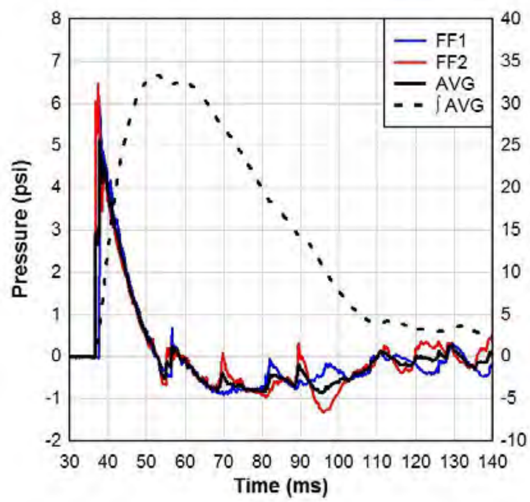
Figure 4-15 plots the recorded incident overpressure data (i.e., by gages FF1 and FF2) and the average of these two gages for each of the three tests.



(a) Test 1.



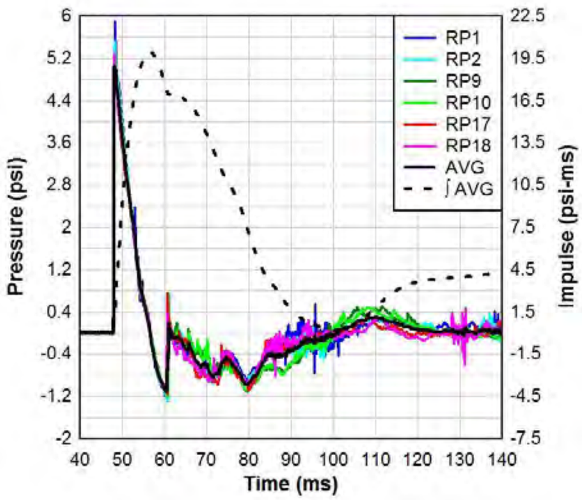
(b) Test 2.



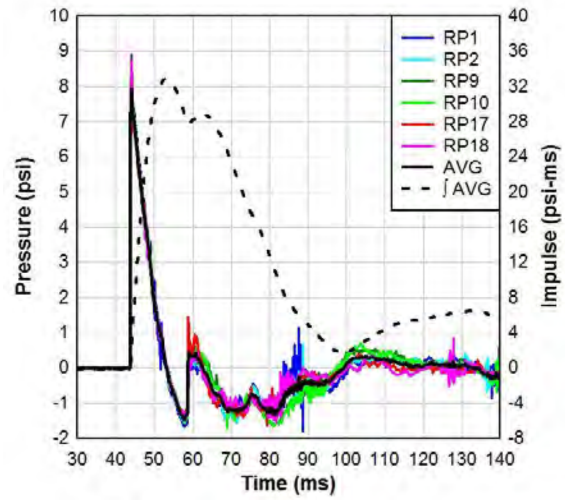
(c) Test 3.

Figure 4-15. Incident Overpressure Data.

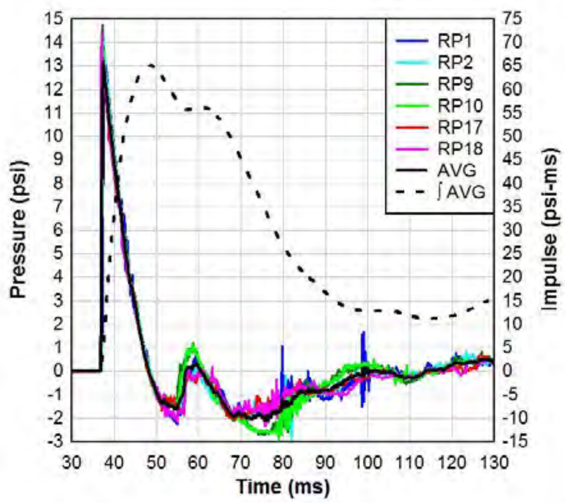
Similarly, Figure 4-16 plots the reflected pressure data recorded at the first-floor front panels (i.e., by gages RP1, RP2, RP9, RP10, RP17, and RP18) and the average of these six gages for each of the three tests. Plots of the remaining pressure histories are included in Appendix D.



(a) Test 1.



(b) Test 2.



(c) Test 3.

Figure 4-16. Reflected Pressure Data at First-Floor Front Panels.

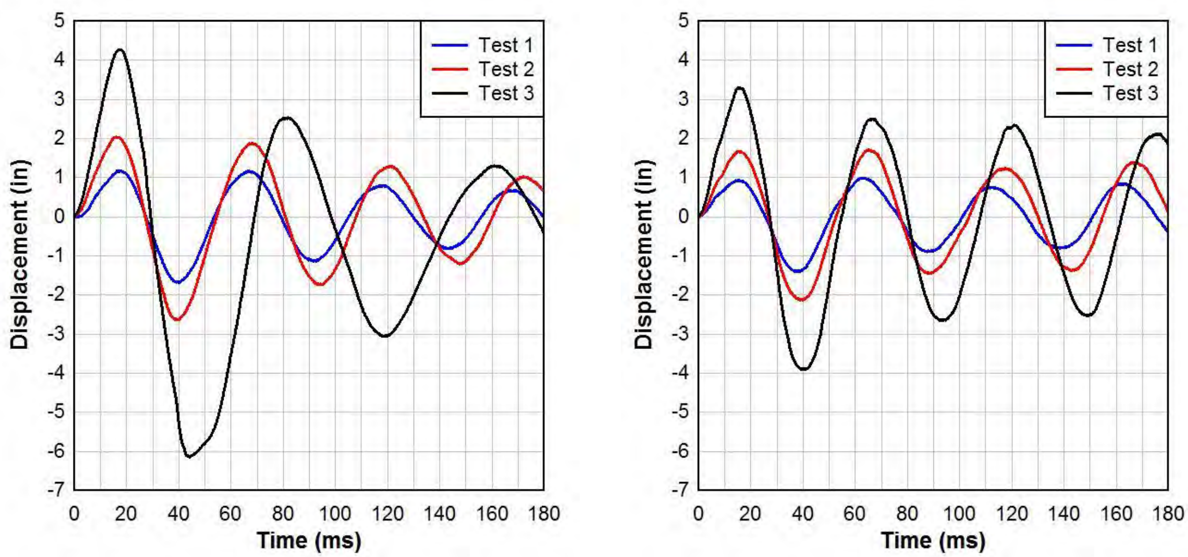
Table 5-1 provides a summary of the incident and peak reflected pressure positive phase data for all three shots. The values shown in Table 5-1 are generated based on the average curves shown in Figure 4-15 and Figure 4-16.

Table 4-1. Pressure Data Summary.

Test	Time of Arrival [ms]	Incident Overpressure [psi]	Incident Impulse [psi-ms]	Peak Reflected Pressure [psi]	Peak Reflected Impulse [psi-ms]
1	48.1	2.41	10.9	5.05	19.9
2	43.6	3.45	18.0	7.94	32.9
3	36.7	5.15	33.3	13.2	65.2

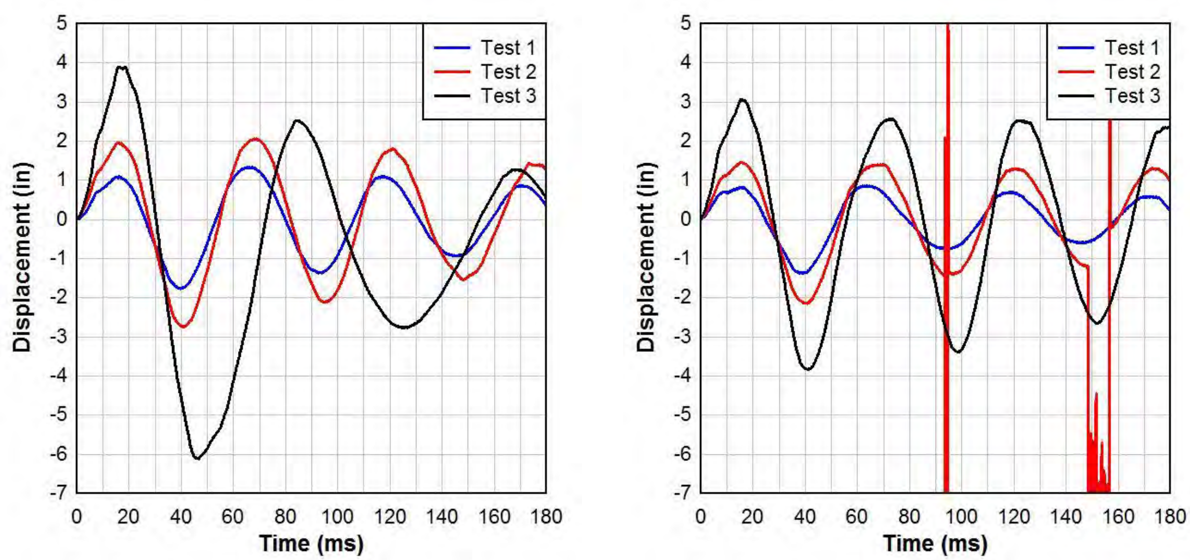
4.2.2 Displacement

Figure 4-17, Figure 4-18, and Figure 4-19 plot the recorded panel displacements at the front panel of the Grade V1, Grade E1, and Grade V4 test structures, respectively. Plots of the remaining displacement histories are included in Appendix D.



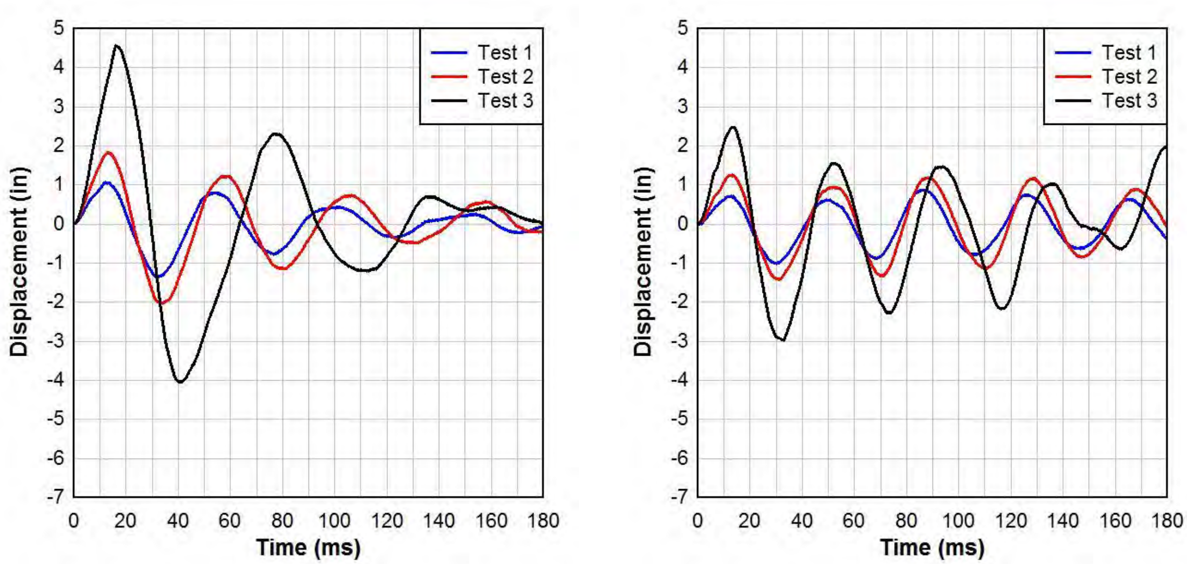
(a) 1st Floor Front Panel (DG2). *(b) 2nd Floor Front Panel (DG4).*

Figure 4-17. Displacement Data for Grade V1 Structure.



(a) 1st Floor Front Panel (DG13). *(b) 2nd Floor Front Panel (DG15).*

Figure 4-18. Displacement Data for Grade E1 Structure.



(a) 1st Floor Front Panel (DG24). *(b) 2nd Floor Front Panel (DG26).*

Figure 4-19. Displacement Data for Grade V4 Structure.

Table 4-2 provides a summary of the peak inbound and rebound displacements for nine locations on each test structure. The values shown in Table 4-2 are peak displacements for the first displacement cycle.

Table 4-2. Peak Displacement Data Summary.

Location	Test	Structure Grade: V1 Inbound [in]	V1 Rebound [in]	E1 Inbound [in]	E1 Rebound [in]	V4 Inbound [in]	V4 Rebound [in]
1st Floor Front (DG2, DG13, DG24)	1	1.18	-1.68	1.09	-1.77	1.07	-1.36
	2	2.04	-2.64	1.96	-2.75	1.83	-2.04
	3	4.28	-6.15	3.90	-6.12	4.57	-4.05
2nd Floor Front (DG4, DG15, DG26)	1	0.93	-1.41	0.83	-1.38	0.71	-1.02
	2	1.71	-2.13	1.47	-2.15	1.26	-1.42
	3	3.30	-3.91	3.07	-3.84	2.47	-2.98
1st Floor Side (DG5, DG19, DG30)	1	0.55	-0.84	0.62	-1.09	0.51	-0.57
	2	0.92	-1.18	1.01	-1.66	0.73	-0.86
	3	1.67	-1.94	1.97	-2.78	1.36	-1.45
2nd Floor Side (DG10, DG18, DG29)	1	0.51	-1.01	0.52	-0.96	0.41	-0.71
	2	1.33	-1.57	0.81	-1.52	0.65	-1.06
	3	1.46	-2.57	1.51	-2.79	1.09	-1.79
Window Jamb (DG6, DG20, DG31)	1	0.42	-0.53	0.47	-0.71	0.41	-0.44
	2	0.72	-0.83	0.77	-1.06	0.63	-0.68
	3	1.27	-1.53	1.50	-1.92	1.20	-1.12
Window Head (DG7, DG21, DG32)	1	0.65	-0.83	0.72	-0.89	0.36	-0.45
	2	1.07	-1.39	1.23	-1.52	0.56	-0.71
	3	1.98	-2.24	2.33	-2.88	1.10	-1.23
Door Jamb (DG8, DG16, DG27)	1	0.45	-0.61	0.45	-0.64	0.34	-0.48
	2	0.76	-0.95	0.76	-1.01	0.59	-0.69
	3	1.43	-1.52	1.43	-1.59	1.08	-1.15
Door Head (DG9, DG17, DG28)	1	0.60	-0.89	0.65	-0.96	0.29	-0.47
	2	1.06	-1.39	1.15	-1.54	0.54	-0.66
	3	2.06	-2.22	2.17	-2.56	1.10	-1.15
Roof (DG11, DG22, DG33)	1	0.57	-0.66	0.59	-0.65	0.63	-0.91
	2	0.92	-1.03	0.70	-0.81	1.02	-1.43
	3	1.47	-1.85	1.33	-1.62	1.83	-2.28

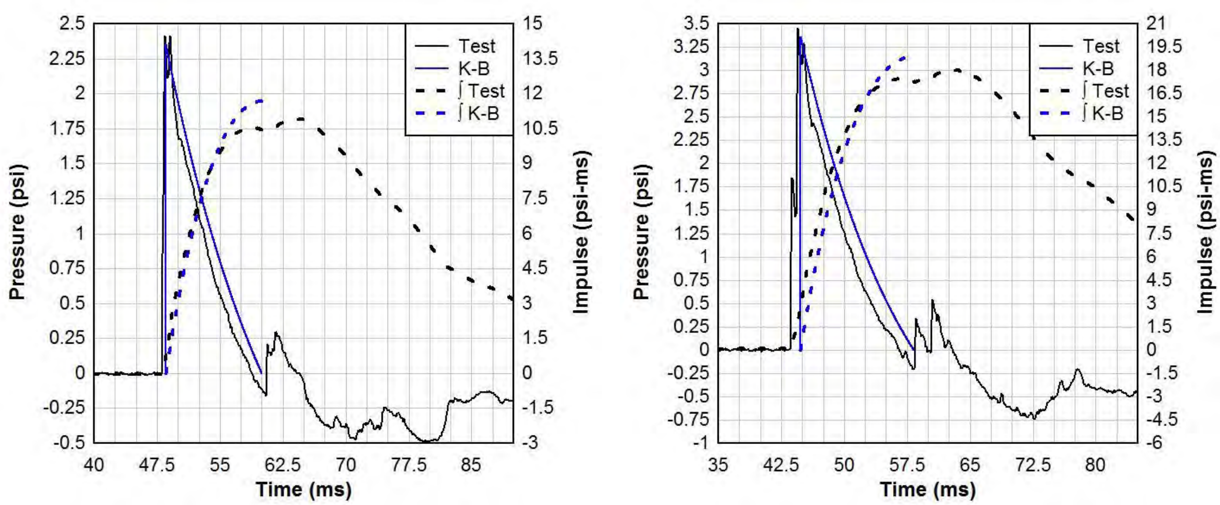
CHAPTER 5

TEST DATA COMPARISONS

This test data obtained from the three blast tests described herein is compared with analytical methods commonly used to design blast-resistant structures for airblast loading. The chapter opens with comparing the recorded airblast pressures with the Kingery-Bulmash equations [9]. Next, the recorded displacement response of the constituent panels of the CLT test structures is compared with idealized SDOF dynamic analysis calculations. The chapter is concluded by drawing conclusions concerning the use of these analytical models to design CLT structures for airblast loading.

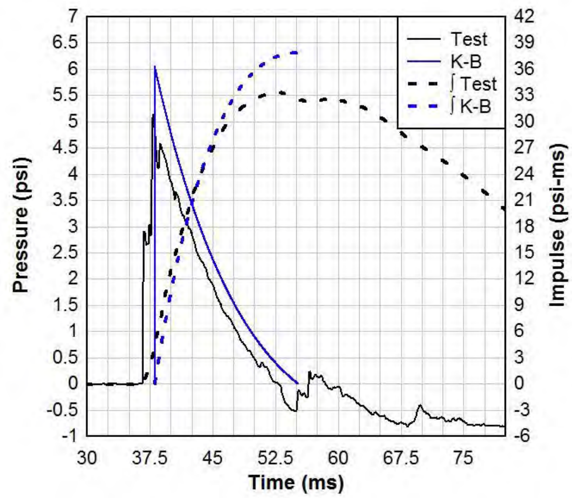
5.1 AIRBLAST LOADING

Figure 5-1 compares the average curve shown in Figure 4-15 with that generated using the Kingery-Bulmash (K-B) equations assuming an aboveground hemispherical surface burst. In general, the measured and computed data compare well in terms of peak pressure, positive phase impulse, and time of arrival.



(a) Test 1. *(b) Test 2.*

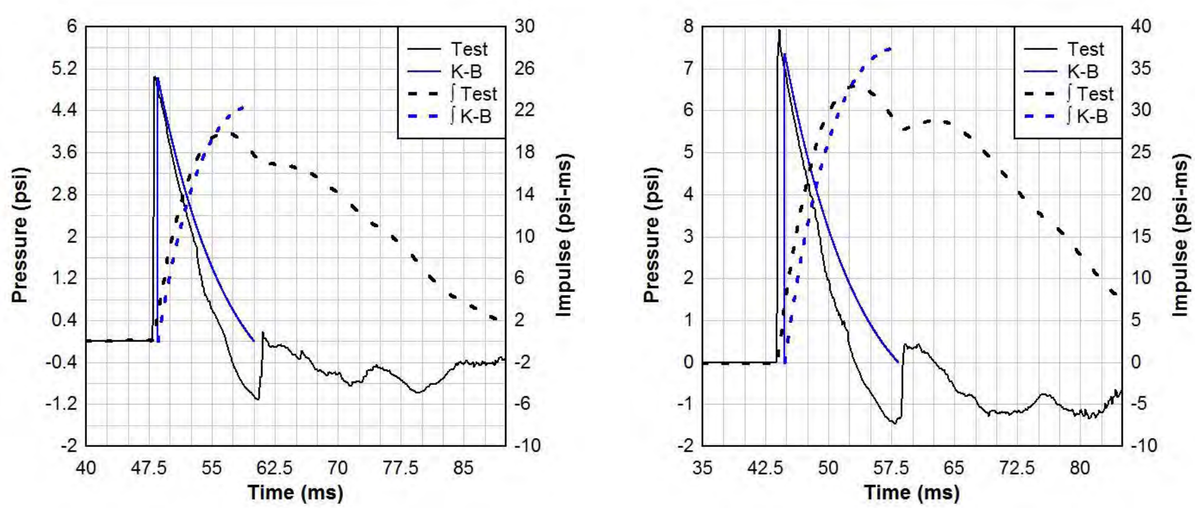
Figure 5-1. Incident Overpressure Data Comparisons.



(c) Test 3.

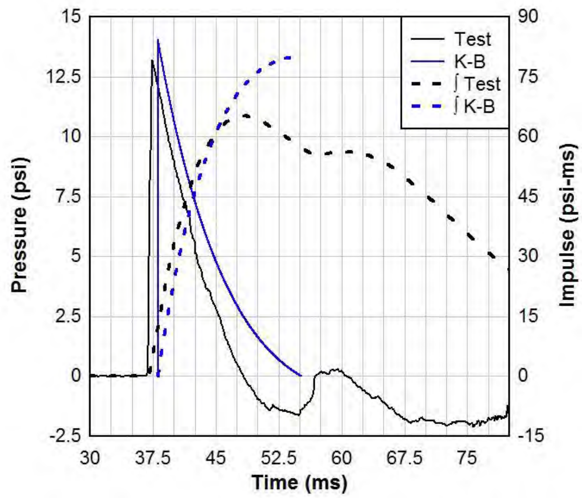
Figure 5-1. Incident Overpressure Data Comparisons. (Cont'd)

Figure 5-2 compares the average curve shown in Figure 4-16 with that computed using the K-B equations. In general, the measured and computed data compare well in terms of peak pressure and time of arrival for all shots. However, it is apparent that the positive phase impulses diverge by a noticeable margin. This divergence is likely due to clearing effects not being accounted for in the K-B-generated curve.



(a) Test 1. *(b) Test 2.*

Figure 5-2. Reflected Pressure Data Comparisons at First-Floor Front Panels.



(c) Shot 3.

Figure 5-2. Reflected Pressure Data Comparisons at First-Floor Front Panels. (Cont'd)

Table 5-1 provides a summary of the peak incident and reflected pressure positive phase data for all three shots.

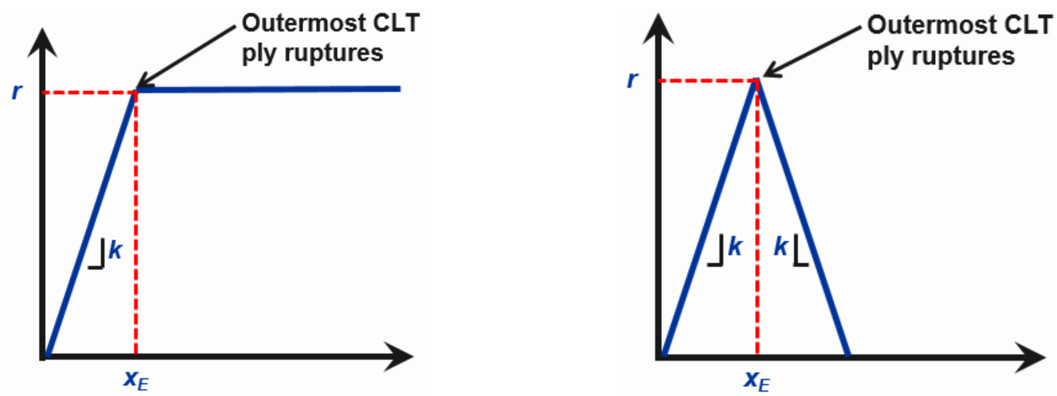
Table 5-1. Pressure Data Comparison with Kingery-Bulmash Equations.

Test	Time of Arrival [ms]		Incident Overpressure [psi]		Incident Impulse [psi-ms]		Peak Reflected Pressure [psi]		Peak Reflected Impulse [psi-ms]	
	Test[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]
1	48.1	48.6	2.41	2.35	10.9	11.7	5.05	5.03	19.9	22.5
2	43.6	44.8	3.45	3.37	18.0	18.9	7.94	7.36	32.9	37.5
3	36.7	38.1	5.15	6.06	33.3	37.9	13.2	14.1	65.2	80.0

[1] Taken from average curves shown in Figure 4-15 and Figure 4-16.
[2] As computed by the Kingery-Bulmash equations assuming aboveground hemispherical surface burst.

5.2 STRUCTURAL RESPONSE

A series of SDOF dynamic analyses were performed using the pressure histories recorded for each test and the background testing information described in Chapter 2. Two different resistance functions were employed: (1) elasto-plastic and (2) post-peak softening equal to the negative value of the elastic stiffness. These idealized resistance functions are shown in Figure 5-3.



(a) Elasto-Plastic (EP). *(b) Post-Peak Softening (SOFT).*

Figure 5-3. Idealized Resistance Functions used in SDOF Dynamic Analysis.

The following assumptions were employed in these analyses:

- The boundary conditions were idealized as follows:
  - End 1: Out-of-plane and in-plane translation restrained.
  - End 2: Out-of-plane translation restrained only.
- The parameters used to construct the resistance function (i.e., r, k, x_E) were computed using the shear analogy model and the characteristic values listed in Table 1 of ANSI/APA PRG 320-2012. This r value was increased by a dynamic increase factor of 1.25 (i.e., see Section 2.2.2) and the 0.85 conservatism reduction factor specified in Annex A of PRG 320 for bending strength was not applied.
- CLT panel density was assumed to be 35 pcf for all grades of CLT tested.
- The mass of the 3-ply CLT panel, window covering (i.e., two pieces of ¾-inch thick plywood), and door were assumed to be 12 psf, 4.5 psf, and 8 psf, respectively.
- The mass of used in the SDOF calculation. m, was assumed to be uniformly distributed over the airblast-load-applied-area (i.e., the product of L and b_{trib}).
- The width of panel used to resist airblast loads around openings, b_{eff}, was set equal to half the opening length but not greater than the distance from the edge of the opening to the nearest panel splice.

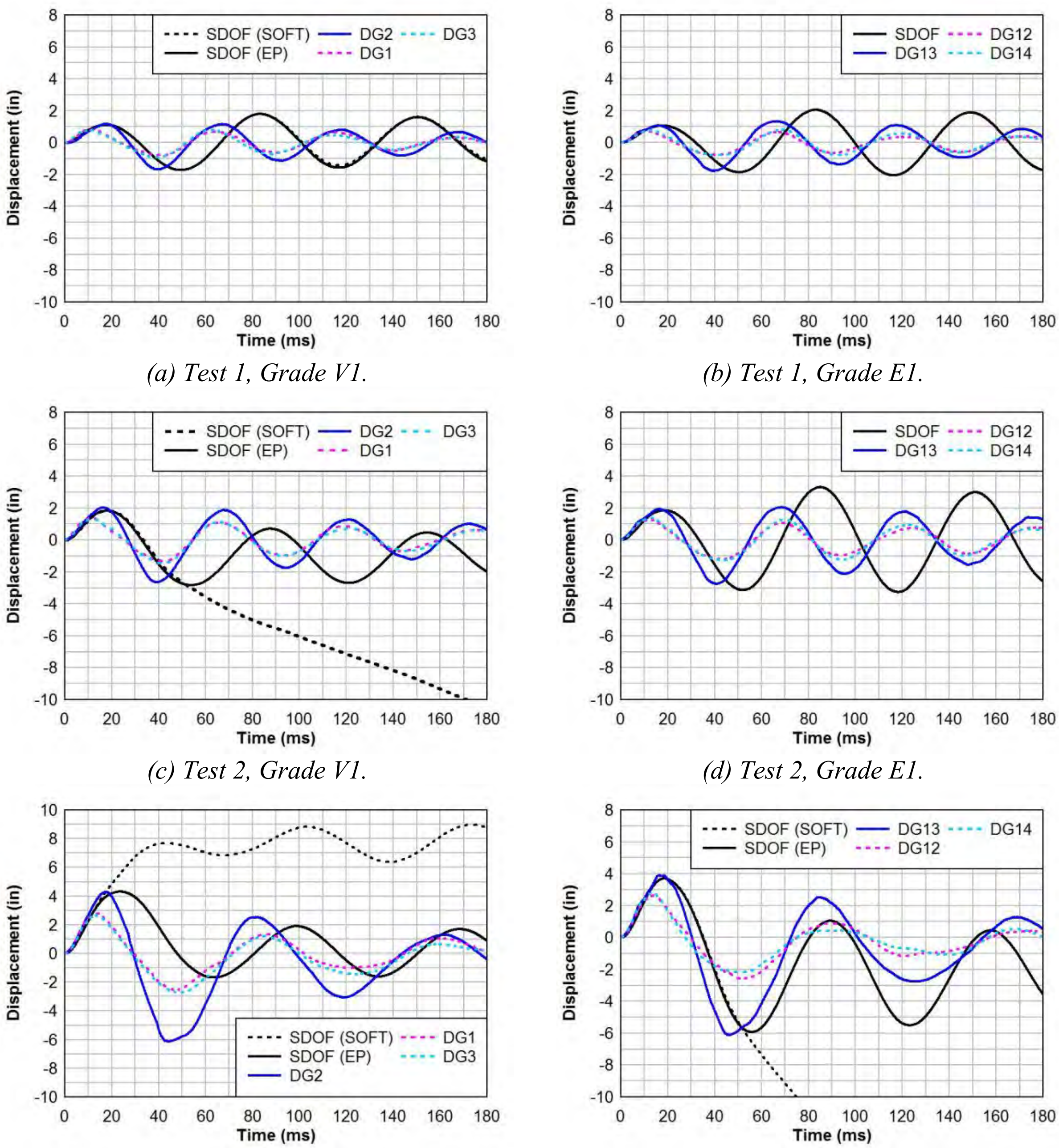
- Viscous damping was applied. The fraction of critical damping was assumed to be 2-percent.

The resulting SDOF dynamic analysis parameters for all cases considered based on the above assumptions are shown in Table 5-2.

Table 5-2. SDOF Dynamic Analysis Parameters.

Grade	DG	Blast Load	Description	L [ft]	b_{eff} [ft]	b_{trib} [ft]	m [psi-ms^2/in]	k [psi/in]	r [psi]	x_E [in]
V1	2	RP1-RP2 AVG	1st floor front	12.00	1.00	1.00	216.2	1.45	2.49	1.72
	4	RP3	2nd floor front	12.00	1.00	1.00	216.2	1.45	2.49	1.72
	5	RP4	1st floor left	12.00	1.00	1.00	216.2	1.45	2.49	1.72
	6	RP5	window jamb	12.00	1.75	3.50	196.5	0.72	1.25	1.72
	7	RP5	window head	3.50	1.75	2.63	171.1	4.80	1.31	0.27
	8	RP6	door jamb	12.00	1.81	3.50	195.1	0.75	1.29	1.72
	9	RP6	door head	3.38	1.69	2.53	192.1	5.53	1.41	0.26
	10	RP7	2nd floor right	12.00	1.00	1.00	216.2	1.45	2.49	1.72
	11	RP8	roof	13.67	1.00	1.00	216.2	0.88	1.92	2.18
E1	13	RP9-RP10 AVG	1st floor front	12.00	1.00	1.00	216.2	1.50	5.39	3.58
	15	RP11	2nd floor front	12.00	1.00	1.00	216.2	1.50	5.39	3.58
	16	RP12	door jamb	12.00	1.81	3.50	195.1	0.78	2.80	3.58
	17	RP12	door head	3.38	1.69	2.53	192.1	4.78	1.34	0.28
	18	RP13	2nd floor left	12.00	1.00	1.00	216.2	1.50	5.39	3.58
	19	RP14	1st floor right	12.00	1.00	1.00	216.2	1.50	5.39	3.58
	20	RP15	window jamb	12.00	1.75	3.50	196.5	0.75	2.70	3.58
	21	RP15	window head	3.50	1.75	2.63	171.1	4.14	1.25	0.30
	22	RP16	roof	13.67	1.00	1.00	216.2	0.92	4.16	4.52
V4	24	RP17-RP18 AVG	1st floor front	10.00	1.00	1.00	216.2	2.01	3.09	1.54
	26	RP19	2nd floor front	10.00	1.00	1.00	216.2	2.01	3.09	1.54
	27	RP20	door jamb	10.00	1.81	3.50	190.9	1.04	1.60	1.54
	28	RP20	door head	3.38	1.69	2.53	192.1	4.32	2.08	0.48
	29	RP21	2nd floor left	10.00	1.00	1.00	216.2	2.01	3.09	1.54
	30	RP22	1st floor right	10.00	1.00	1.00	216.2	2.01	3.09	1.54
	31	RP23	window jamb	10.00	1.75	3.50	192.5	1.00	1.55	1.54
	32	RP23	window head	3.50	1.75	2.63	171.1	3.75	1.94	0.52
	33	RP24	roof	13.67	1.00	1.00	216.2	0.61	1.65	2.70

Figure 5-4 and Figure 5-5 show comparisons of how the SDOF dynamic analysis results obtained using these resistance functions compared with the test data for the front panels for a CLT made up of visually graded lamella (i.e., Grade V1) and CLT made up of MSR lamella (i.e., Grade E1).

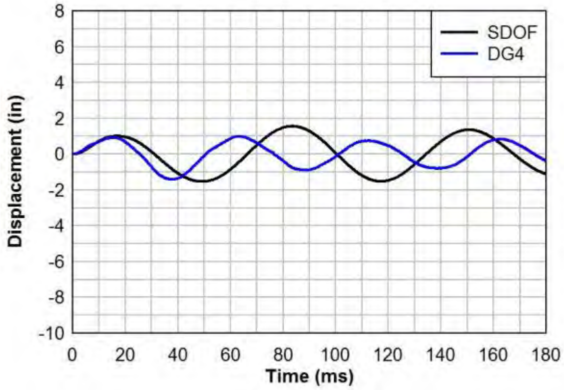


(a) Test 1, Grade V1. *(b) Test 1, Grade E1.*

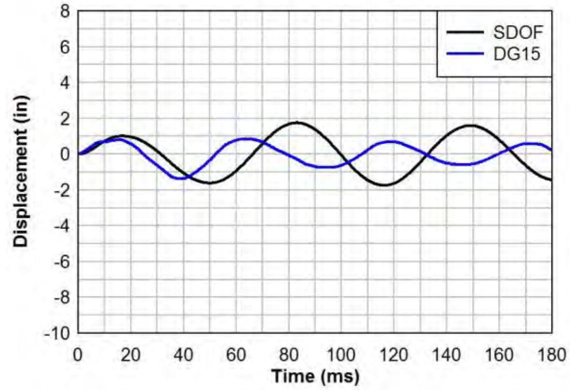
(c) Test 2, Grade V1. *(d) Test 2, Grade E1.*

(e) Test 3, Grade V1. *(f) Test 3, Grade E1.*

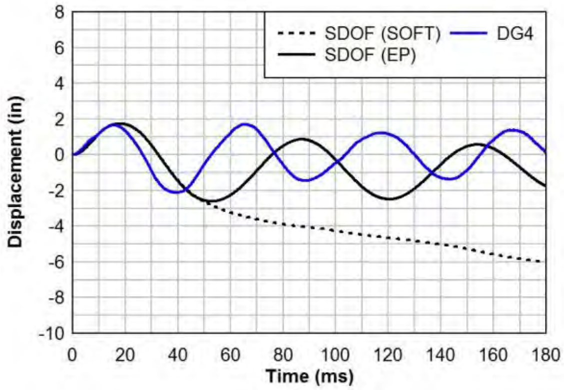
Figure 5-4. First-Floor Front Panel Displacement Comparisons.



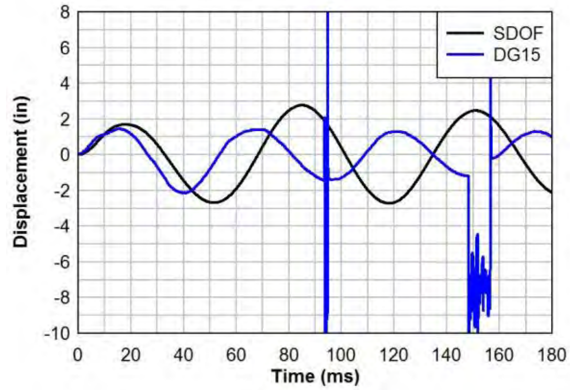
(a) Test 1, Grade V1.



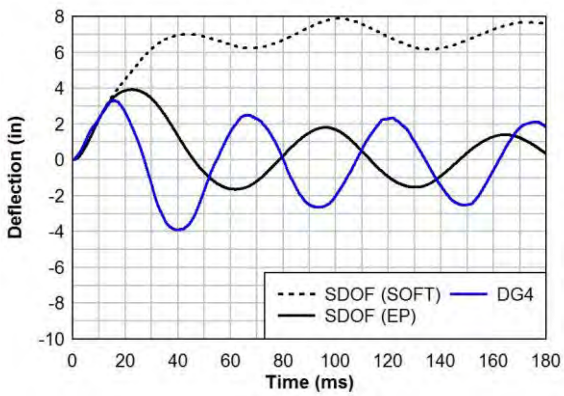
(b) Test 1, Grade E1.



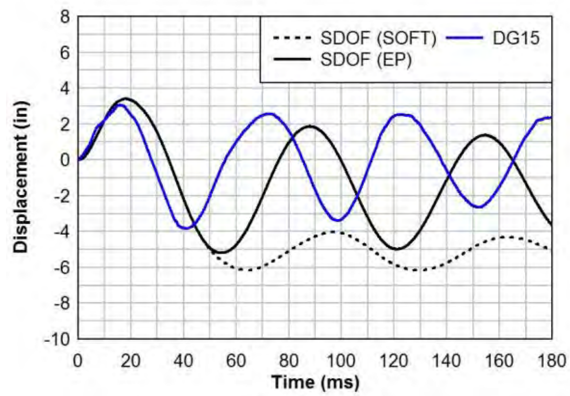
(c) Test 2, Grade V1.



(d) Test 2, Grade E1.



(e) Test 3, Grade V1.



(f) Test 3, Grade E1.

Figure 5-5. Second-Floor Front Panel Displacement Comparisons.

Table 5-3, Table 5-4, and Table 5-5 record the results of these SDOF dynamic analyses for the Grade V1, Grade E1, and Grade V4 test structures, respectively, and compare the computed values with those recorded in the tests. The elasto-plastic resistance function is used to compute the SDOF values included in these tables. Several notes are provided concerning the values placed in blue and red in the table:

- Where the difference between the test and computed displacement exceeded 20 percent of the test value, the difference percentage is highlighted in blue (i.e., the SDOF was at least

20 percent greater than the test value) or red (i.e., the SDOF was at least 20 percent less than the test value).

- Where the peak ductility, μ, associated with the SDOF analysis exceeded that computed for the first-floor front panel during Test 3 (i.e., the only panel and shot combination where actual rupture of the panels was observed), these values are highlighted in red.

Table 5-3. Grade V1 Test Structure Displacement Summary.

DG	Blast Load	Shot	1st Inbound Displacment				1st Rebound Displacment				μ[3]
			Test [in]	*SDOF*[1] *[in]*	*% Diff.*	μ_{in}[2]	*Test [in]*	*SDOF*[1] *[in]*	*% Diff.*	μ_{rb}[2]	
2	RP1-RP2 AVG	1	1.18	1.11	-6.1%	0.65	-1.68	-1.72	2.5%	1.00	**1.00**
		2	2.04	1.83	-10.2%	1.06	-2.64	-2.82	7.0%	1.64	**1.64**
		3	4.28	4.31	0.7%	2.51	-6.15	-1.68	-72.7%	0.98	**2.51**
4	RP3	1	0.93	1.01	7.9%	0.58	-1.41	-1.53	8.5%	0.89	**0.89**
		2	1.71	1.74	2.0%	1.01	-2.13	-2.61	22.5%	1.52	**1.52**
		3	3.30	3.92	18.7%	2.28	-3.91	-1.65	-57.7%	0.96	**2.28**
5	RP4	1	0.55	0.62	13.7%	0.36	-0.84	-0.92	9.5%	0.54	**0.54**
		2	0.92	1.01	9.3%	0.59	-1.18	-1.49	25.7%	0.86	**0.86**
		3	1.67	1.83	9.8%	1.07	-1.94	-2.46	26.8%	1.43	**1.43**
6	RP5	1	0.42	0.83	99.2%	0.48	-0.53	-1.25	137.0%	0.73	**0.73**
		2	0.72	1.41	96.6%	0.82	-0.83	-2.11	154.8%	1.23	**1.23**
		3	1.27	2.66	109.9%	1.55	-1.53	-2.69	76.5%	1.57	**1.57**
7	RP5	1	0.65	0.33	-49.3%	1.22	-0.83	-0.32	-61.2%	1.19	**1.22**
		2	1.07	0.66	-38.3%	2.44	-1.39	-0.06	-95.7%	0.22	**2.44**
		3	1.98	2.06	4.1%	7.63	-2.24	N/A[4]	N/A[4]	N/A[4]	**7.63**
8	RP6	1	0.45	1.10	142.9%	0.64	-0.61	-1.50	143.9%	0.87	**0.87**
		2	0.76	1.81	138.9%	1.05	-0.95	-2.36	148.7%	1.37	**1.37**
		3	1.43	3.56	149.2%	2.07	-1.52	-1.86	22.3%	1.08	**2.07**
9	RP6	1	0.60	0.36	-39.9%	1.38	-0.89	-0.49	-45.1%	1.88	**1.88**
		2	1.06	0.81	-23.5%	3.12	-1.39	-0.59	-57.7%	2.27	**3.12**
		3	2.06	2.73	32.5%	10.50	-2.22	N/A[4]	N/A[4]	N/A[4]	**10.50**
10	RP7	1	0.51	0.68	34.2%	0.40	-1.01	-1.05	4.0%	0.61	**0.61**
		2	1.33	1.14	-14.3%	0.66	-1.57	-1.66	5.8%	0.97	**0.97**
		3	1.46	2.04	39.0%	1.18	-2.57	-2.41	-6.1%	1.40	**1.40**
11	RP8	1	0.57	0.82	44.8%	0.38	-0.66	-1.00	52.4%	0.46	**0.46**
		2	0.92	1.32	42.8%	0.60	-1.03	-1.73	68.2%	0.79	**0.79**
		3	1.47	2.30	56.8%	1.06	-1.85	-3.21	73.4%	1.47	**1.47**

1 Elasto-plastic (EP) resistance function used for SDOF values shown in this table.
2 Ductility equal to the SDOF displacement divided by the corresponding x_E value in Table 5-2.
3 Maximum of μ_{in} and μ_{rb}.
4 No computed rebound displacement.

Table 5-4. Grade E1 Test Structure Displacement Summary.

DG	Blast Load	Shot	1st Inbound Displacment				1st Rebound Displacment				μ[3]
			Test [in]	*SDOF[1] [in]*	*% Diff.*	μ_{in}[2]	*Test [in]*	*SDOF[1] [in]*	*% Diff.*	μ_{rb}[2]	
13	RP9-RP10 AVG	1	1.09	1.06	-3.3%	0.30	-1.77	-1.85	4.6%	0.52	**0.52**
		2	1.96	1.85	-5.7%	0.52	-2.75	-3.14	14.2%	0.88	**0.88**
		3	3.90	3.70	-5.0%	1.03	-6.12	-5.93	-3.2%	1.66	**1.66**
15	RP11	1	0.83	1.01	21.8%	0.28	-1.38	-1.62	17.3%	0.45	**0.45**
		2	1.47	1.69	15.1%	0.47	-2.15	-2.69	25.1%	0.75	**0.75**
		3	3.07	3.40	10.5%	0.95	-3.84	-5.20	35.3%	1.45	**1.45**
16	RP12	1	0.45	1.07	139.1%	0.30	-0.64	-1.51	135.4%	0.42	**0.42**
		2	0.76	1.80	138.3%	0.50	-1.01	-2.51	148.2%	0.70	**0.70**
		3	1.43	3.27	129.3%	0.91	-1.59	-4.70	196.0%	1.31	**1.31**
17	RP12	1	0.65	0.39	-40.3%	1.39	-0.96	-0.63	-34.6%	2.25	**2.25**
		2	1.15	0.90	-22.1%	3.21	-1.54	-0.66	-57.3%	2.36	**3.21**
		3	2.17	2.91	34.1%	10.39	-2.56	N/A[4]	N/A[4]	N/A[4]	**10.39**
18	RP13	1	0.52	0.67	29.8%	0.19	-0.96	-1.03	6.8%	0.29	**0.29**
		2	0.81	1.11	36.4%	0.31	-1.52	-1.64	7.5%	0.46	**0.46**
		3	1.51	2.02	33.6%	0.57	-2.79	-2.85	1.9%	0.80	**0.80**
19	RP14	1	0.62	0.73	17.4%	0.20	-1.09	-1.18	8.6%	0.33	**0.33**
		2	1.01	1.18	17.3%	0.33	-1.66	-1.84	10.9%	0.51	**0.51**
		3	1.97	2.19	10.9%	0.61	-2.78	-3.14	12.8%	0.88	**0.88**
20	RP15	1	0.47	0.89	90.5%	0.25	-0.71	-1.28	79.9%	0.36	**0.36**
		2	0.77	1.49	92.6%	0.42	-1.06	-2.16	104.7%	0.60	**0.60**
		3	1.50	2.75	83.3%	0.77	-1.92	-4.03	110.3%	1.13	**1.13**
21	RP15	1	0.72	0.39	-45.5%	1.30	-0.89	-0.45	-49.6%	1.50	**1.50**
		2	1.23	0.79	-35.9%	2.63	-1.52	-0.21	-86.2%	0.70	**2.63**
		3	2.33	2.52	8.1%	8.40	-2.88	N/A[4]	N/A[4]	N/A[4]	**8.40**
22	RP16	1	0.59	0.82	38.8%	0.18	-0.65	-1.03	57.9%	0.23	**0.23**
		2	0.70	1.29	85.3%	0.29	-0.81	-1.78	118.9%	0.39	**0.39**
		3	1.33	2.34	75.7%	0.52	-1.62	-3.37	107.6%	0.75	**0.75**

[1] Elasto-plastic (EP) resistance function used for SDOF values shown in this table.
[2] Ductility equal to the SDOF displacement divided by the corresponding x_E value in Table 5-2.
[3] Maximum of μ_{in} and μ_{rb}.
[4] No computed rebound displacement.

Table 5-5. Grade V4 Test Structure Displacement Summary.

DG	Blast Load	Shot	1st Inbound Displacment				1st Rebound Displacment				μ[3]
			Test [in]	*SDOF[1] [in]*	*% Diff.*	μ_{in}[2]	*Test [in]*	*SDOF[1] [in]*	*% Diff.*	μ_{rb}[2]	
24	RP17-RP18 AVG	1	1.07	0.93	-13.2%	0.60	-1.36	-1.47	8.2%	0.95	**0.95**
		2	1.83	1.56	-14.9%	1.01	-2.04	-2.44	19.8%	1.58	**1.58**
		3	4.57	3.64	-20.4%	2.36	-4.05	-1.03	-74.6%	0.67	**2.36**
26	RP19	1	0.71	0.89	24.3%	0.58	-1.02	-1.34	32.0%	0.87	**0.87**
		2	1.26	1.47	17.1%	0.96	-1.42	-2.22	56.2%	1.44	**1.44**
		3	2.47	3.40	37.4%	2.21	-2.98	-1.10	-63.0%	0.72	**2.21**
27	RP20	1	0.34	0.95	176.5%	0.62	-0.48	-1.47	203.5%	0.95	**0.95**
		2	0.59	1.55	163.4%	1.01	-0.69	-2.39	248.4%	1.55	**1.55**
		3	1.08	3.17	192.8%	2.06	-1.15	-1.53	32.6%	0.99	**2.06**
28	RP20	1	0.29	0.41	40.6%	0.85	-0.47	-0.73	56.1%	1.52	**1.52**
		2	0.54	0.66	22.3%	1.38	-0.66	-0.90	36.9%	1.88	**1.88**
		3	1.10	1.95	77.6%	4.06	-1.15	-0.17	-85.2%	0.35	**4.06**
29	RP21	1	0.41	0.59	44.4%	0.38	-0.71	-0.96	34.6%	0.62	**0.62**
		2	0.65	0.96	48.1%	0.62	-1.06	-1.48	39.0%	0.96	**0.96**
		3	1.09	1.79	64.2%	1.16	-1.79	-2.08	15.8%	1.35	**1.35**
30	RP22	1	0.51	0.55	8.0%	0.36	-0.57	-0.77	33.9%	0.50	**0.50**
		2	0.73	0.88	21.1%	0.57	-0.86	-1.16	35.5%	0.76	**0.76**
		3	1.36	1.65	21.6%	1.07	-1.45	-1.65	13.7%	1.07	**1.07**
31	RP23	1	0.41	0.75	84.8%	0.49	-0.44	-1.10	147.3%	0.71	**0.71**
		2	0.63	1.24	96.5%	0.81	-0.68	-1.82	166.1%	1.18	**1.18**
		3	1.20	2.36	96.8%	1.53	-1.12	-2.31	106.8%	1.50	**1.53**
32	RP23	1	0.36	0.39	7.1%	0.75	-0.45	-0.52	14.5%	1.00	**1.00**
		2	0.56	0.60	6.7%	1.15	-0.71	-0.55	-22.4%	1.06	**1.15**
		3	1.10	1.56	41.8%	3.00	-1.23	N/A[4]	N/A[4]	N/A[4]	**3.00**
33	RP24	1	0.63	0.85	33.9%	0.31	-0.91	-1.20	32.2%	0.45	**0.45**
		2	1.02	1.45	41.4%	0.54	-1.43	-2.13	48.7%	0.79	**0.79**
		3	1.83	2.74	49.7%	1.02	-2.28	-4.50	97.1%	1.67	**1.67**

[1] Elasto-plastic (EP) resistance function used for SDOF values shown in this table.
[2] Ductility equal to the SDOF displacement divided by the corresponding x_E value in Table 5-2.
[3] Maximum of μ_{in} and μ_{rb}.
[4] No computed rebound displacement.

5.3 OBSERVATIONS

The following general observations are made based on the above comparisons and the visual observations recorded in Chapter 4:

(1) In general, the SDOF dynamic analyses predict a displacement that exceeds that measured in the test. Two notable exceptions to this rule are:

   a. *Above openings*: See (4) below for more commentary concerning this location.

   b. *At front panels for Test 3 on the V-grade structures*: Due to the high coefficient of variation associated with the bending strength of V-grade CLT (see Section 2.1.2), its characteristic bending strength is significantly smaller than its average bending strength. Thus, the SDOF calculations poorly approximate the response of the V-grade CLT panels to airblast loading when the panel ruptures or is on the verge of rupturing.

(2) Figure 5-4 and Figure 5-5 indicate small discrepancies in the test and computed fundamental period values. It appears these discrepancies are more pronounced for the Grade V1 panels and when the SDOF calculation predicts a ductility greater than one. These discrepancies can be due to several factors: (1) poor approximation of panel mass, (2) simplified and idealized boundary conditions, and (3) ignoring the effect of axial load on the stiffness of the panel.

(3) In many cases, the rebound response exceeds the inbound response. This response is not unexpected with lightweight systems exposed to far-field airblast loads and displaced either within or shortly beyond their elastic limit.

(4) The SDOF dynamic analysis is clearly a coarse approximation for the truly multi-degree-of-freedom interaction found at openings. The SDOF dynamic analysis does not account for the flexibility of the jamb in the head/sill calculations, thus generally underpredicting the peak displacement with this condition. Also, applying the airblast load over the entire tributary area of the jamb instantaneously is conservative and yields much larger jamb displacements than recorded in the tests.

(5) Although minimal damage was observed in all panels except for the first-floor front panels following Test 3, ductility ratios often exceed one in the SDOF dynamic analyses. Reasons for this apparent contradiction include:

   a. The panels are stronger than the characteristic (i.e., 5-percent exclusion) values in PRG 320, particularly the visually graded panels (see Section 2.1.2).

   b. Two-way action and panel fixity (i.e., see roof panel connection in Section 3.1.3.4) serve to augment panel strength.

   c. For minor strength direction bending (i.e., at door opening head and window opening head and sill, the strength of the panel prescribed by PRG 320 only considers the middle ply for a 3-ply panel. While this approximation is perhaps appropriate for small displacements because crosswise boards are not necessarily in firm contact, for an ultimate load state brought about by airblast loading, it is possible these boards will be in contact and thus transfer compression forces, increasing the depth of the lever arm, and significantly increasing the moment strength of the panel.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

A series of live blast tests was performed on three two-story, single-bay CLT structures at Tyndall Air Force Base. The structures, included anchorage to an existing concrete slab, were constructed in full over a period of eight days. Each structure was constructed using a different grade of CLT (i.e., grade designations V1, E1, and V4) and included window and door openings consistent with an actual building. Self-tapping screws and adhesive anchors were utilized in concert with steel angles to connect the constituent panels of each structure.

Three shots were performed to demonstrate the effectiveness of CLT over a spectrum of airblast loads. The first two shots were designed to stress the CLT structures within their respective elastic limits. The third shot was designed to push the structures beyond their elastic limits such that post-peak response could be observed. Reflected pressure and peak displacements were recorded at front, side, and roof faces using a total of sixty-two gages to thoroughly document the response of the structures in time.

For the first two tests, peak recorded displacements were consistent with pre-test predictions indicating the efficacy of the design assumptions and methodology in predicting elastic response of CLT to dynamic loads. Furthermore, results from the third test indicated a controlled response in which localized panel rupture was observed but connection integrity and load carrying ability were not compromised for each of the three structures tested.

6.2 CONCLUSIONS

Based on the results of this testing effort, the following general conclusions are made:

- The rebound response of CLT often controls over its inbound response, thus underlying the importance of considering the negative phase of the airblast loading when designing CLT components and systems for airblast loading.

- Visually graded CLT panels demonstrate significantly greater out-of-plane bending strength than that associated with the characteristic values defined in PRG 320.

- Localized CLT panel rupture can be sustained without adverse consequences to the CLT system’s connections and load carrying ability. Further testing can be used to investigate the impact of localized CLT panel rupture for different conditions (e.g., different in-plane axial loads, different connection configurations, etc.).

- An SDOF dynamic analysis can be used to approximate peak displacements in 3-ply CLT panels without openings provided the mean out-of-plane strength of the CLT panel can be approximated.

- An SDOF dynamic analysis is not well-suited to approximate peak displacements in CLT panels with openings. A more refined analytical model with more degrees of freedom is necessary to approximate peak displacements in these circumstances.

- The minor strength direction bending strength values for 3-ply CLT panels in Annex A of PRG 320 may be too conservative from an ultimate response perspective. Further testing to justify more representative peak bending strengths in the minor strength direction may allow for airblast-loaded structures to be designed more economically.

APPENDIX A

REFERENCES

[1] ANSI/APA PRG 320-2012, "Standard for Performance-Rated Cross-Laminated Timber", The Engineered Wood Association, 2012.

[2] UFC 4-010-01, "DoD Minimum Antiterrorism Standards for Buildings, with Change 1", U.S. Army Corps of Engineers, 2013.

[3] Nagy, E. and M.K. Weaver, "Quasi-Static Out-of-Plane Testing of CLT and NLT Panels", Karagozian & Case, Inc., Glendale, CA, Report No. TR-16-42.1, 2016.

[4] Gagnon, S. and M. Popovski, "Structural Design of Cross-Laminated Timber Elements", Chapter 3 in *CLT Handbook*, FPInnovations, Canada, 2011.

[5] Lowak, M.J., "Static and Dynamic Testing of Cross-Laminated Timber Panels", Baker Engineering and Risk Consultants, Inc., San Antonio, TX, Project No. 01-05261-001-15, 2015.

[6] Lowak, M.J., "Additional Static and Dynamic Testing of Cross-Laminated Timber Panels", Baker Engineering and Risk Consultants, Inc., San Antonio, TX, Project No. 01-05261-002-15, 2015.

[7] ANSI/AWC NDS-2015, "National Design Specification for Wood Construction, 2015 Edition", American Wood Council, 2014.

[8] UFC 4-010-02, "DoD Minimum Antiterrorism Standoff Distances for Buildings", U.S. Army Corps of Engineers, 2012. *(Limited Distribution)*

[9] Kingery, N.C. and G. Bulmash, "Airblast Parameters from TNT Spherical Air Burst and Hemispherical Surface Burst", U.S. Army Ballistic Research Laboratory, Aberdeen Proving Ground, MD, Report No. ARBRL-TR-02555, 1984. *(Limited Distribution)*

APPENDIX B

CONSTRUCTION DRAWINGS OF CLT TEST STRUCTURES

GENERAL NOTES

GENERAL

1. THE CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS AND DIMENSIONS AT THE SITE BEFORE STARTING WORK.
2. TYPICAL DETAILS AND GENERAL NOTES ARE APPLICABLE UNLESS OTHERWISE DETAILED OR NOTED ON THE DRAWINGS.
3. EXISTING CONDITIONS AS SHOWN ON THE DRAWINGS ARE FOR REFERENCE ONLY. CONTRACTOR IS REQUIRED TO VERIFY ALL EXISTING CONDITIONS AND DIMENSION DURING FIELD SURVEYS.
4. THE CONTRACTOR IS RESPONSIBLE FOR ALL TEMPORARY SHORING AND BRACING.
5. PROVIDE NON-SHRINK GROUT IN ACCORDANCE WITH ASTM C1107.

STRUCTURAL STEEL:

1. STRUCTURAL STEEL SHALL CONFORM TO ASTM A36.
2. MATERIALS AND WORKMANSHIP SHALL CONFORM TO THE SPECIFICATIONS OF THE AMERICAN INSTITUTE OF STEEL CONSTRUCTION.
3. USE HILTI HY-200R ADHESIVE BY HILTI WITH ASTM A36 THREADED ROD WHERE "POST-INSTALLED ADHESIVE ANCHOR" IS INDICATED ON THE DRAWINGS. INSTALL IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS. NUTS SHALL BE IN ACCORDANCE WITH ASTM A563, GRADE A, HEAVY HEX, AND WASHERS SHALL BE IN ACCORDANCE WITH ASTM F436.

WOOD FRAMING:

1. CLT STRUCTURE SPECIMENS SHALL BE CONSTRUCTED WITH THE ANSI/APA PRG 320-2012 GRADE INDICATED BELOW:

A. #1: GRADE VI BY DR JOHNSON.
B. #2: GRADE E1 BY NORDIC.
C. #3: GRADE V4 BY SMARTLAM.

2. FRAMING LUMBER SHALL BE DOUGLAS FIR LARCH, GRADE MARKED No. 2 OR APPROVED EQUAL.
3. PLYWOOD SHALL BE GRADE MARKED STRUCTURAL I. ORIENTED STRAND BOARD (OSB) MAY BE SUBSTITUTED FOR PLYWOOD. OSB SHALL HAVE THE SAME PANEL SPAN RATING AND SHALL BE OF THE SAME THICKNESS AS THE SPECIFIED PLYWOOD. ALL PLYWOOD/OSB SHALL BE BONDED WITH EXTERIOR GLUE.
4. UNLESS NOTED OTHERWISE, USE ASSY SK SCREWS BY MYTICON FOR STEEL-TO-WOOD CONNECTIONS AND ASSY ECO FOR WOOD-TO-WOOD CONNECTIONS WITH THE DIAMETER & LENGTH AS INDICATED.
5. SCREWS SHALL BE INSTALLED AT LEAST ONE INCH FROM CENTER OF SCREW TO CRACKS, CHECKS, OR GAPS IN OUTER PLY OF CLT. INSTALLER SHALL CONSULT WITH DESIGNER WHERE THIS IS NOT POSSIBLE.

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700 North Brand Blvd., Suite 700
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Tel: 818-240-1919 Fax: 818-240-4966
—
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Mark	Date	Appr.	Description
2	09-19-2016	MW	MISC REVISIONS
1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

GENERAL NOTES

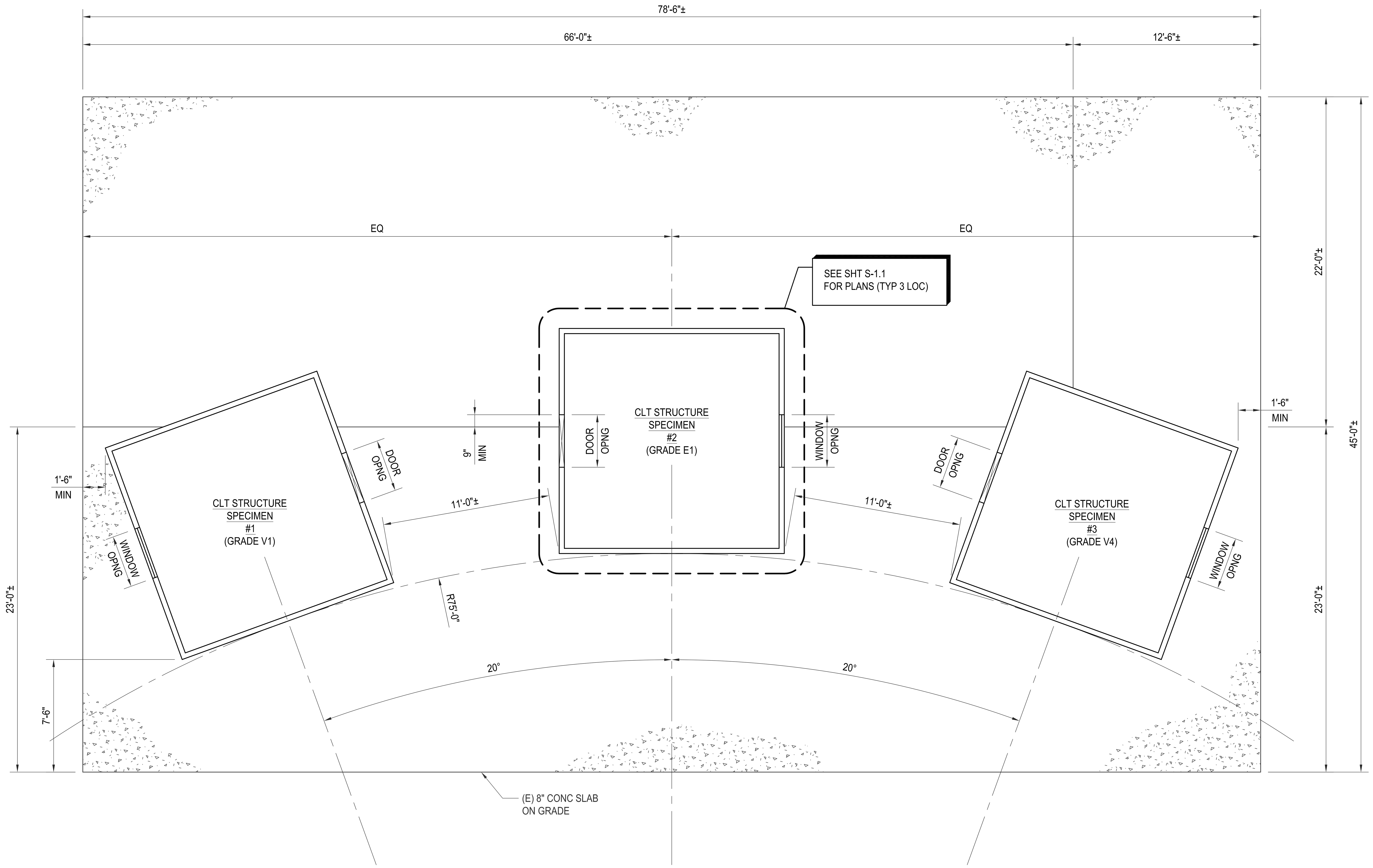
DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

JOB NO.	FILE NAME:
2271_2015	S001_2271.DWG

SHEET NUMBER

S-0.1

May 23, 2017



A
S-1.0
SITE PLAN
SCALE: 1/4"=1'-0"

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Mark	Date	Appr.	Description
2	09-19-2016	MW	MISC REVISIONS
1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

SITE PLAN

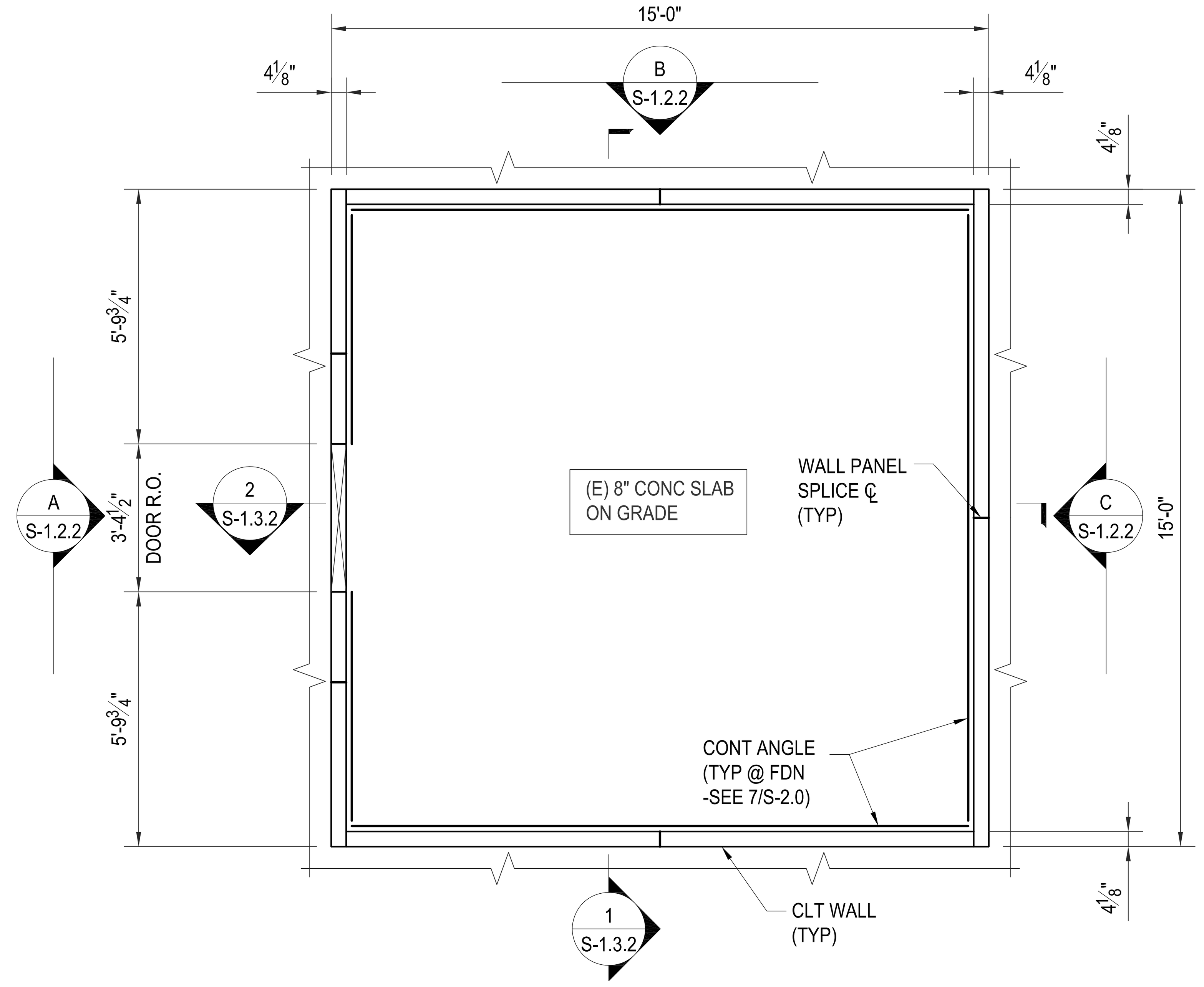
DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

JOB NO.	FILE NAME:
2271_2015	S100_2271.DWG

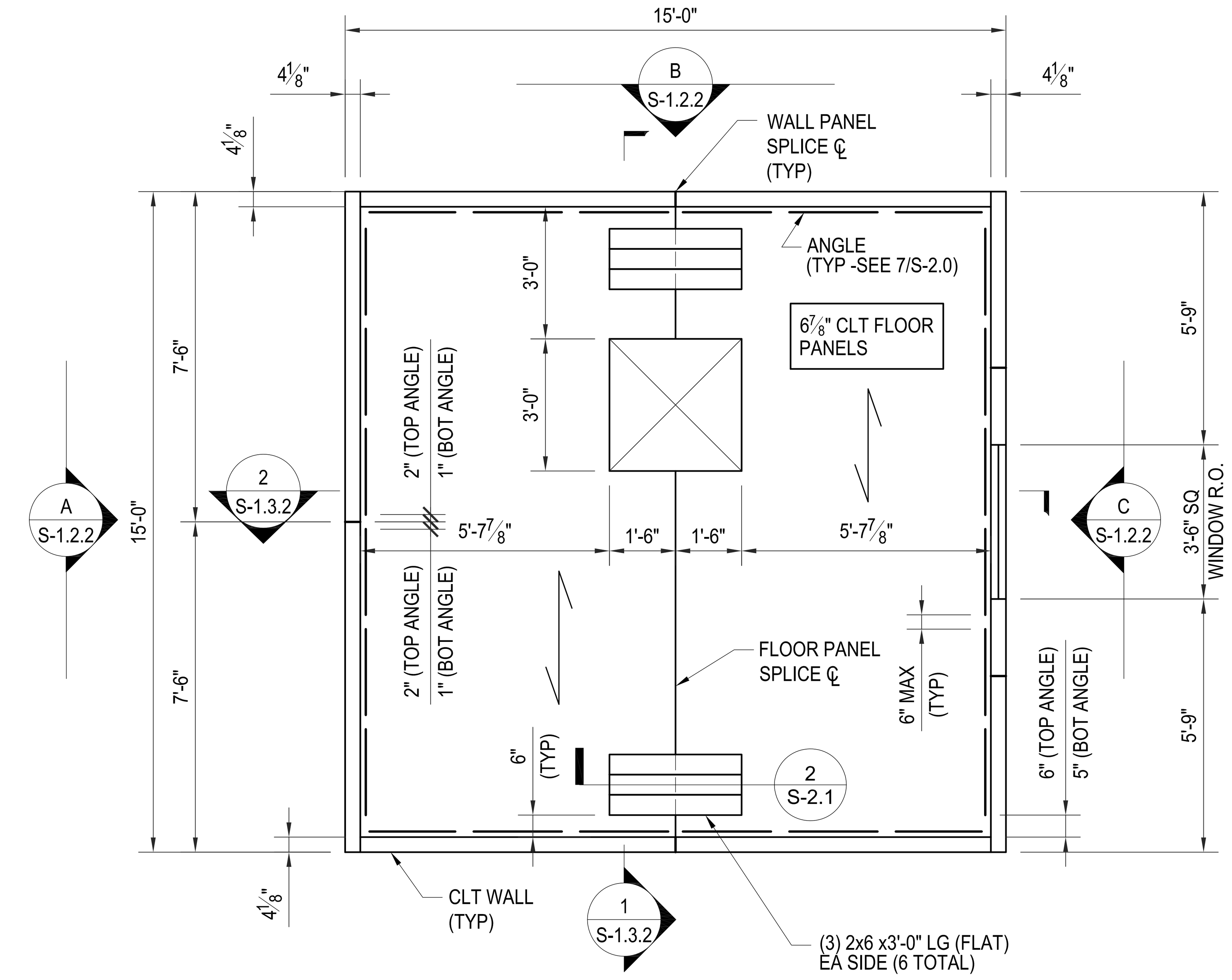
SHEET NUMBER

S-1.0

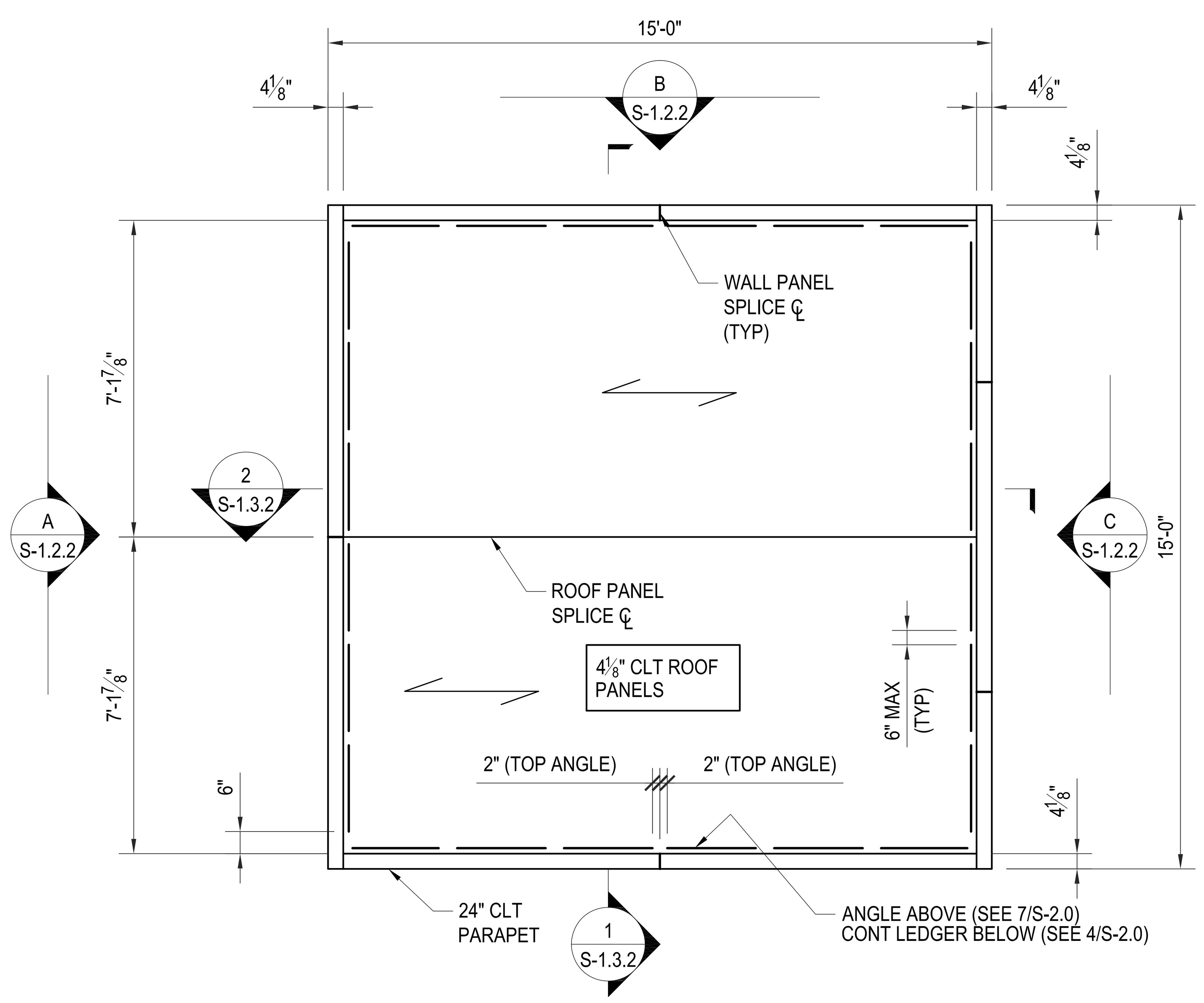
May 23, 2017



A S-1.1 GROUND FLOOR PLAN
SCALE: 3/8"=1'-0"



B S-1.1 SECOND FLOOR PLAN
SCALE: 3/8"=1'-0"



C S-1.1 ROOF PLAN
SCALE: 3/8"=1'-0"

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Mark	Date	Appr.	Description
3	05-24-2017	MW	MISC REVISIONS
2	09-19-2016	MW	MISC REVISIONS
1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

PLANS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

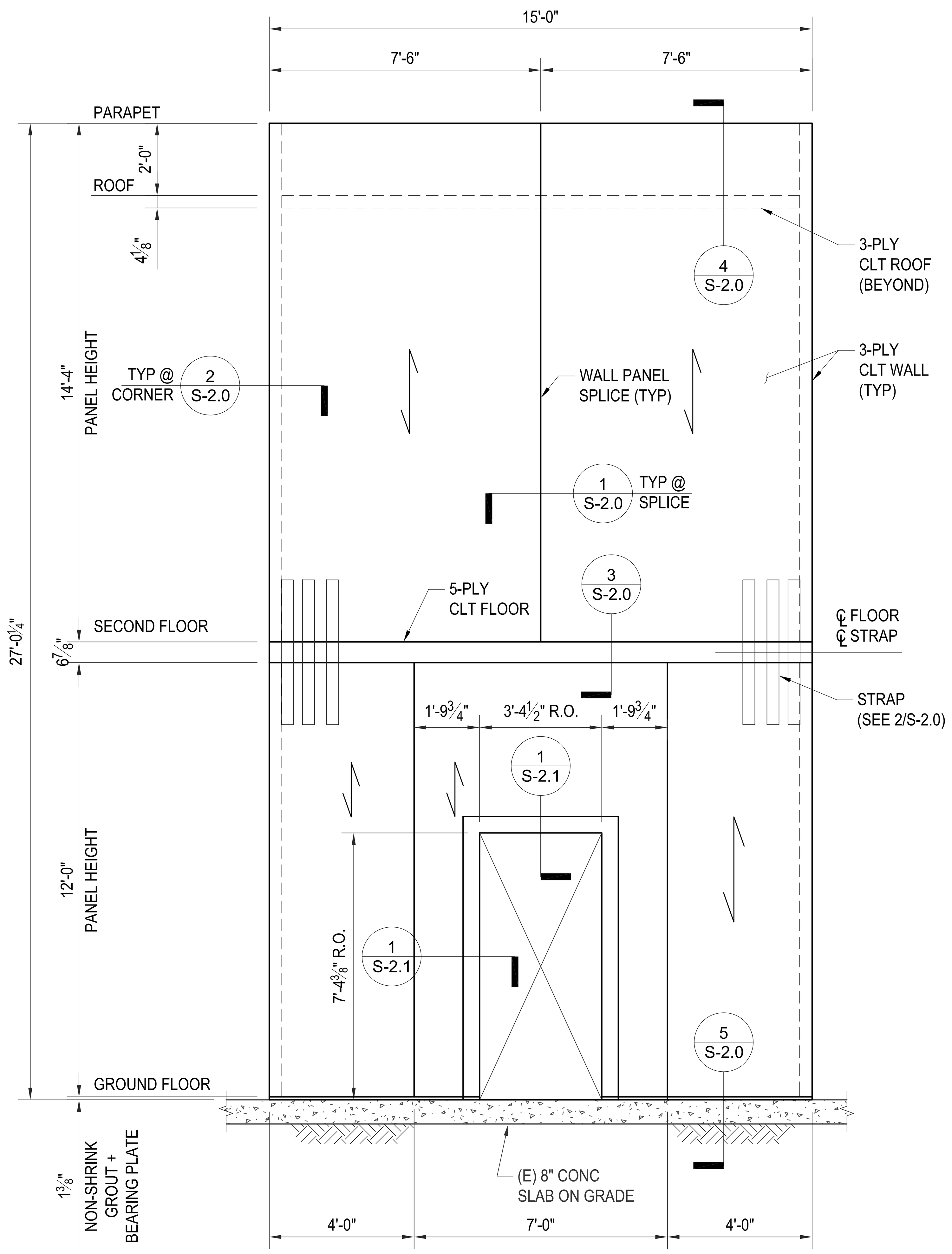
JOB NO. 2271_2015

FILE NAME: S101_2271.DWG

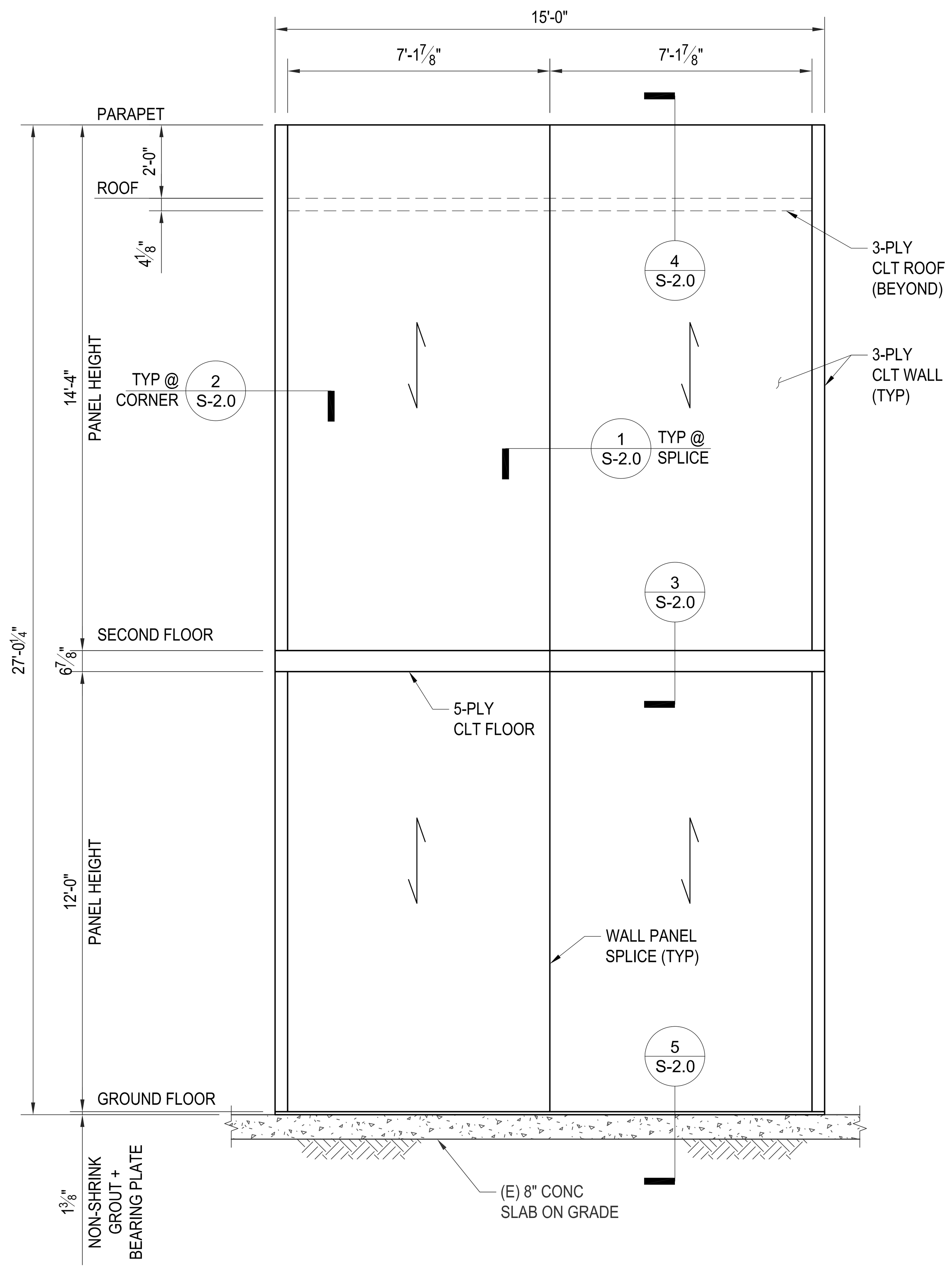
SHEET NUMBER

S-1.1 (3)

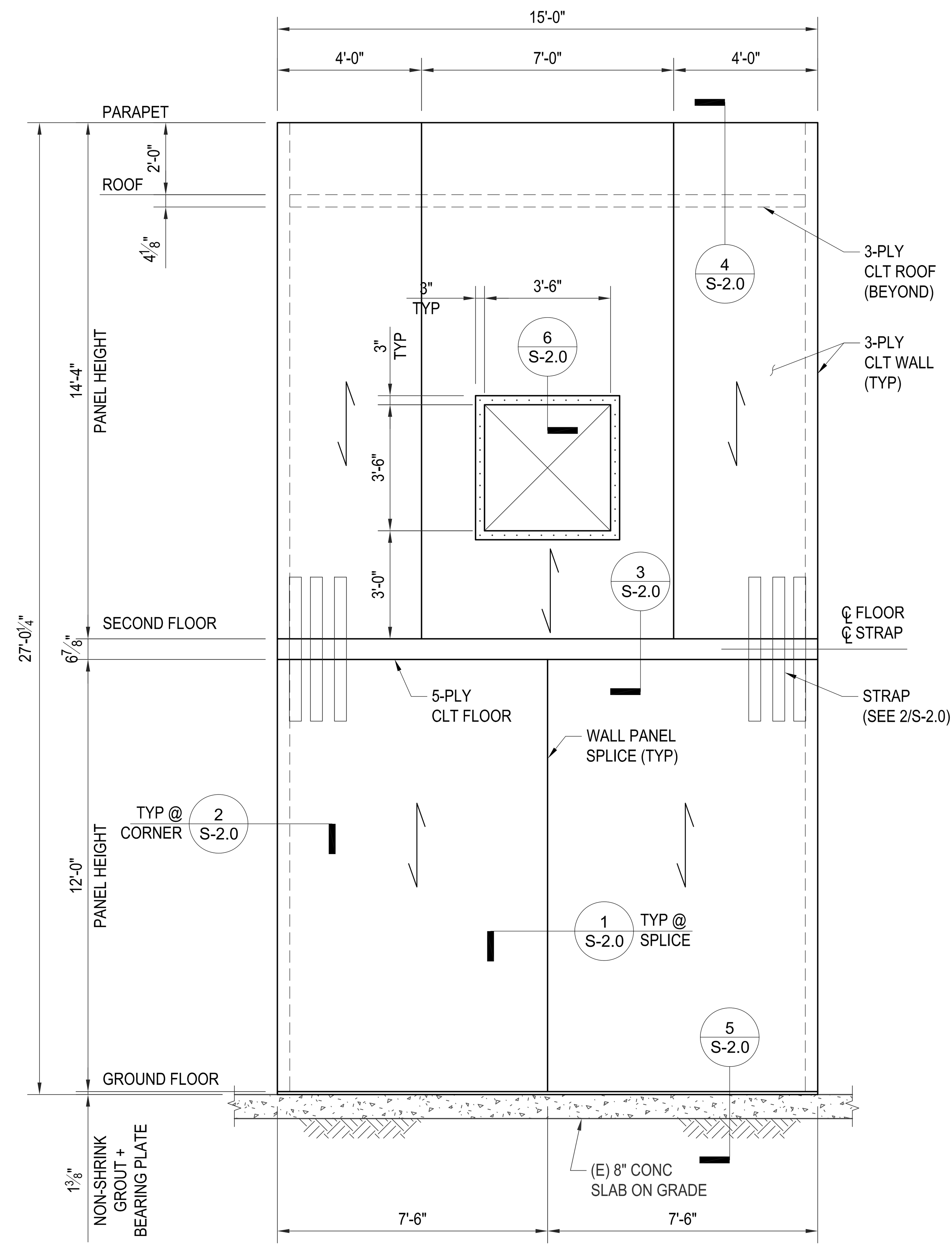
May 23, 2017



A S-1.2.1 SIDE ELEVATION (WITH DOOR)
SCALE: 3/8"=1'-0"



B S-1.2.1 FRONT / REAR ELEVATION
SCALE: 3/8"=1'-0"



C S-1.2.1 SIDE ELEVATION (WITH WINDOW)
SCALE: 3/8"=1'-0"

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Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

ELEVATIONS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

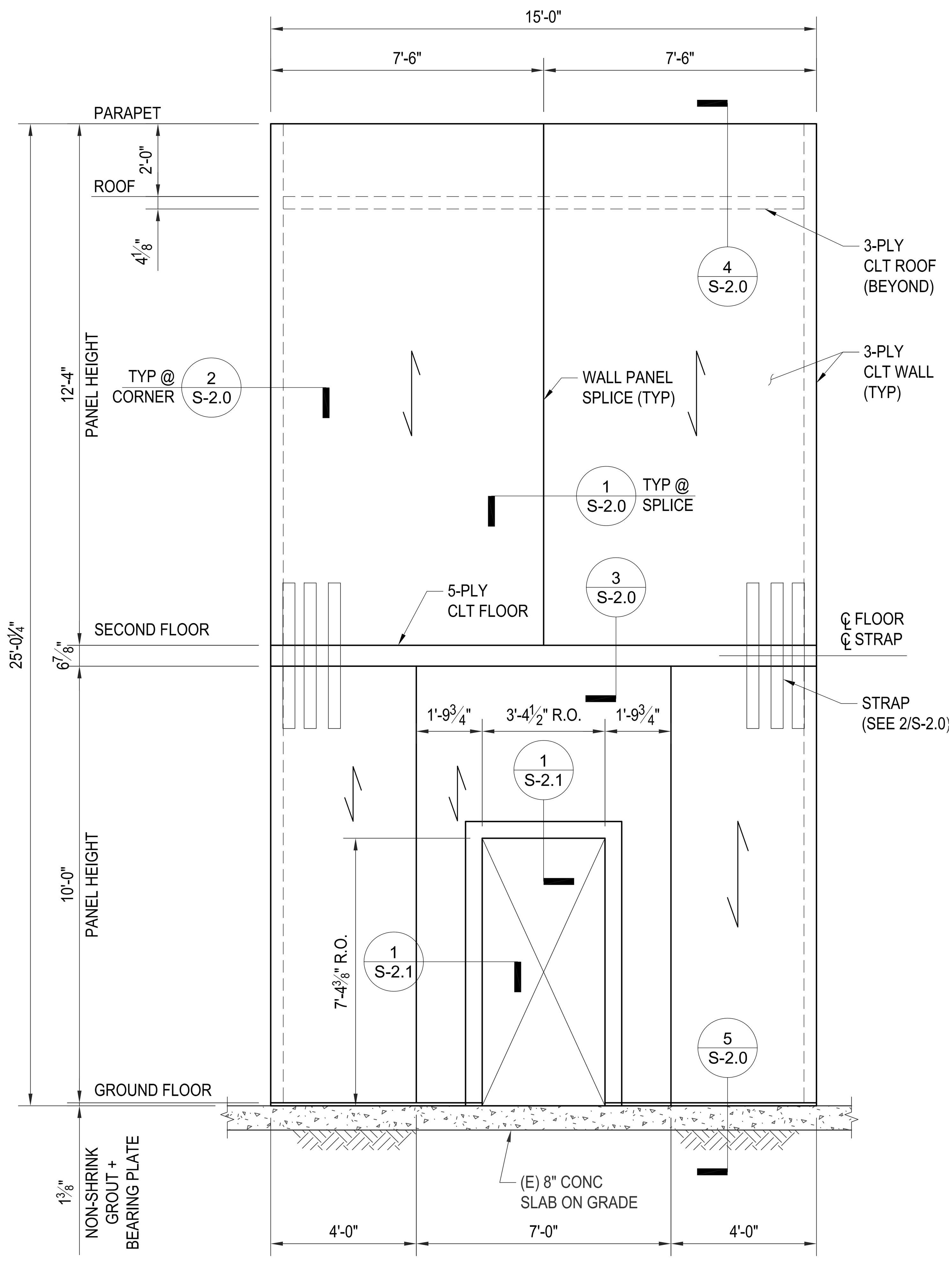
JOB NO.	FILE NAME:
2271_2015	S10201_2271.DWG

SHEET NUMBER

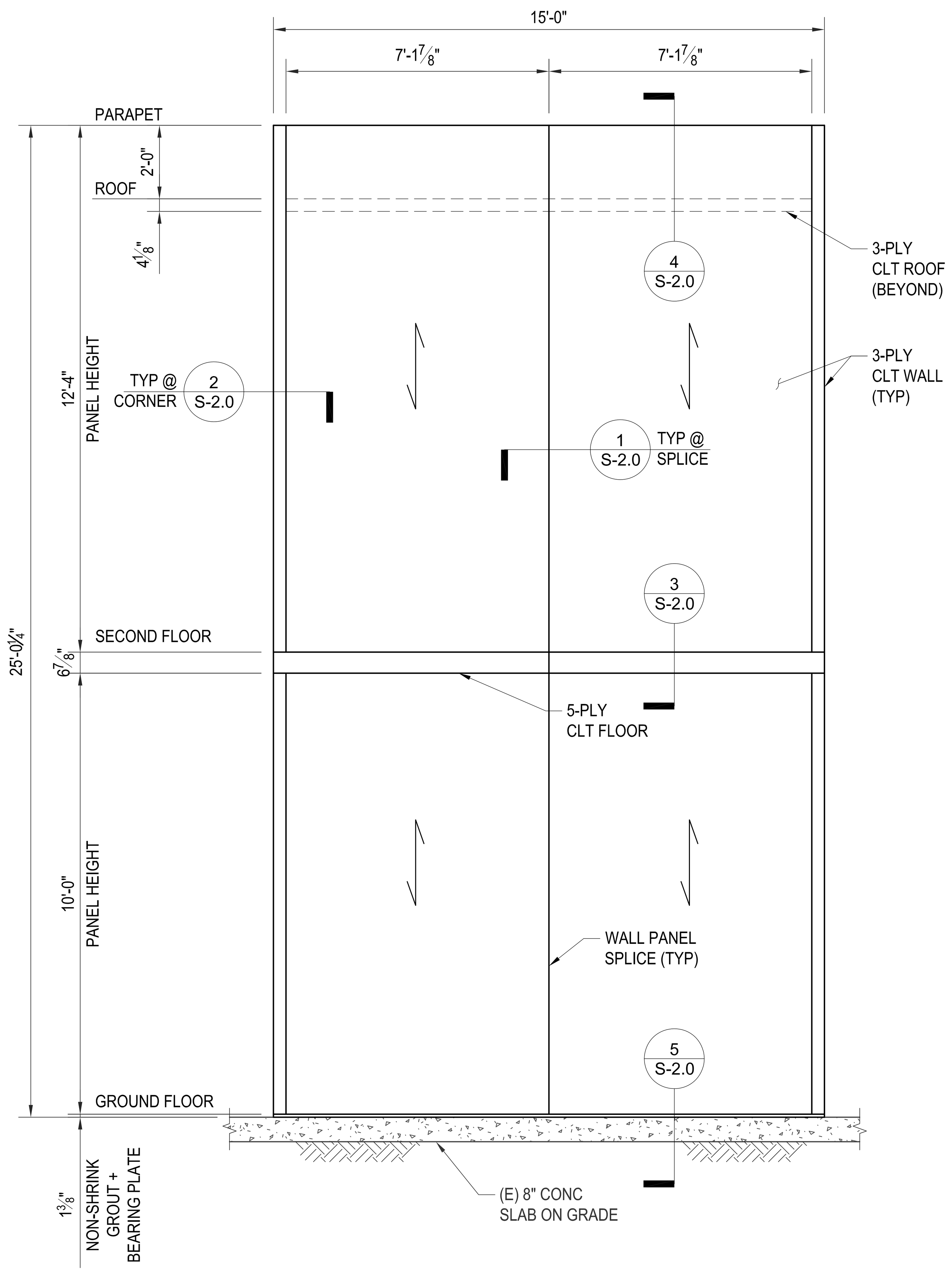
S-1.2.1

3

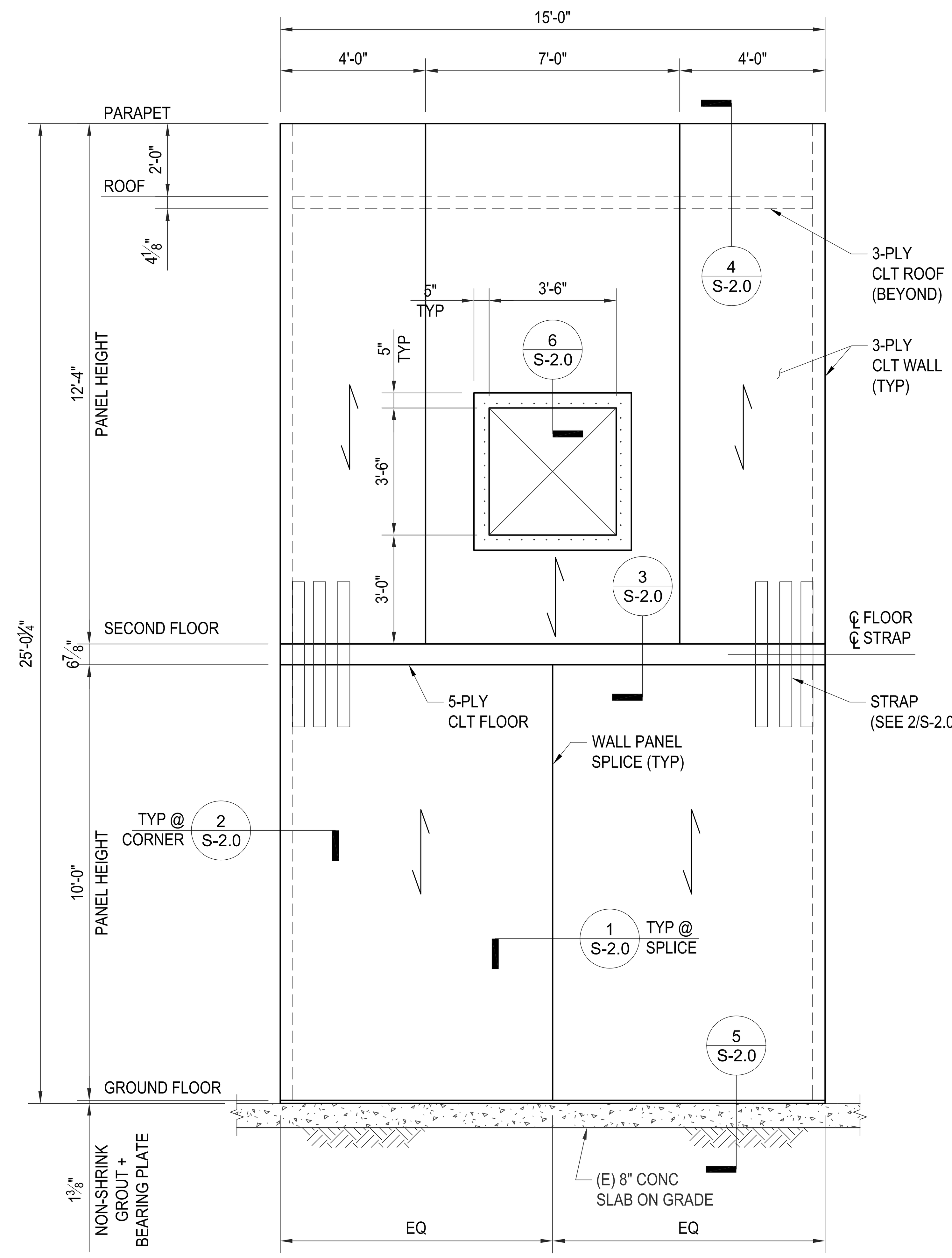
May 23, 2017



A
S-1.2.2
SIDE ELEVATION (WITH DOOR)
SCALE: 3/8"=1'-0"



B
S-1.2.2
FRONT / REAR ELEVATION
SCALE: 3/8"=1'-0"



C
S-1.2.2
SIDE ELEVATION (WITH WINDOW)
SCALE: 3/8"=1'-0"

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1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

ELEVATIONS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

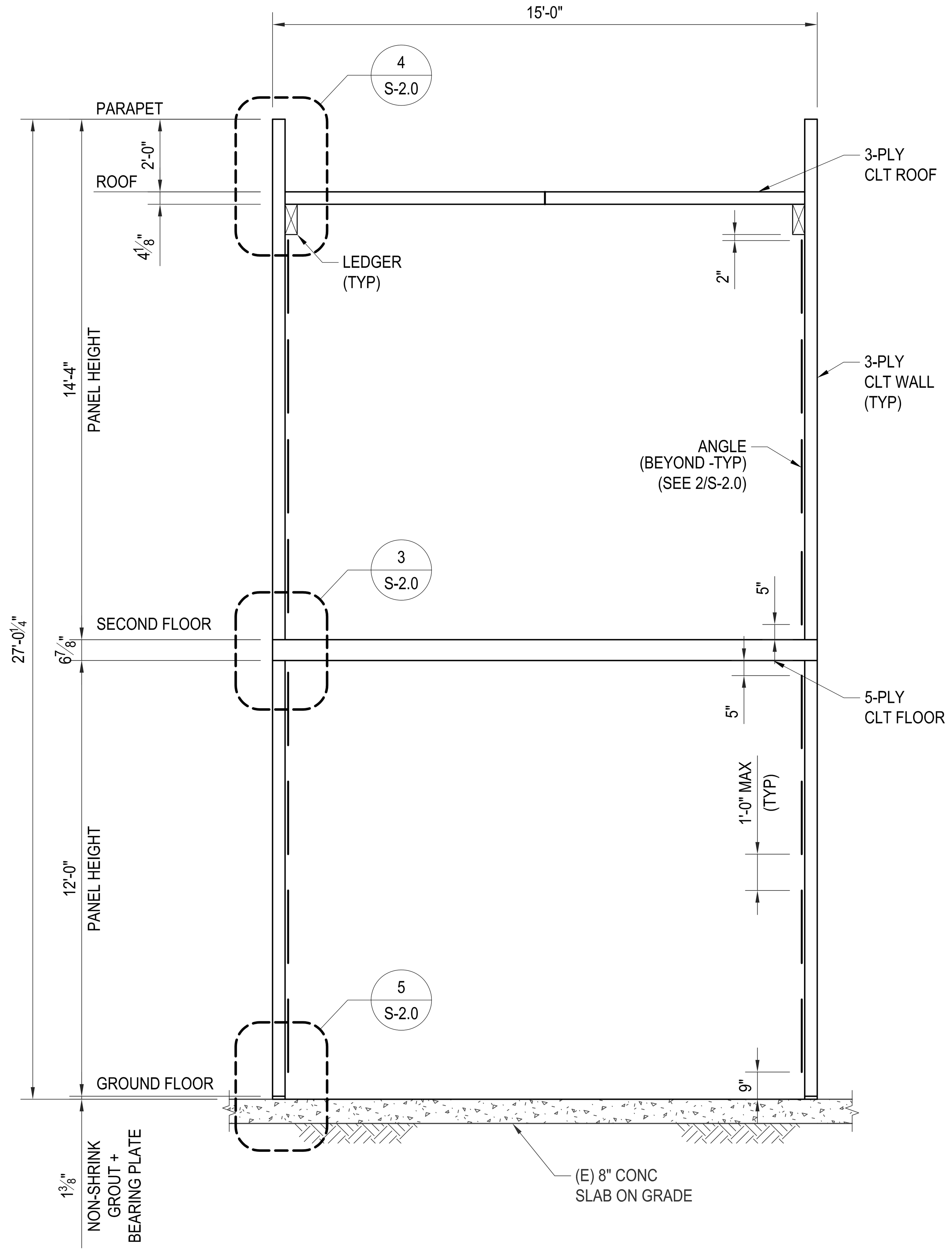
JOB NO.	FILE NAME:
2271_2015	S10202_2271.DWG

SHEET NUMBER

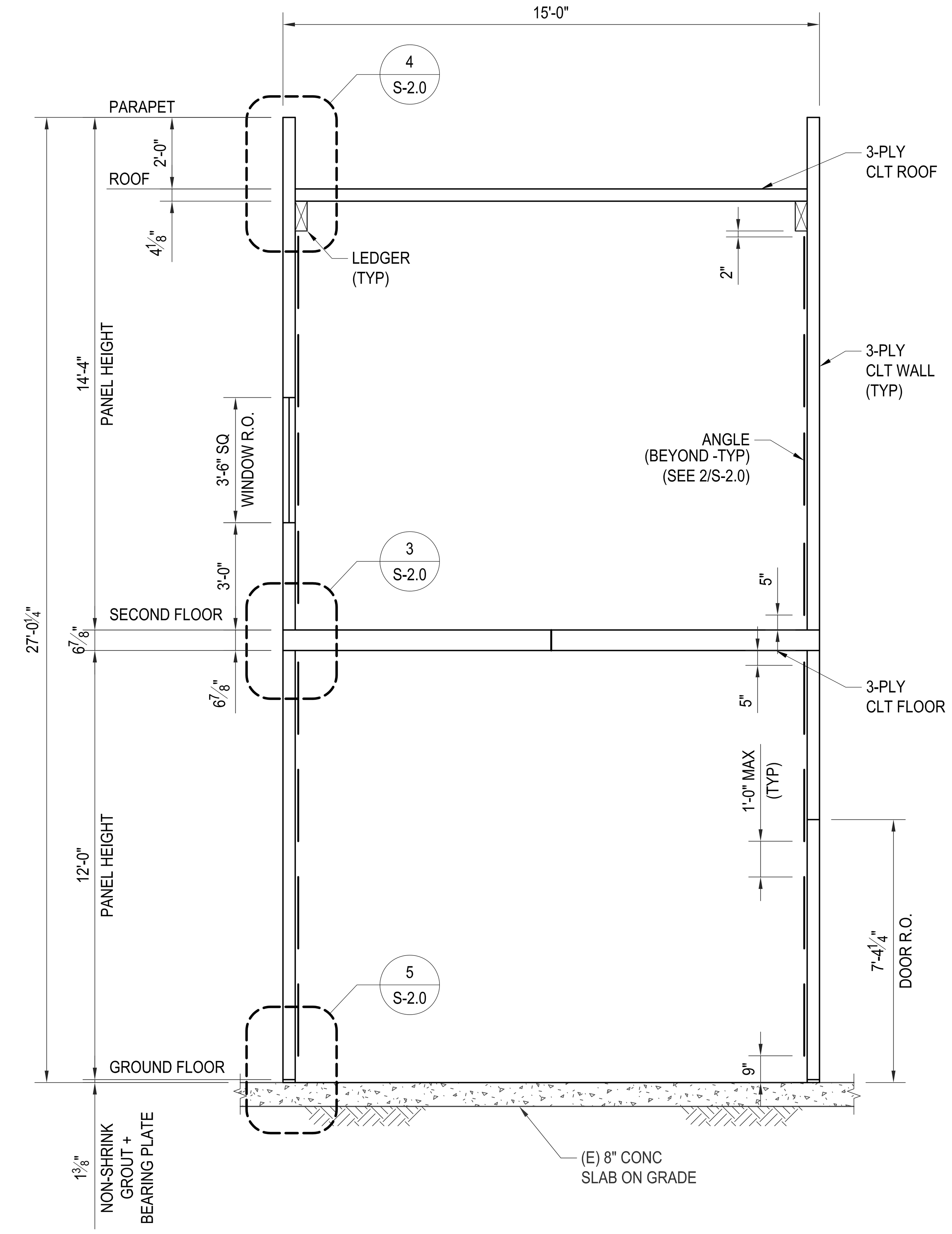
S-1.2.2

3

May 23, 2017



1 S-1.3.1 SECTION
SCALE: 3/8"=1'-0"



2 S-1.3.1 SECTION
SCALE: 3/8"=1'-0"

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1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

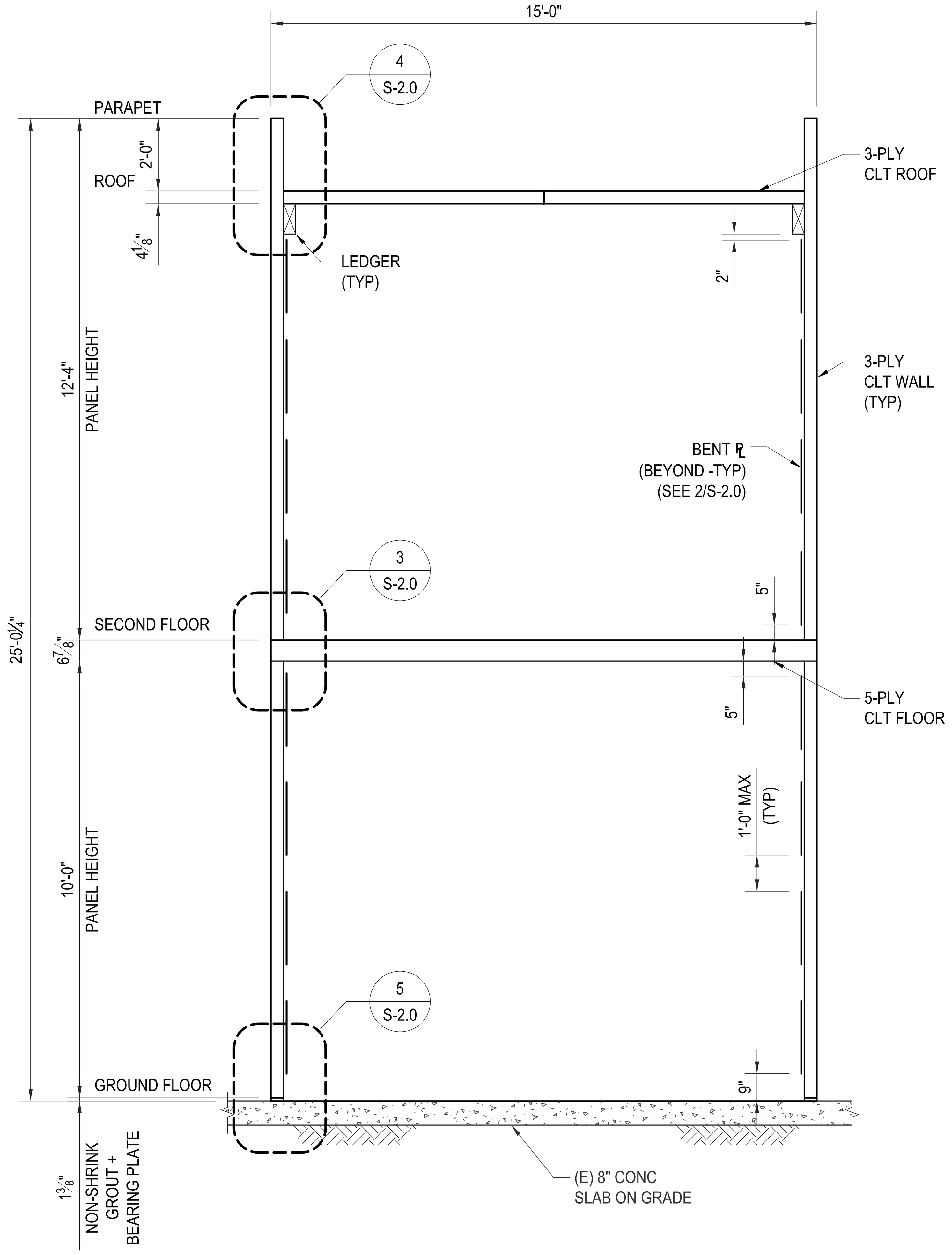
SECTIONS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

JOB NO. 2271_2015
FILE NAME: S10301_2271.DWG

SHEET NUMBER
S-1.3.1 3

May 23, 2017



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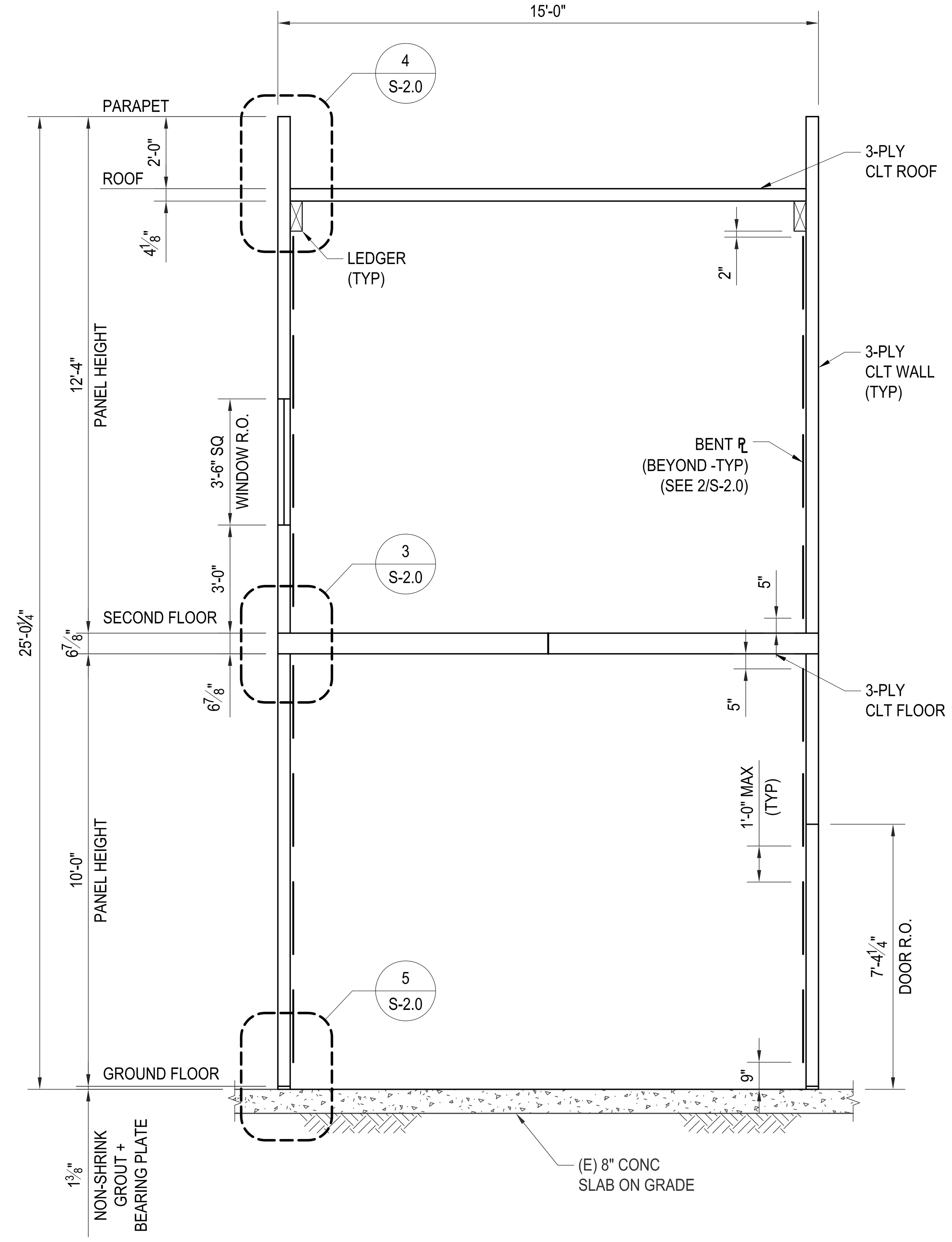
S-1.3.2
SECTION
SCALE: 3/8"=1'-0"



2
S-1.3.2
SECTION
SCALE: 3/8"=1'-0"

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1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT
MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

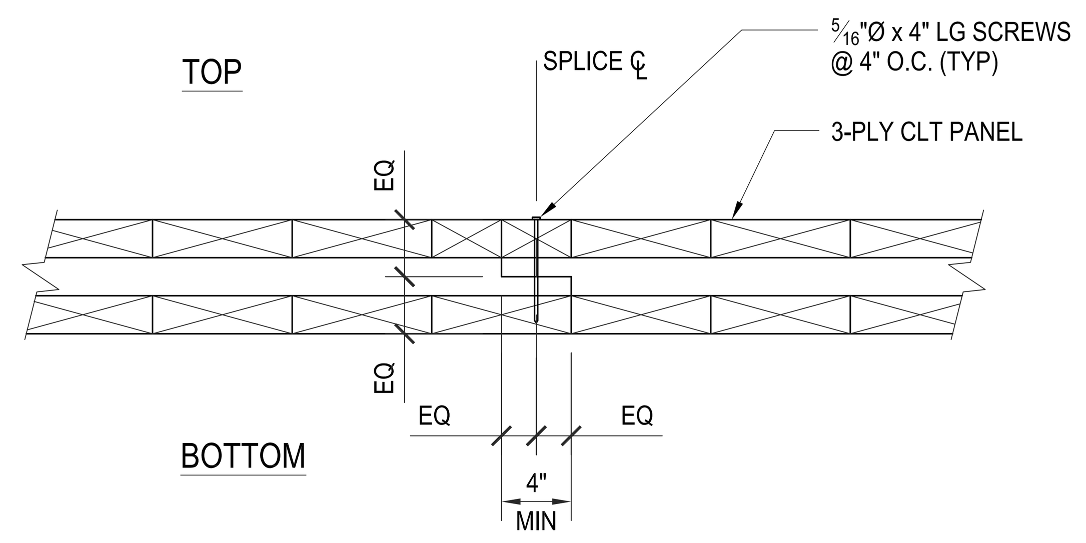
SHEET TITLE
SECTIONS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

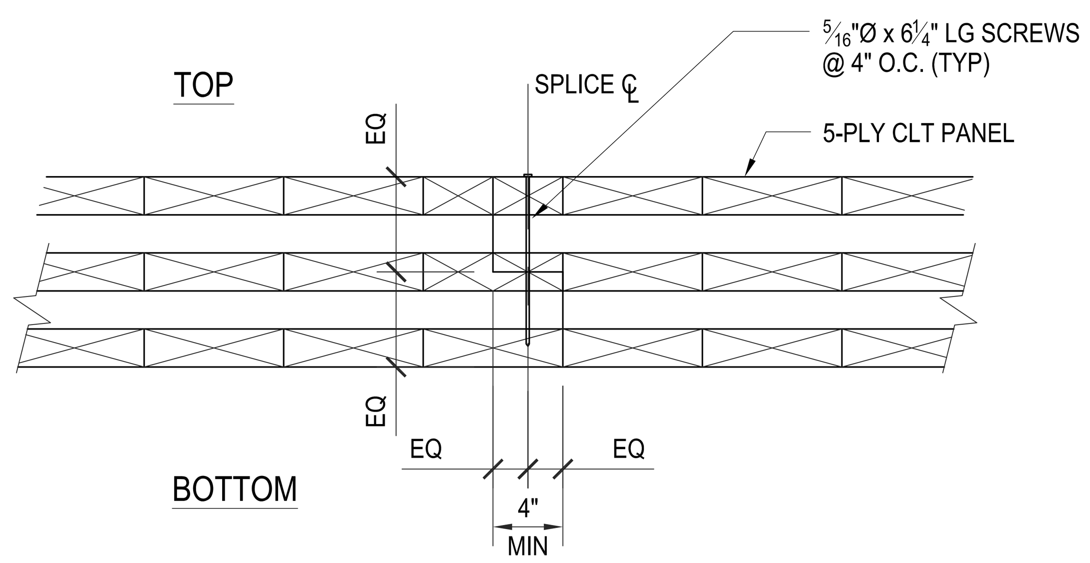
JOB NO. 2271_2015
FILE NAME: S10302_2271.DWG

SHEET NUMBER
S-1.3.2
3

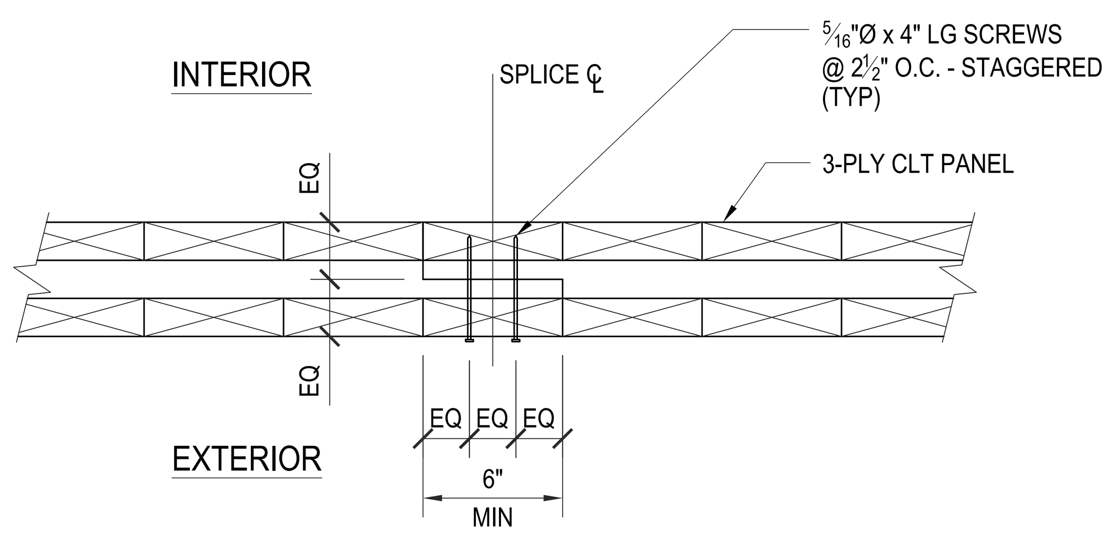
May 23, 2017



ROOF PANEL SPLICE

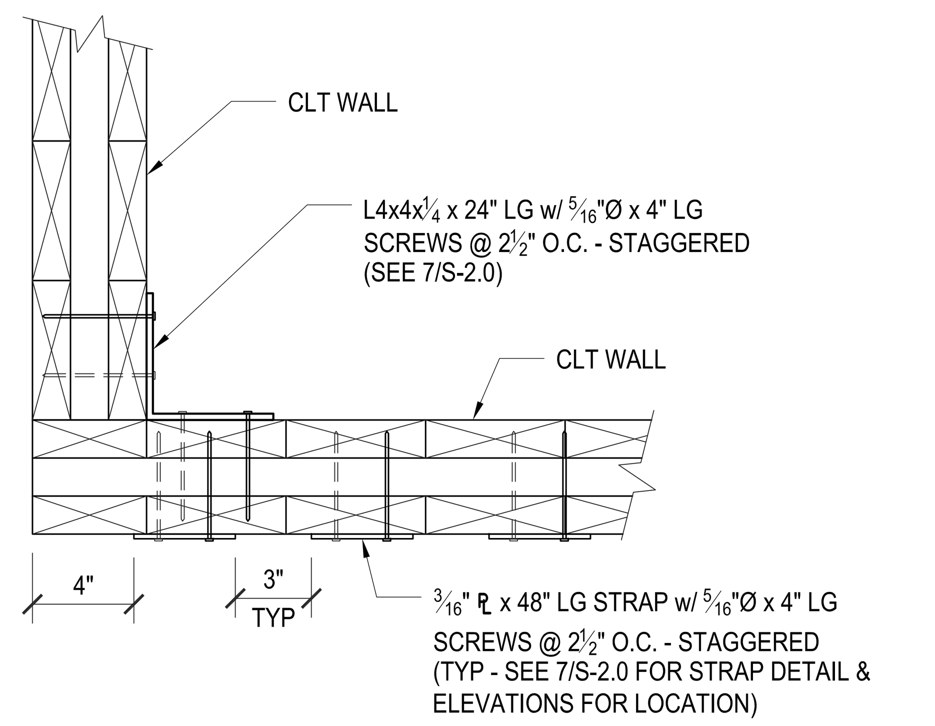


FLOOR PANEL SPLICE

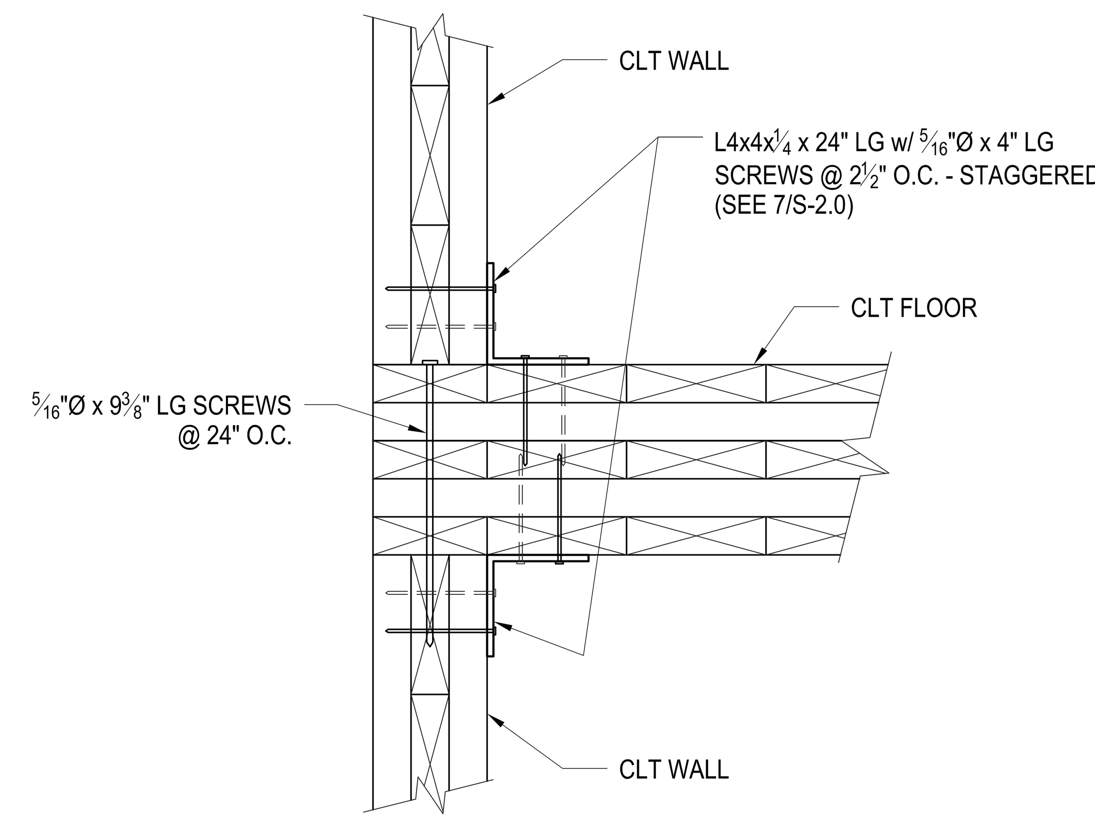


WALL PANEL SPLICE

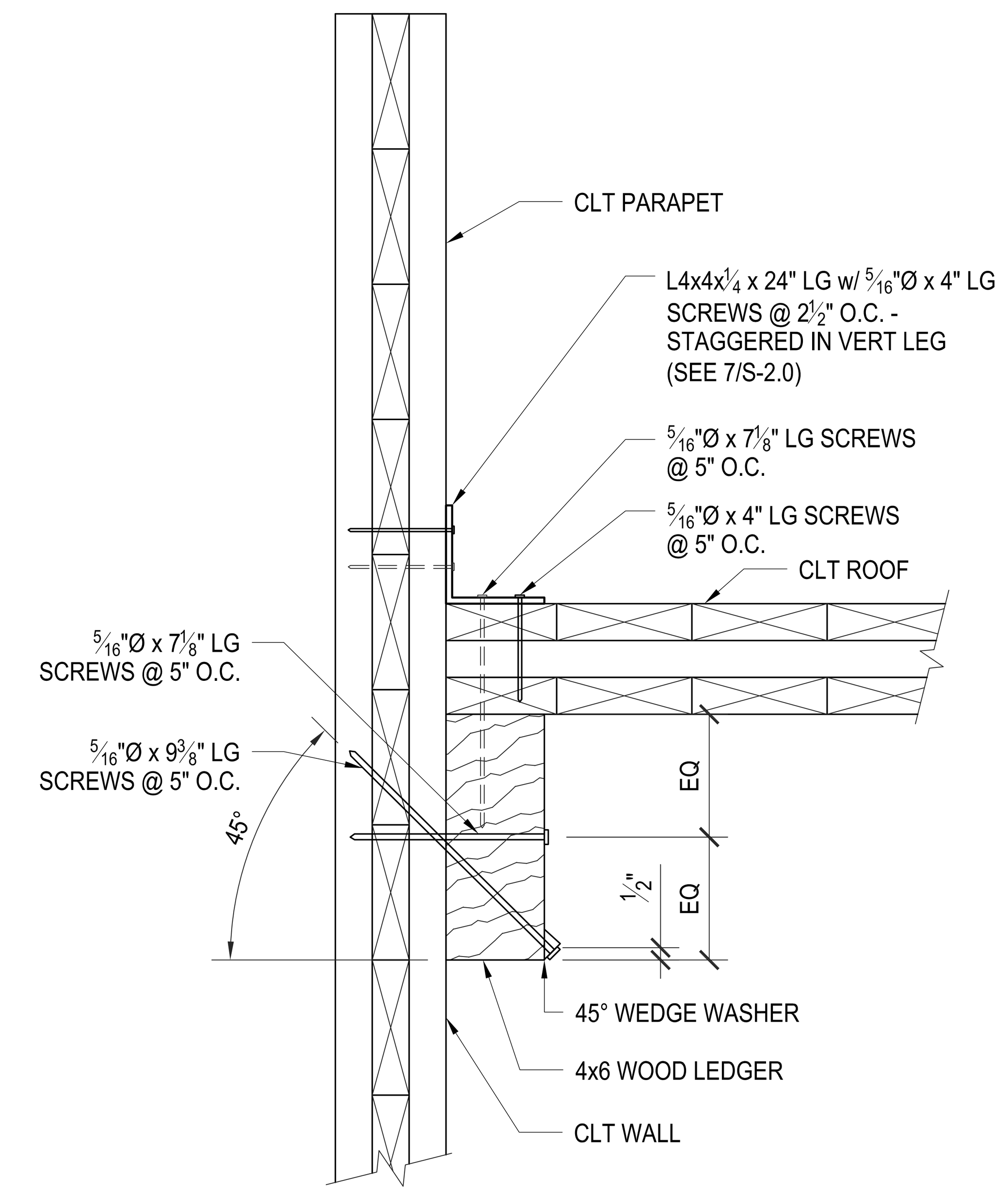
1 SPLICE DETAIL
S-2.0 SCALE: 2"=1'-0"



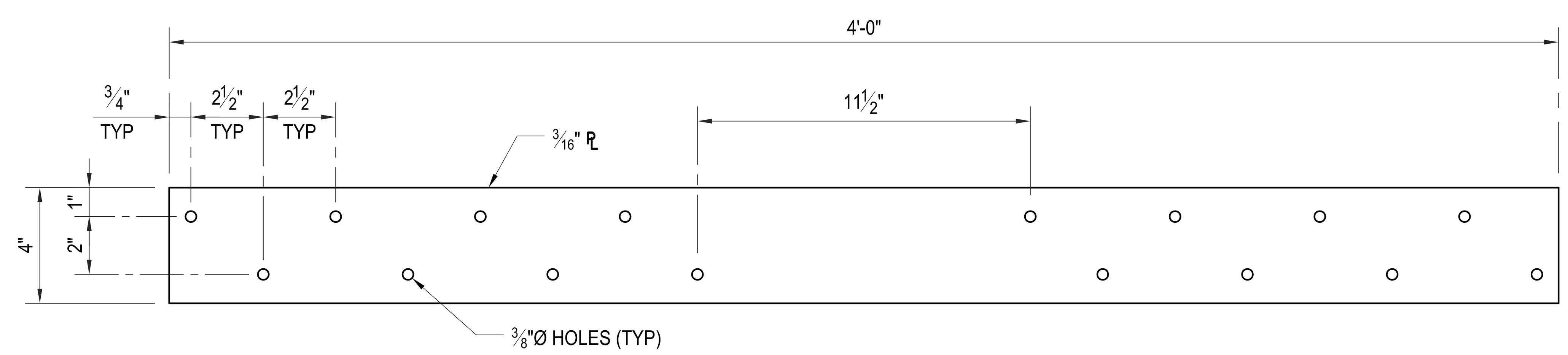
2 PLAN DETAIL (TYP @ WALL CORNER)
S-2.0 SCALE: 2"=1'-0"



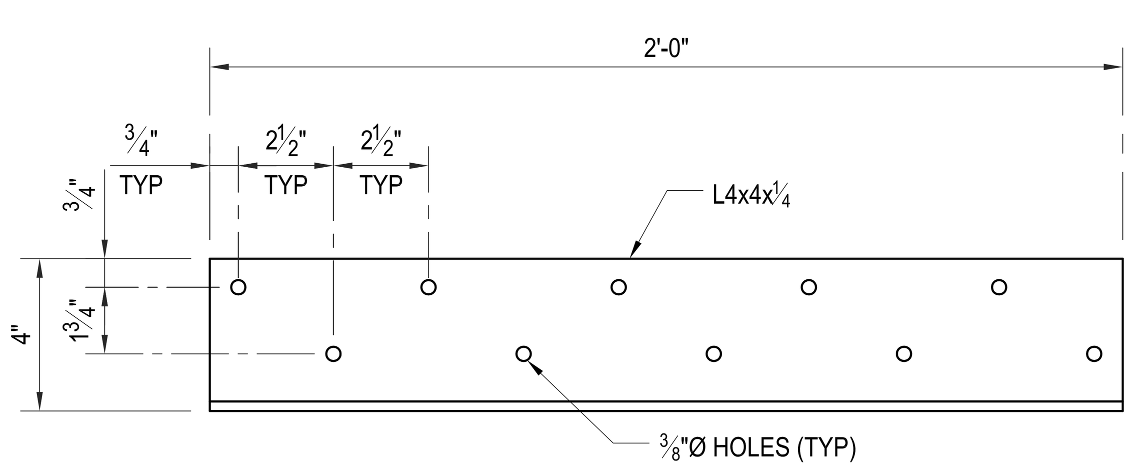
3 SECTION (TYP @ 2ND FLOOR)
S-2.0 SCALE: 2"=1'-0"



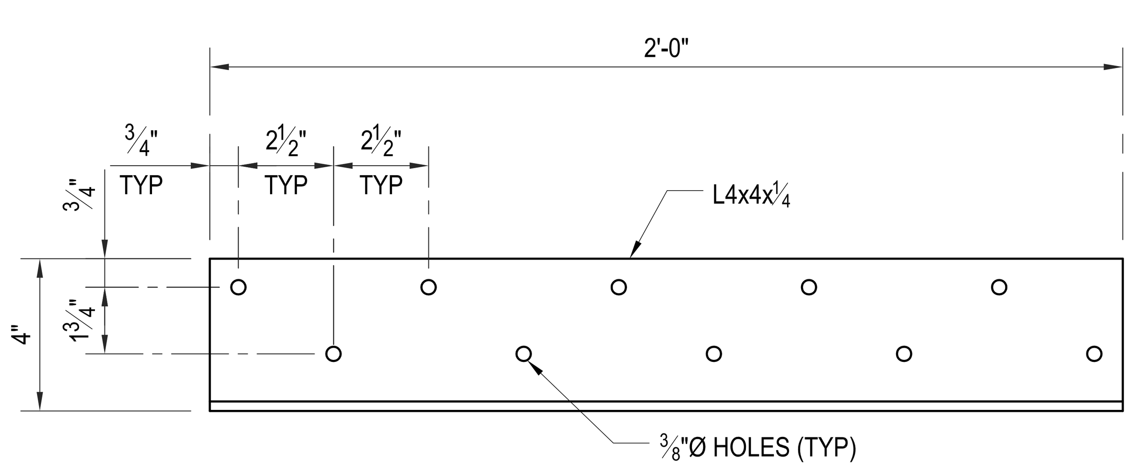
4 SECTION (TYP @ ROOF)
S-2.0 SCALE: 2"=1'-0"



TYPICAL STRAP

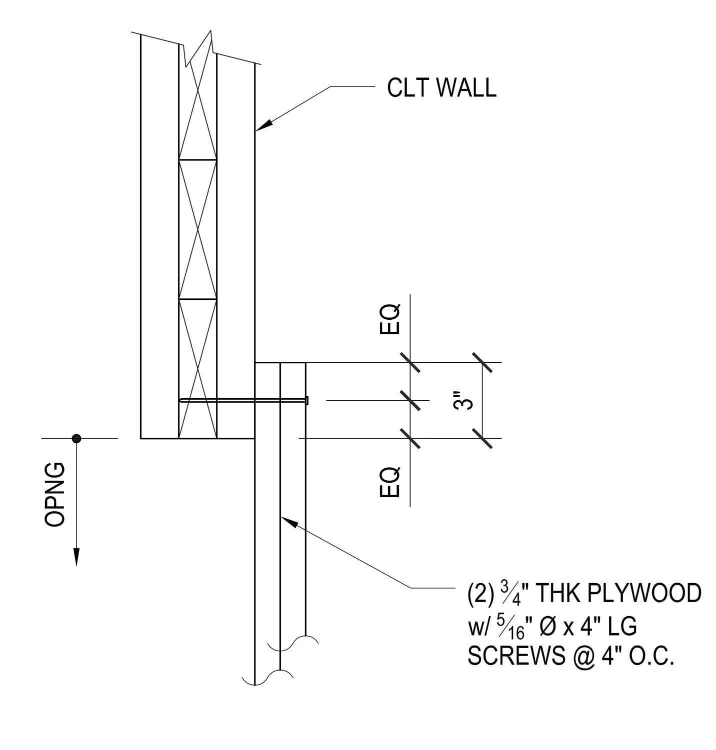


HORIZ LEG

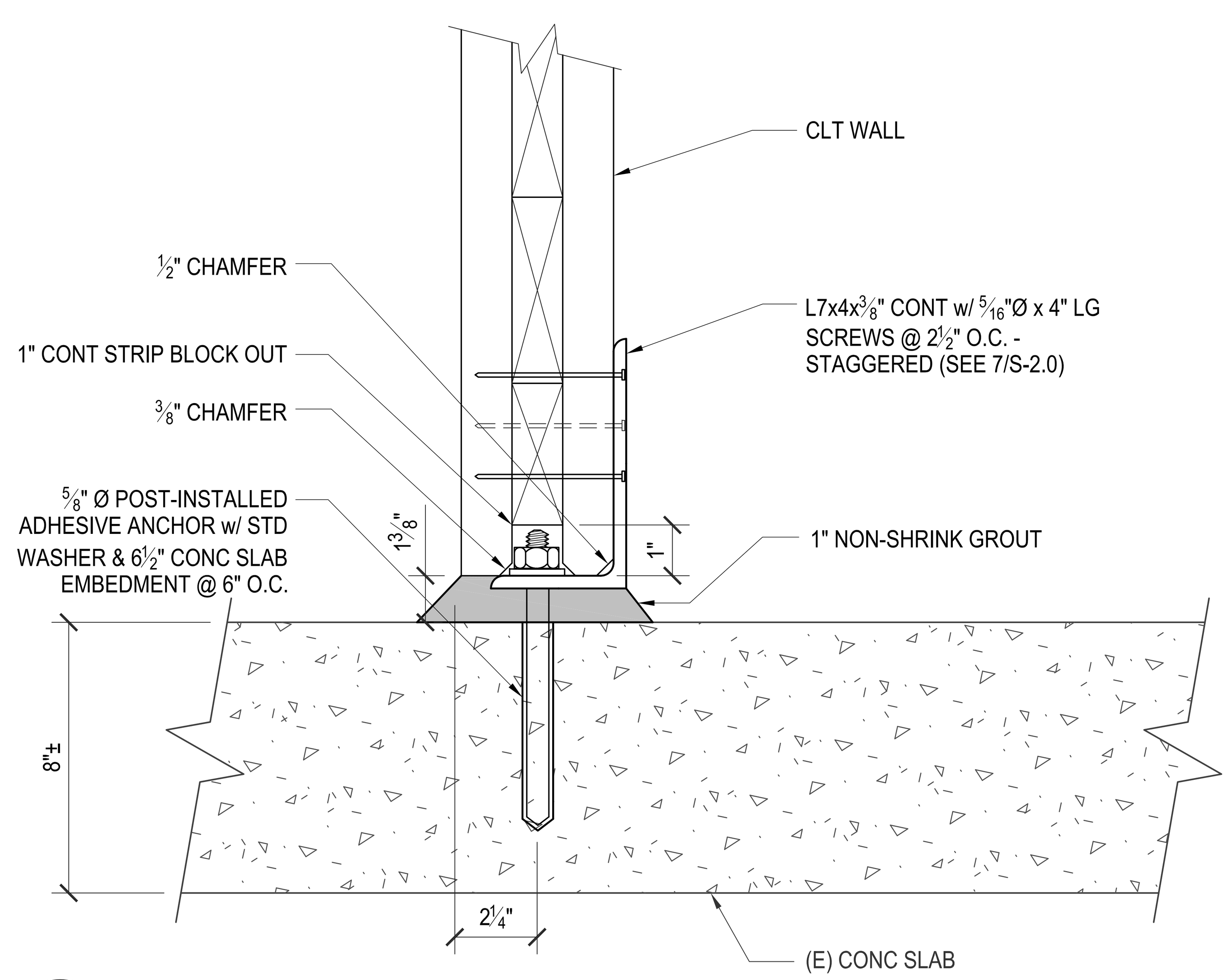


VERT LEG

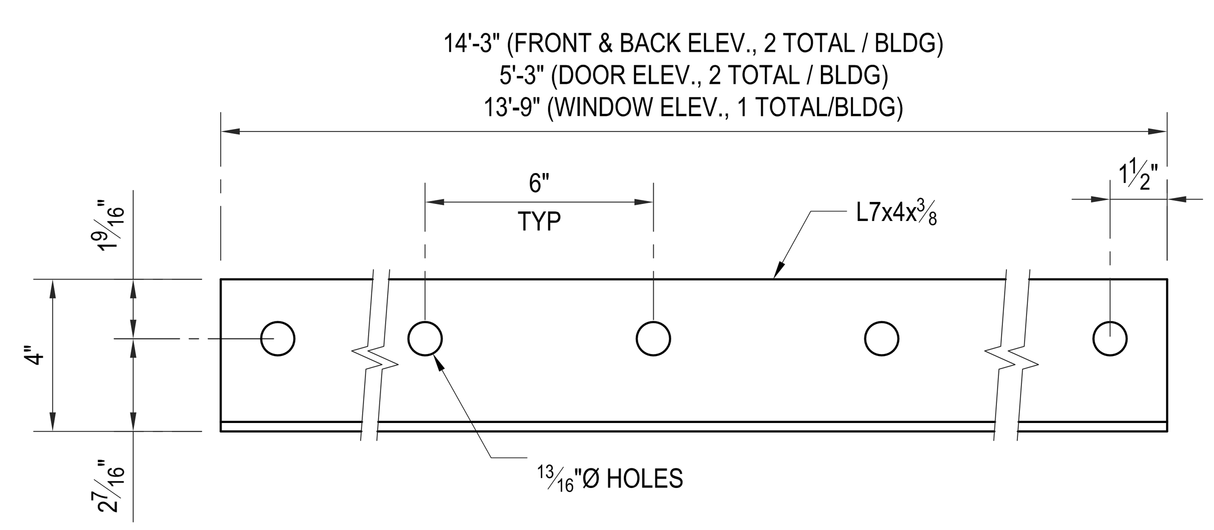
AT 2ND FLOOR & ROOF



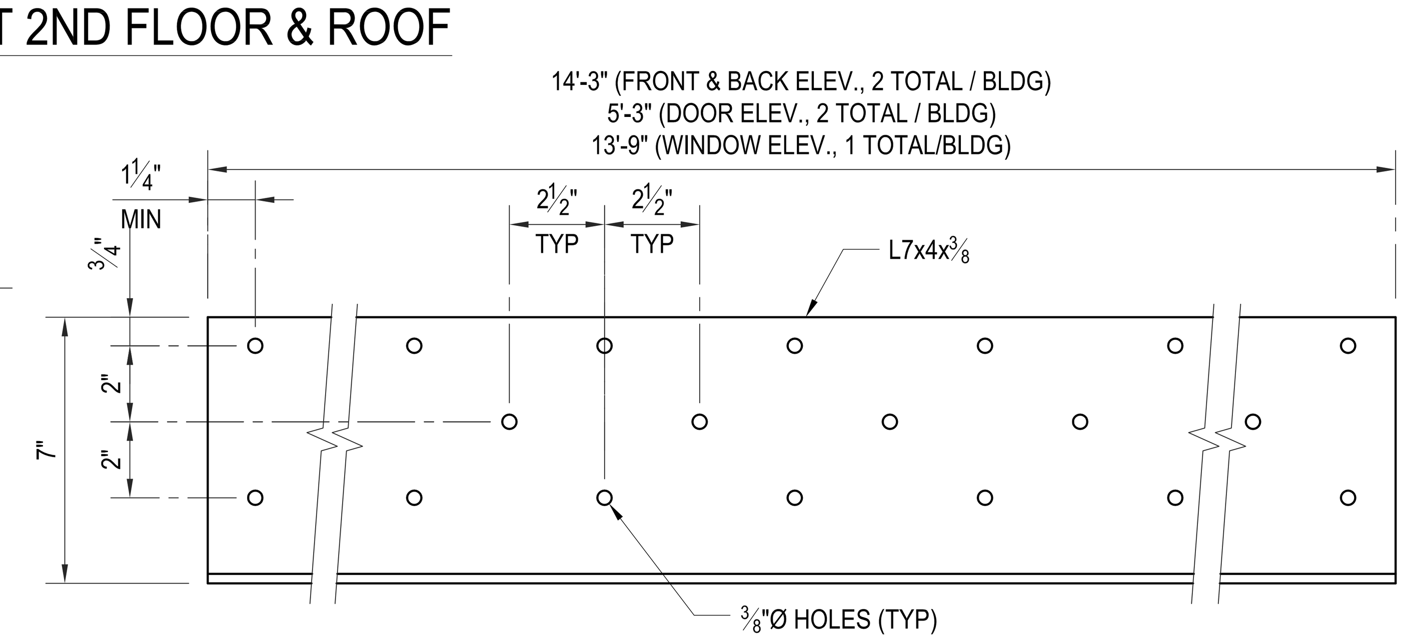
6 DETAIL (TYP @ WALL OPNG)
S-2.0 SCALE: 2"=1'-0"



5 SECTION (TYP @ FLOOR)
S-2.0 SCALE: 3"=1'-0"



HORIZ LEG



VERT LEG

AT FOUNDATION

7 DETAIL (PLATE SCREW SPCG)
S-2.0 SCALE: 2"=1'-0"

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2	09-19-2016	MW	MISC REVISIONS
1	08-24-2016	MW	MISC REVISIONS

Revisions

PROJECT
MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

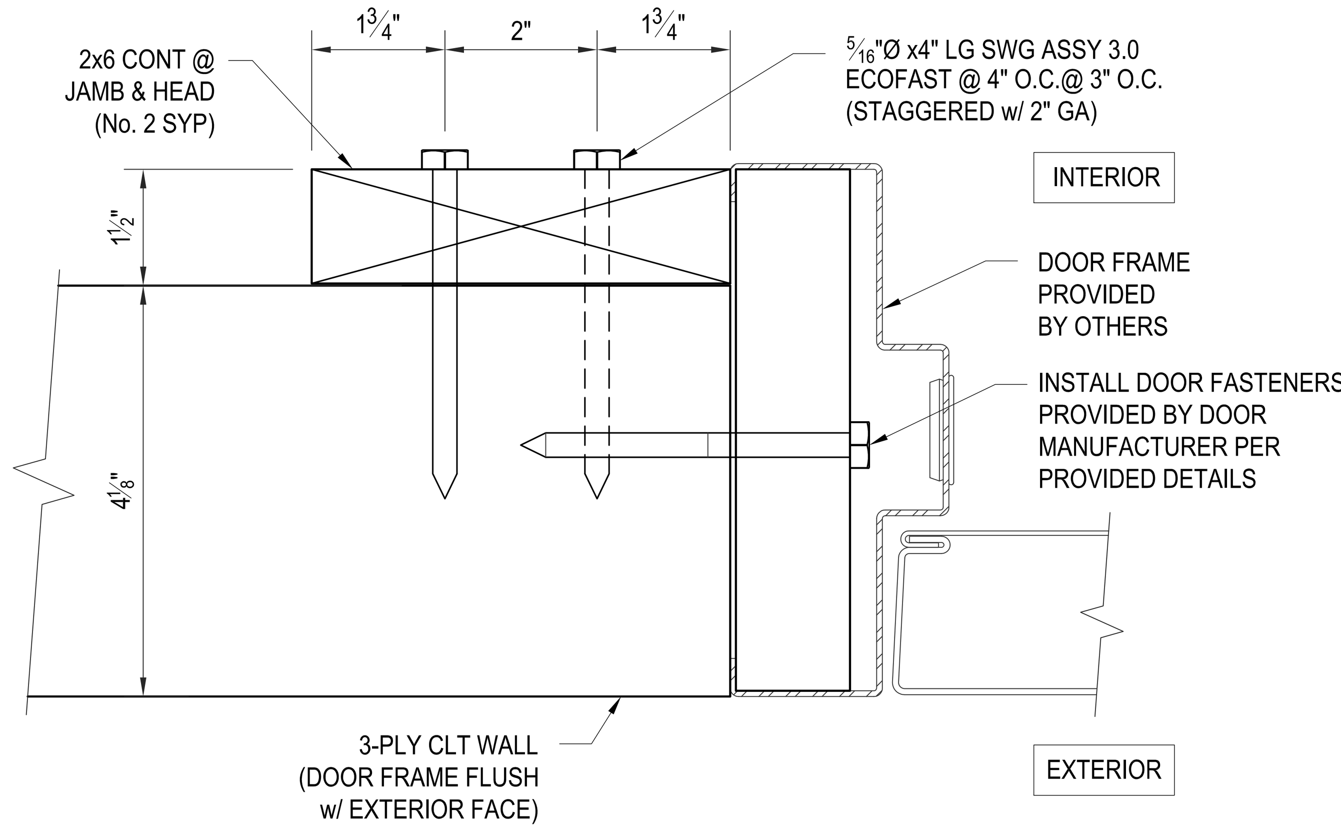
SHEET TITLE
DETAILS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

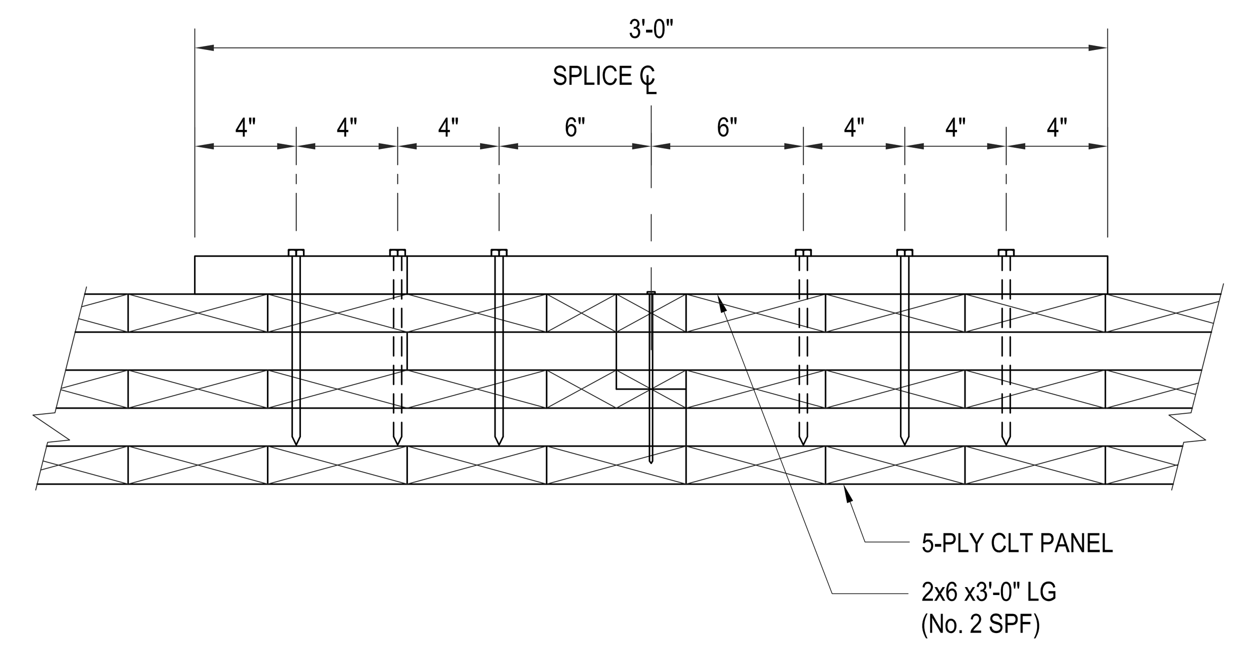
JOB NO. 2271_2015
FILE NAME: S200_2271.DWG

SHEET NUMBER
S-2.0

May 23, 2017



1 S-2.1 PLAN DETAIL @ DOOR JAMB
SCALE: 6"=1'-0"



2 S-2.1 SECTION
SCALE: 2"=1'-0"

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Mark	Date	Appr.	Description

Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

DETAILS

DRAWN	CHECKED	APPROVED	DATE ISSUED
CJ	MW	LT	09/19/2016

JOB NO. 2271_2015

FILE NAME: S201_2271.DWG

SHEET NUMBER

S-2.1

3

May 23, 2017

APPENDIX C

AS-BUILT DRAWINGS OF DOORS

AMBICO
SPECIALIZED DOORS · FRAMES · WINDOWS

AMBICO INTERNATIONAL LIMITED
532 MONTREAL ROAD, SUITE 112
OTTAWA, ON, CANADA K1K 4R4
Phone (613) 746-4663 FAX (613) 746-4721
TOLL FREE PHONE # 1-888-423-2224
TOLL FREE FAX # 1-800-465-8561
WEBSITE http://www.ambico.com
E-Mail specialized@ambico.com

Shop Drawing Submittal

Date	Page
September 09, 2016	1
Order Number	
ORAI13141	

AI

Sold To:	Ship To:
AMERICAN DIRECT 11000 LAKEVIEW AVENUE, LENEXA,, KS 66219 USA	Jacobs Technology Tyndall Air Force Base 104 Research Rd., BLDG 9742 48 HRS NOTICE Attn: Casey O'Laughlin 816.844.5596 Tyndall Air Force Base, FL 32403 USA

Fax Number:	9136775576	chrisw@americandirectco.com
Attention:	Chris Wilson	
Job Name:	Remake Openings ORAI12560	
Customer PO#:	111729	
Order Date:	August 15, 2016	
Order Confirmation Number:	ORAI13141	
Ambico Project Manager:	Ben Soulis	
Ambico Product:	*BLAST RESITANT CHARGE WEIGHT II FRAMES & DOORS*	

Attached is a copy of our shop drawings for the above noted order. The shop drawing is issued for your review and approval. Please confirm the following:

_____ Revise and resubmit
_____ Approved as noted
_____ Approved without changes

Signature:_______________ Date:__________

Fabrication of material will begin only when the shop drawing is returned with the above noted information. In order to maintain our current lead time we require the shop drawing returned approved by the date noted below.

Current Lead Time 7 weeks Approval Required by:

Please refer to our Order Confirmation Number when making inquiries.

Cc: Ambico rep Agency: House
House

SHOP DRAWING LINE LISTING

OPENING INFORMATION					FRAME INFORMATION			DOOR INFORMATION				NOTES		
TAG	QTY	PERF. RATING / LABEL	REBATE WIDTH / REBATE HEIGHT	SWING / HW SET	FRAME ELEV / FRAME Ga.	FRAME MATERIAL / FRAME FINISH	ANCHOR / JAMB DEPTH	DOOR ELEV / DOOR Ga	DOOR MATERIAL / DOOR FINISH	DOOR THICK / DOOR CORE	CL TOP/BOT / DOOR EDGE	FRAME REMARKS	DOOR REMARKS	REV # 0 / REV DATE
128	1	BLAST RESISTANT	36 in	RHR	3 SIDED SINGLE	GALVANEAL	P+D (Bolts By Ambico)	FLUSH	GALVANEAL	1.75 in				0
			86 in	09	14	PRIME	5.75 in	14	PRIME	BLAST RESISTANT	LOCKSEAM			15-Aug-16
501	1	BLAST RESISTANT	36 in	RHR	3 SIDED SINGLE	GALVANEAL	P+D (Bolts By Ambico)	FLUSH	GALVANEAL	1.75 in				0
			86 in	12.1	14	PRIME	5.75 in	14	PRIME	BLAST RESISTANT	LOCKSEAM			15-Aug-16
502	1	BLAST RESISTANT	36 in	LHR	3 SIDED SINGLE	GALVANEAL	P+D (Bolts By Ambico)	FLUSH	GALVANEAL	1.75 in				0
			86 in	12.2	14	PRIME	5.75 in	14	PRIME	BLAST RESISTANT	LOCKSEAM			15-Aug-16

1120 CUMMINGS AVENUE
OTTAWA, ONTARIO, CANADA
K1J 7R8
TEL# : 1 (888) 423-2224
FAX# : 1 (800) 465-8561

ORDER NUMBER: ORAI13141
CUSTOMERID: A18898
CUSTOMER PO: 111729
CUSTOMER NAME: AMERICAN DIRECT
JOB NAME: JOB NAME: 119698 PAL Candlewood Suites Restone Arsenal

SHIP TO ADDRESS : TBA

PROJECT MANAGER: B SOULIS

ORDER DATE : 15-Aug-16
SUBMITTED DATE : N/A
APPROVAL DATE : N/A

Page #: 1 Of: 6

AMBICO
Specialized Steel Doors & Frames
Specialized Wood Doors
1120 CUMMINGS AVENUE
OTTAWA, ONTARIO, CANADA
K1J 7R8
TEL# : 1 (888) 423-2224
FAX# : 1 (800) 465-

AMBICO HARDWARE PREPARATION SCHEDULE

ORDER NUMBER: ORAI12560
DATE: 25/06/2015

HARDWARE SET: 09

OPENINGS

QTY	TAG #	RABBET SIZE	FRAME TYPE	DOOR ELEVATION	DOOR THK	HANDING
1	TAG: 128	36" x 86"	3 SIDED SINGLE	FLUSH	1.75"	RHR

HARDWARE

TOTAL QTY	ITEM TYPE	MANUFACTURI	DESCRIPTION
3	HINGES	MCKINNEY	T4A3386 4.50 x 4.50 x.180
1	LOCK	ONITY	HT-24
1	TRIM	ONITY	KHD3-L-626
1	CLOSER	YALE	Reinforced for Pa,Reg Arm

HARDWARE SET: 12.1

OPENINGS

QTY	TAG #	RABBET SIZE	FRAME TYPE	DOOR ELEVATION	DOOR THK	HANDING
1	TAG: 501	36" x 86"	3 SIDED SINGLE	FLUSH	1.75"	RHR

HARDWARE

TOTAL QTY	ITEM TYPE	MANUFACTURI	DESCRIPTION
3	HINGES	MCKINNEY	T4A3386 4.50 x 4.50 x.180
1	LOCK	YALE	PB 4730LN
1	CLOSER	YALE	Reinforced for Pa,Reg Arm

HARDWARE SET: 12.2

OPENINGS

QTY	TAG #	RABBET SIZE	FRAME TYPE	DOOR ELEVATION	DOOR THK	HANDING
1	TAG: 502	36" x 86"	3 SIDED SINGLE	FLUSH	1.75"	LHR

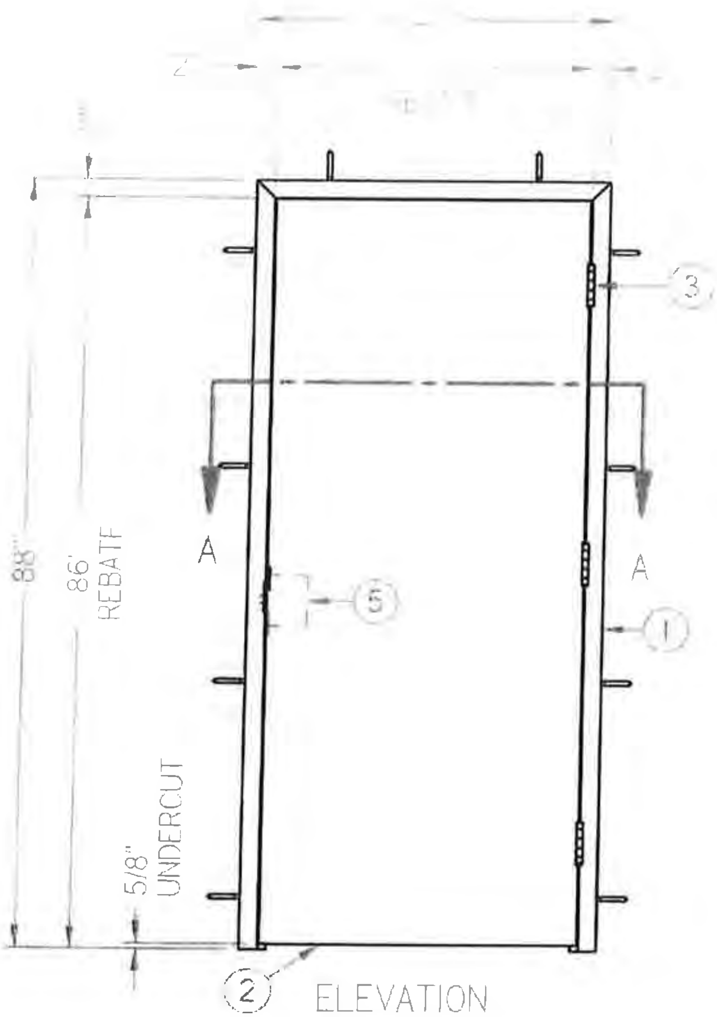
HARDWARE

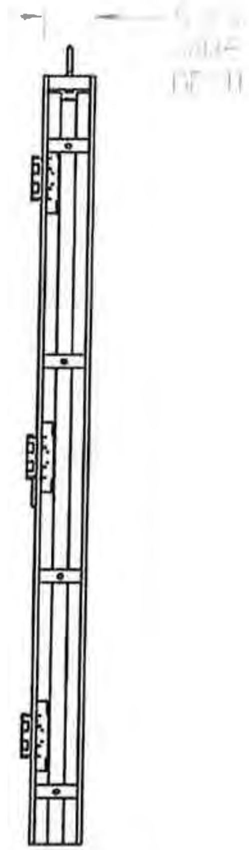
TOTAL QTY	ITEM TYPE	MANUFACTURI	DESCRIPTION
3	HINGES	MCKINNEY	T4A3386 4.50 x 4.50 x.180
1	LOCK	ONITY	HT-24
1	TRIM	ONITY	KHD3-L-626
1	CLOSER	YALE	Reinforced for Pa,Reg Arm

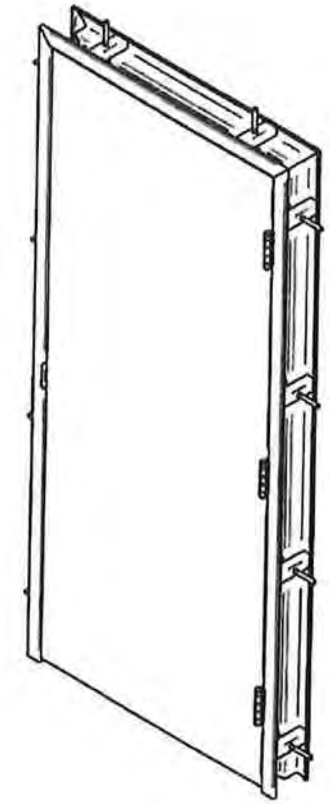
NOTES
ALL HARDWARE SUPPLIED AND INSTALLED BY OTHERS.

2/6

NOTES:
1. ARCHITECT AND/OR CONTRACTOR TO ENSURE WALL CAN RESIST APPLIED LOADING.
2. FRAME AND DOOR SHOWN ARE RHR. LHR IS A MIRROR-IMAGE.







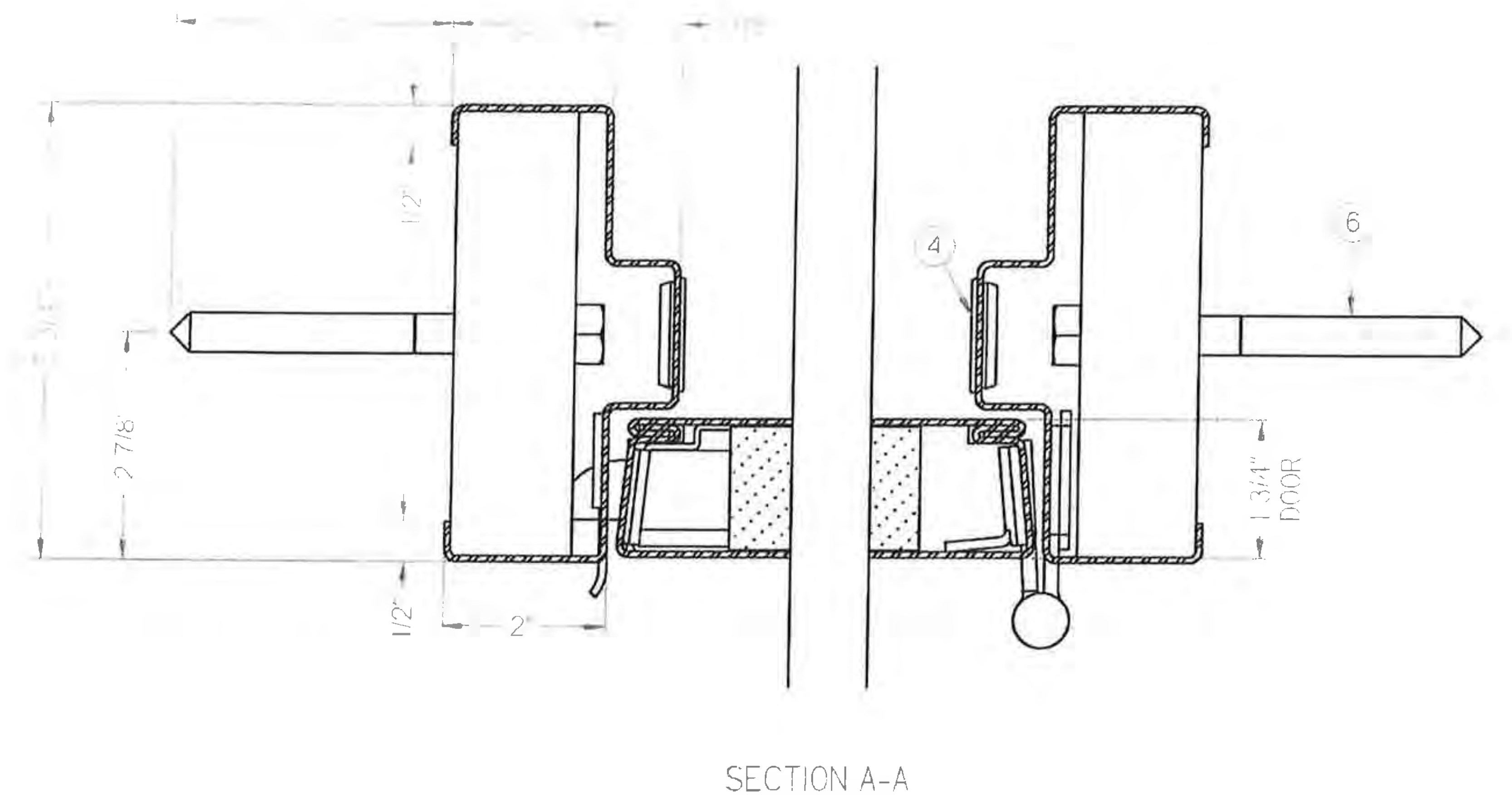
ISOMETRIC VIEW

Item No.	Title	Qty.
1	Blast Resistant Frame	1
2	Blast Resistant Door	1
3	4 1/2" x 4 1/2" Heavy Weight Hinge (By Others)	3
4	Hole Plug for 1.25" Hole	10
5	Mortised Lock (By Others)	1
6	1/2" x 5" Lag Screw	10

AMBICO SPECIALIZED STEEL DOORS & FRAMES

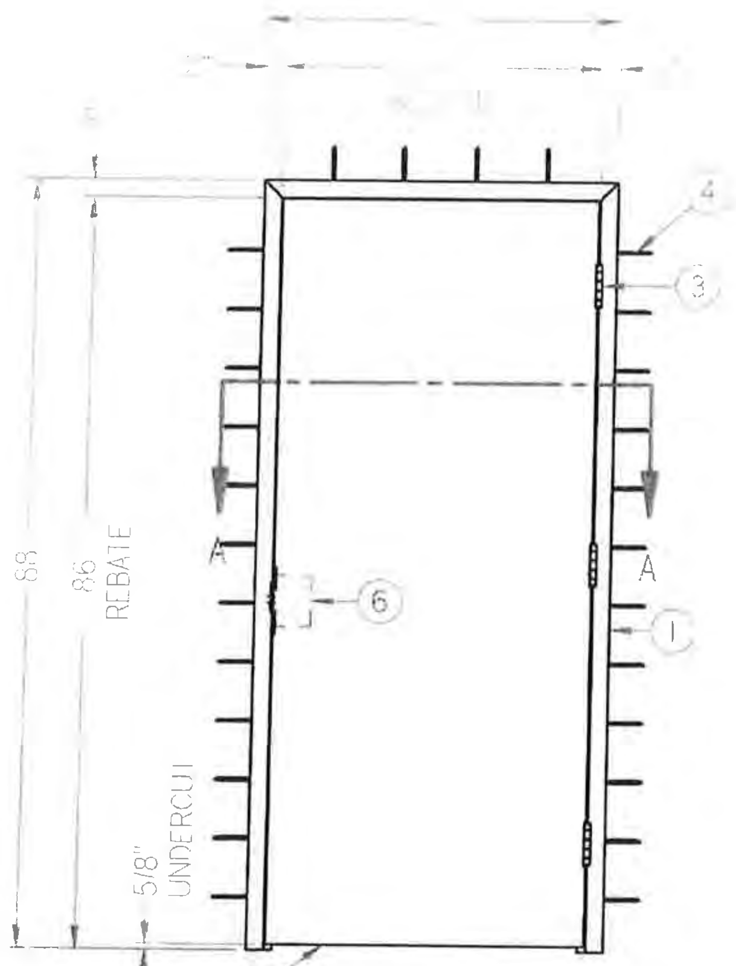
TITLE: CHARGE WEIGHT II, 105' STANDOFF, LOW LEVEL OF PRTOECTION BLAST RESISTANT FRAME AND DOOR - TAGS 501, 502

DATE:	03/02/2015	ORDER NO	
DRAWN BY	tberry	CUSTOMER NO:	A18898
REVISION	0	SHEET	3 OF 6

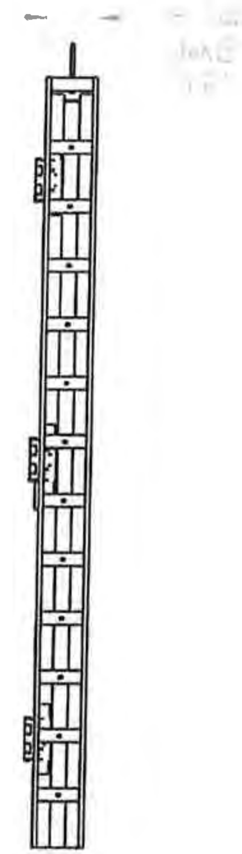


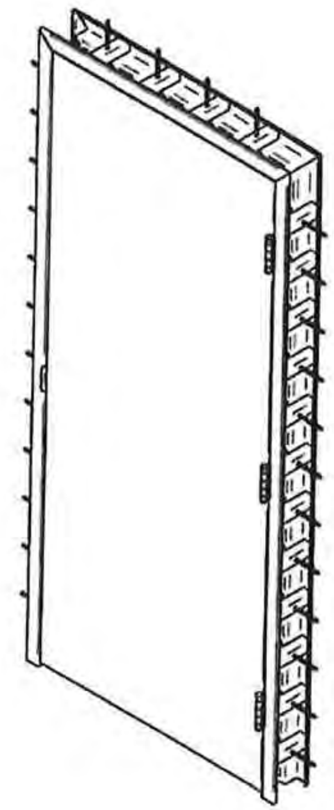
AMBICO SPECIALIZED STEEL DOORS & FRAMES	TITLE: CHARGE WEIGHT II, 105' STANDOFF, LOW LEVEL OF PRTOECTION BLAST RESISTANT FRAME AND DOOR - TAGS 501, 502	DATE 03/02/2015	ORDER NO
		DRAWN BY: tberry	CUSTOMER NO A18898
		REVISION 0	SHEET 4 OF 6

NOTES:
1. ARCHITECT AND/OR CONTRACTOR TO ENSURE WALL CAN RESIST APPLIED LOADING.
2. FRAME AND DOOR SHOWN ARE RHR. LHR IS A MIRROR-IMAGE.



ELEVATION





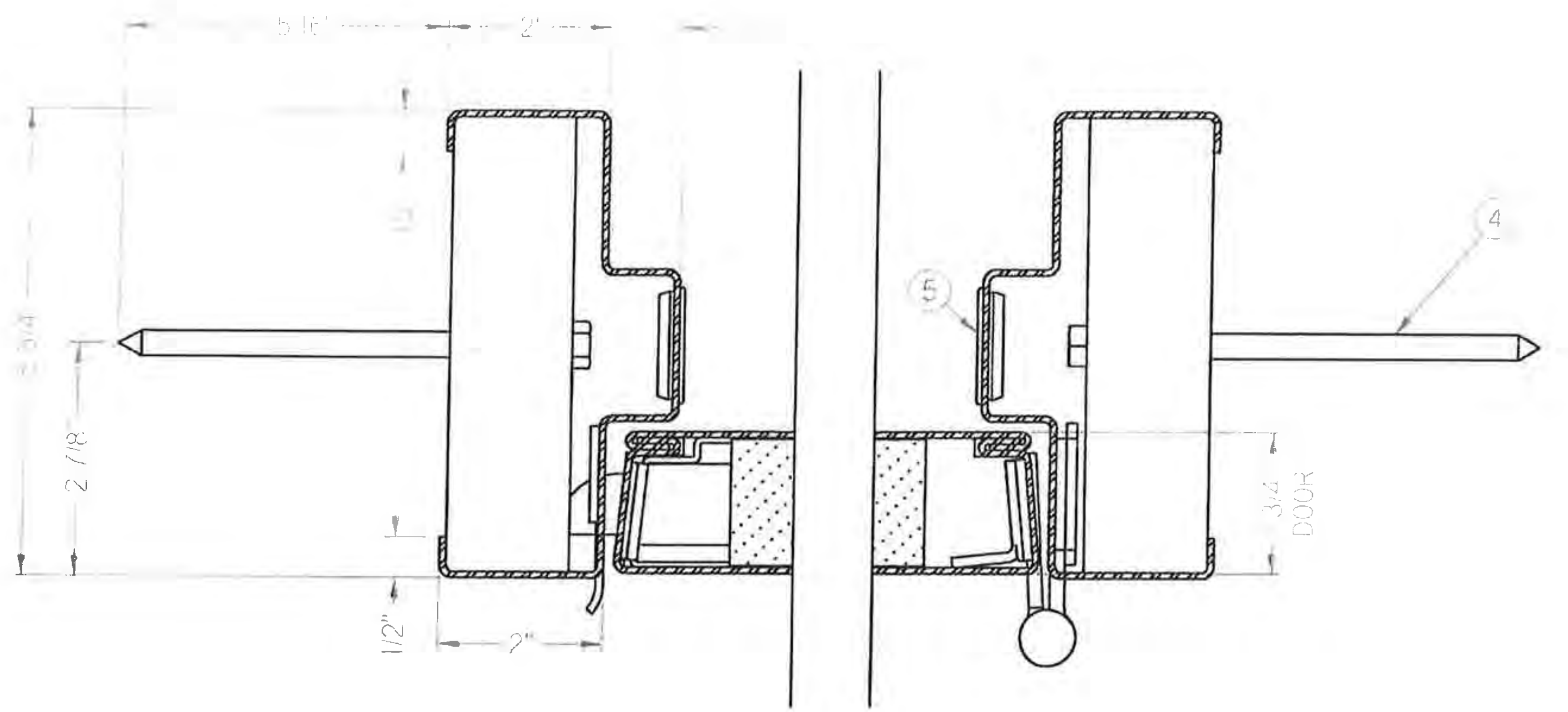
ISOMETRIC VIEW

Item No.	Title	Qty.
1	Blast Resistant Frame	1
2	Blast Resistant Door	1
3	4 1/2" x 4 1/2" Heavy Weight Hinge (By Others)	3
4	5/16" x 5.5" SWG ASSY Kombi Screw	28
5	Hole Plug for 1.25" Hole	28
6	Mortised Lock (By Others)	1

AMBICO
SPECIALIZED STEEL DOORS & FRAMES

TITLE: CHARGE WEIGHT II, 105' STANDOFF, LOW LEVEL OF PRTOECTION BLAST RESISTANT FRAME AND DOOR - TAG 128

DATE	03/02/2015	ORDER NO	
DRAWN BY	tberry	CUSTOMER NO	A18898
REVISION	2	SHEET	5 OF 6



SECTION A-A

AMBICO SPECIALIZED STEEL DOORS & FRAMES	TITLE: CHARGE WEIGHT II, 105' STANDOFF, LOW LEVEL OF PRTOECTION BLAST RESISTANT FRAME AND DOOR - TAG 128	DATE 03/02/2015	ORDER NO
		DRAWN BY: tberry	CUSTOMER NO A18898
		REVISION 2	SHEET 6 OF 6

APPENDIX D

QUICK-LOOK REPORT

Air Force Civil Engineer Center

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Blast Resistance of Cross-Laminated Timber Construction

Casey O'Laughlin, P.E.
Research Civil Engineer, AFCEC Contractor, Jacobs SLG

Overview

- Background
- Resistance Function Development
- Full-Scale Blast Validations
  - Setup
  - Results
    - Full Scale Validation #1
    - Full Scale Validation #2
    - Full Scale Validation #3
- Conclusions

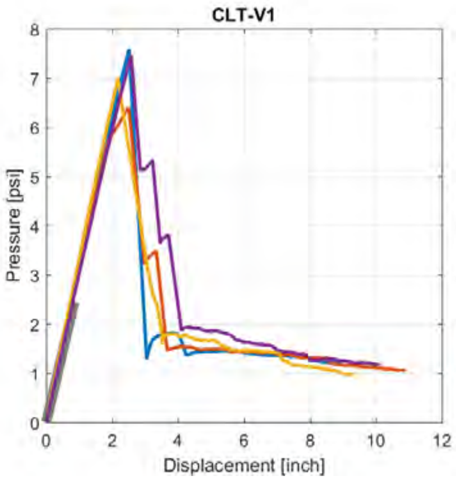
Background

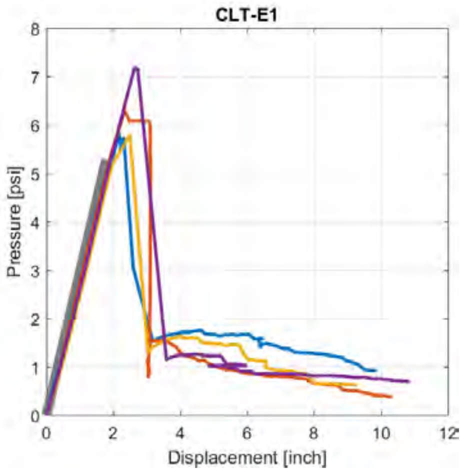
- Cross laminated timber (CLT) is an engineered wood building system consisting of dimensional lumber oriented at right angles to one another and glued to form structural panels
- Objective of effort is the development of blast design criteria for CLT construction
- Karagozian and Case Inc. (K&C) contracted by WoodWorks and worked in conjunction with University of Maine to evaluate blast resistance of CLT panels in static laboratory conditions
- CRADA developed between Karagozian and Case Inc. and AFCEC for execution of full scale blast validations

http://www.woodskyscrapers.com/cross-laminated-timber.html

Resistance Function Development

- Water bladder at the University of Maine used to perform static evaluation of CLT resistance
- Parameters included panel grade, ply number, dimensions, and boundary conditions
- Shock tube testing by PDC and University of Ottawa indicated a dynamic increase factor of between 1.2 and 1.35 for CLT (K&C)



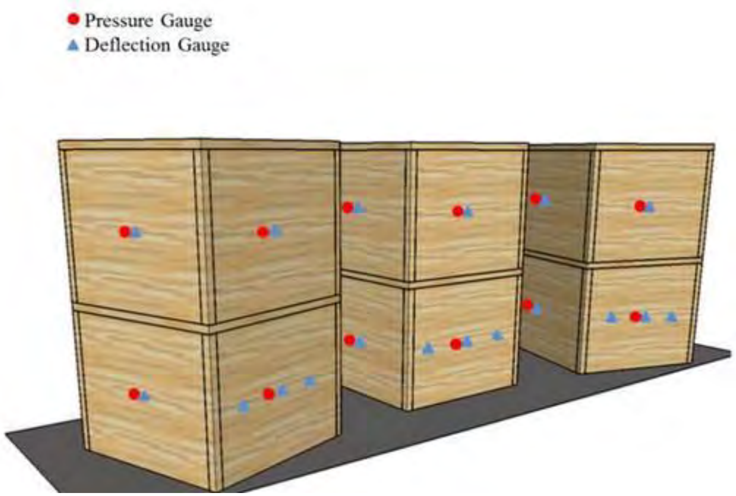


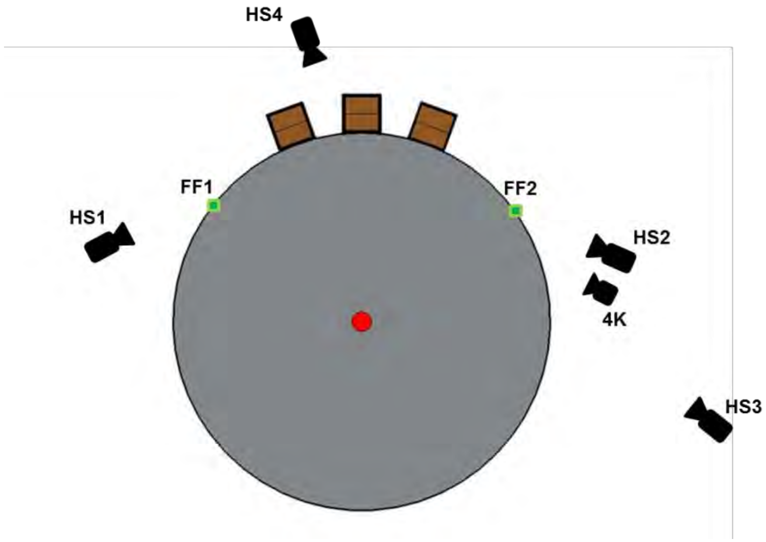
K&C

Full-Scale Blast Validations

Buildings labeled according to grade of CLT panels

Full-Scale Validation Setup





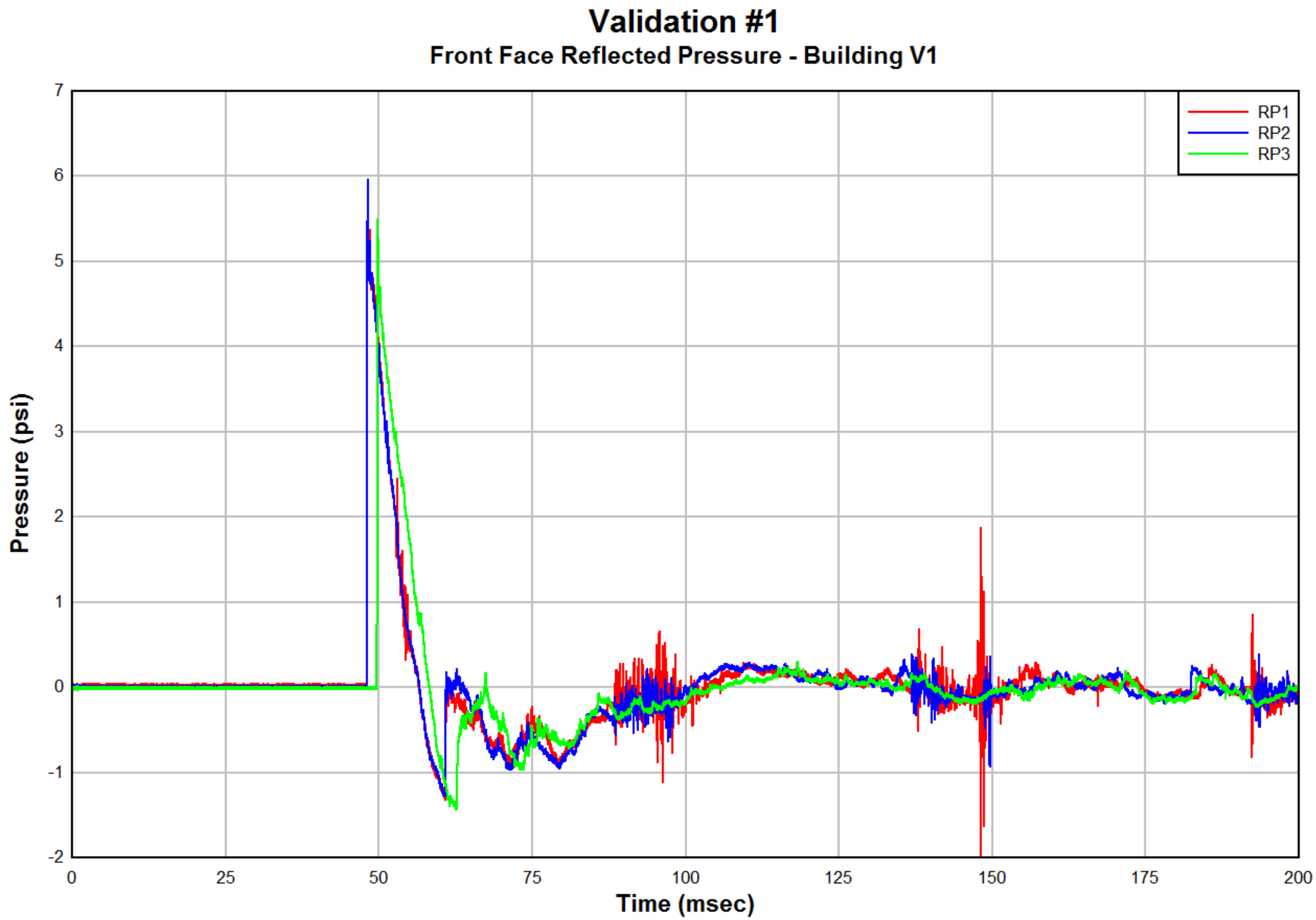
- 62 total gauges
- 24 reflected/incident pressure gauges (8 per building)
- 33 deflection gauges (11 per building)
- 3 internal pressure gauges (1 per building)
- 2 free field incident pressure gauges
- 4 high speed cameras
- 1 4k real-time camera

Full-Scale Validation #1

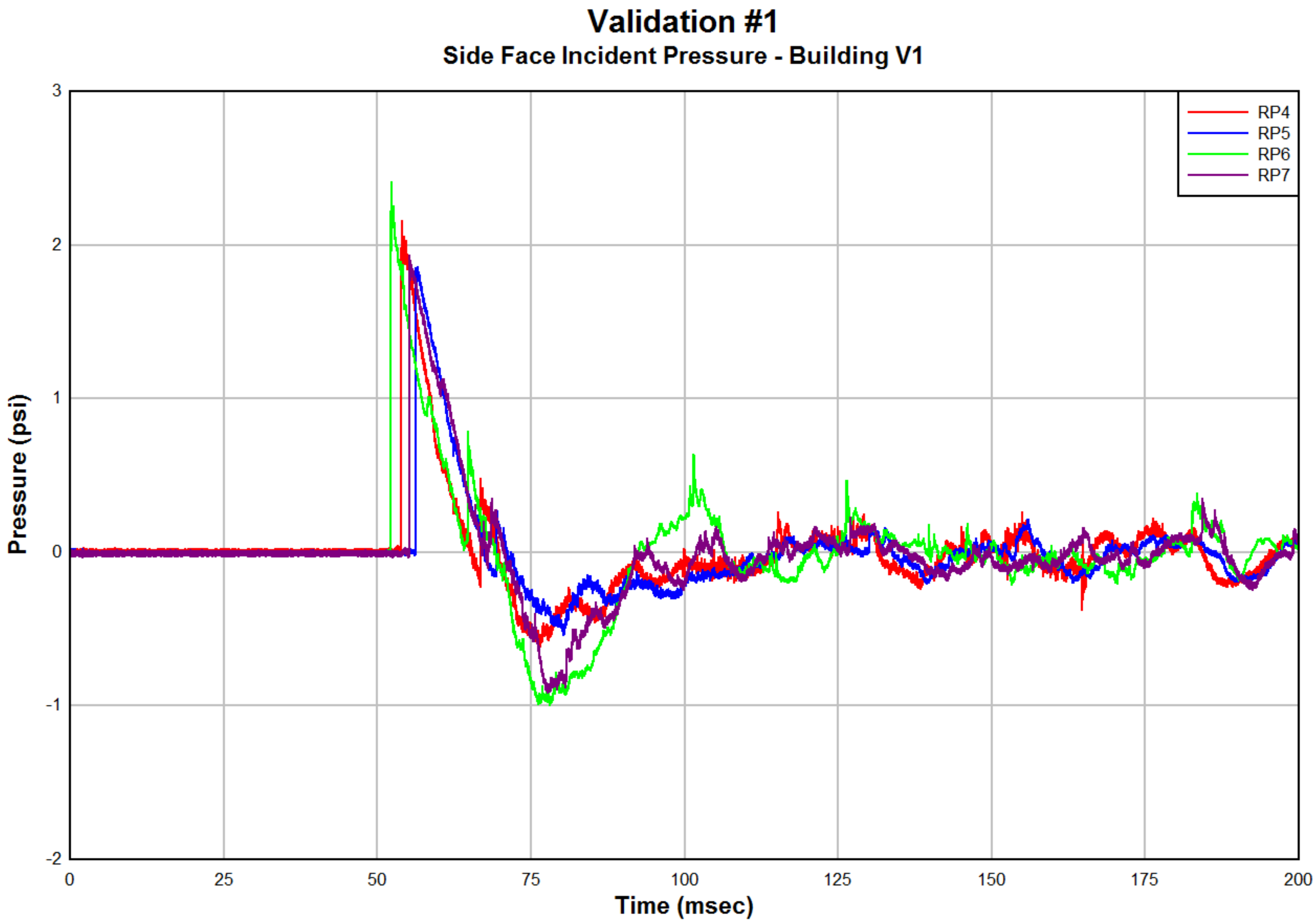
Pre-test

Post-test

Validation #1 – Building V1: Front Face Reflected Pressure Gauges



Validation #1 – Building V1: Side Face Incident Pressure Gauges



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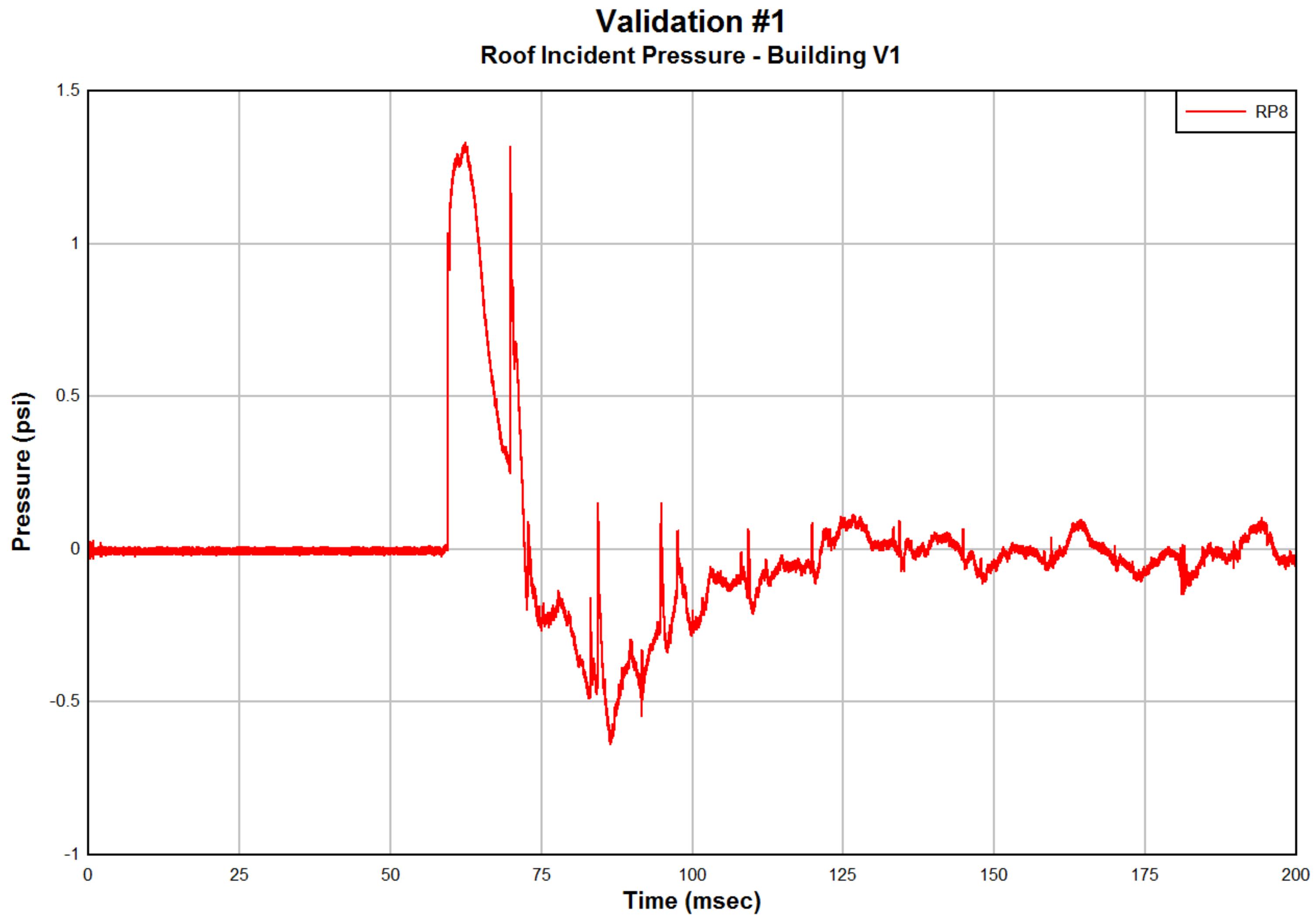
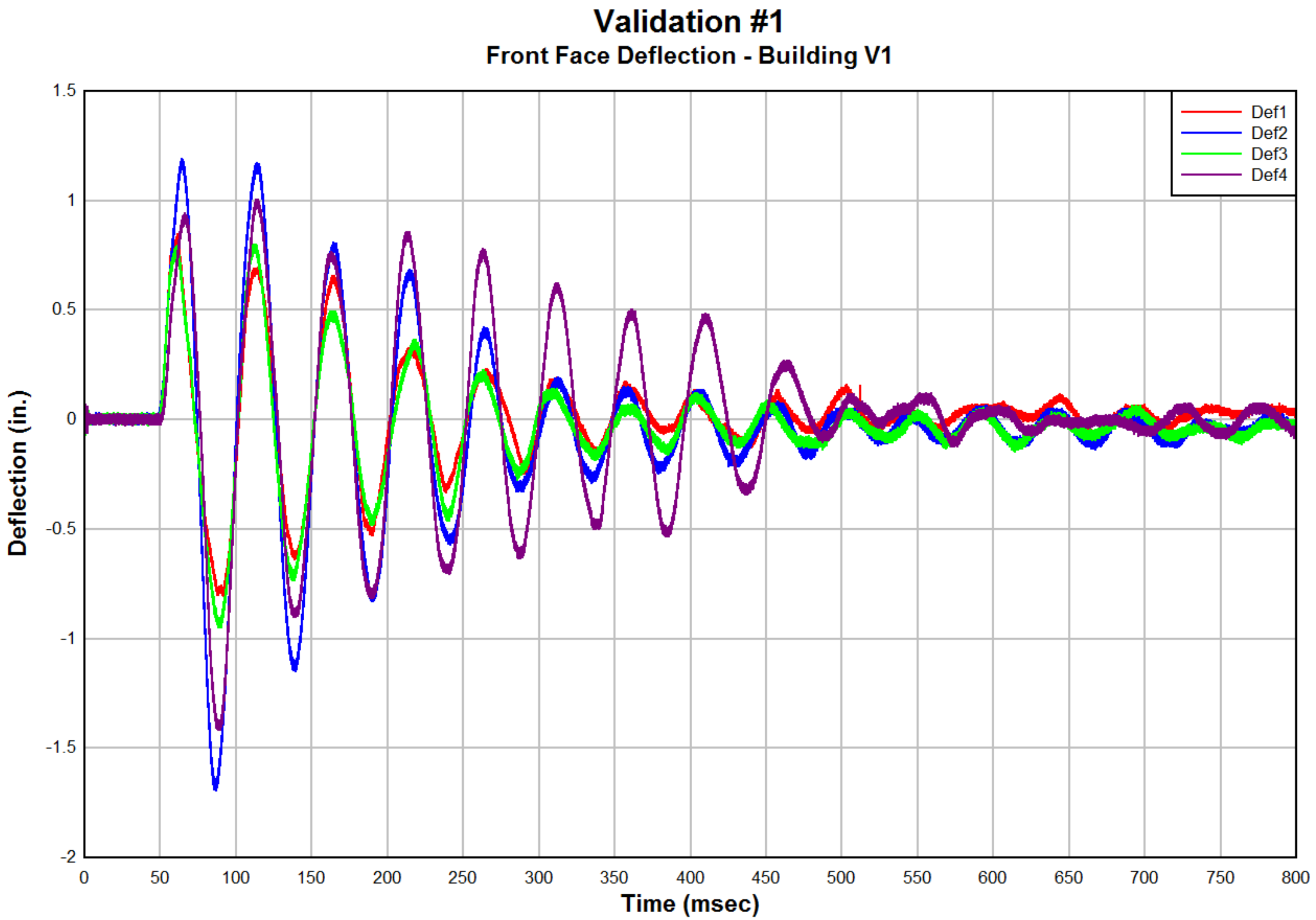
Validation #1 – Building V1: Roof Incident Pressure Gauges



Validation #1 – Building V1: Front Face Deflection Gauges



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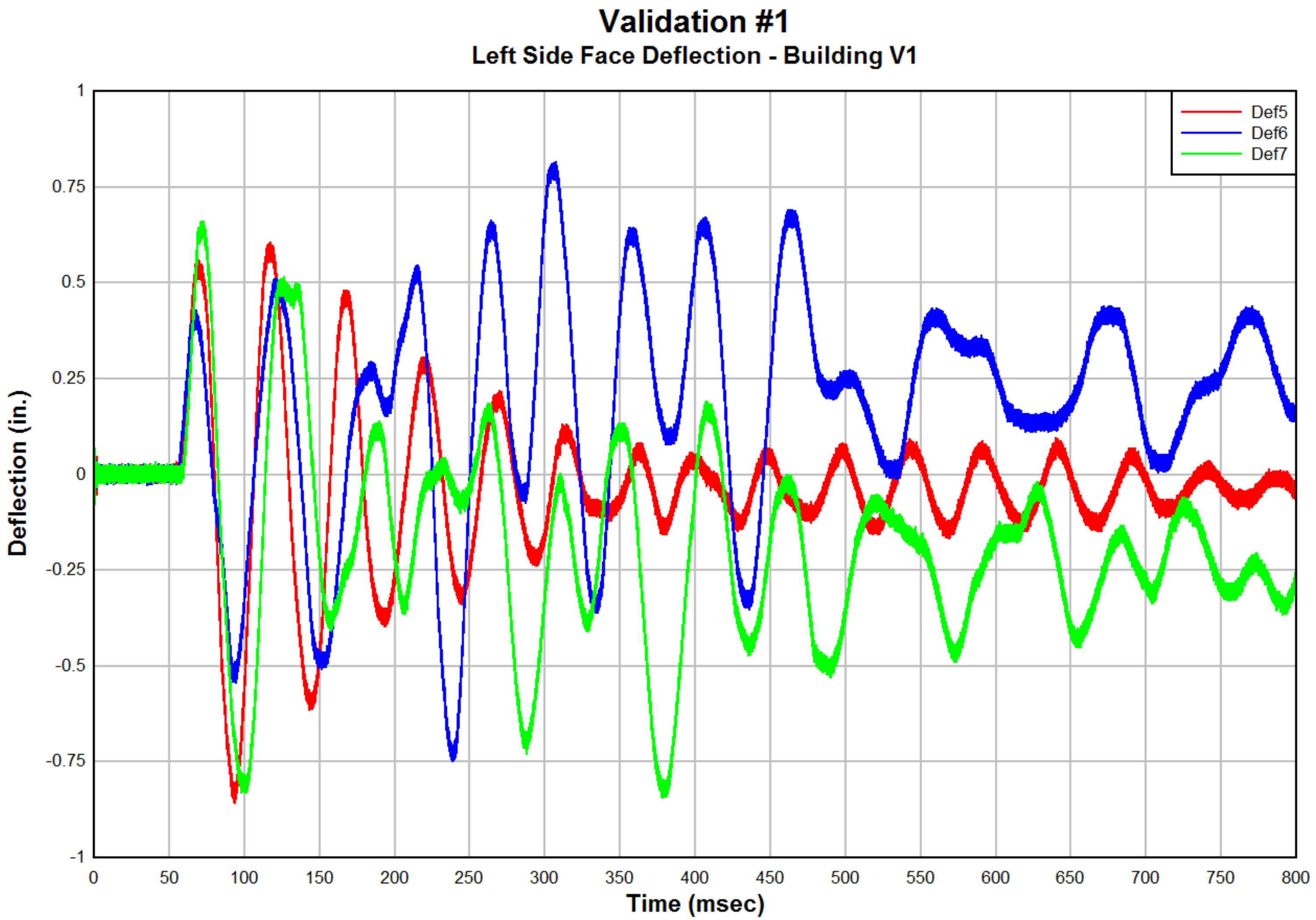
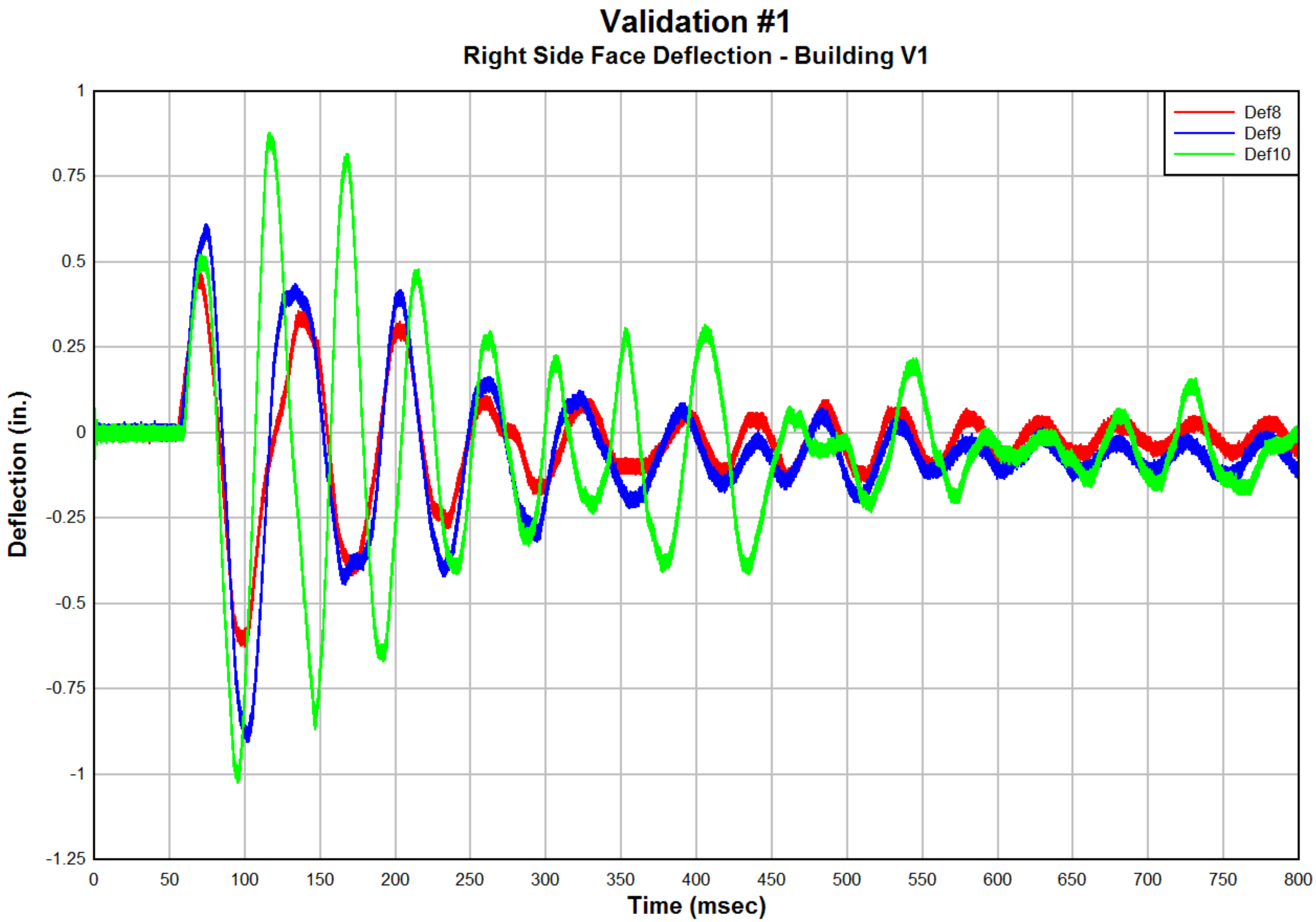
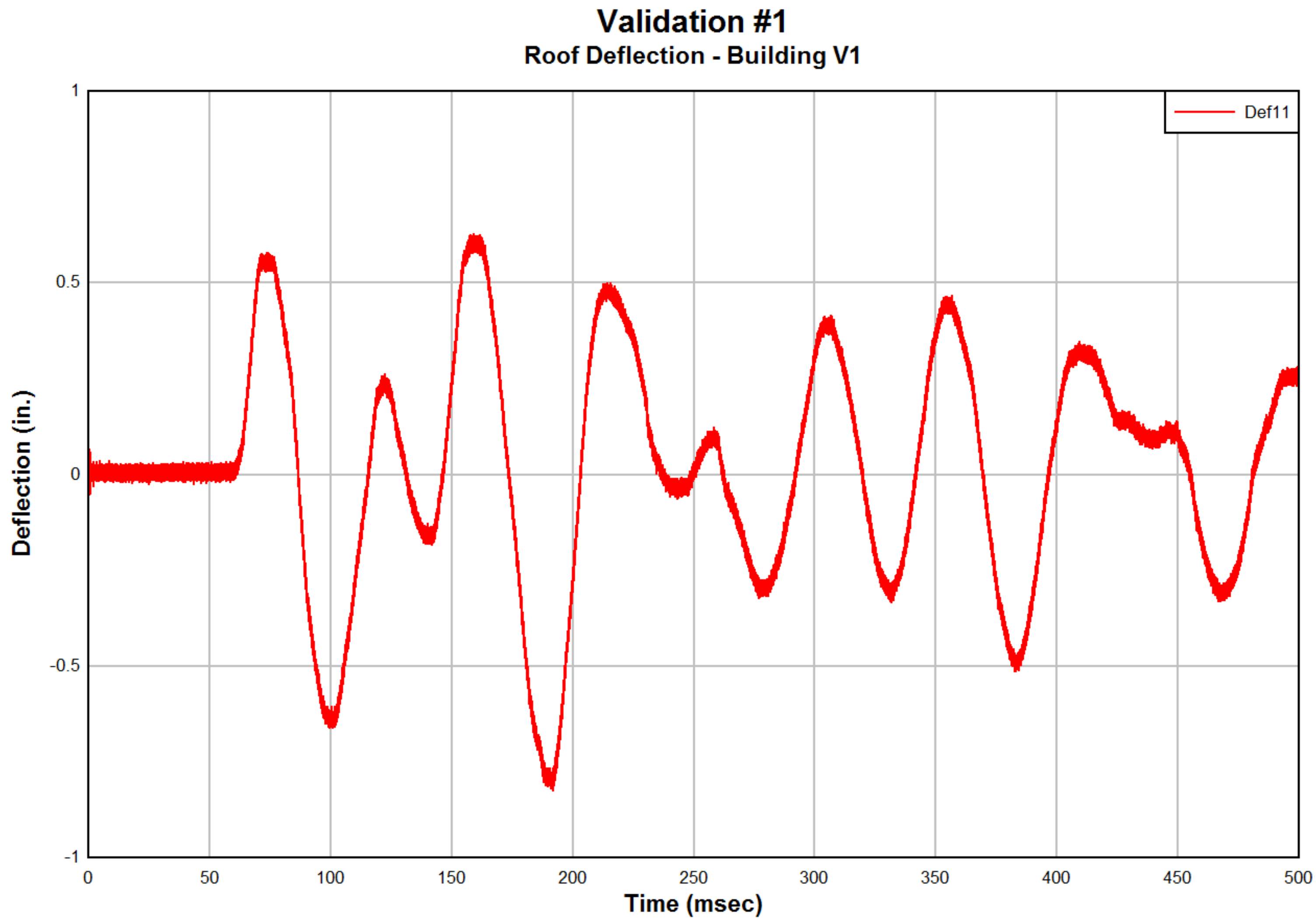
Validation #1 – Building V1: Left Side Face Deflection Gauges



Validation #1 – Building V1: Right Side Face Deflection Gauges



Validation #1 – Building V1: Roof Deflection Gauge



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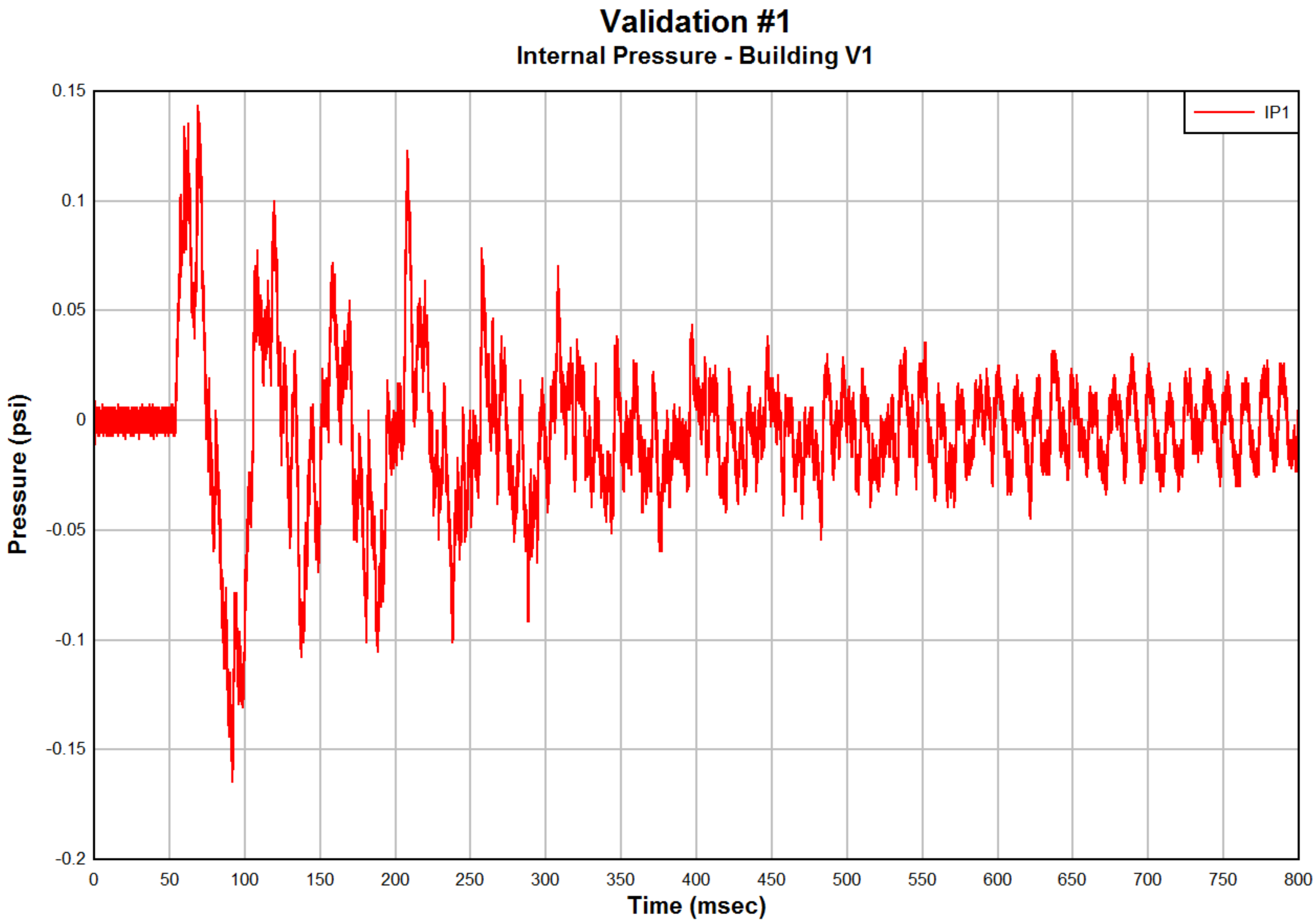
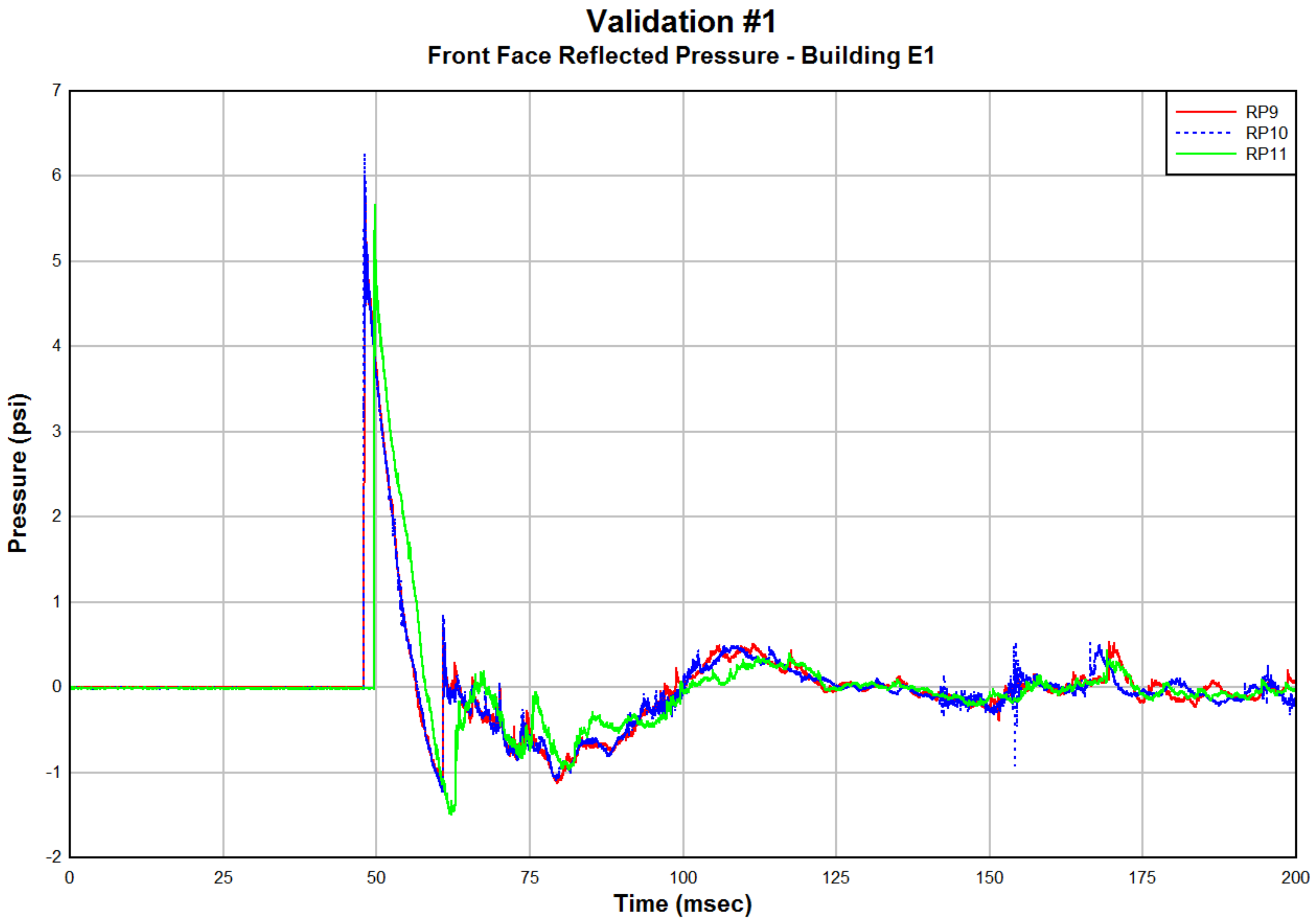
Validation #1 – Building V1: Internal Pressure Gauge



Validation #1 – Building E1: Front Face Reflected Pressure Gauges



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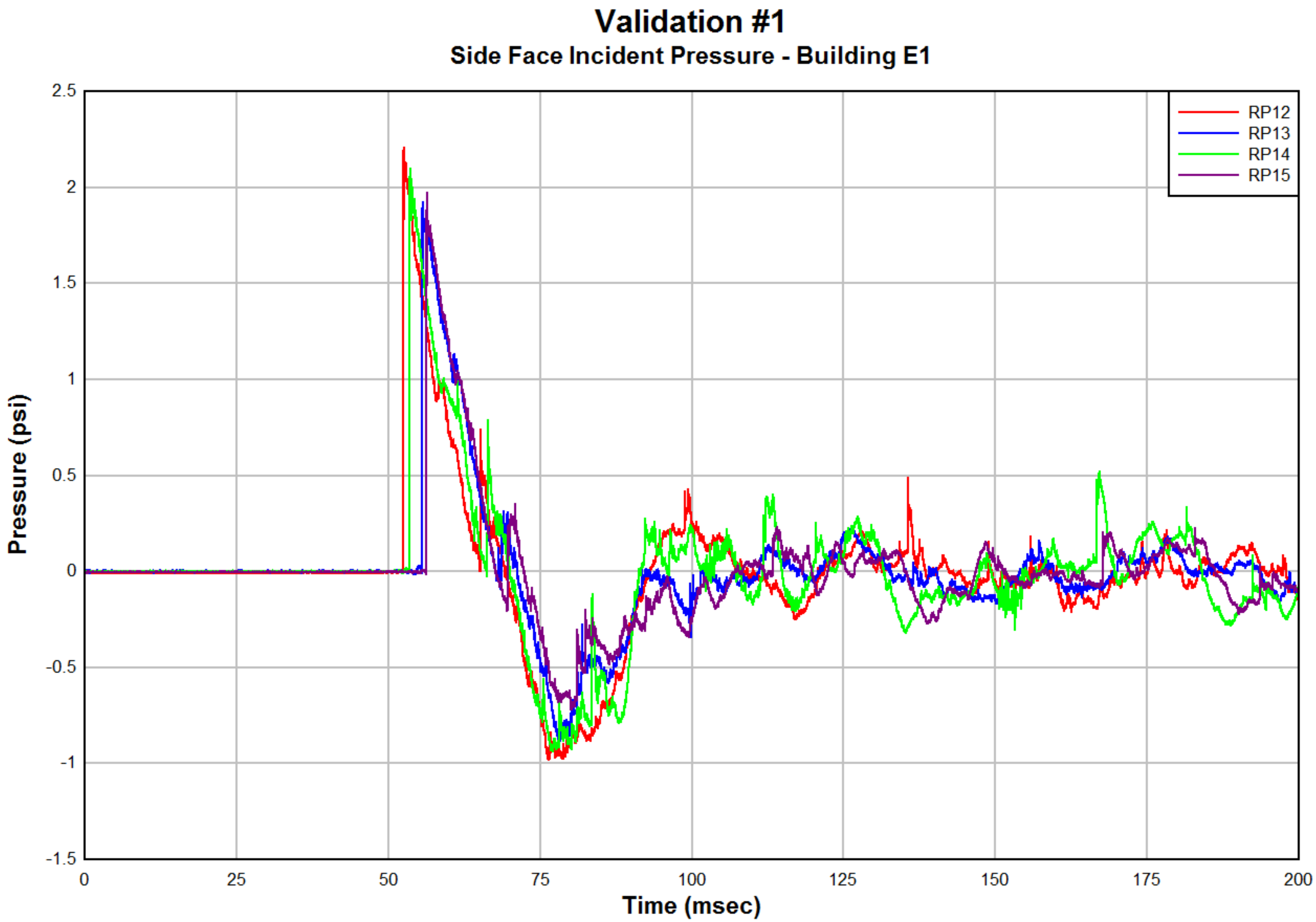

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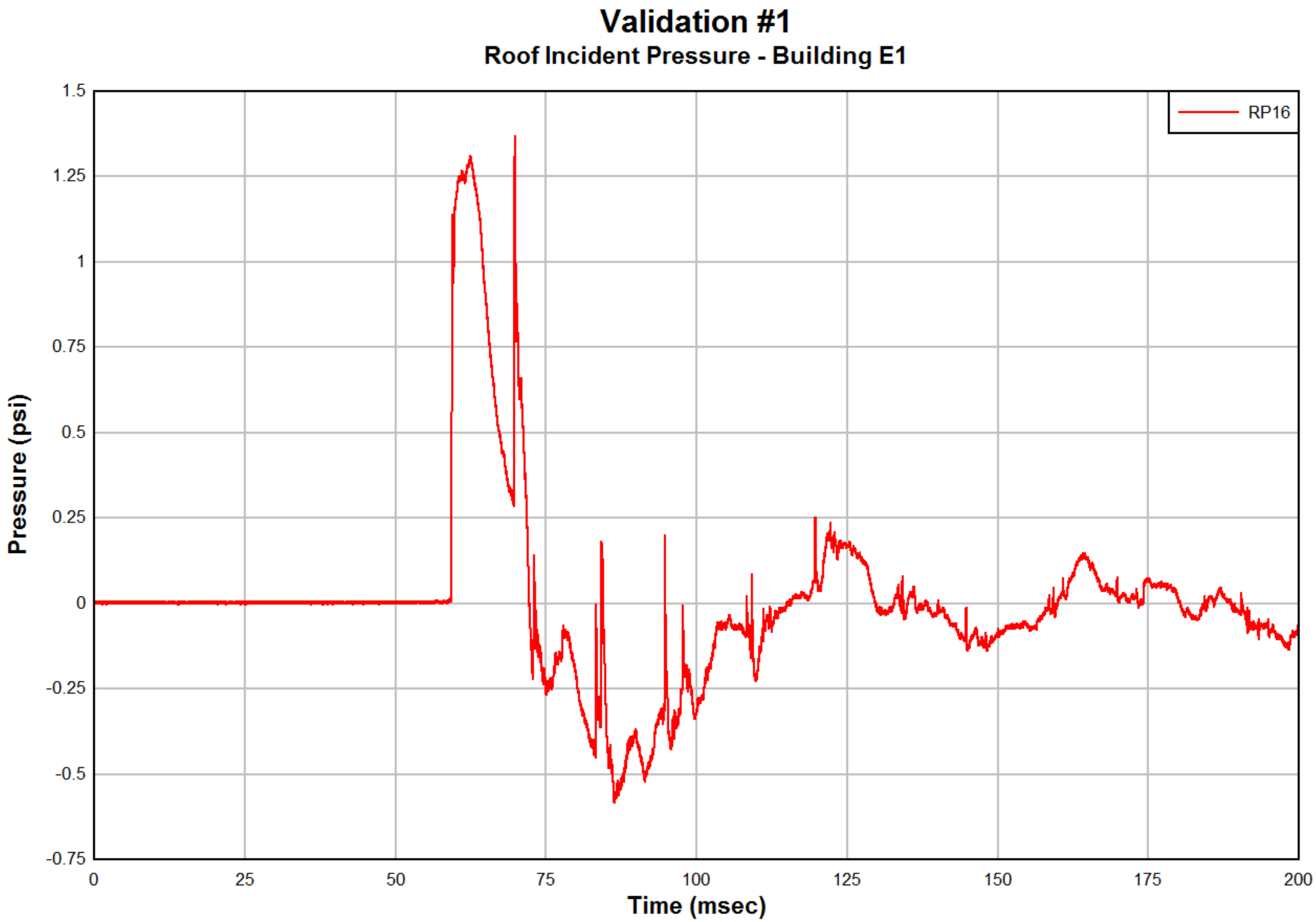

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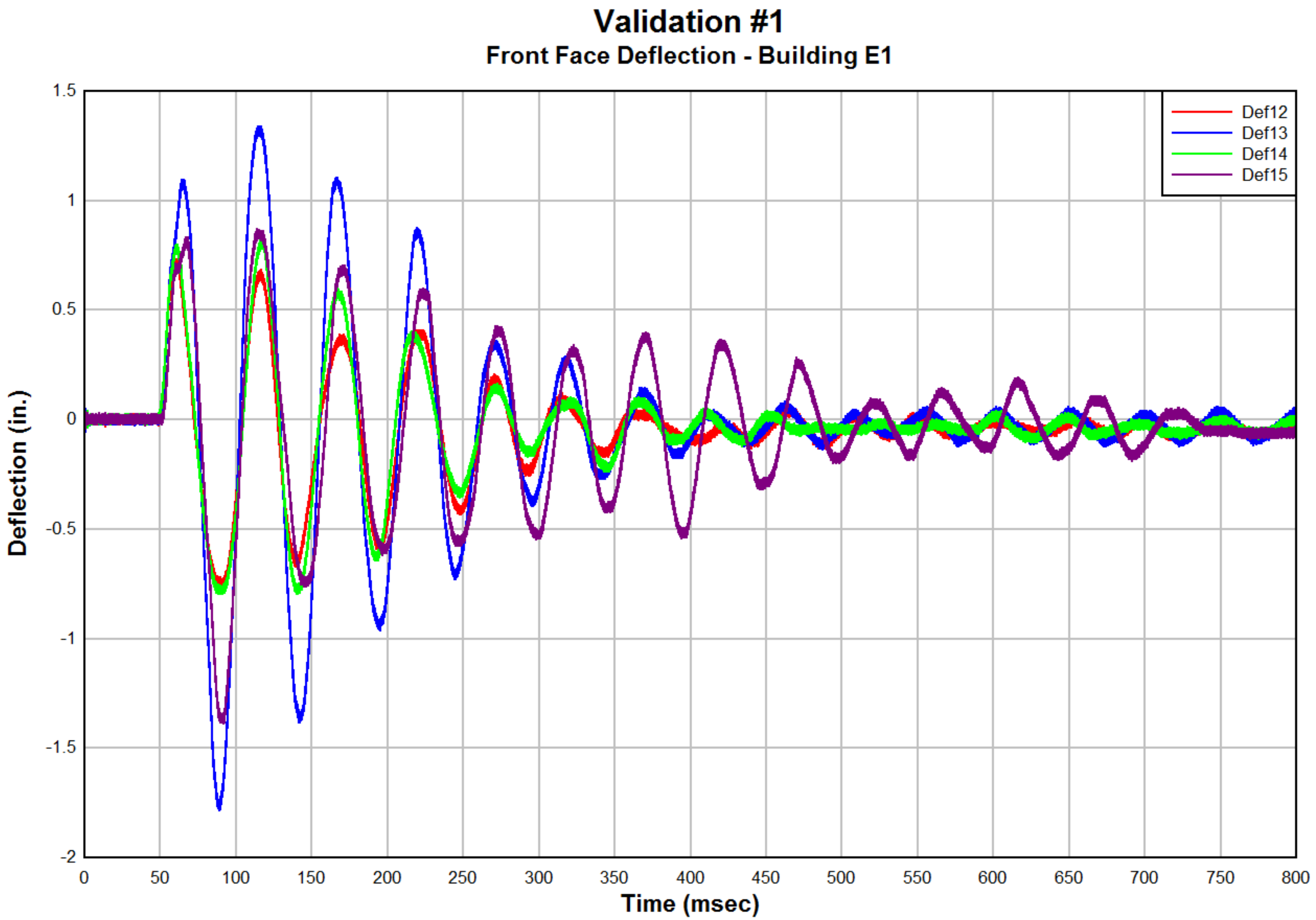

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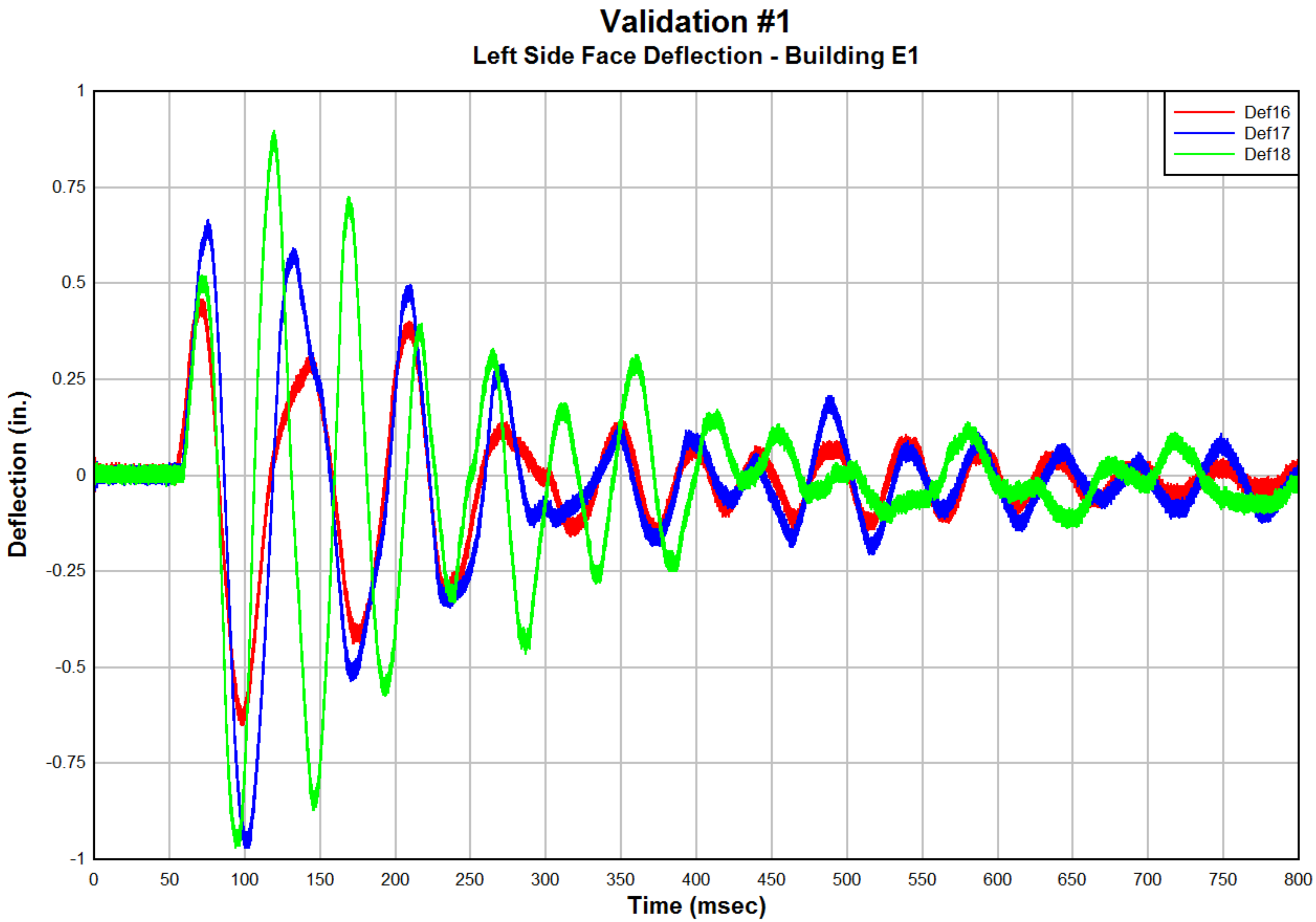
Validation #1 – Building E1: Left Side Face Deflection Gauges



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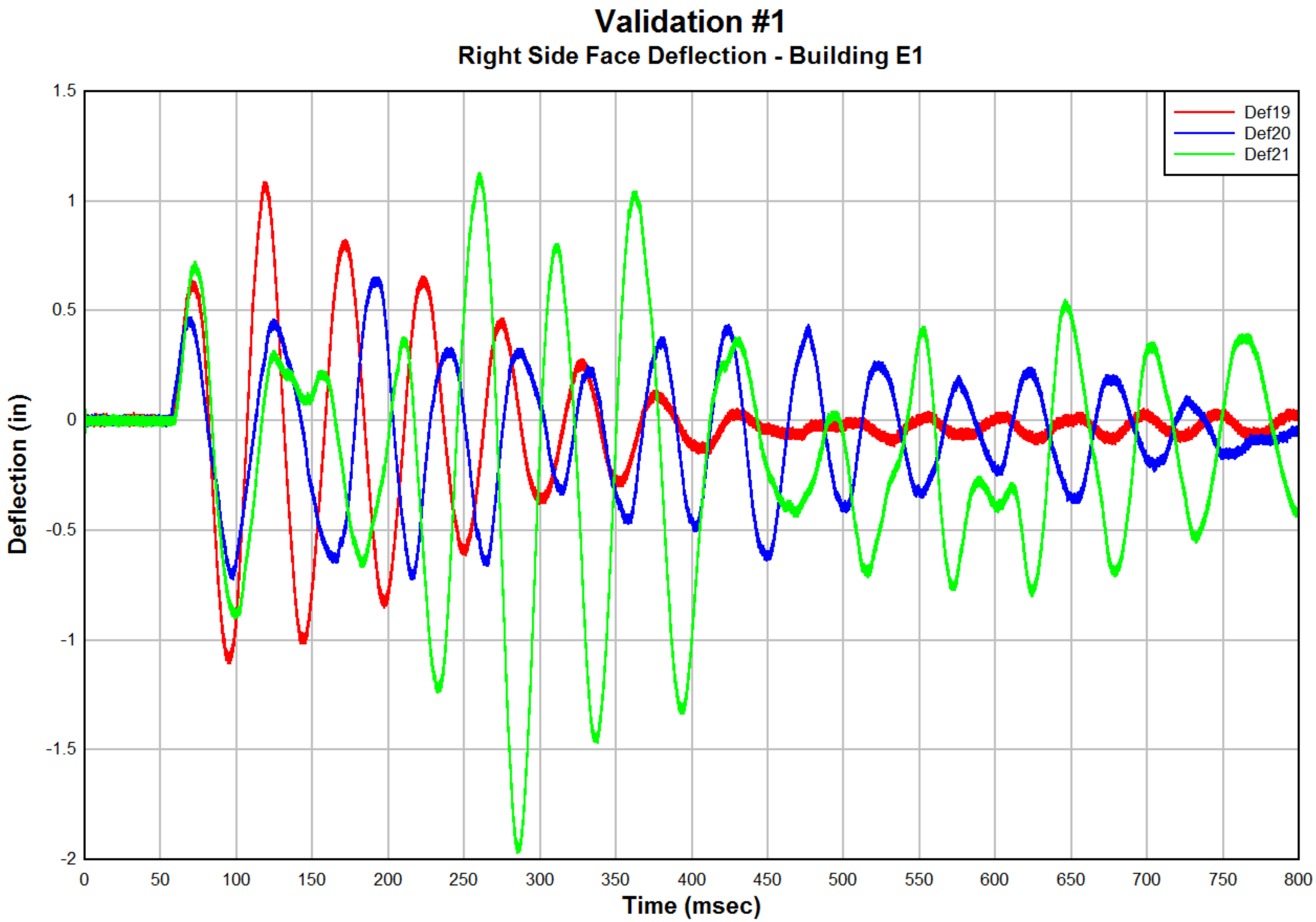

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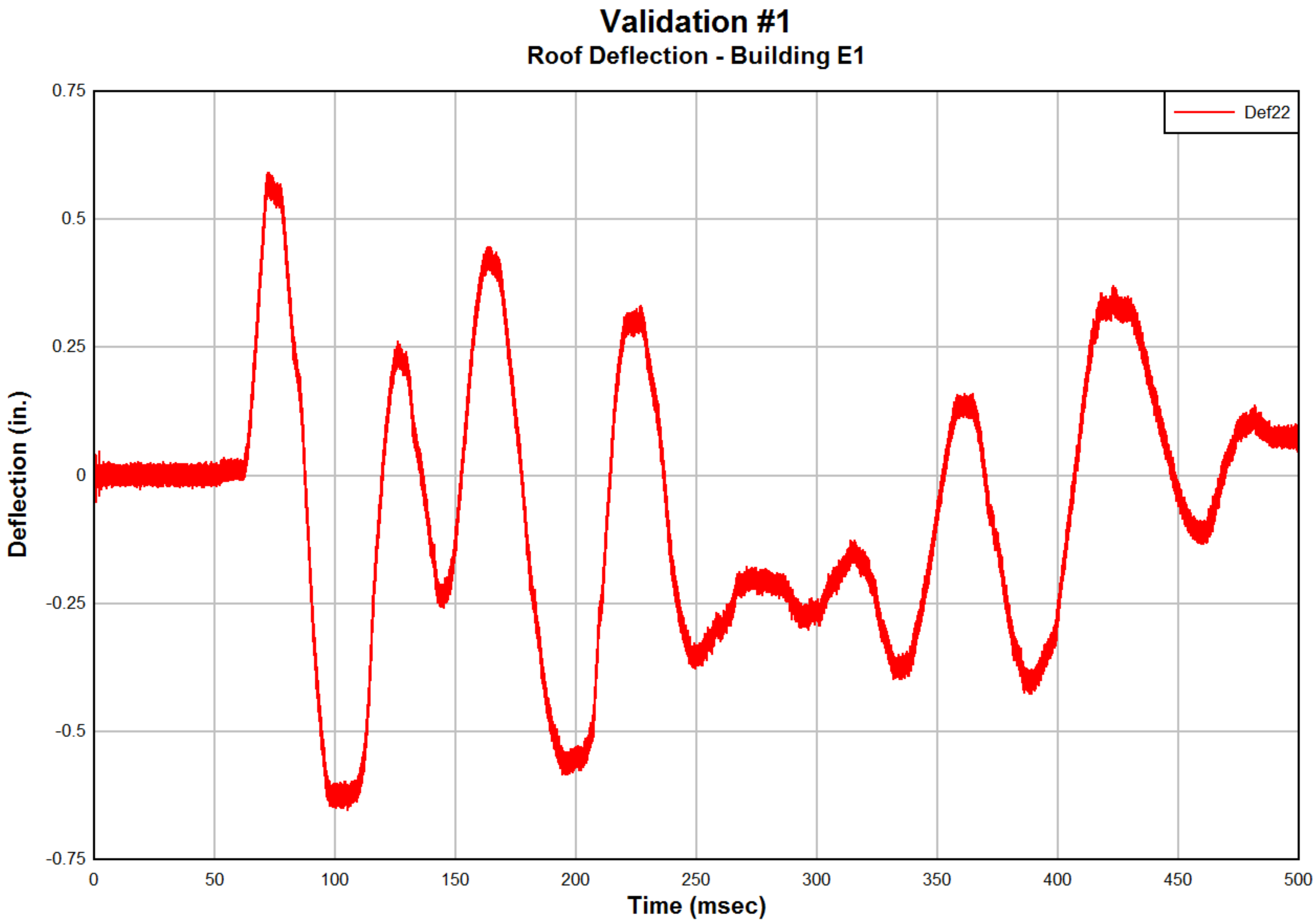
Validation #1 – Building E1: Roof Deflection Gauge



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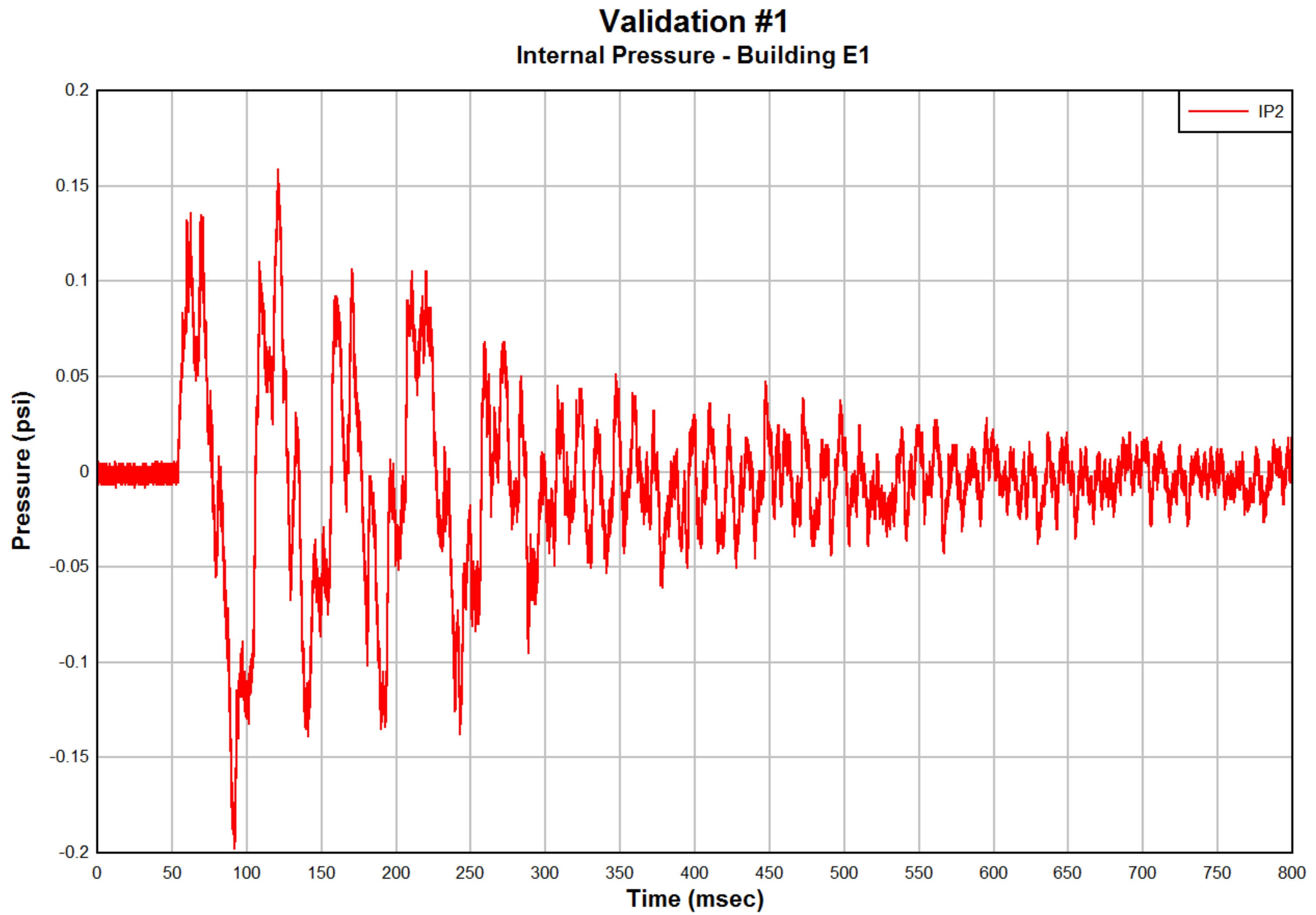
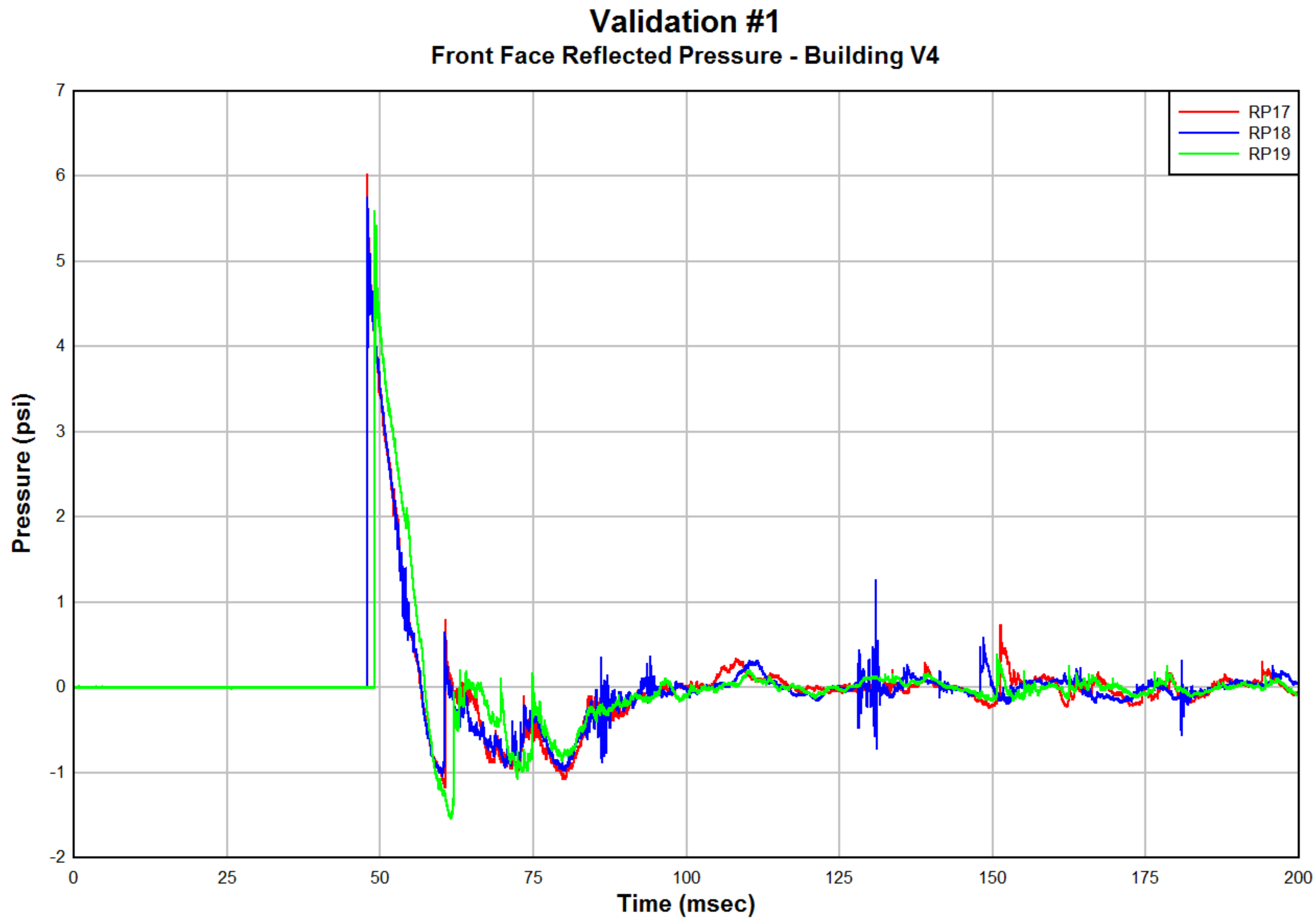
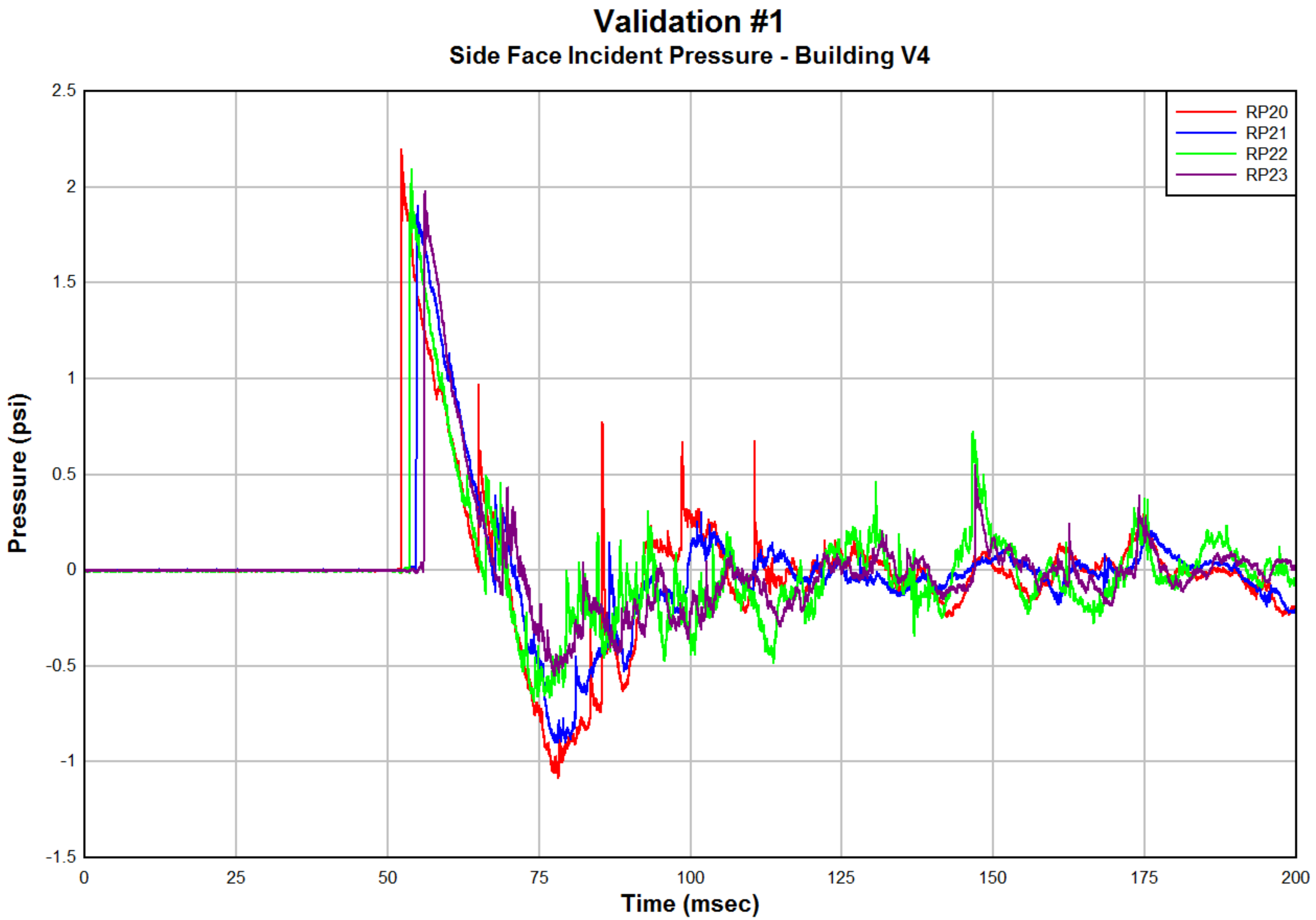


Validation #1 – Building V4: Front Face Reflected Pressure Gauges



Validation #1 – Building V4: Side Face Incident Pressure Gauges



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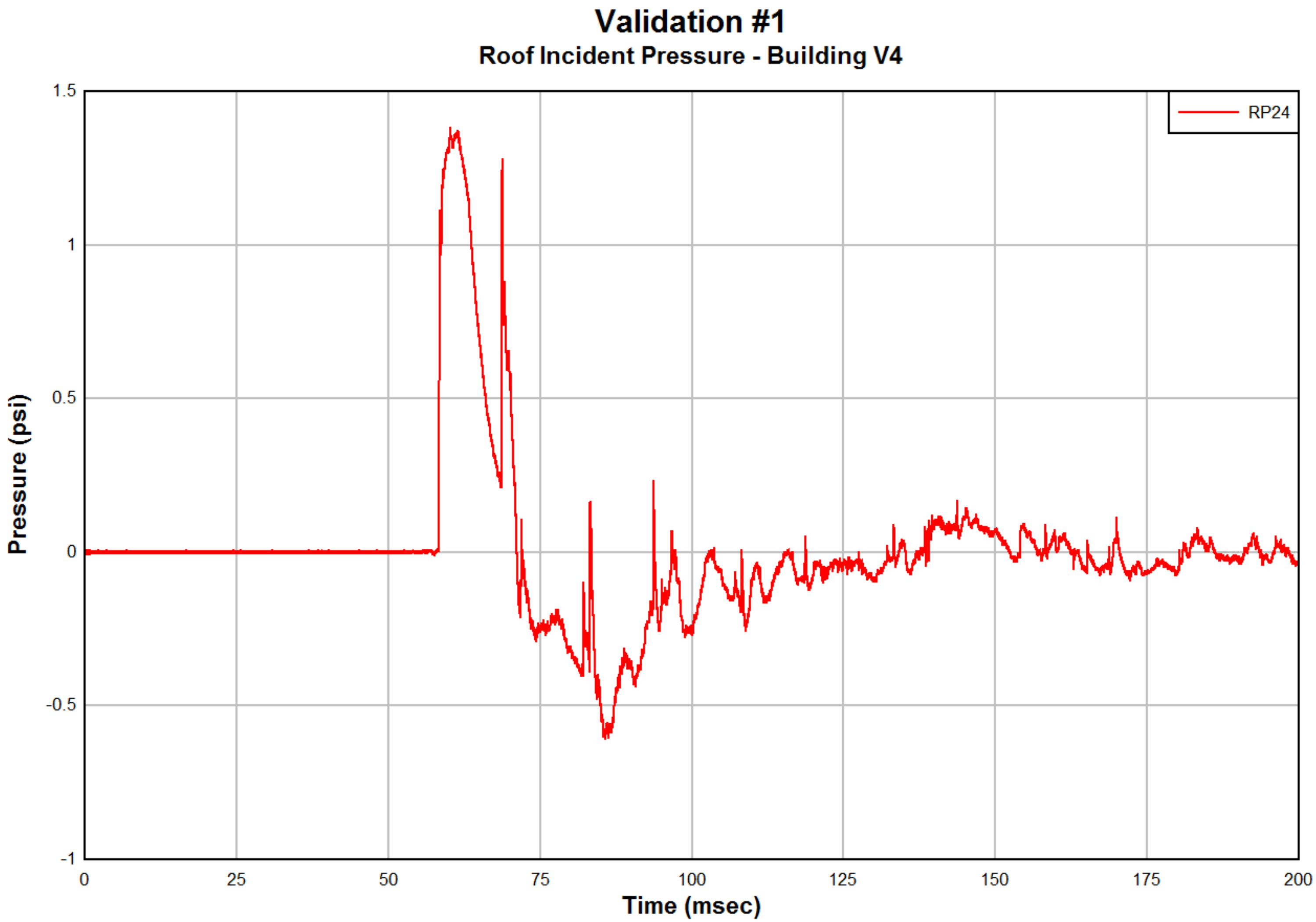

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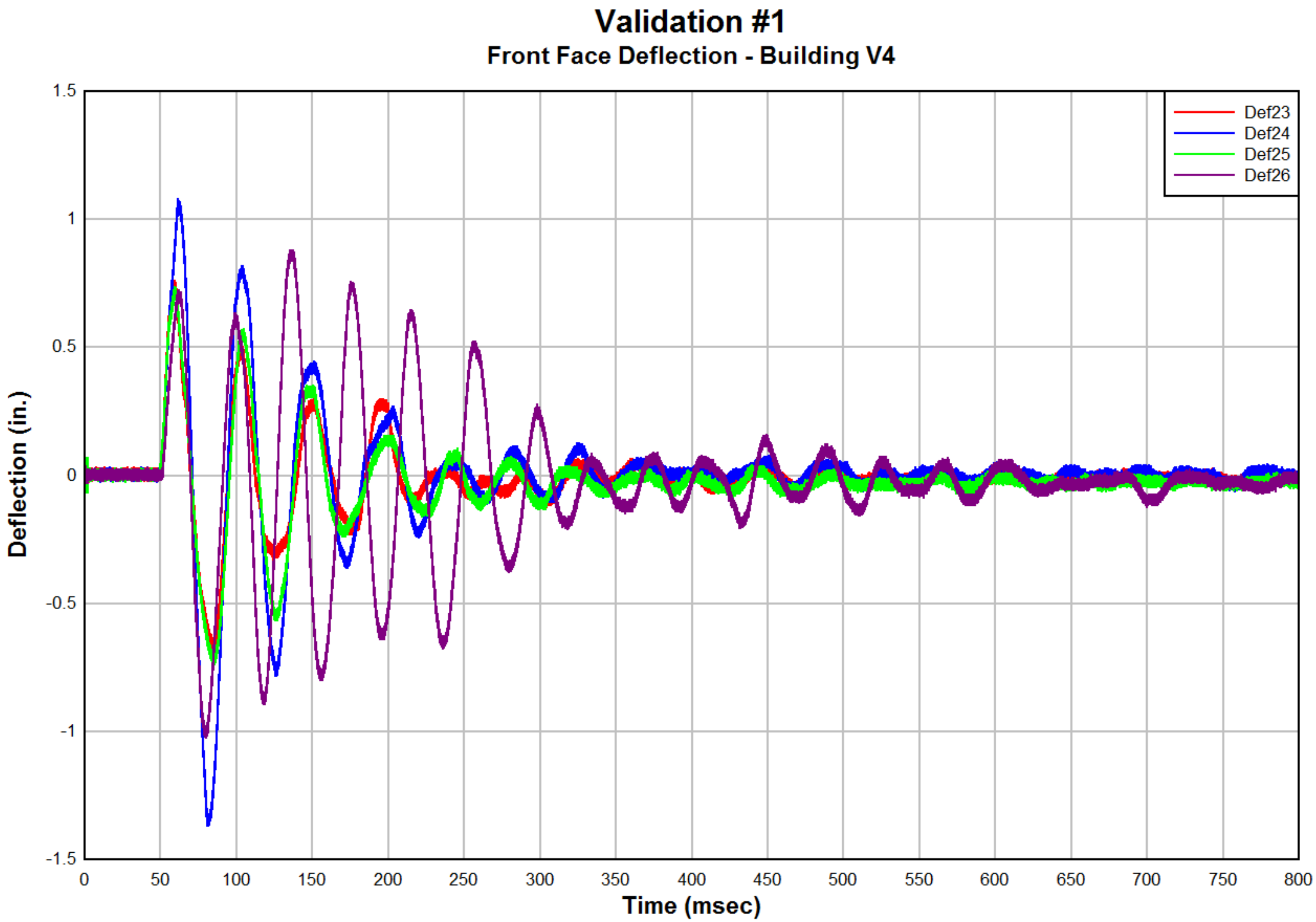

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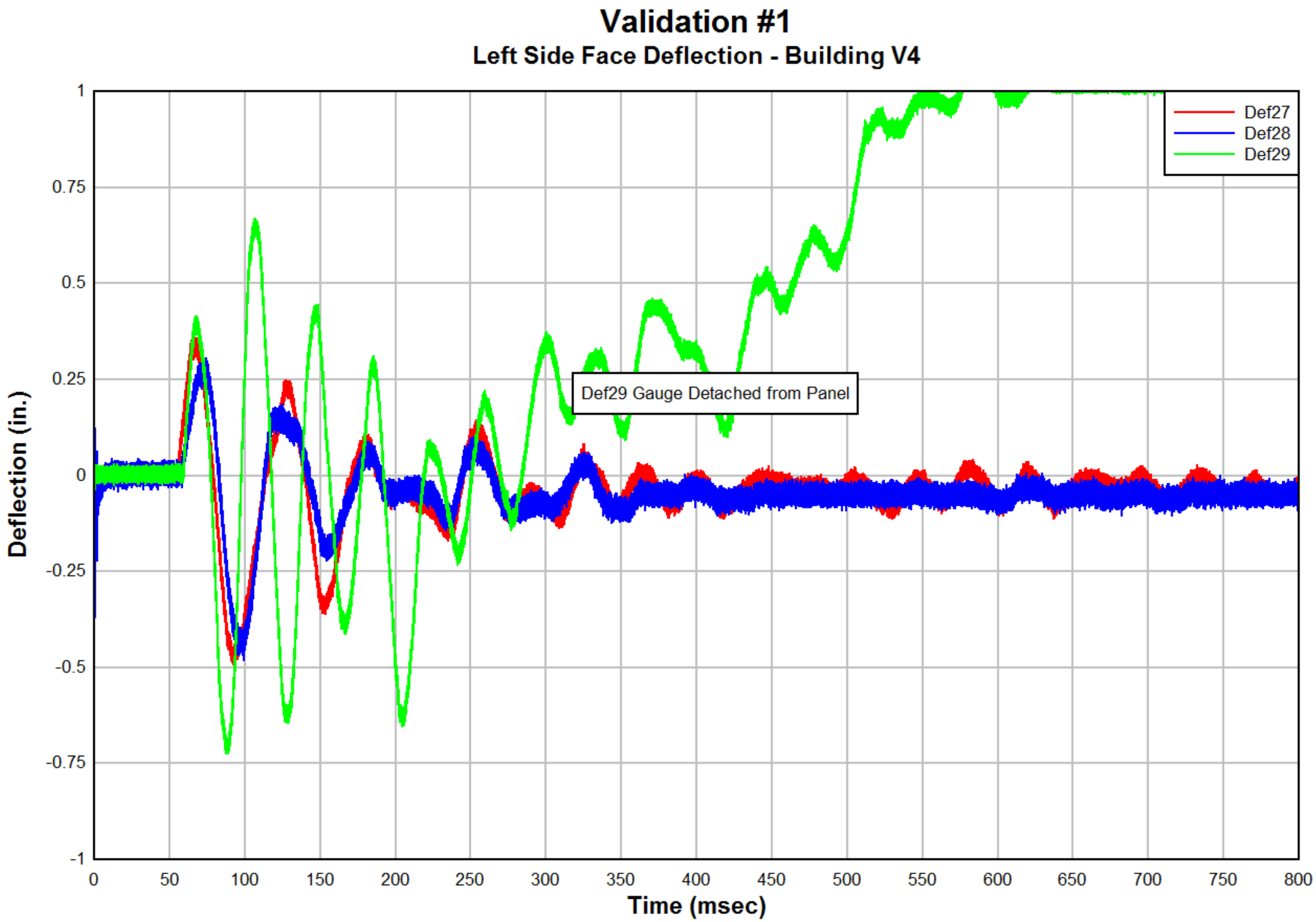
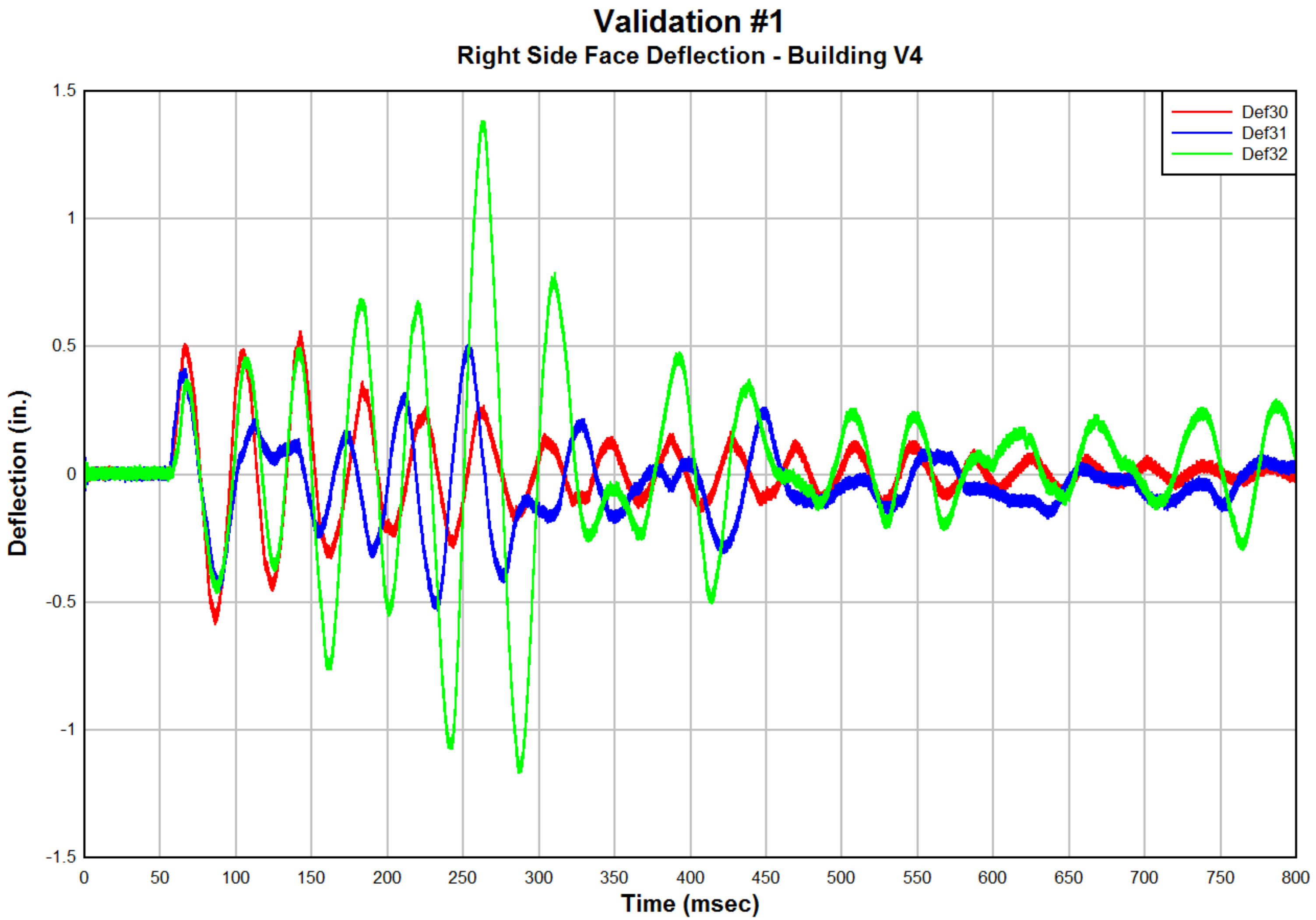
Validation #1 – Building V4: Left Side Face Deflection Gauges



Validation #1 – Building V4: Right Side Face Deflection Gauges



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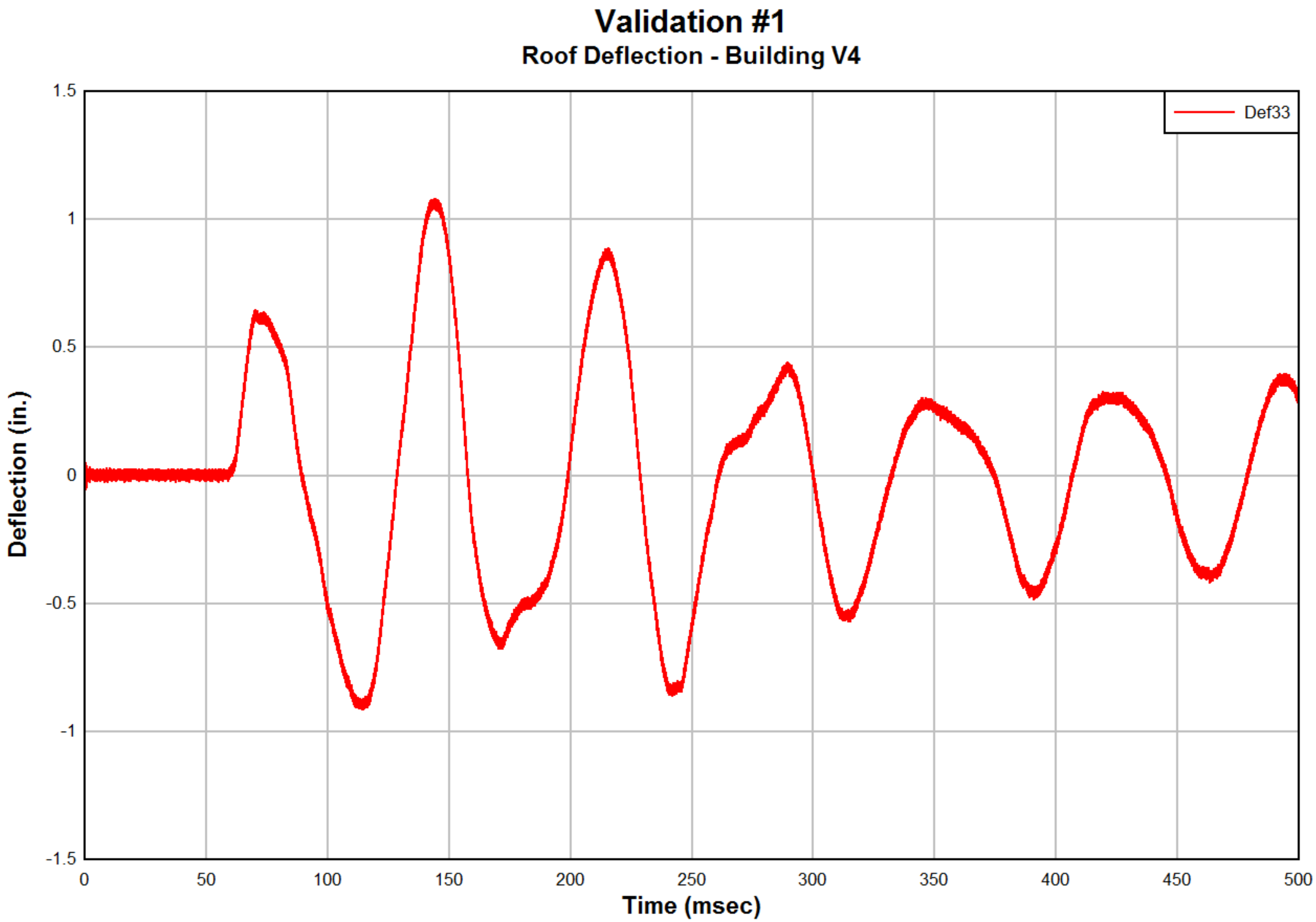

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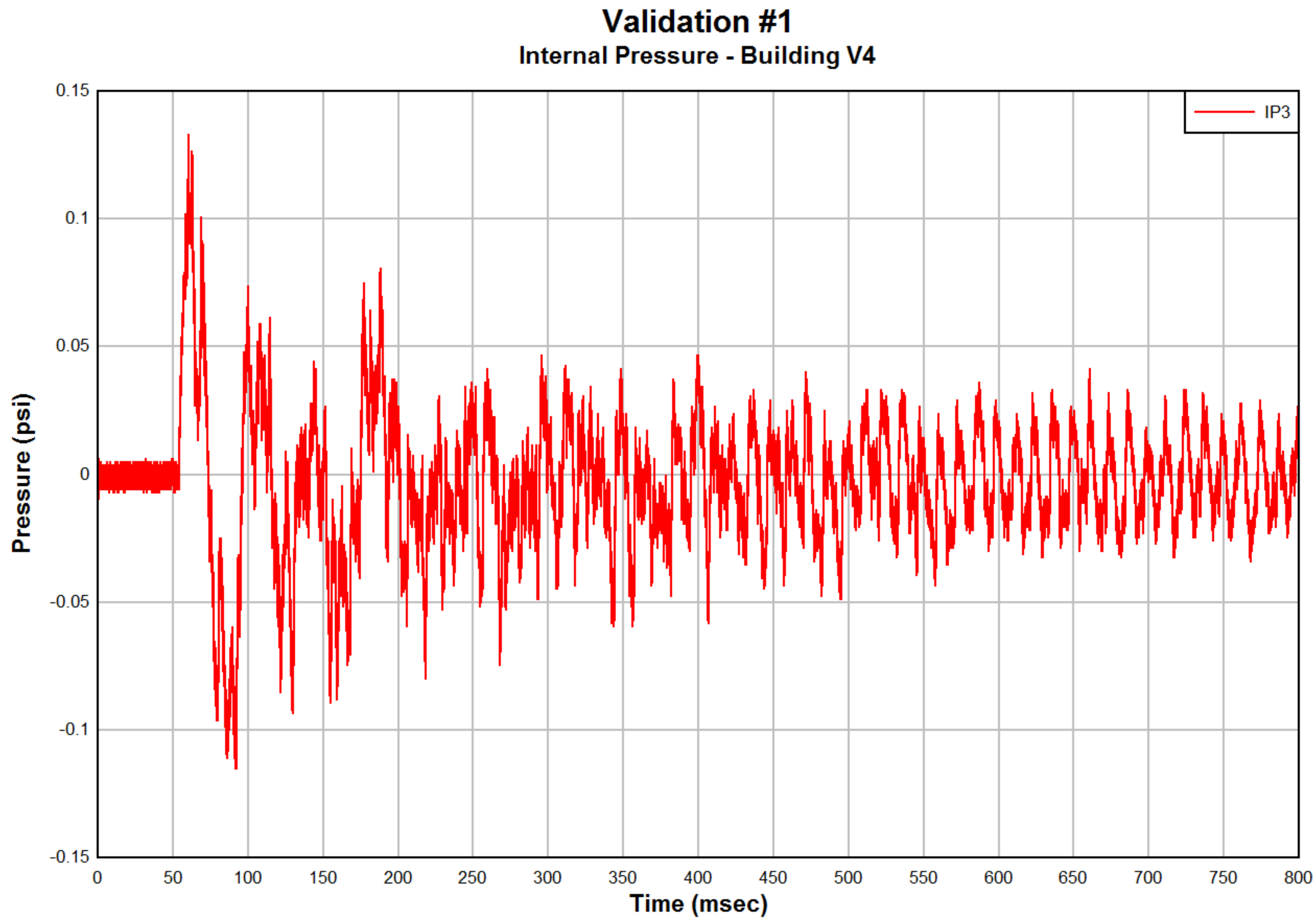
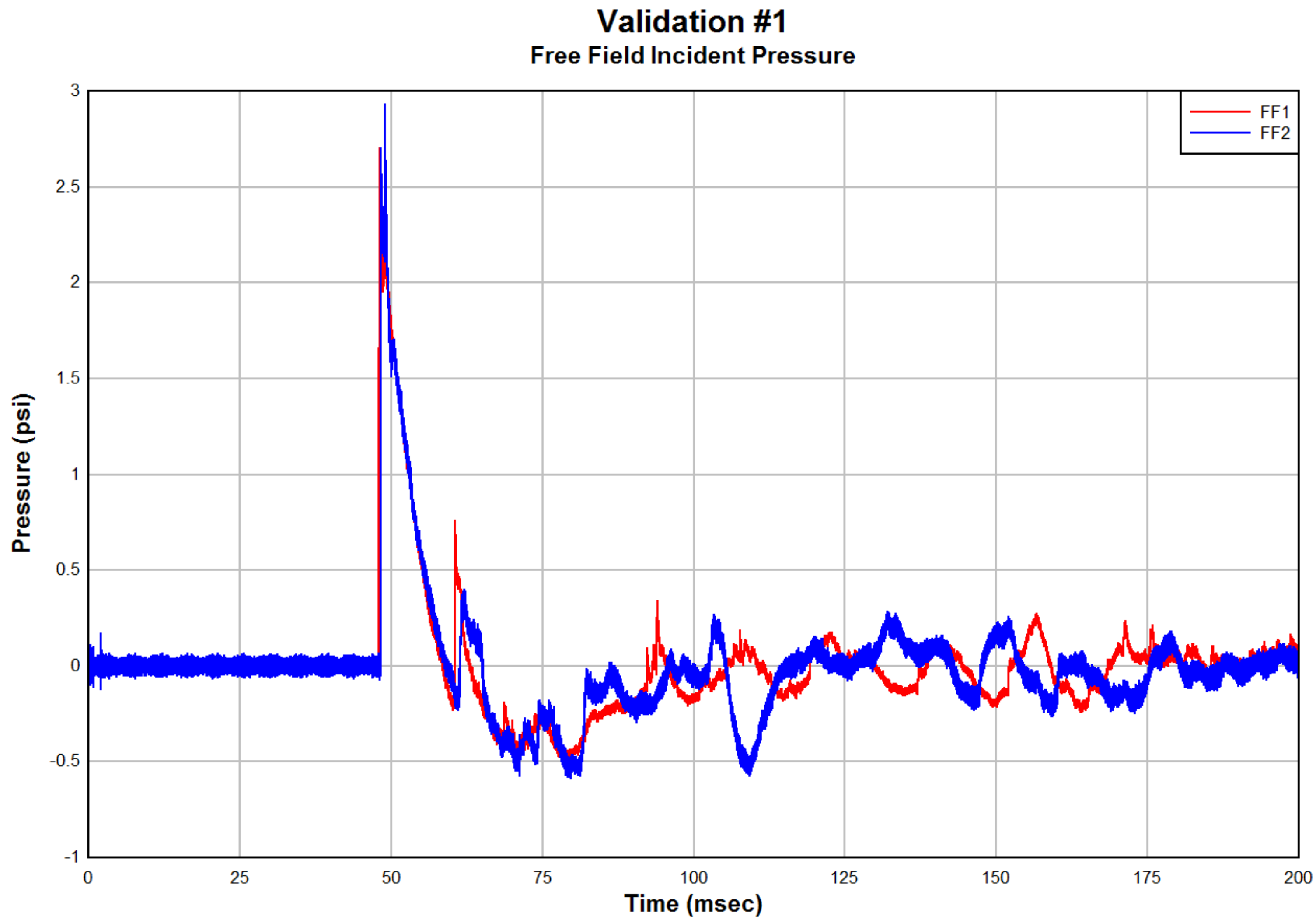


Validation #1: Free Field Incident Pressure Gauges

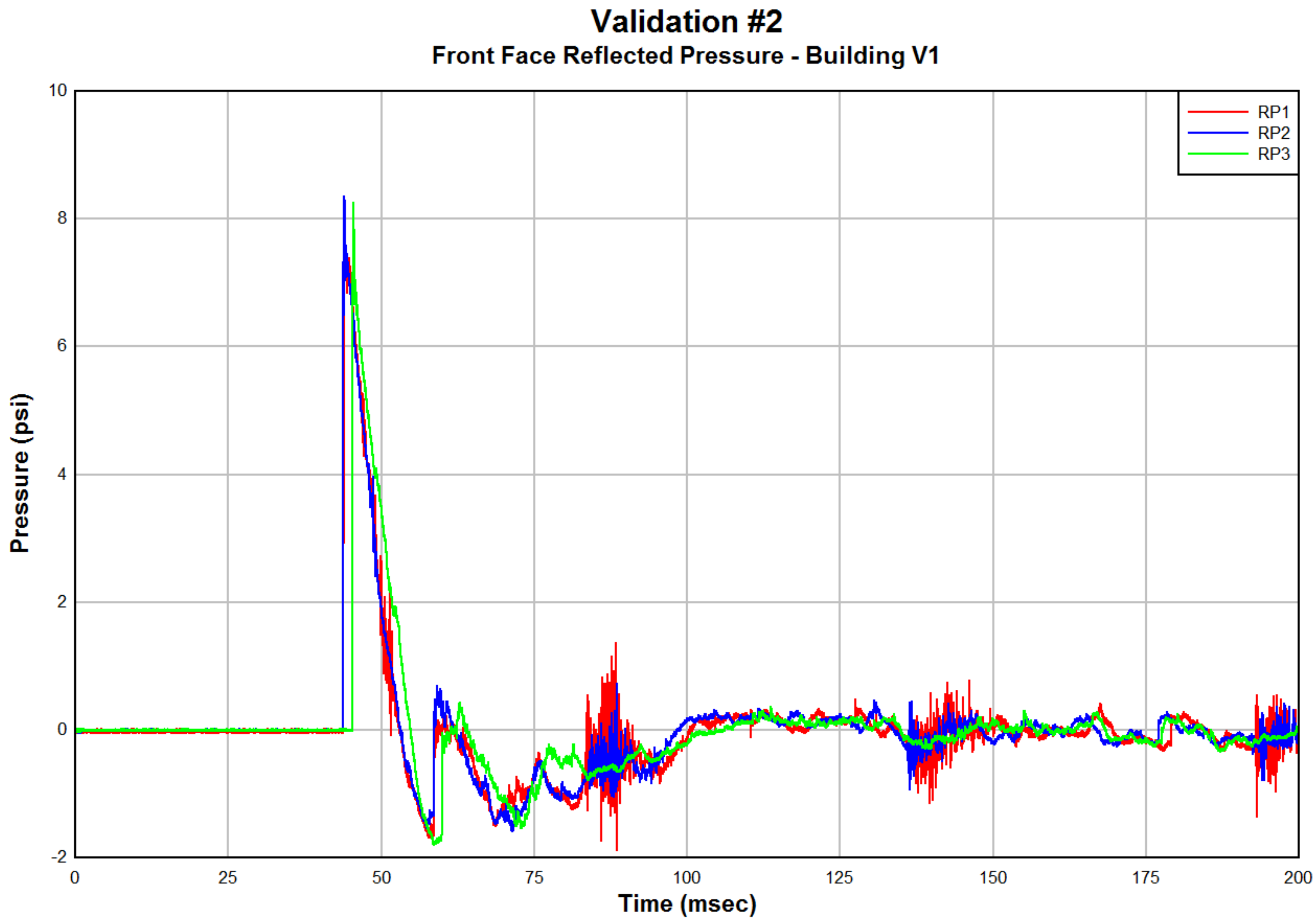


Full-Scale Validation #2

Pre-test

Post-test

Validation #2 – Building V1: Front Face Reflected Pressure Gauges



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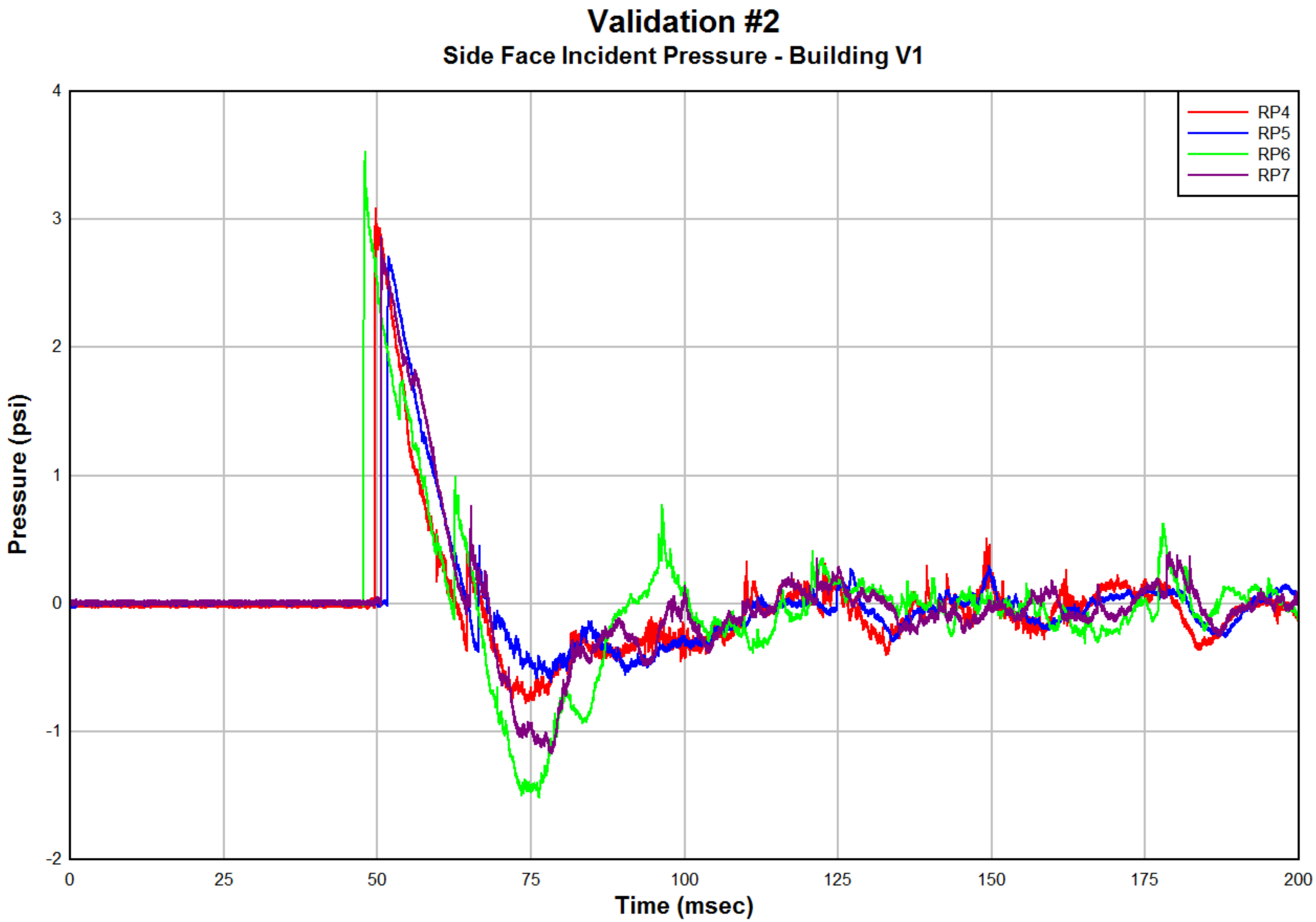

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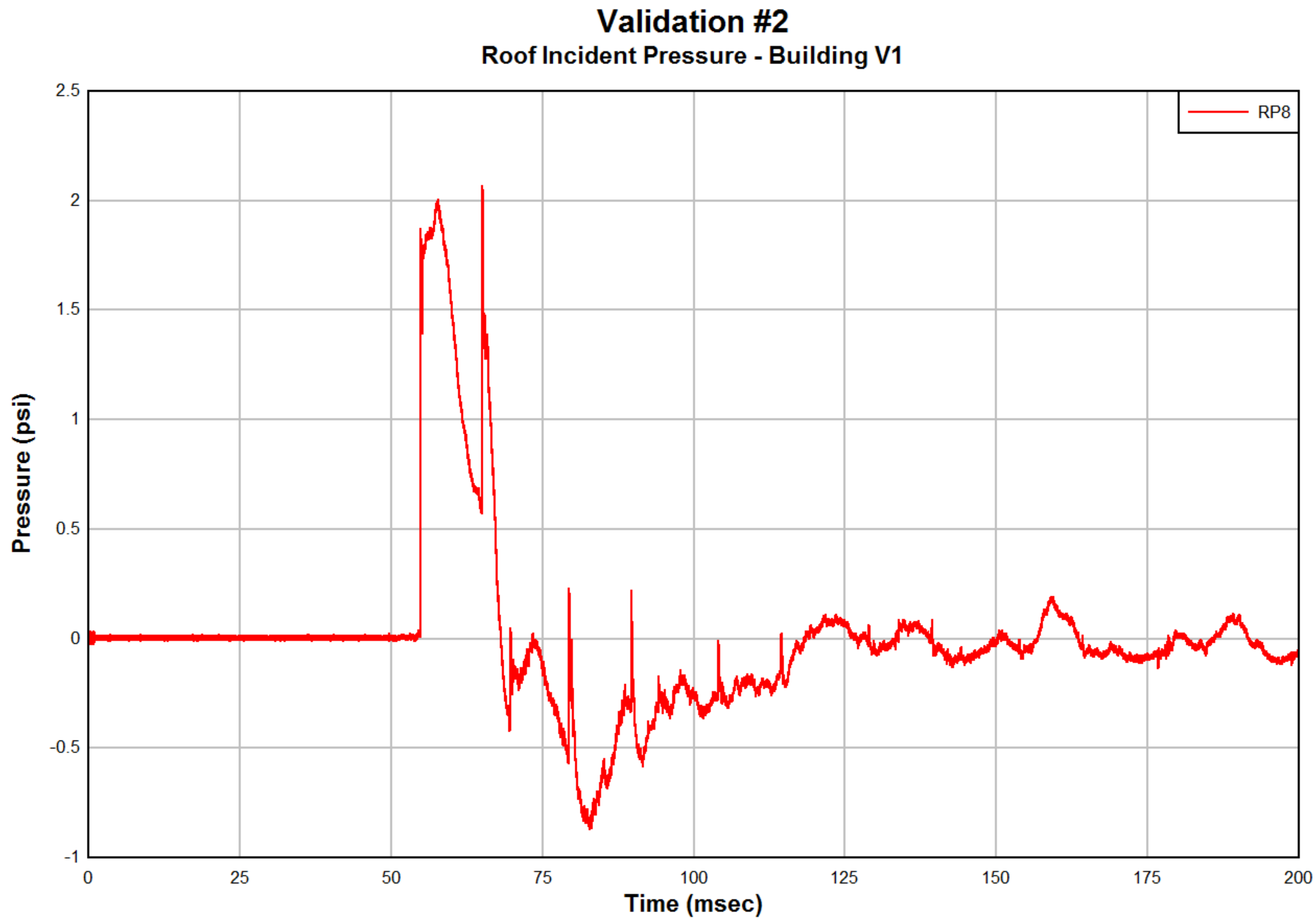

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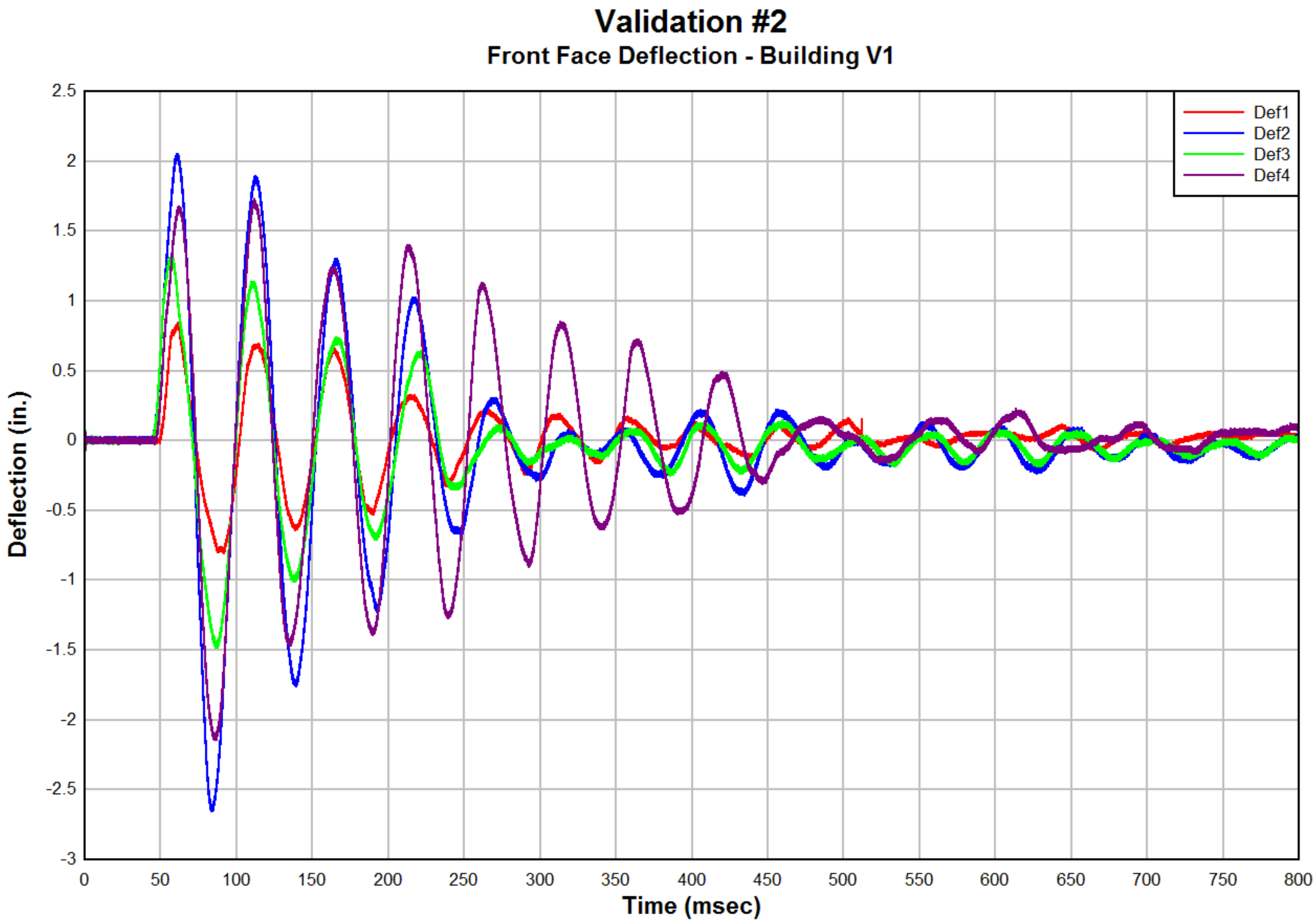

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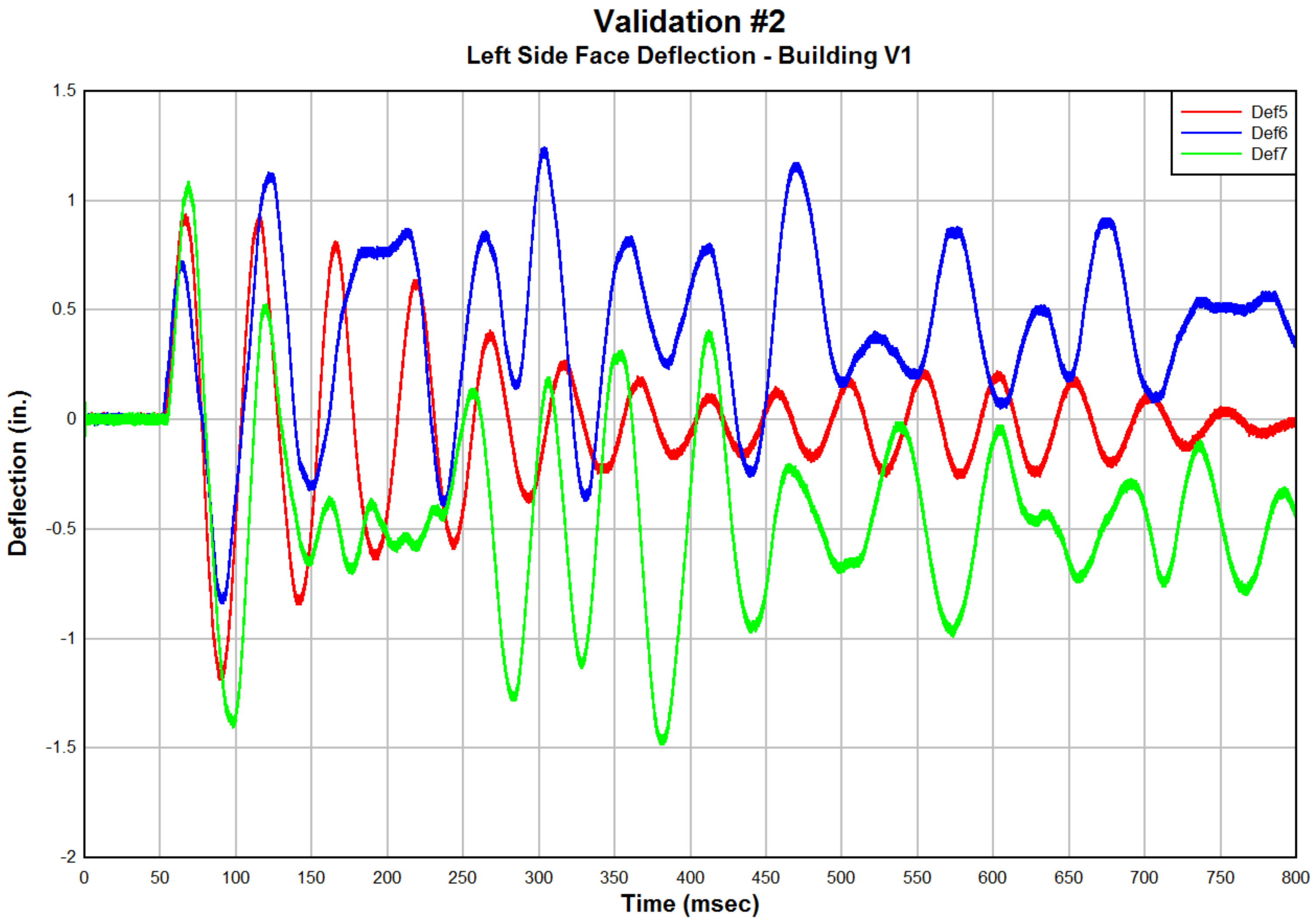

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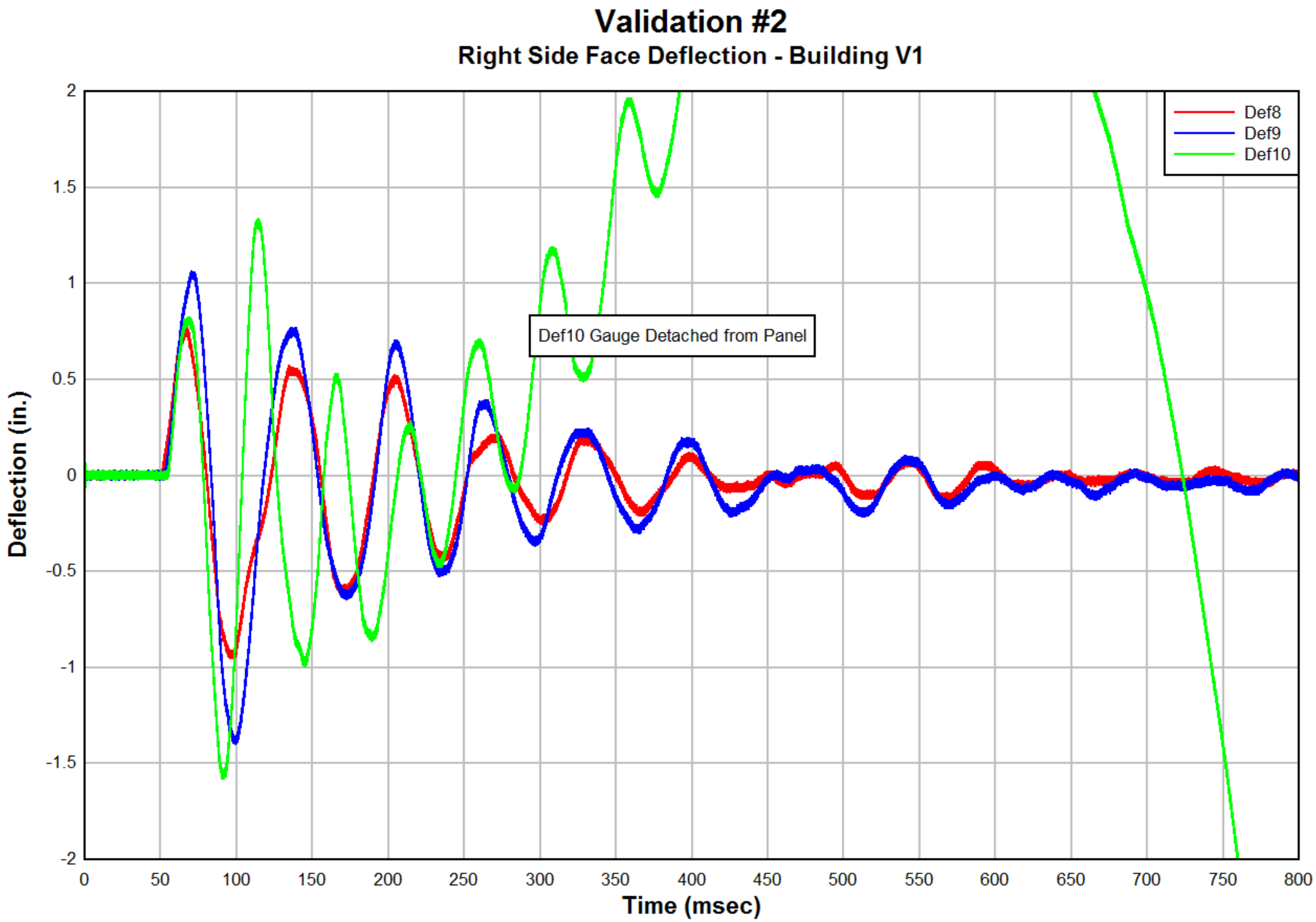

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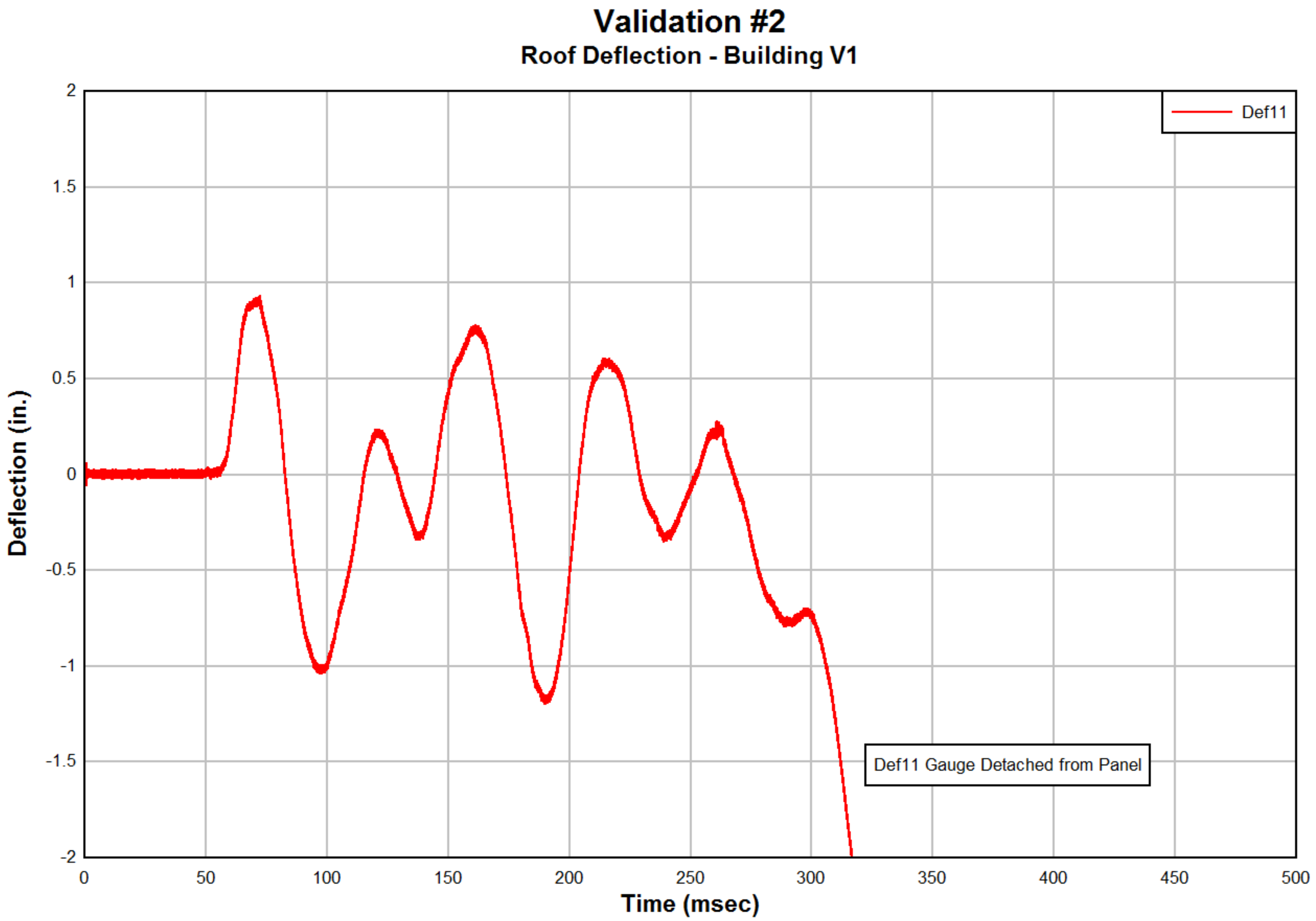
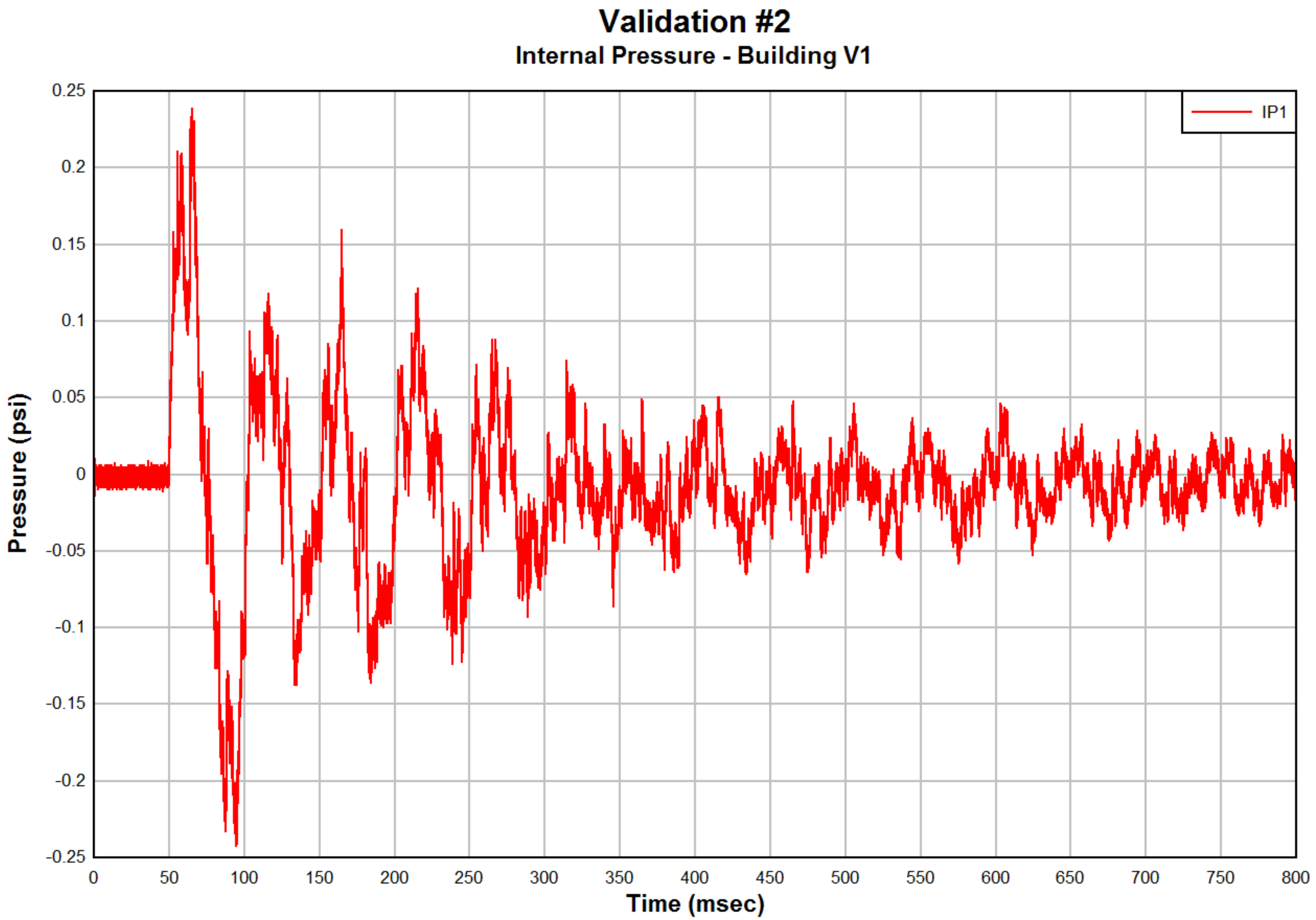
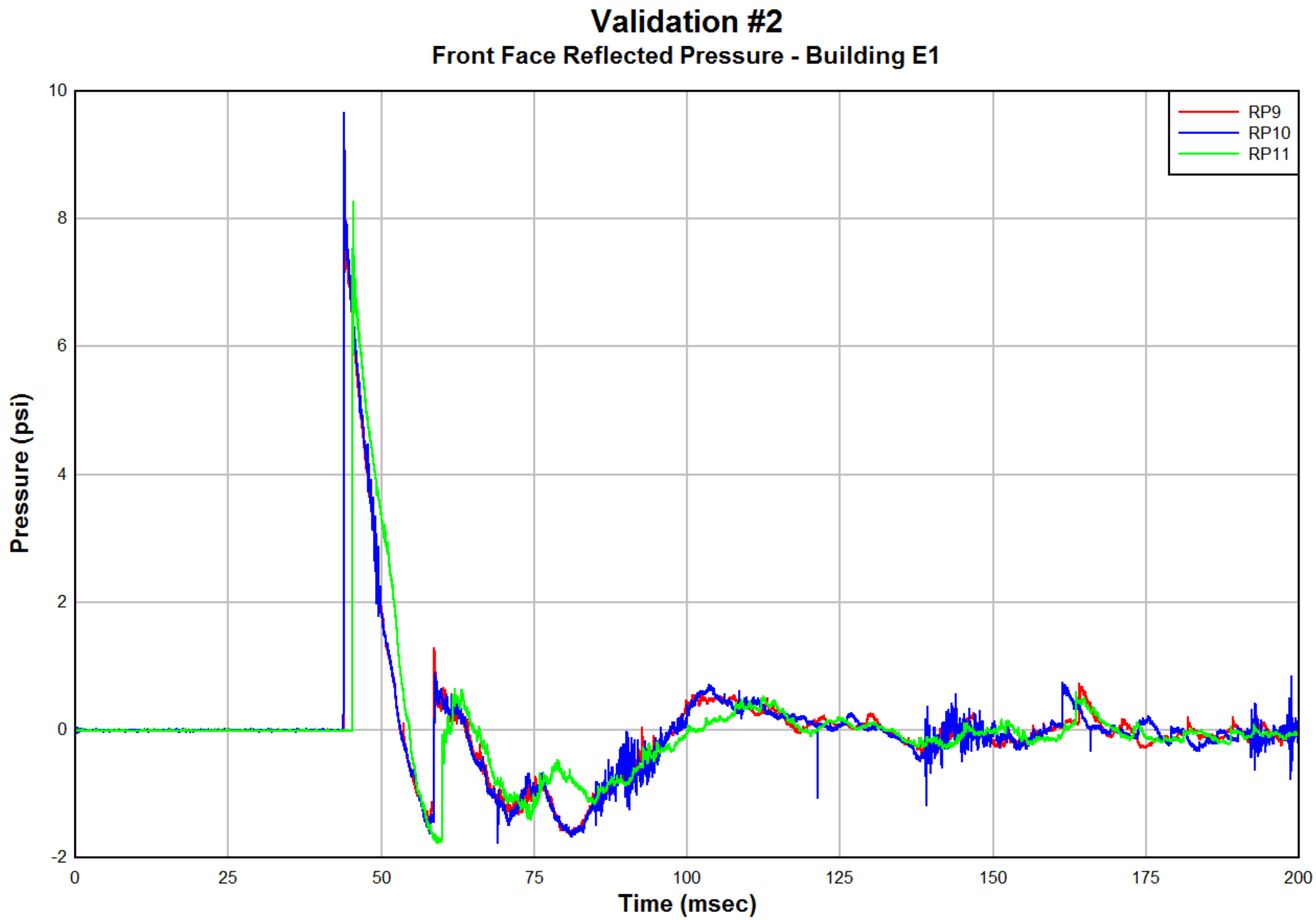


Validation #2 – Building V1: Internal Pressure Gauge



Validation #2 – Building E1: Front Face Reflected Pressure Gauges



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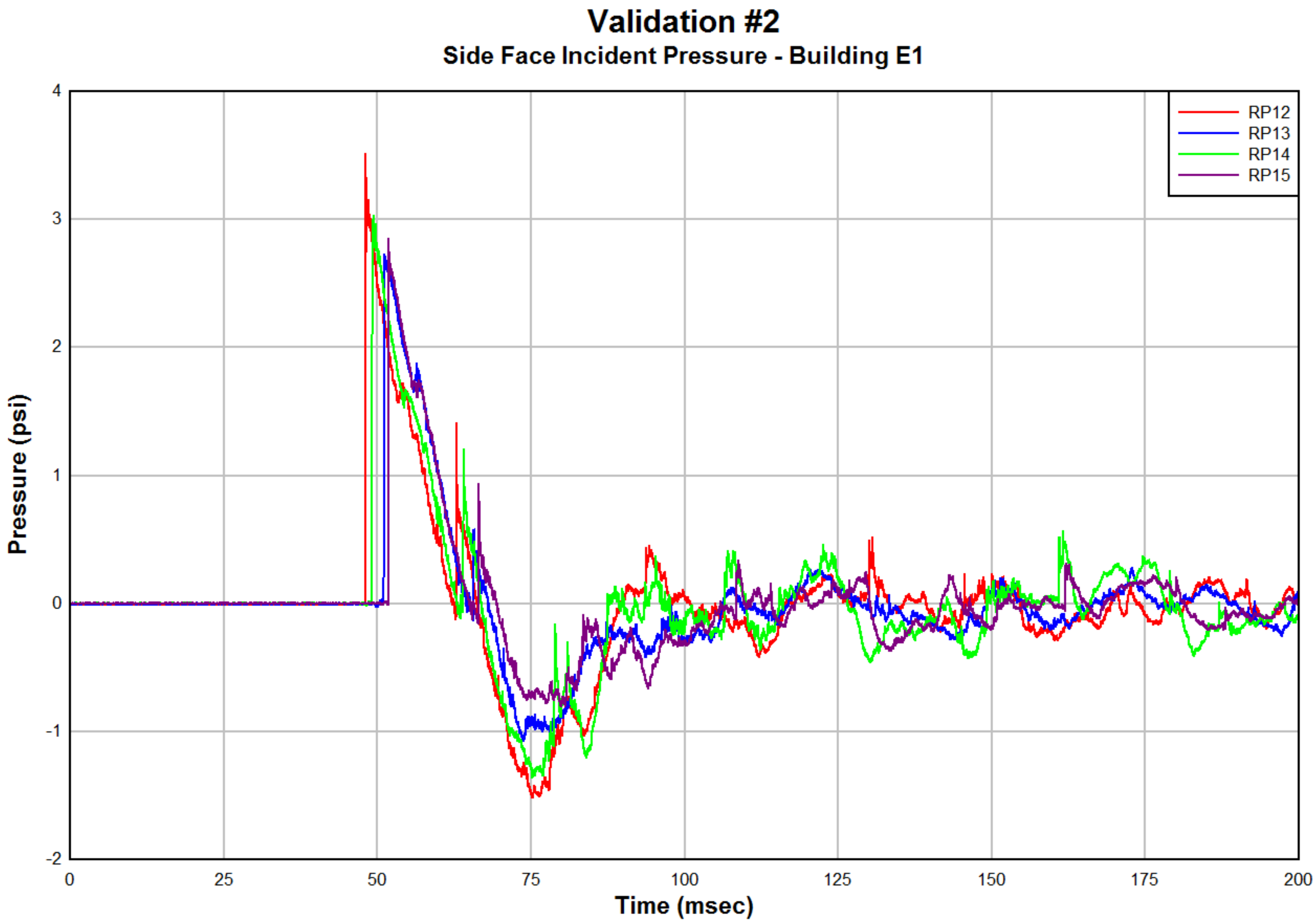

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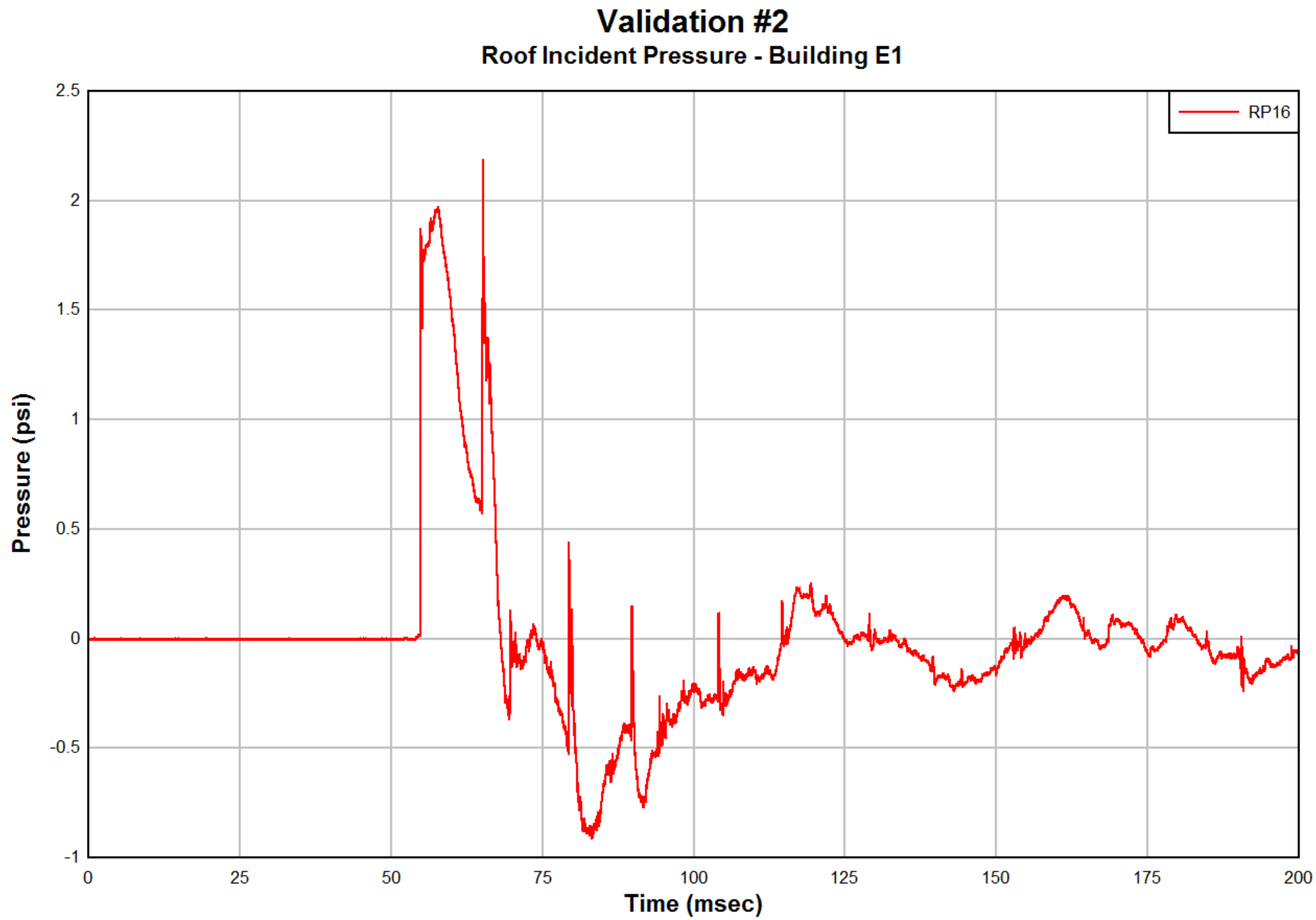
Validation #2 – Building E1: Roof Pressure Gauge



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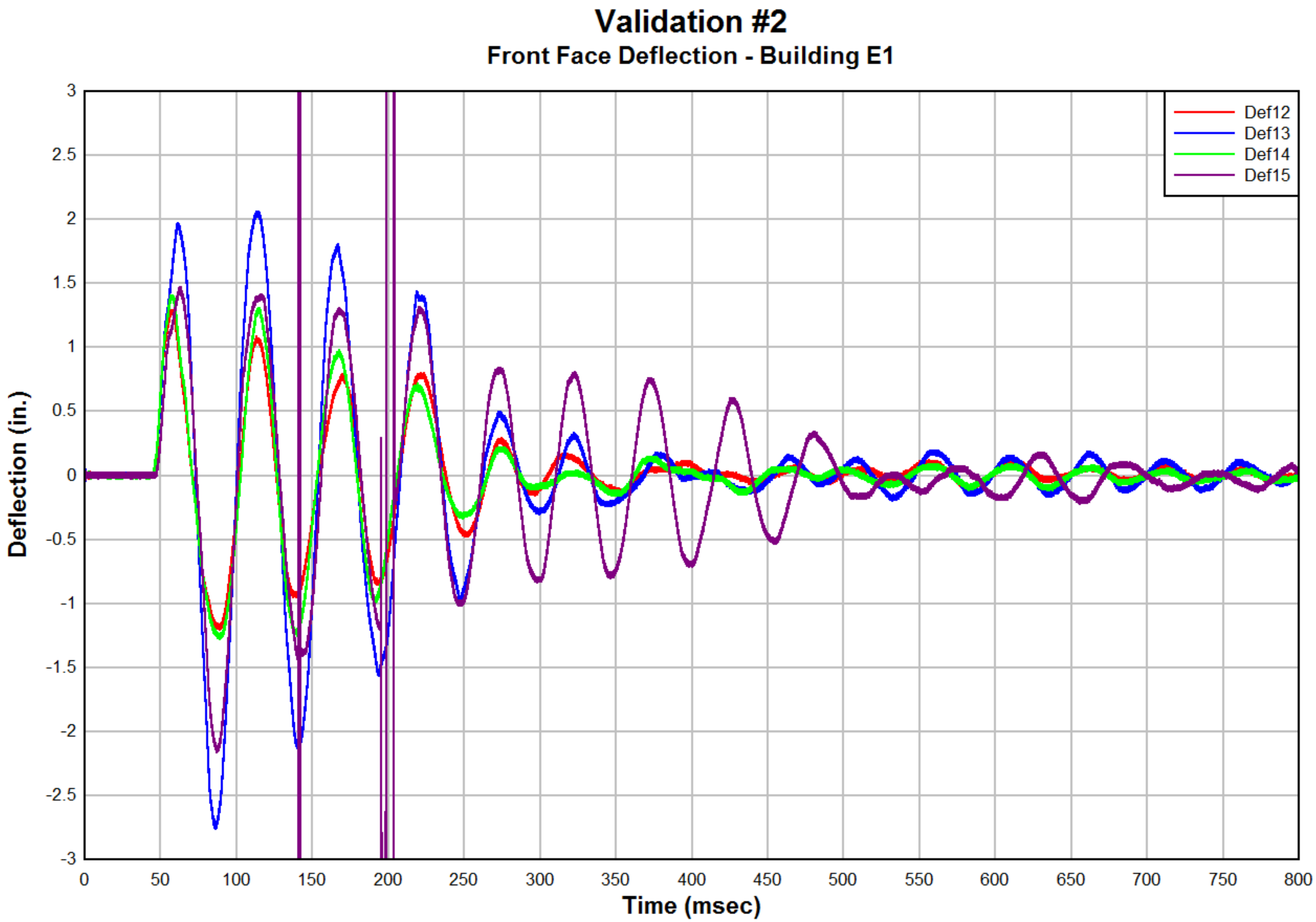
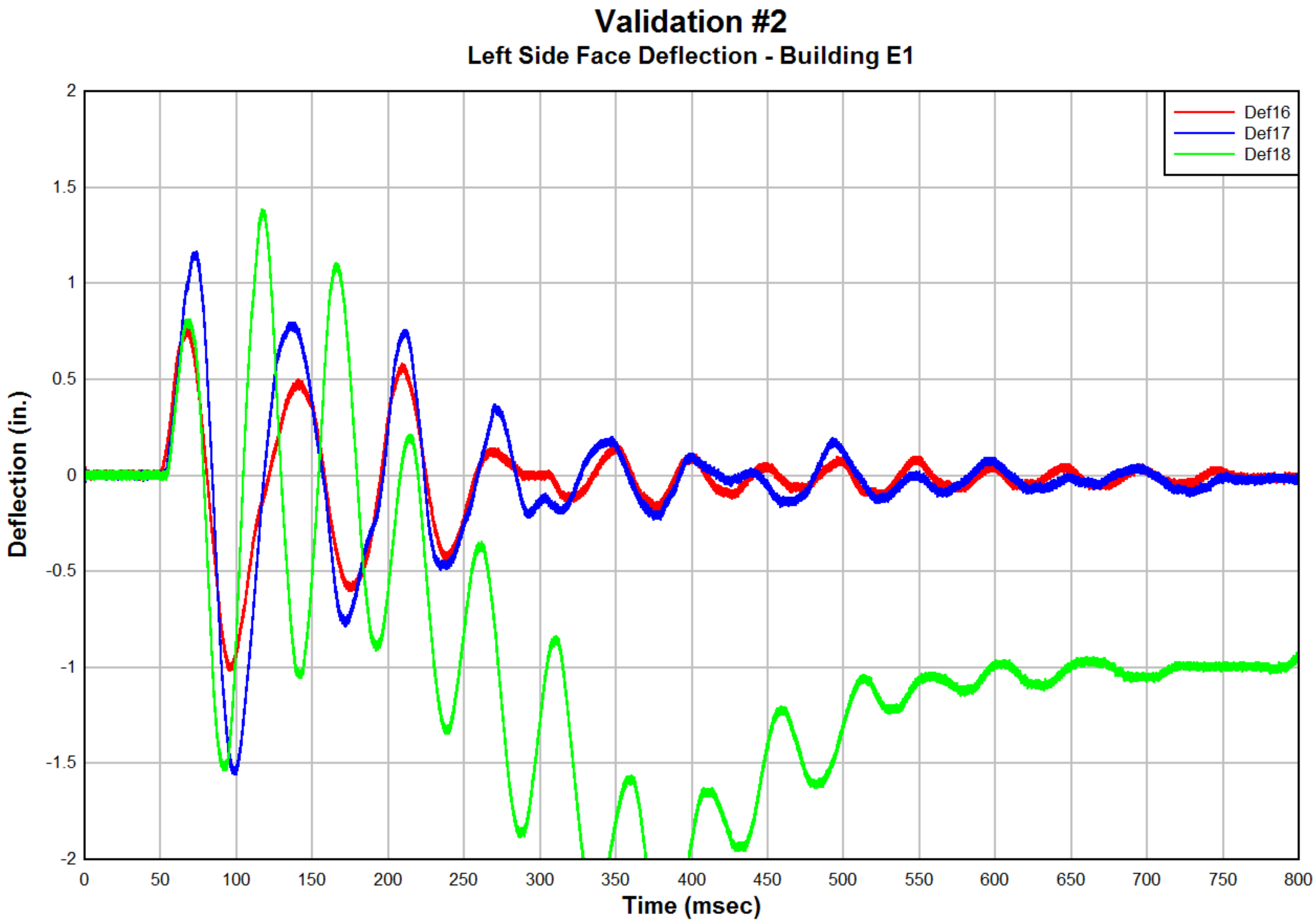
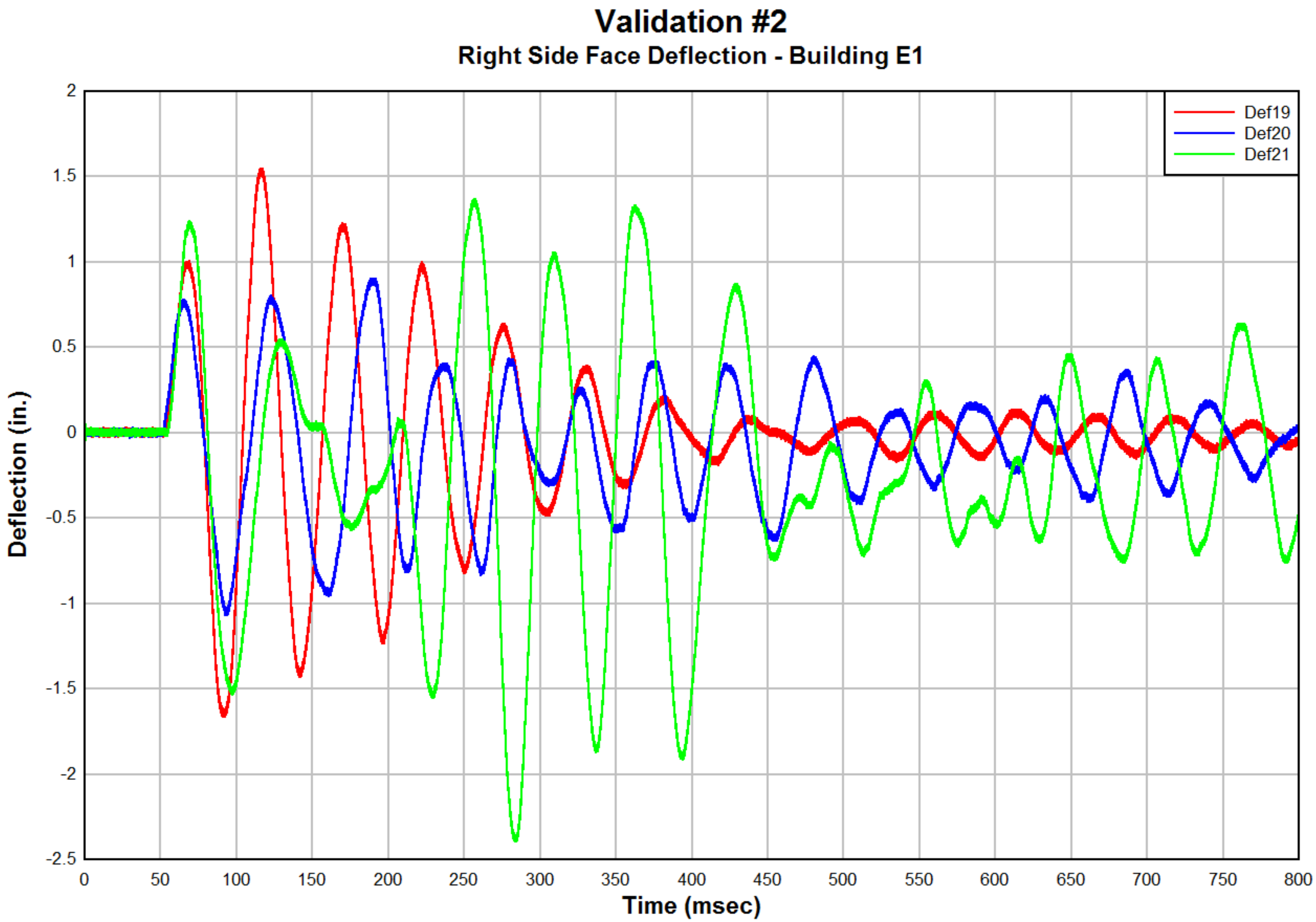


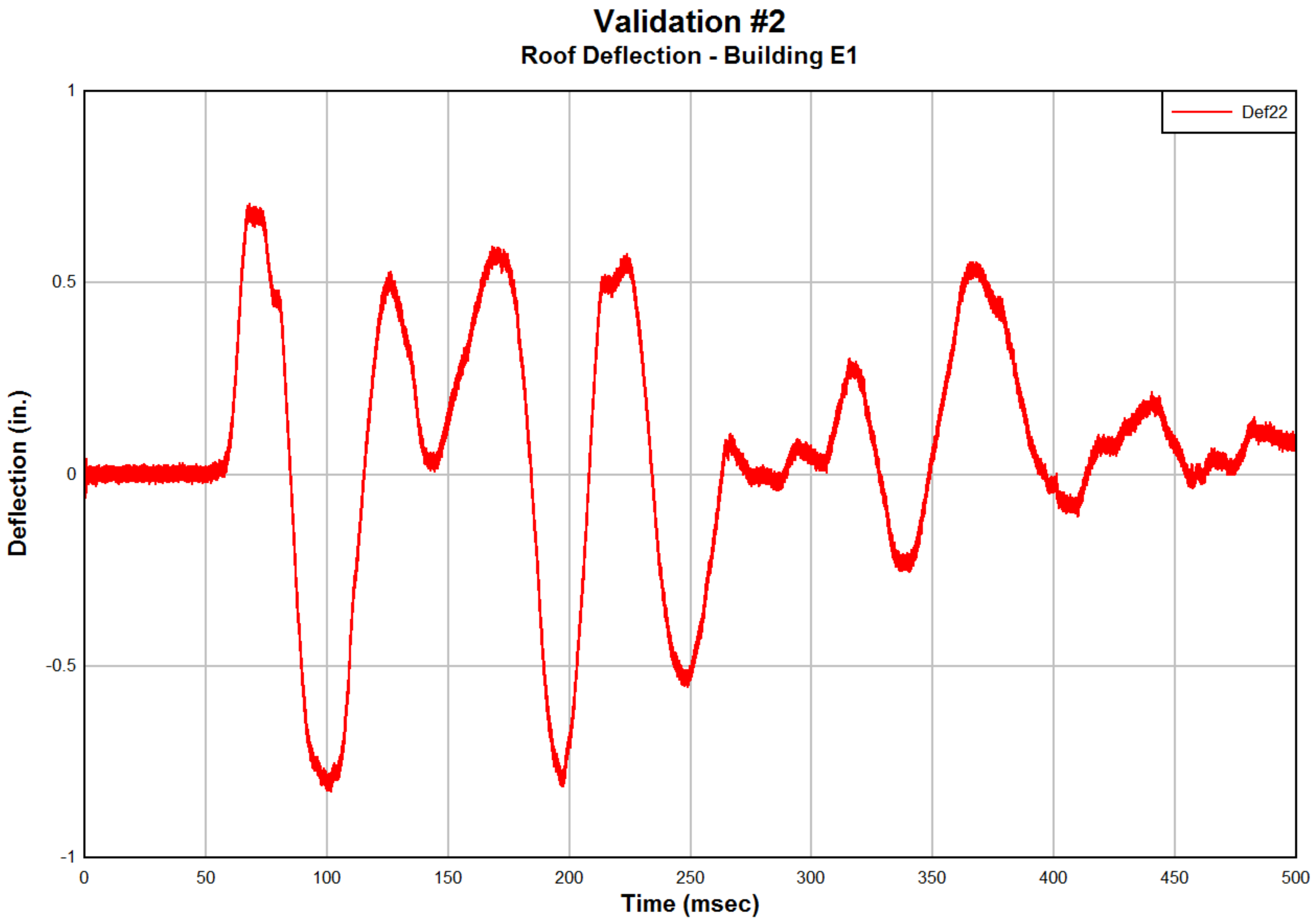
Validation #2 – Building E1: Left Side Face Deflection Gauges



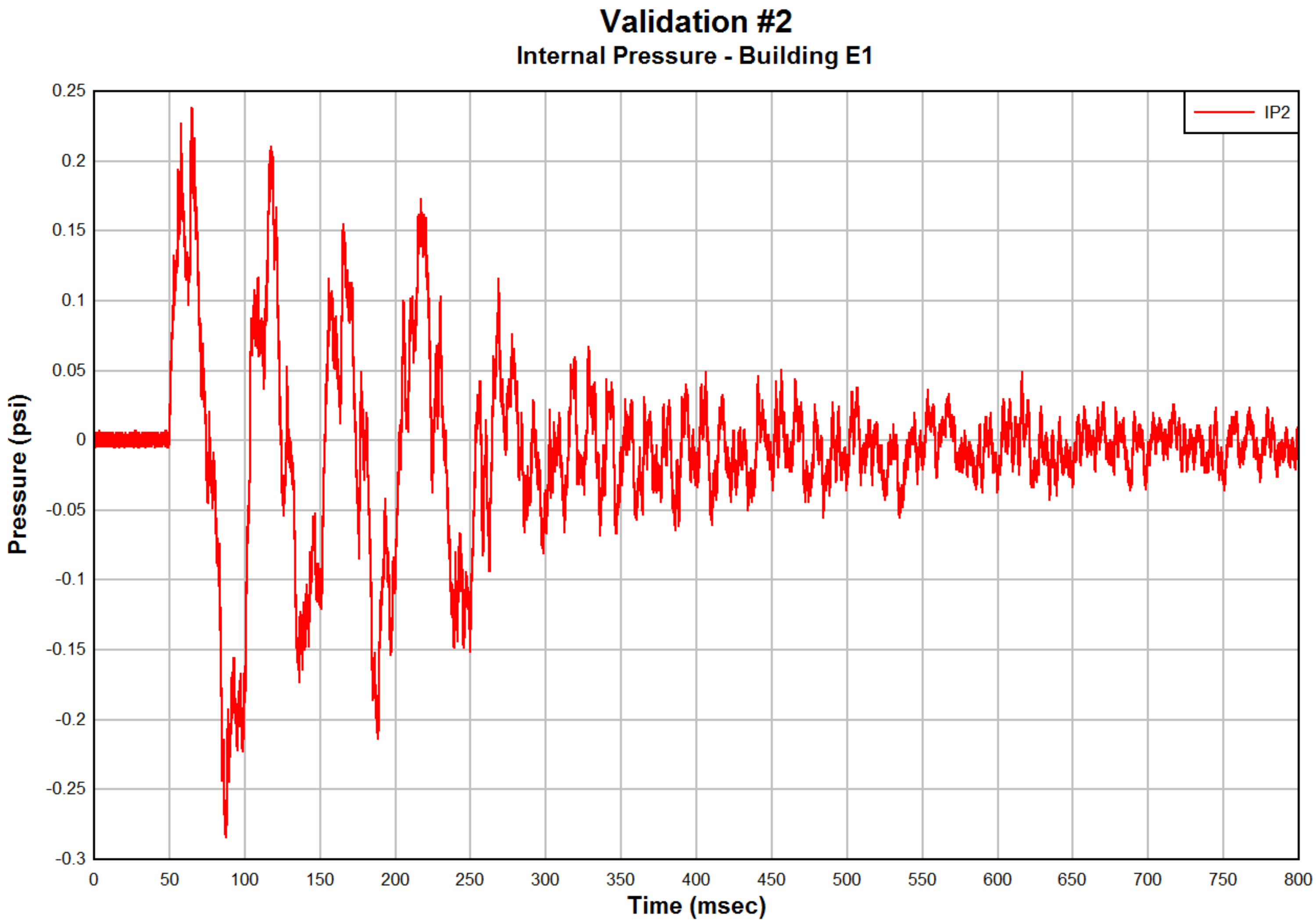
Validation #2 – Building E1: Right Side Face Deflection Gauges



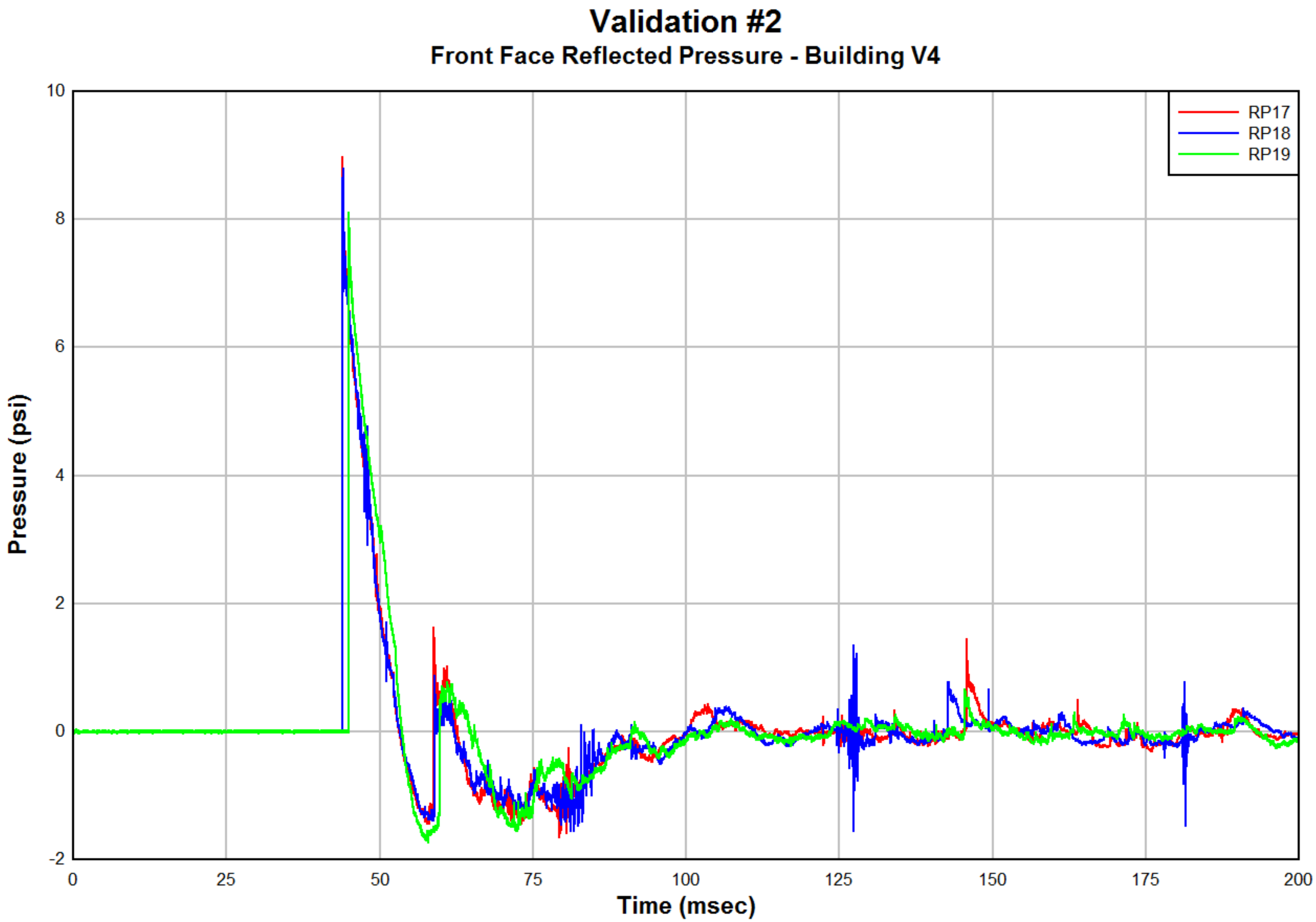
Validation #2 – Building E1: Roof Deflection Gauge



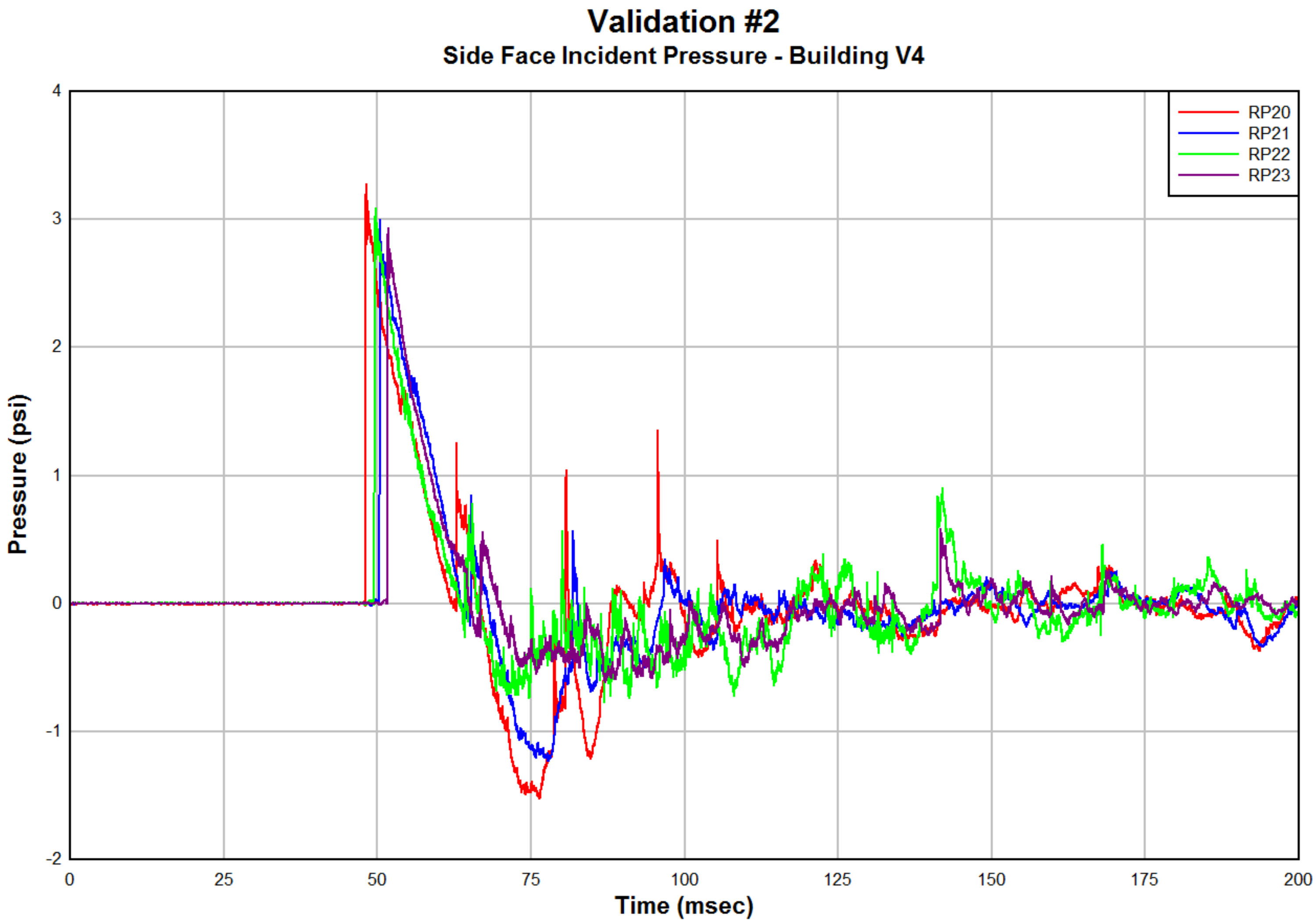
Validation #2– Building E1: Internal Pressure Gauge



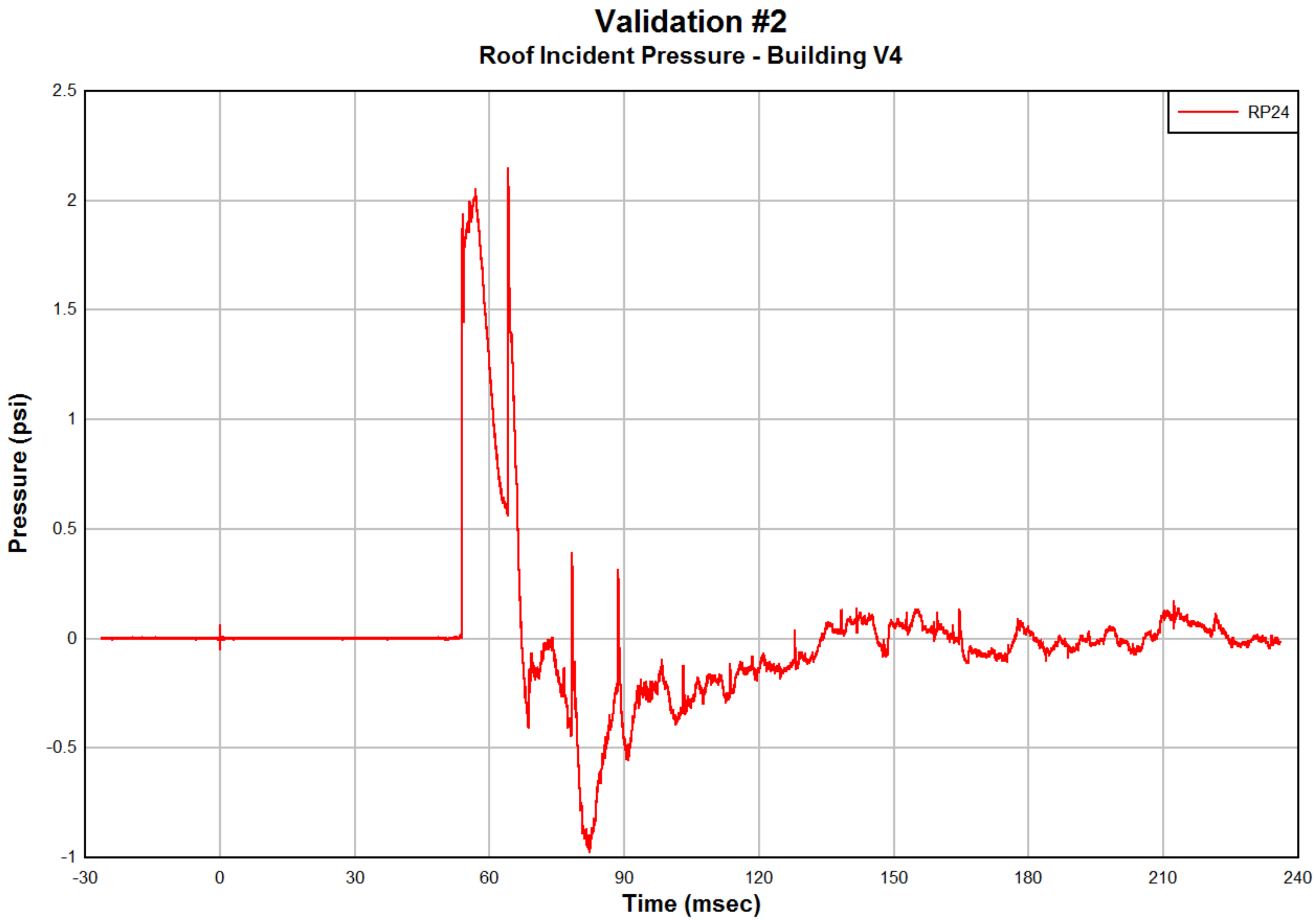
Validation #2 – Building V4: Front Face Reflected Pressure Gauges



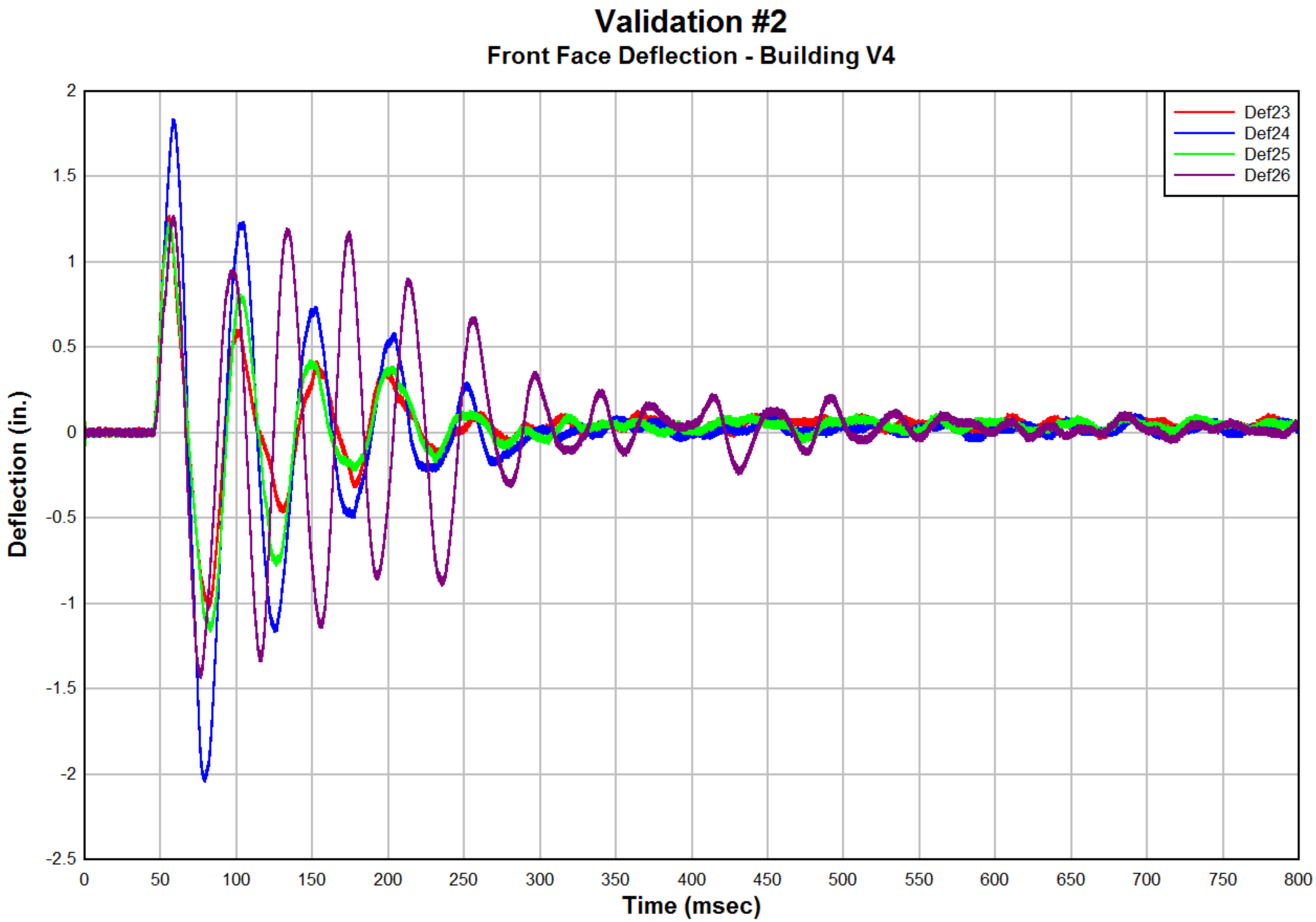
Validation #2 – Building V4: Side Face Incident Pressure Gauges



Validation #2 – Building V4: Roof Pressure Gauge



Validation #2 – Building V4: Front Face Deflection Gauges



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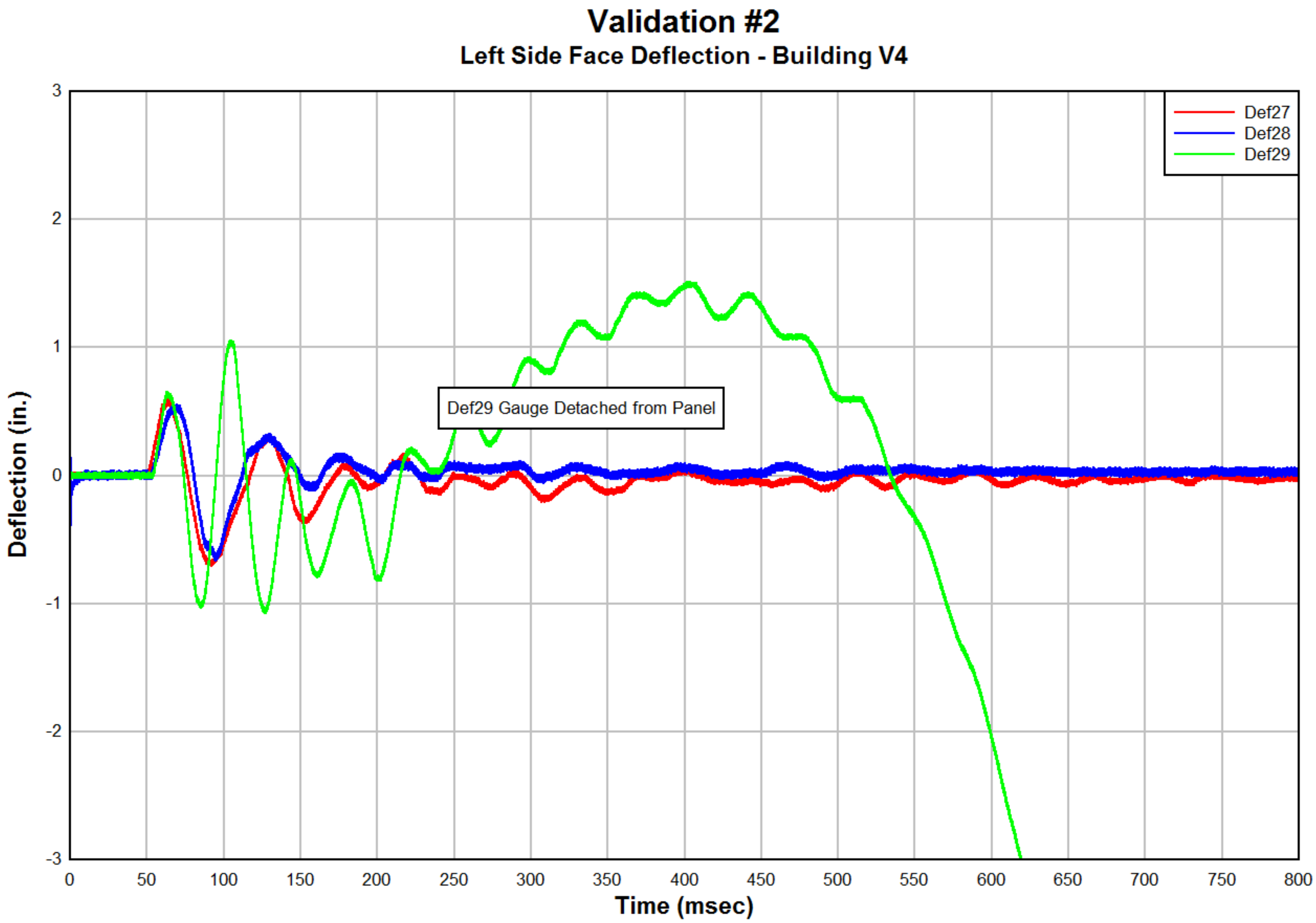
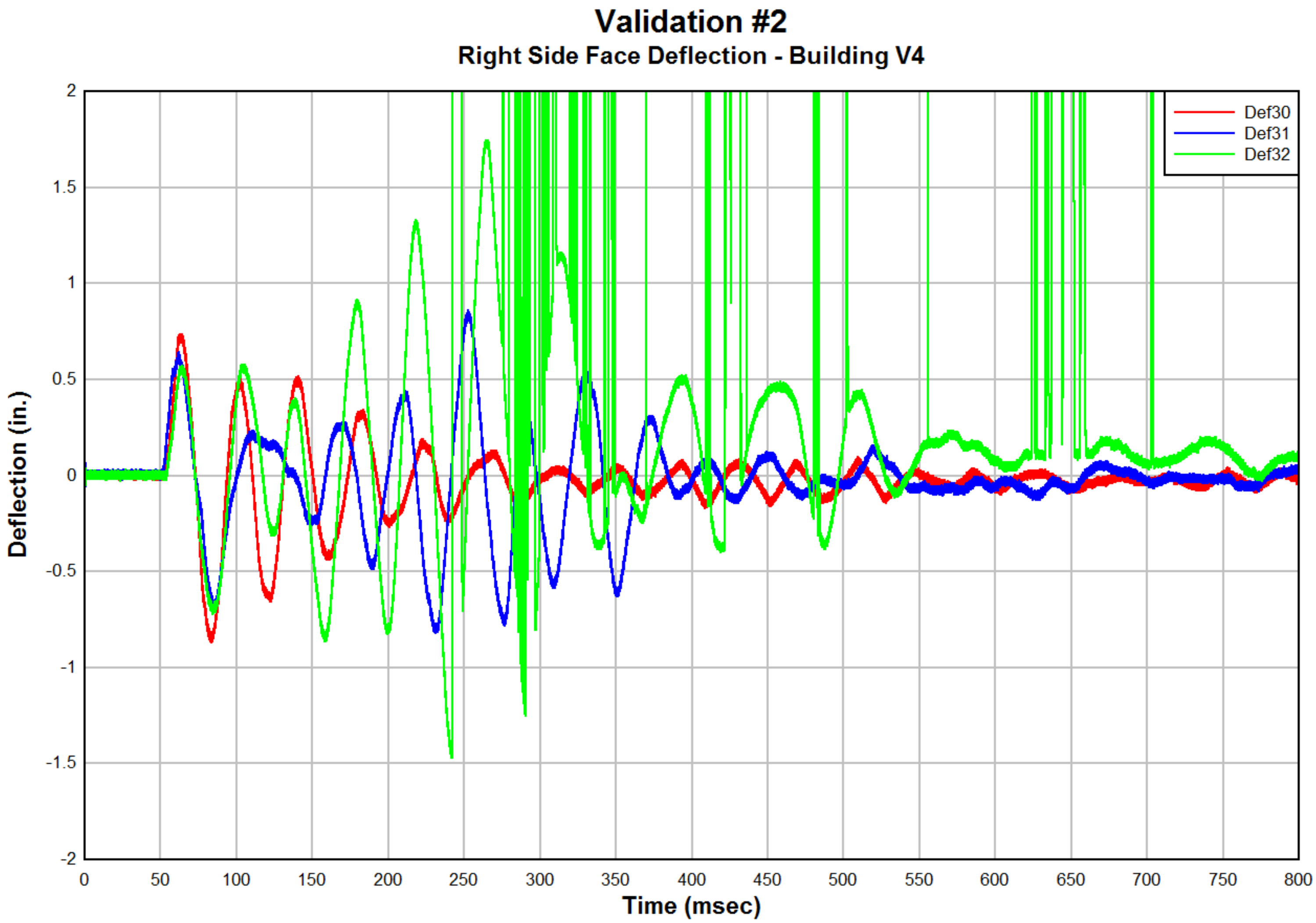
Validation #2 – Building V4: Left Side Face Deflection Gauges



Validation #2 – Building V4: Right Side Face Deflection Gauges



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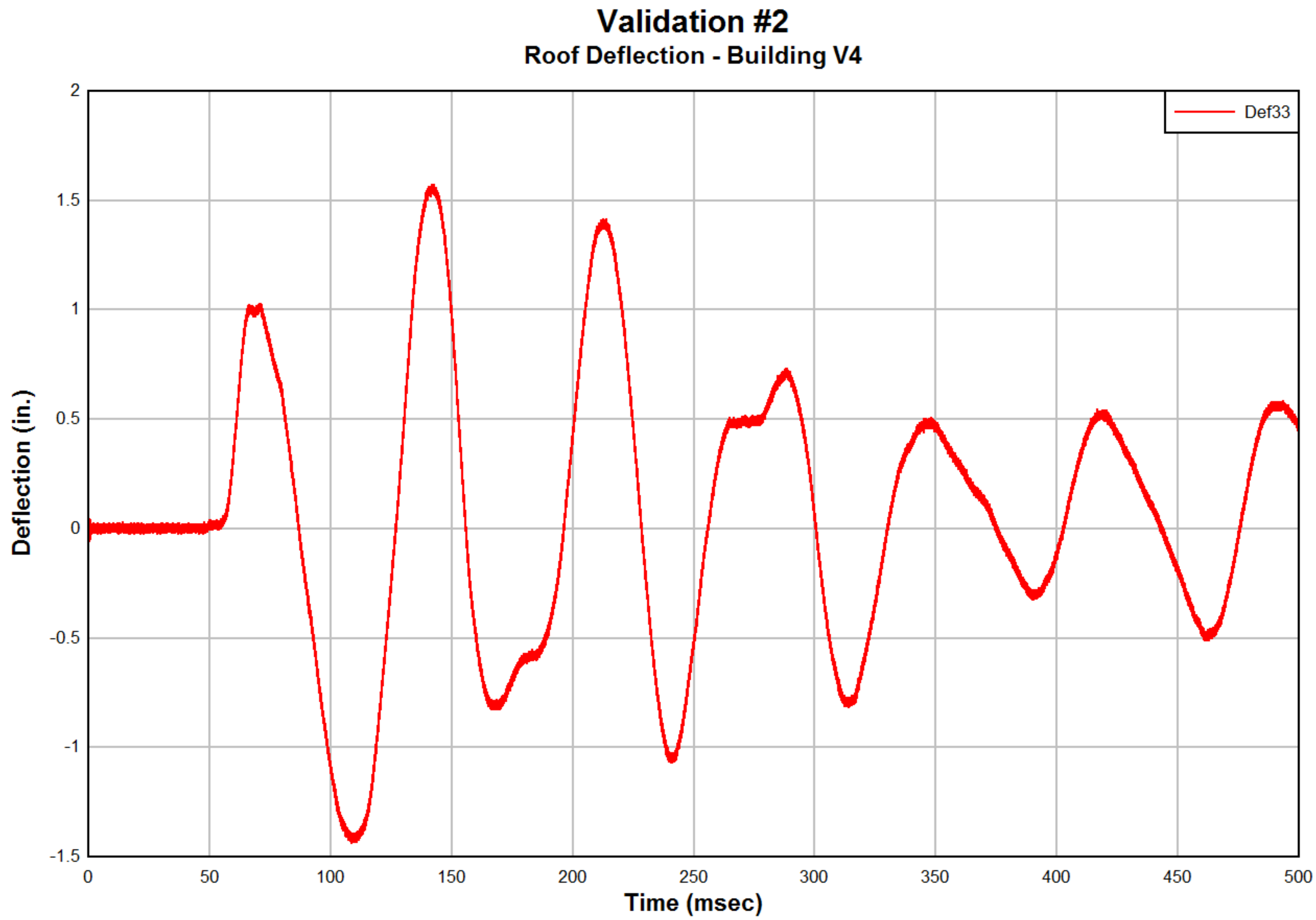
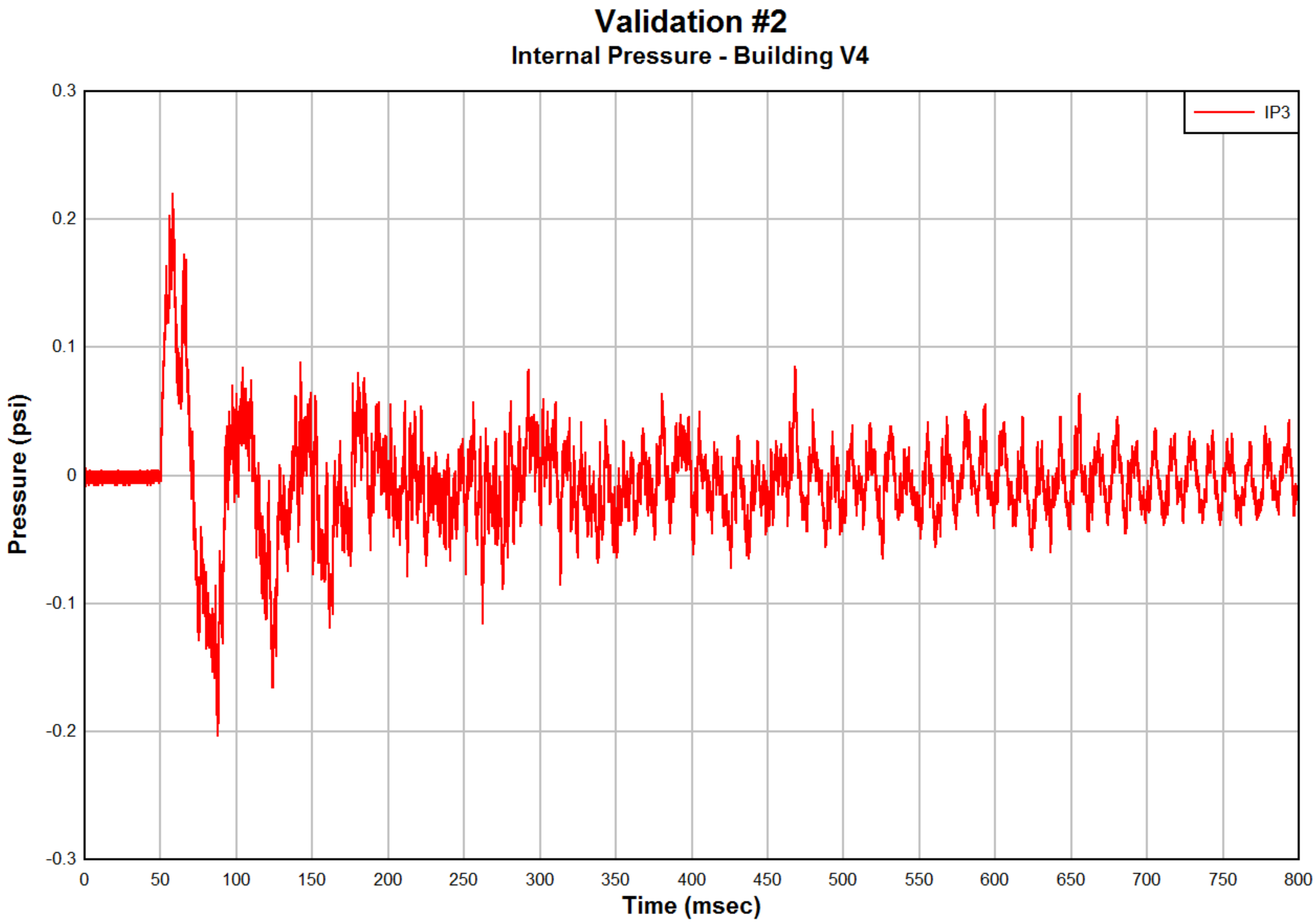
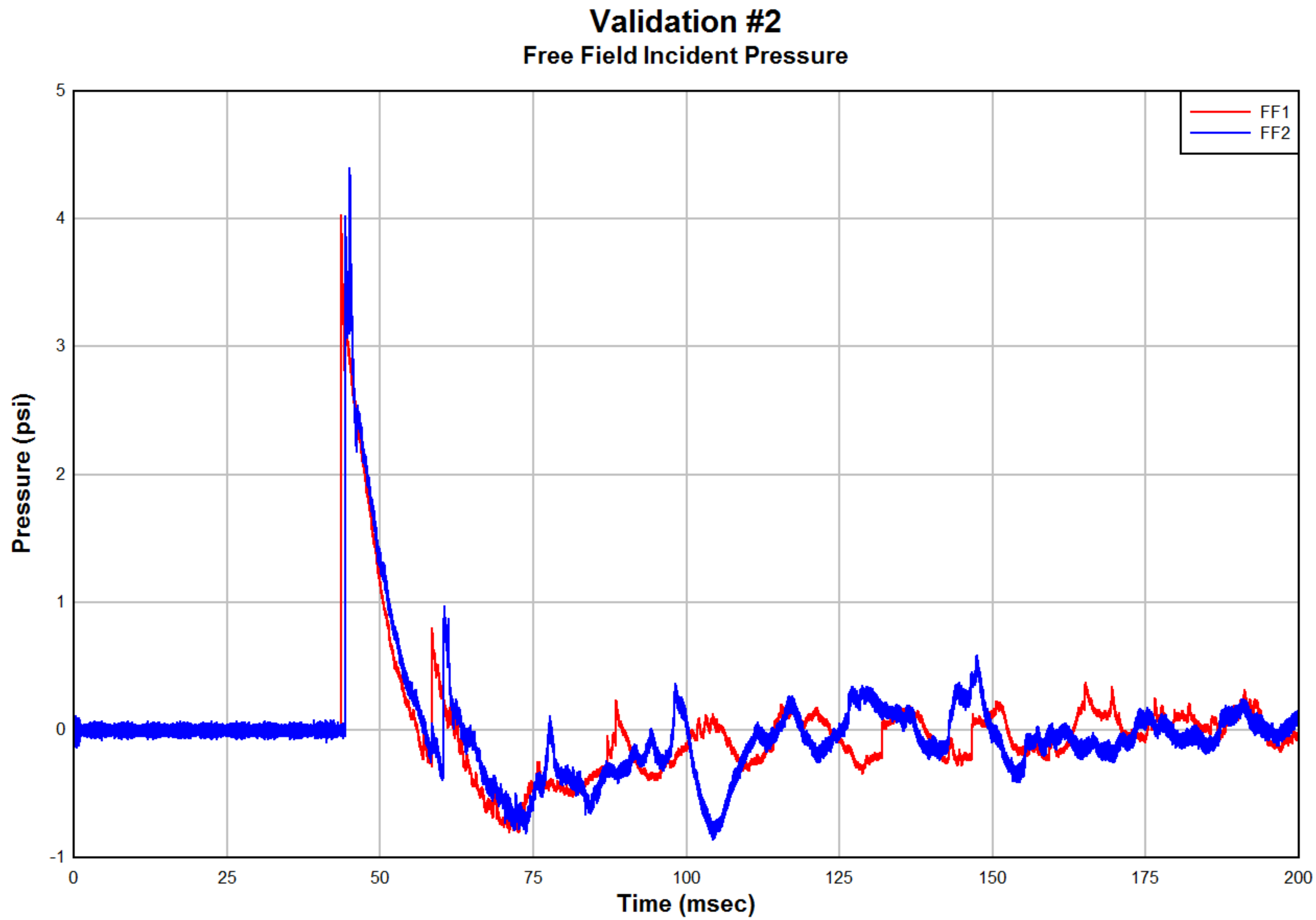
Validation #2 – Building V4: Roof Deflection Gauge



Validation #2 – Building V4: Internal Pressure Gauge



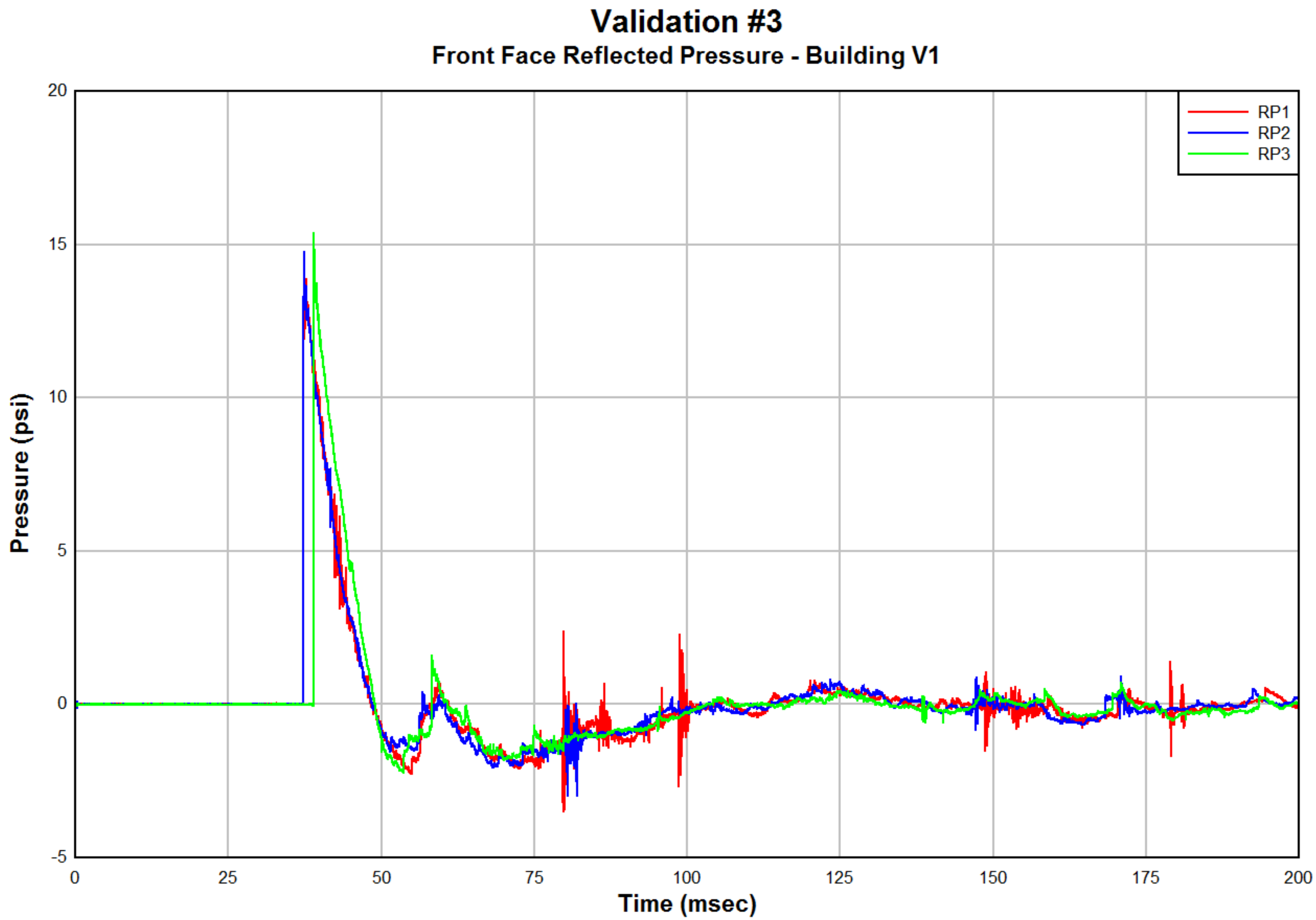
Validation #2: Free Field Incident Pressure Gauges



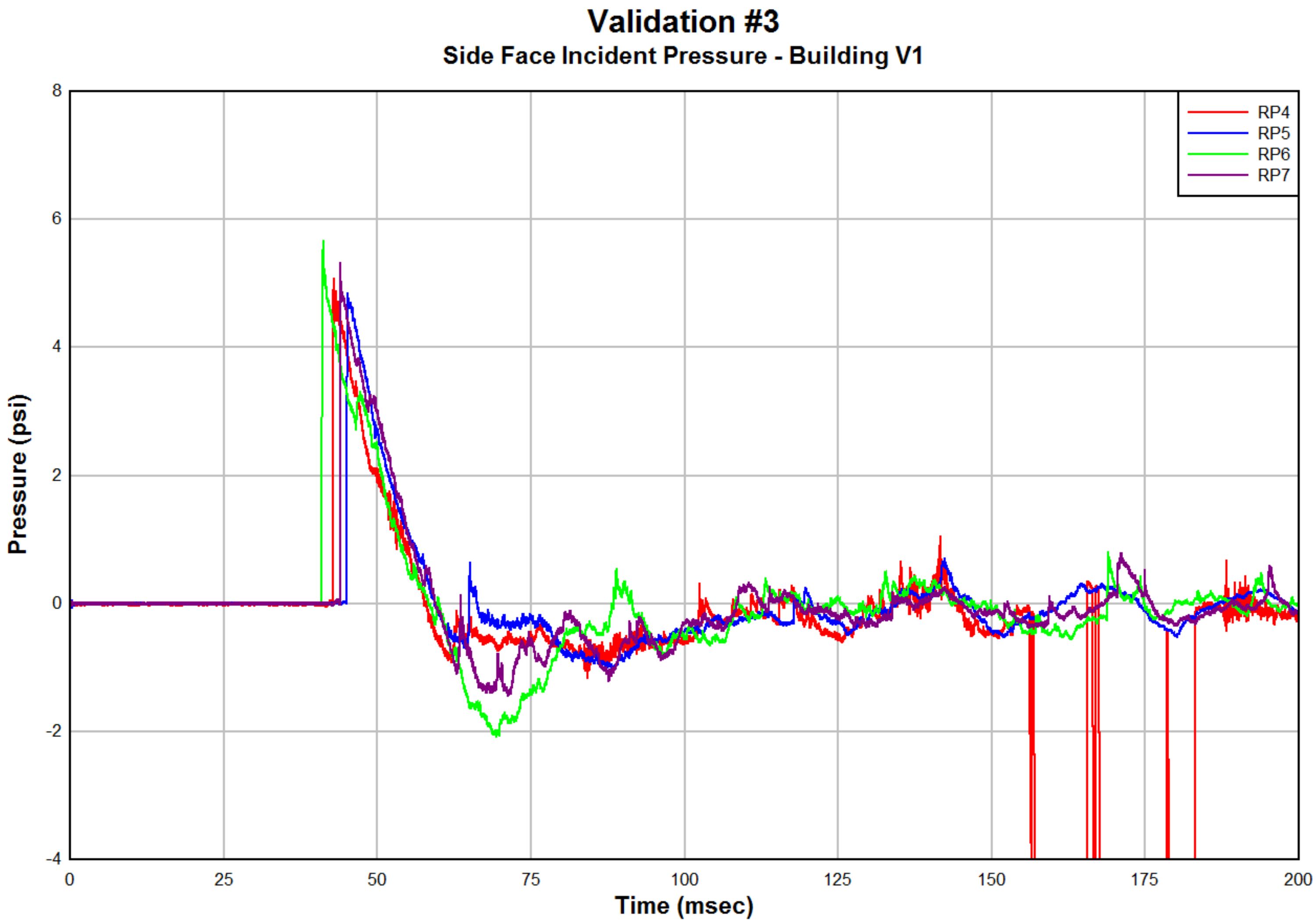
Full-Scale Validation #3

Post-test

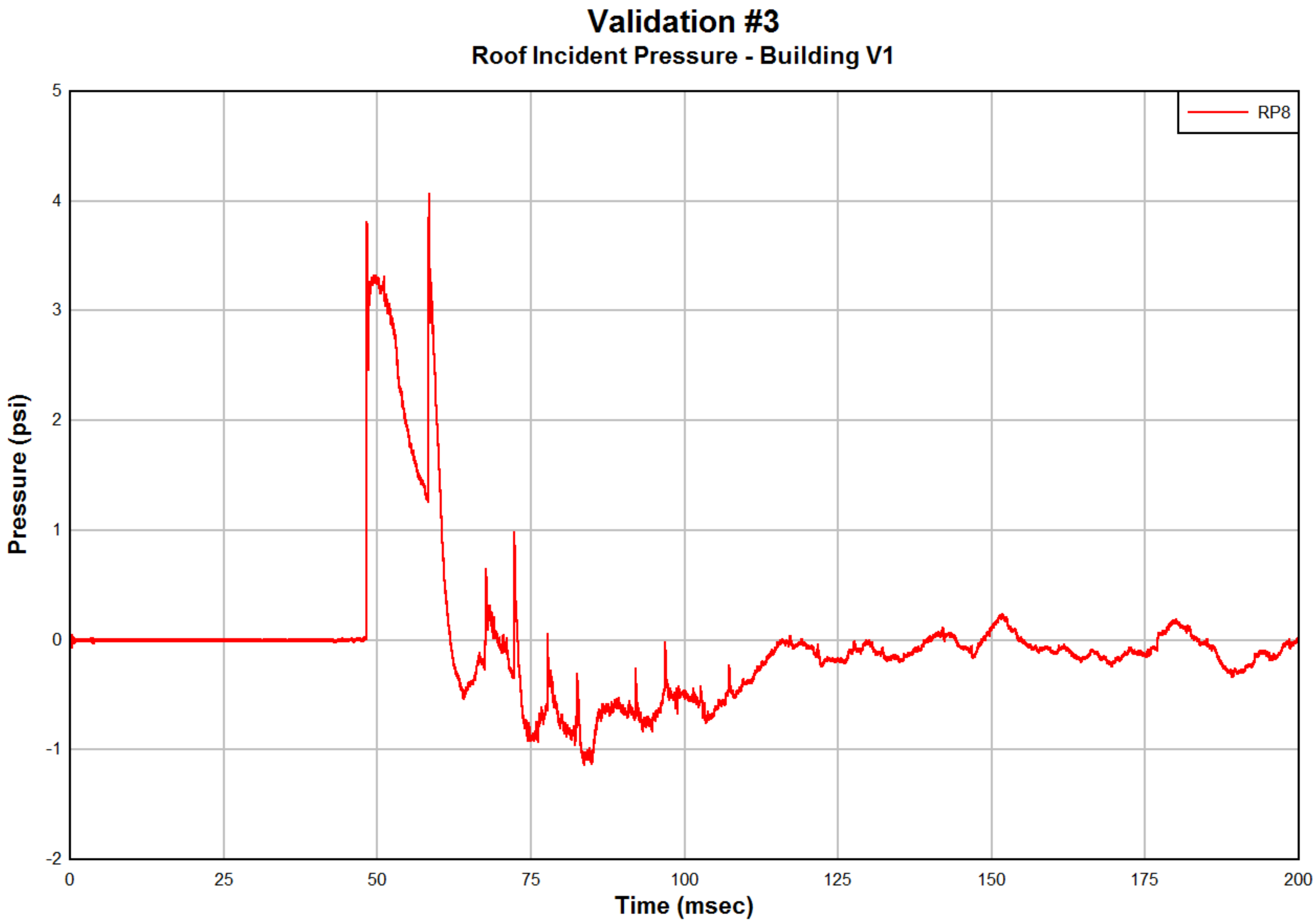
Validation #3 – Building V1: Front Face Reflected Pressure Gauges



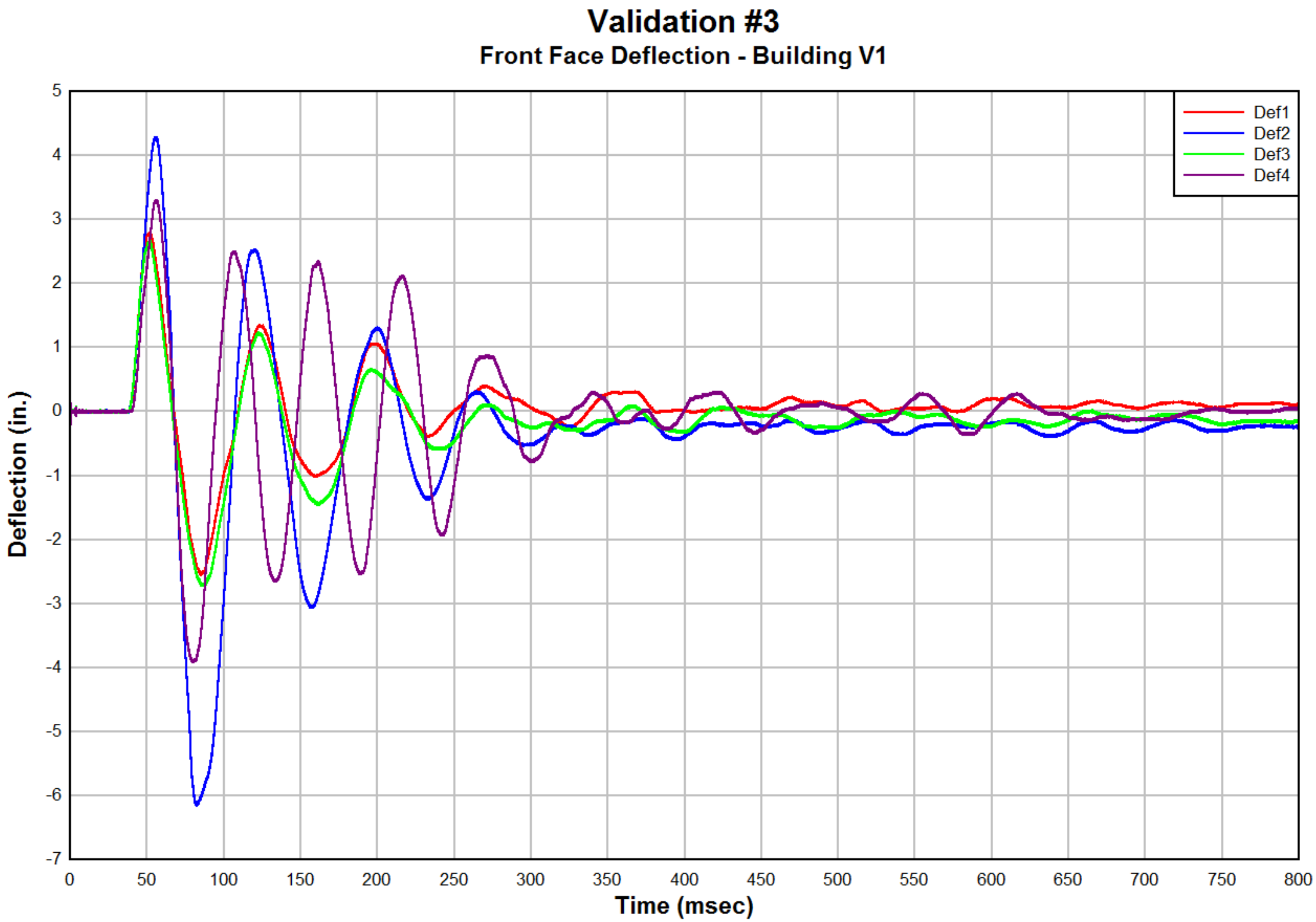
Validation #3 – Building V1: Side Face Incident Pressure Gauges



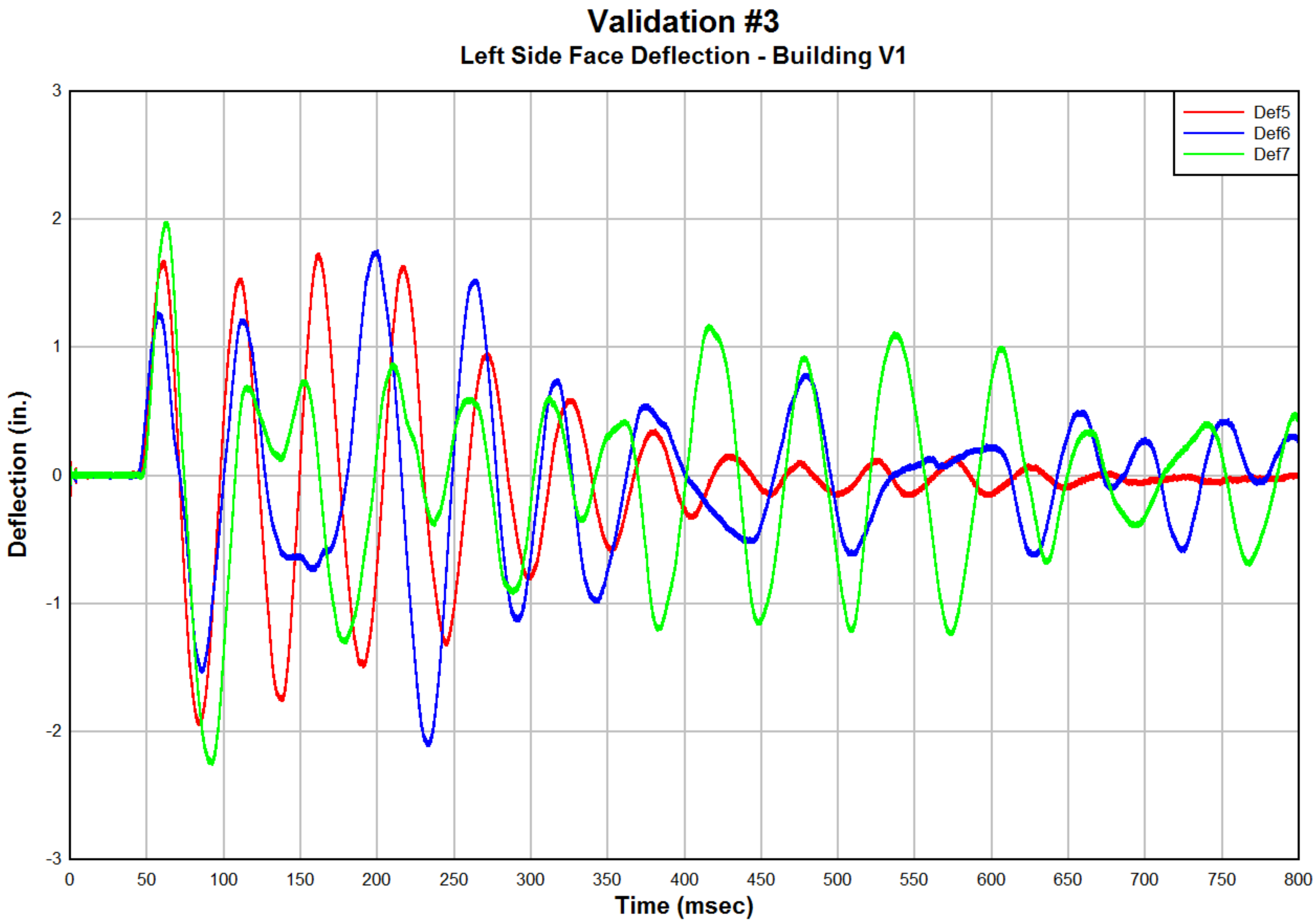
Validation #3 – Building V1: Roof Incident Pressure Gauges



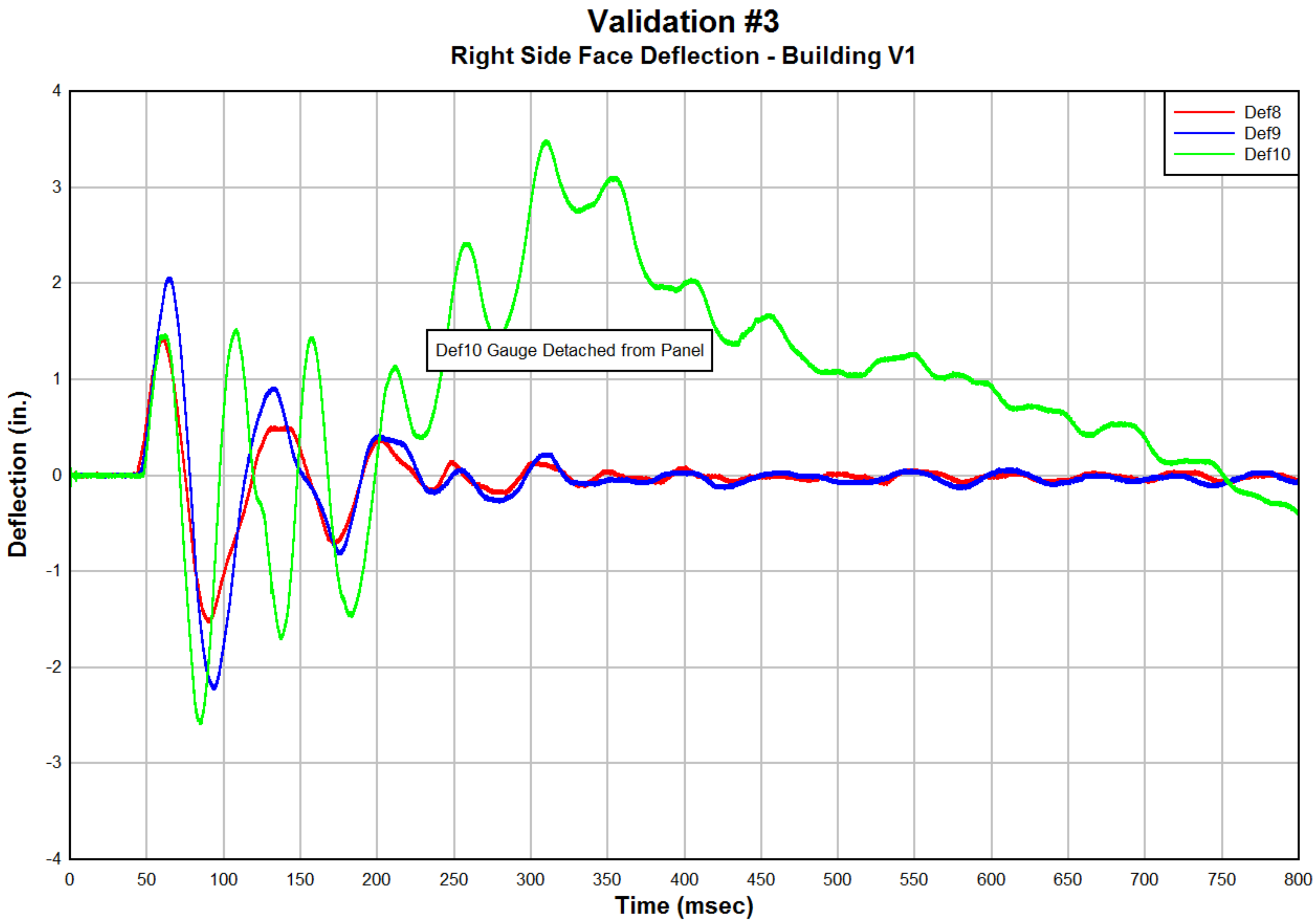
Validation #3 – Building V1: Front Face Deflection Gauges



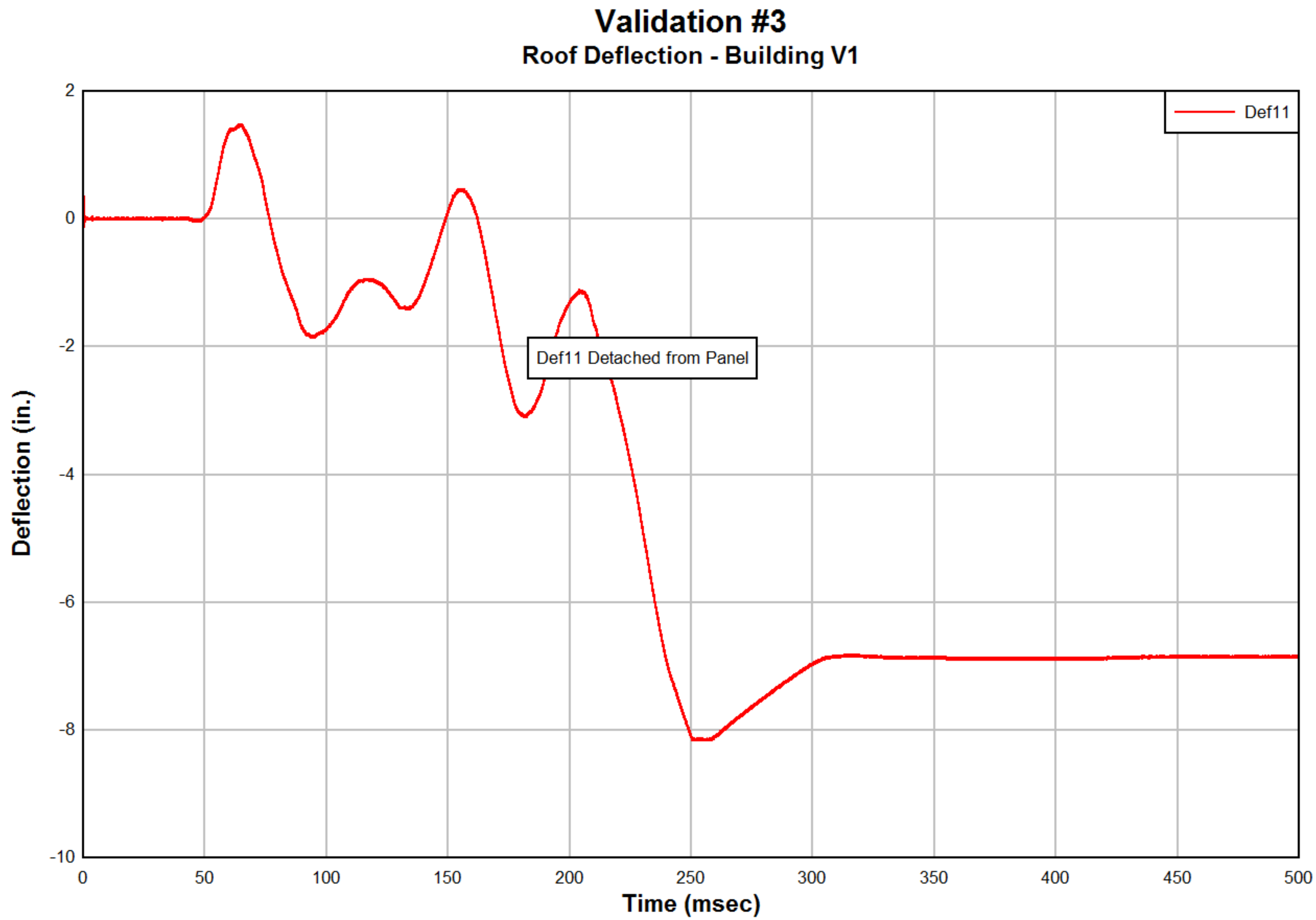
Validation #3 – Building V1: Left Side Face Deflection Gauges



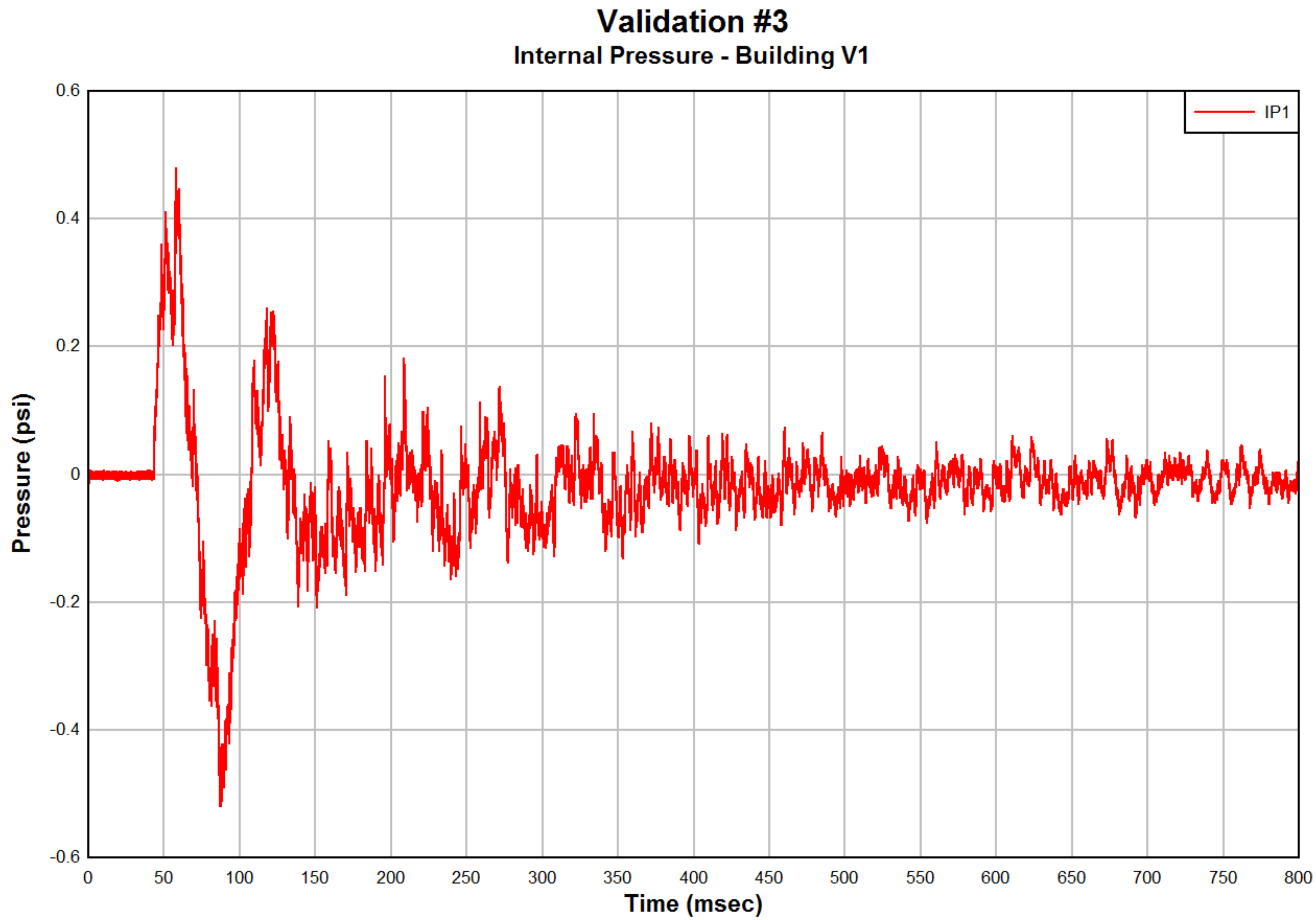
Validation #3 – Building V1: Right Side Face Deflection Gauges



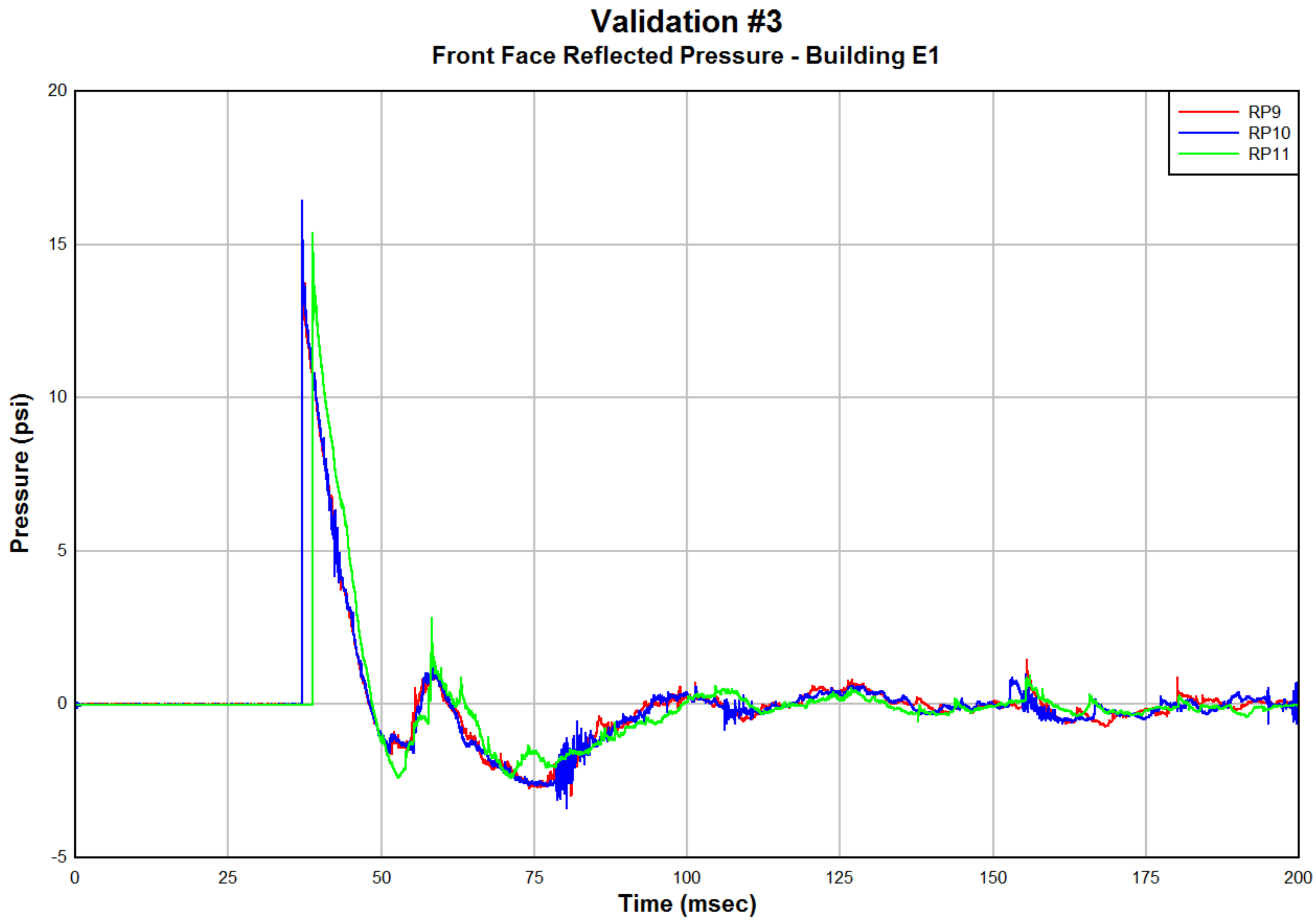
Validation #3 – Building V1: Roof Deflection Gauge



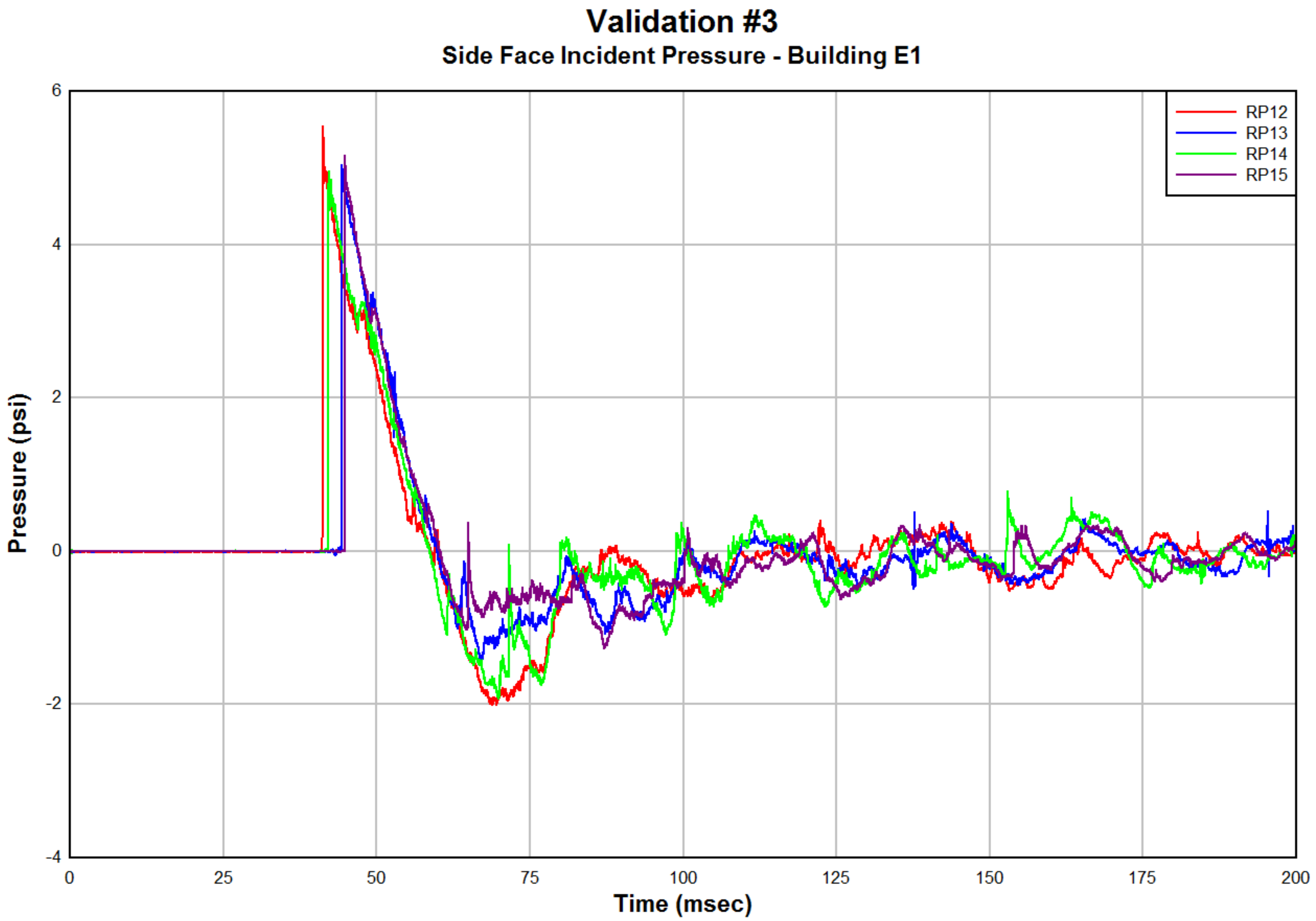
Validation #3 – Building V1: Internal Pressure Gauge



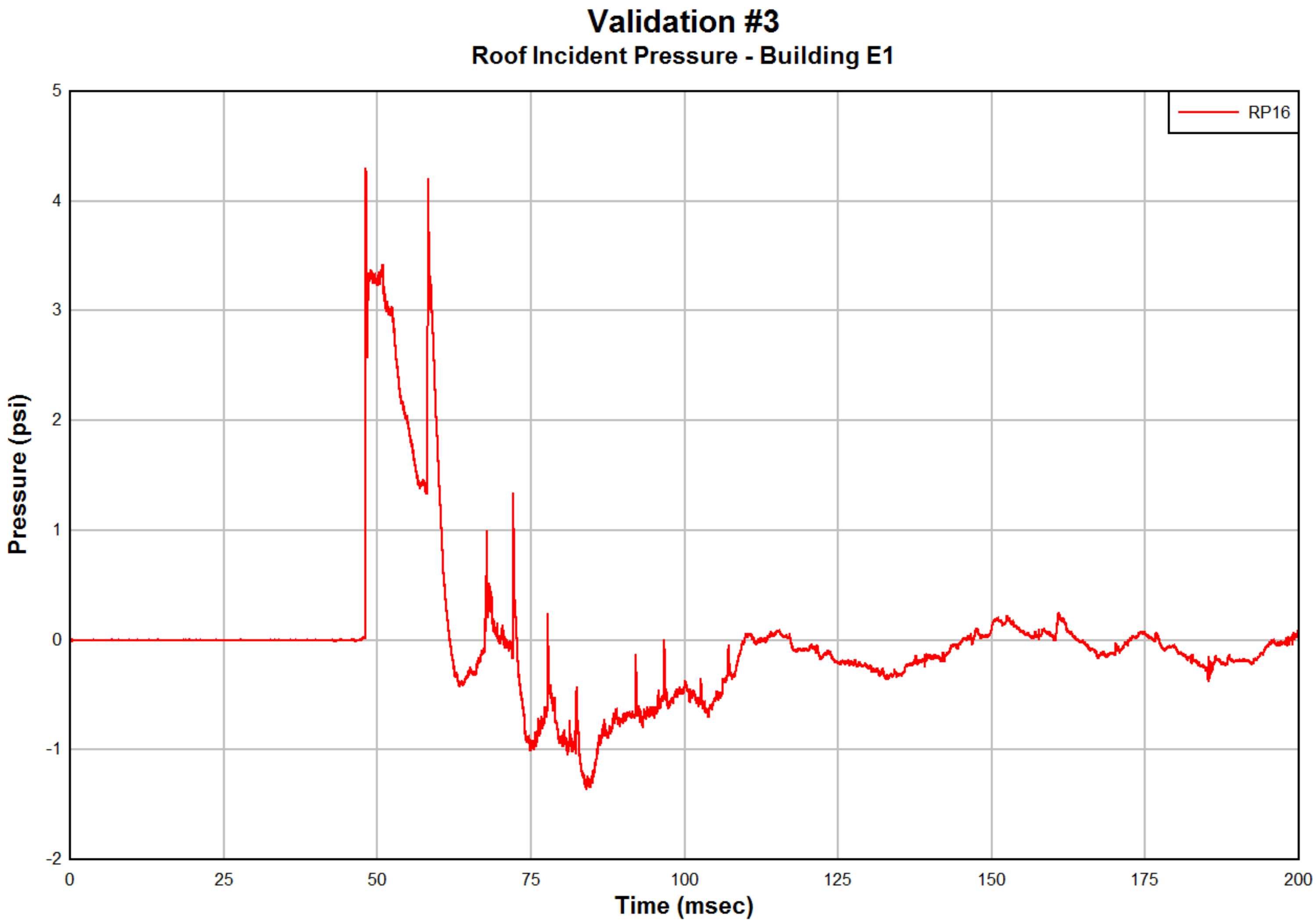
Validation #3 – Building E1: Front Face Reflected Pressure Gauges



Validation #3 – Building E1: Side Face Incident Pressure Gauges



Validation #3 – Building E1: Roof Pressure Gauge



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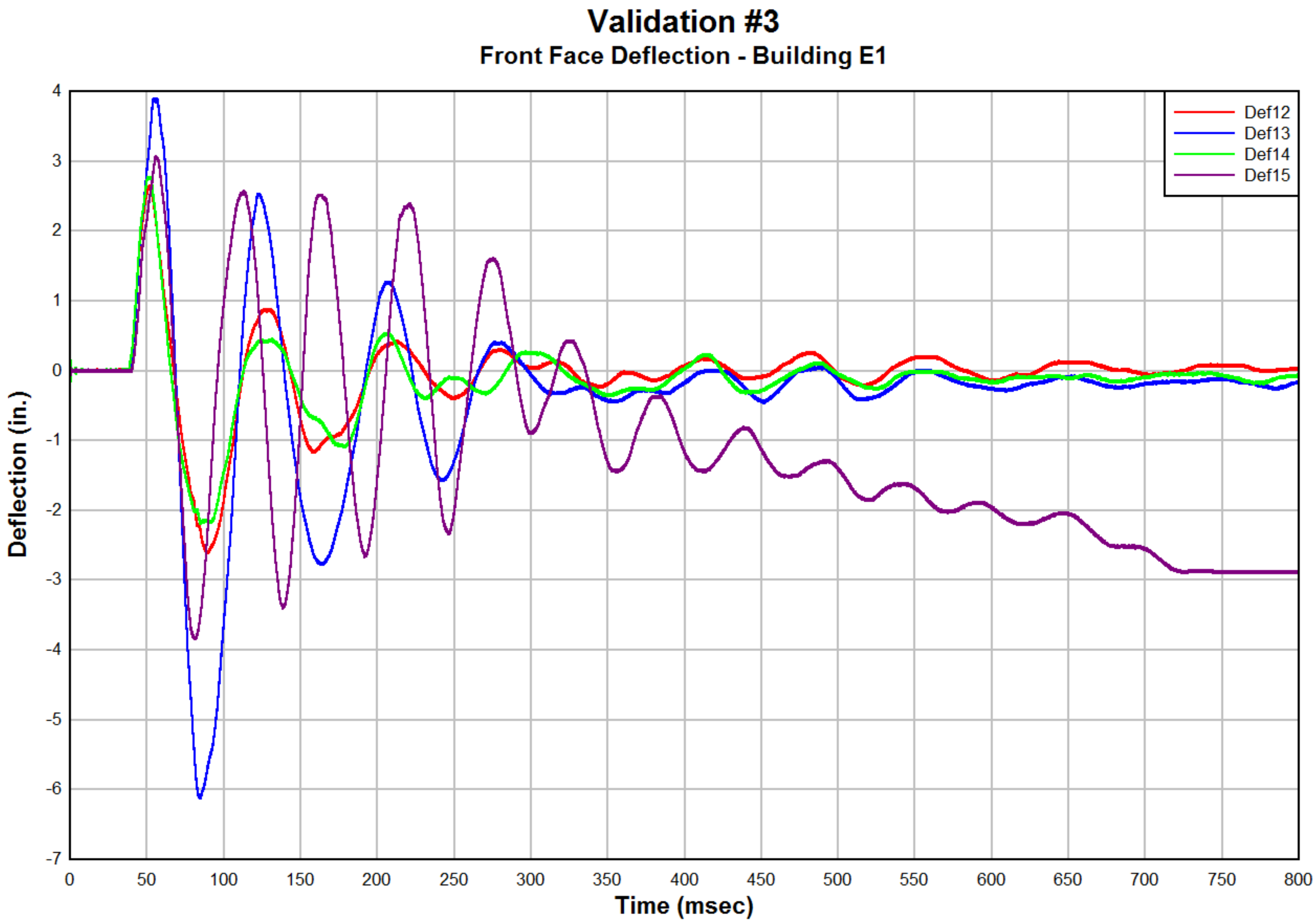
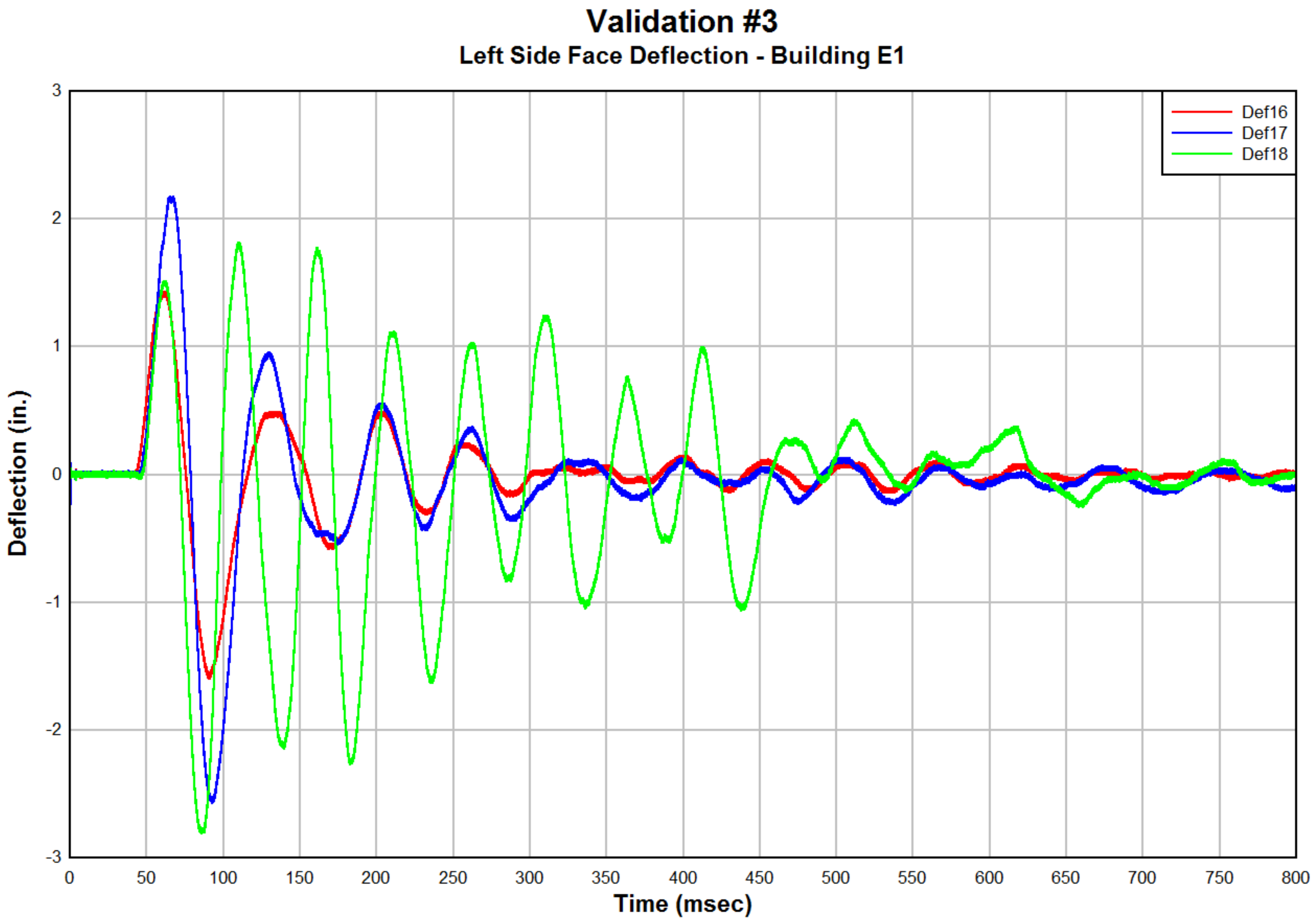
Validation #3 – Building E1: Front Face Deflection Gauges



Validation #3 – Building E1: Left Side Face Deflection Gauges



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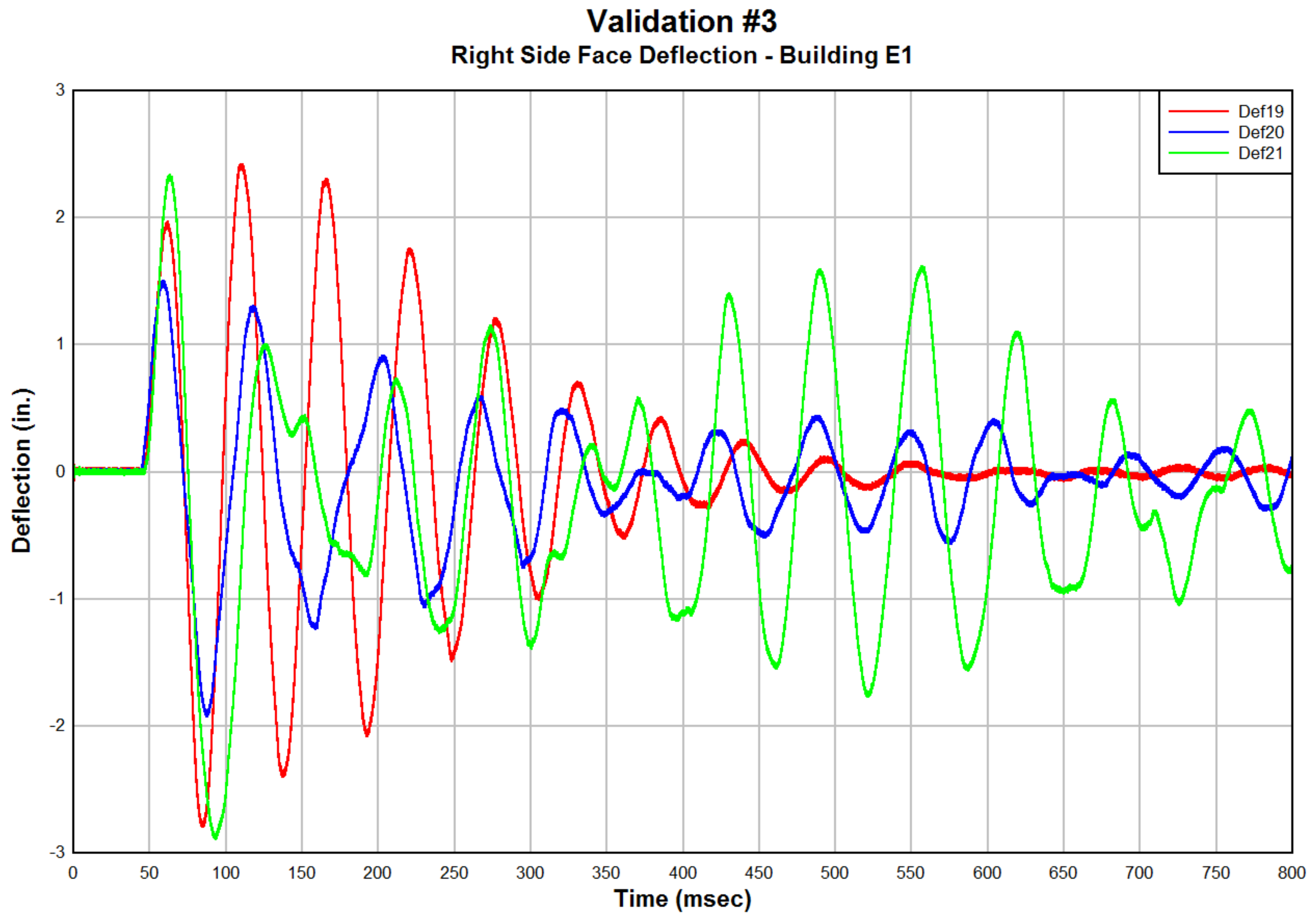
Validation #3 – Building E1: Right Side Face Deflection Gauges



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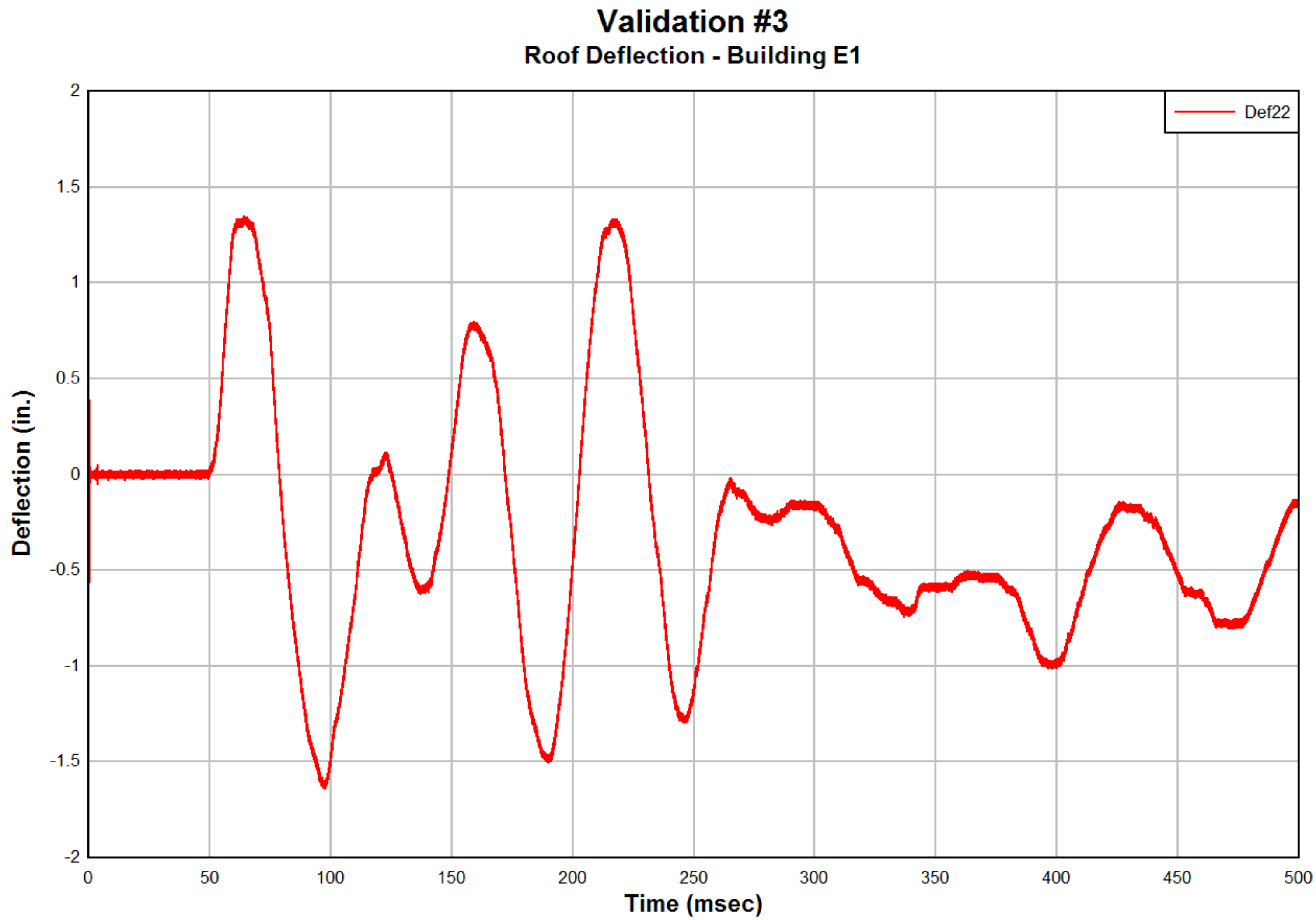
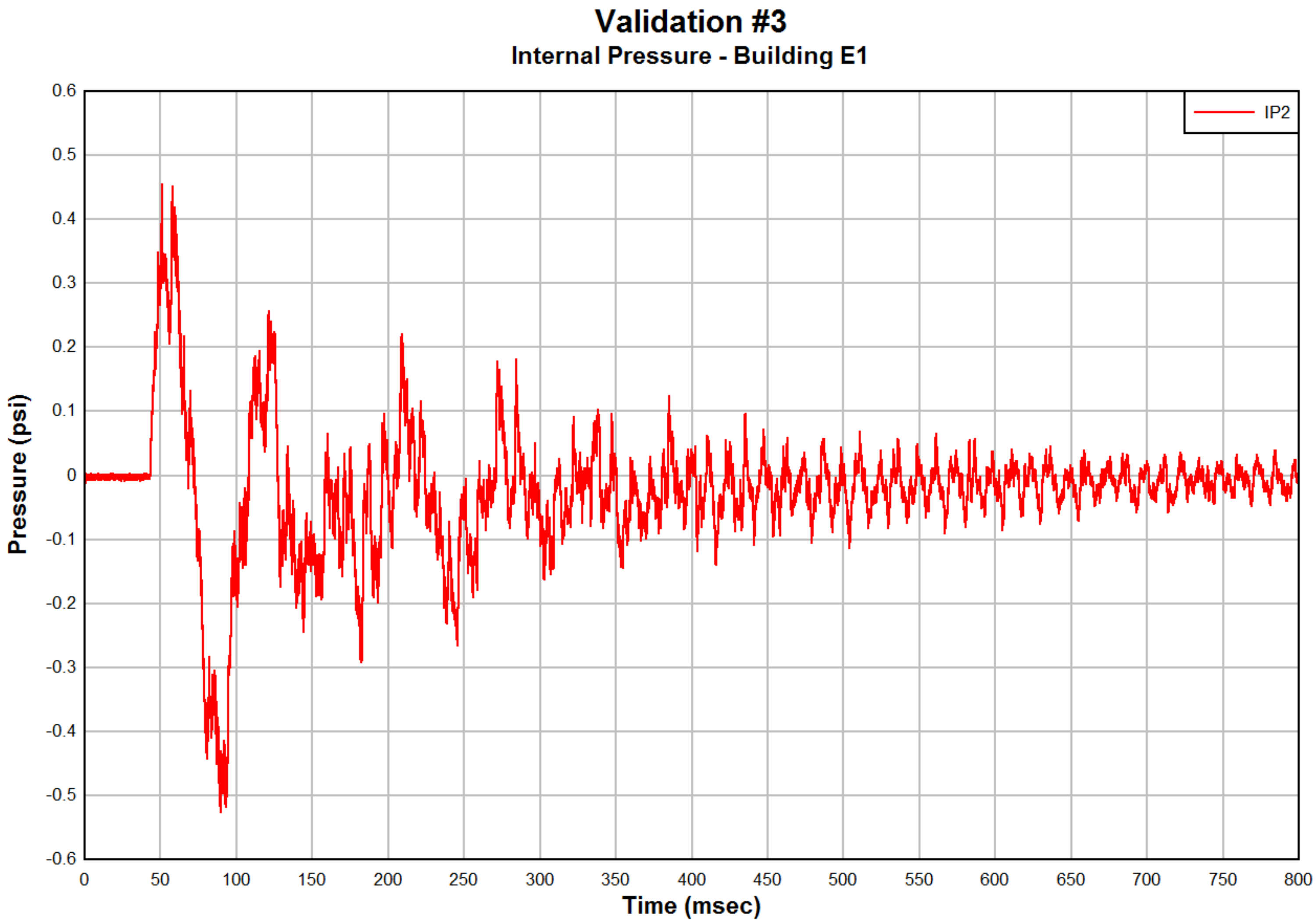
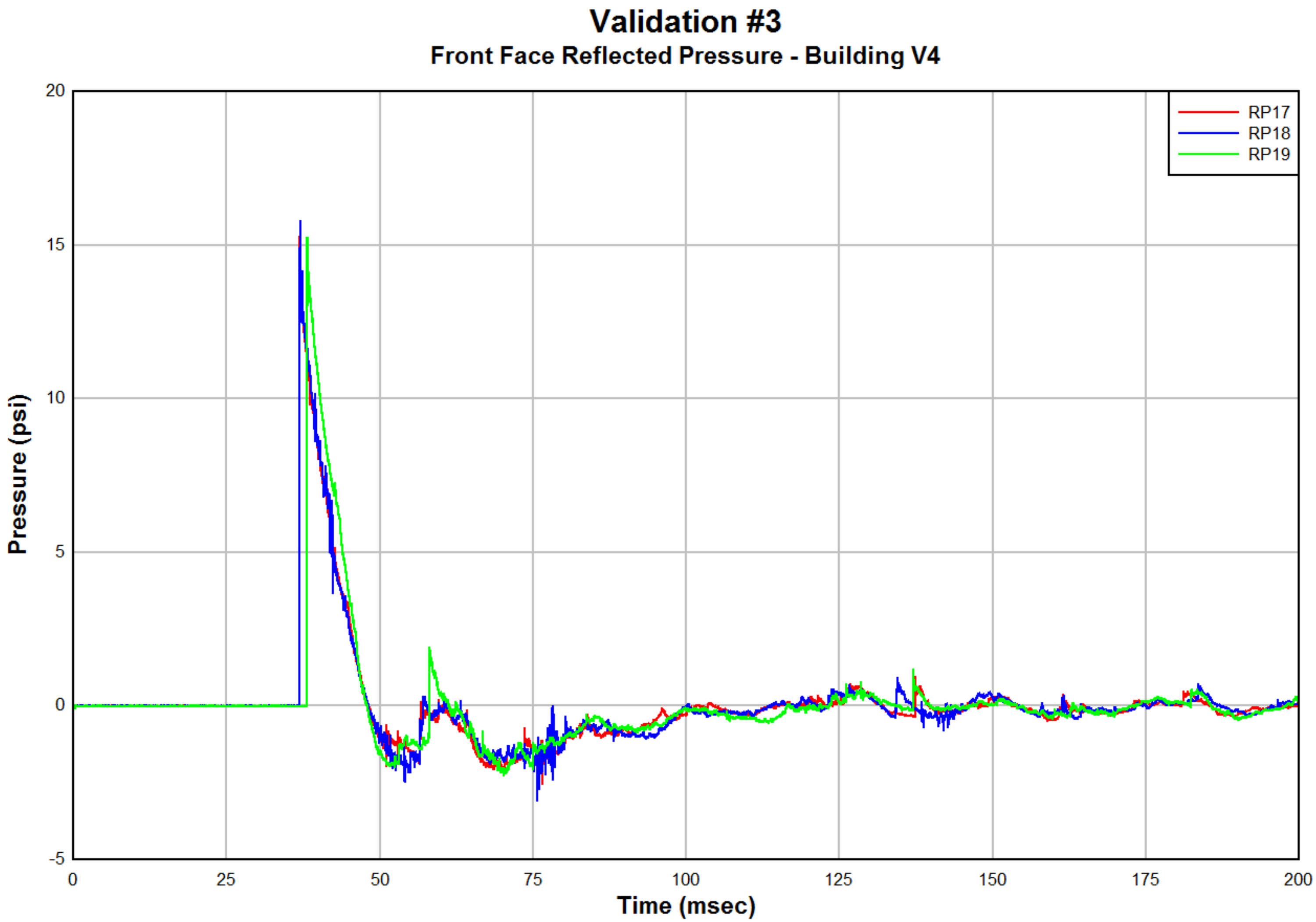
Validation #3 – Building E1: Roof Deflection Gauge



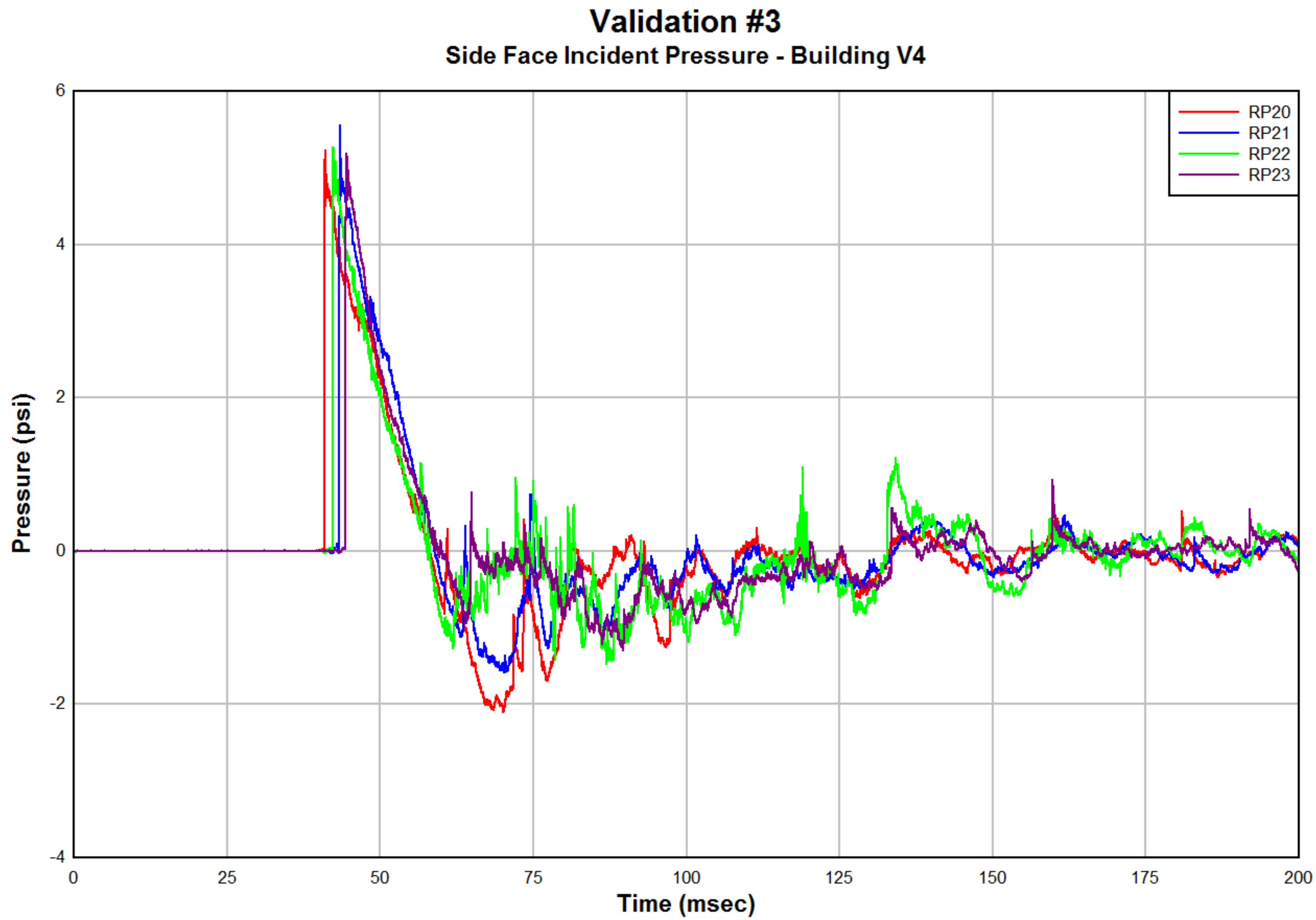
Validation #3– Building E1: Internal Pressure Gauge



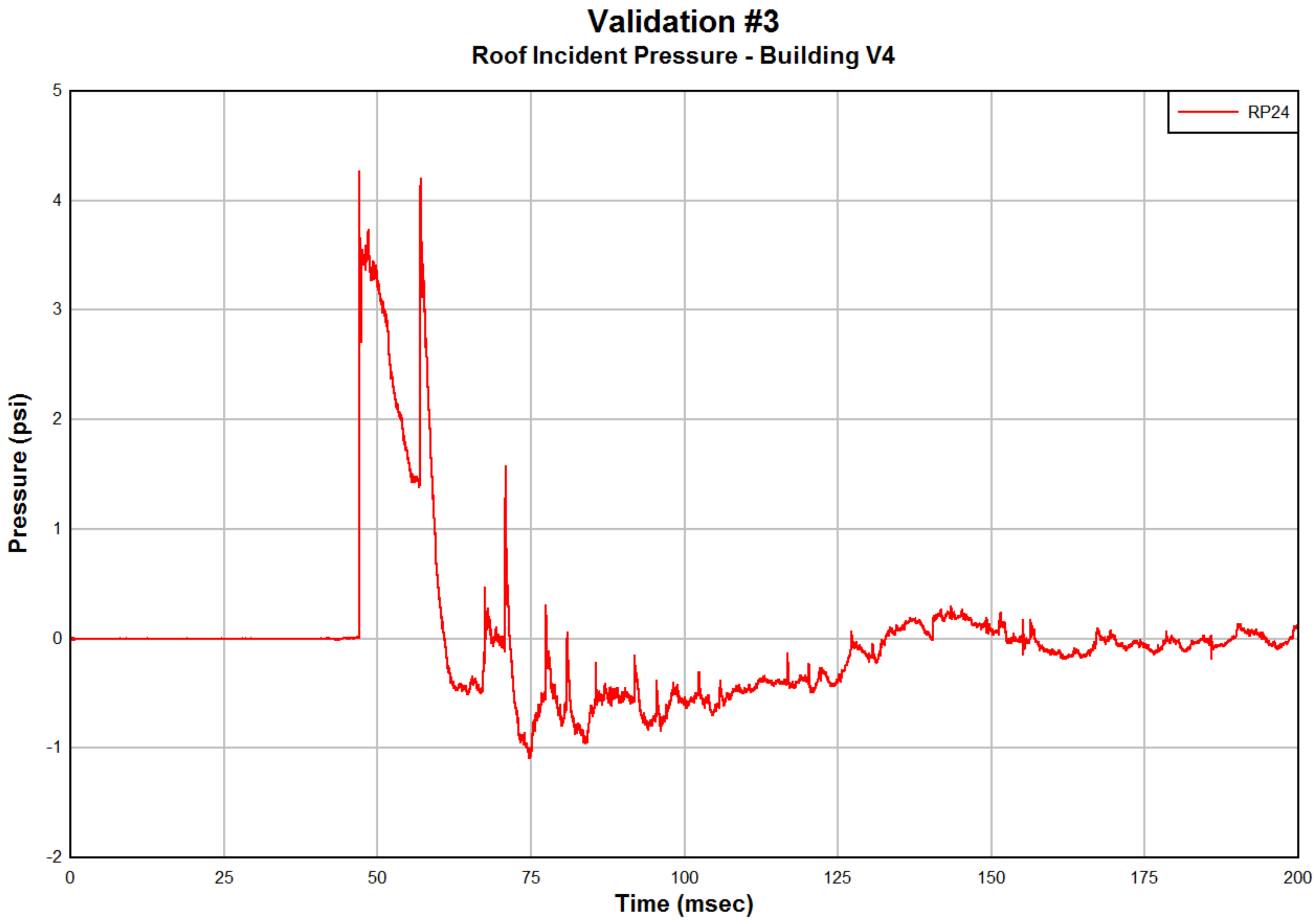
Validation #3 – Building V4: Front Face Reflected Pressure Gauges



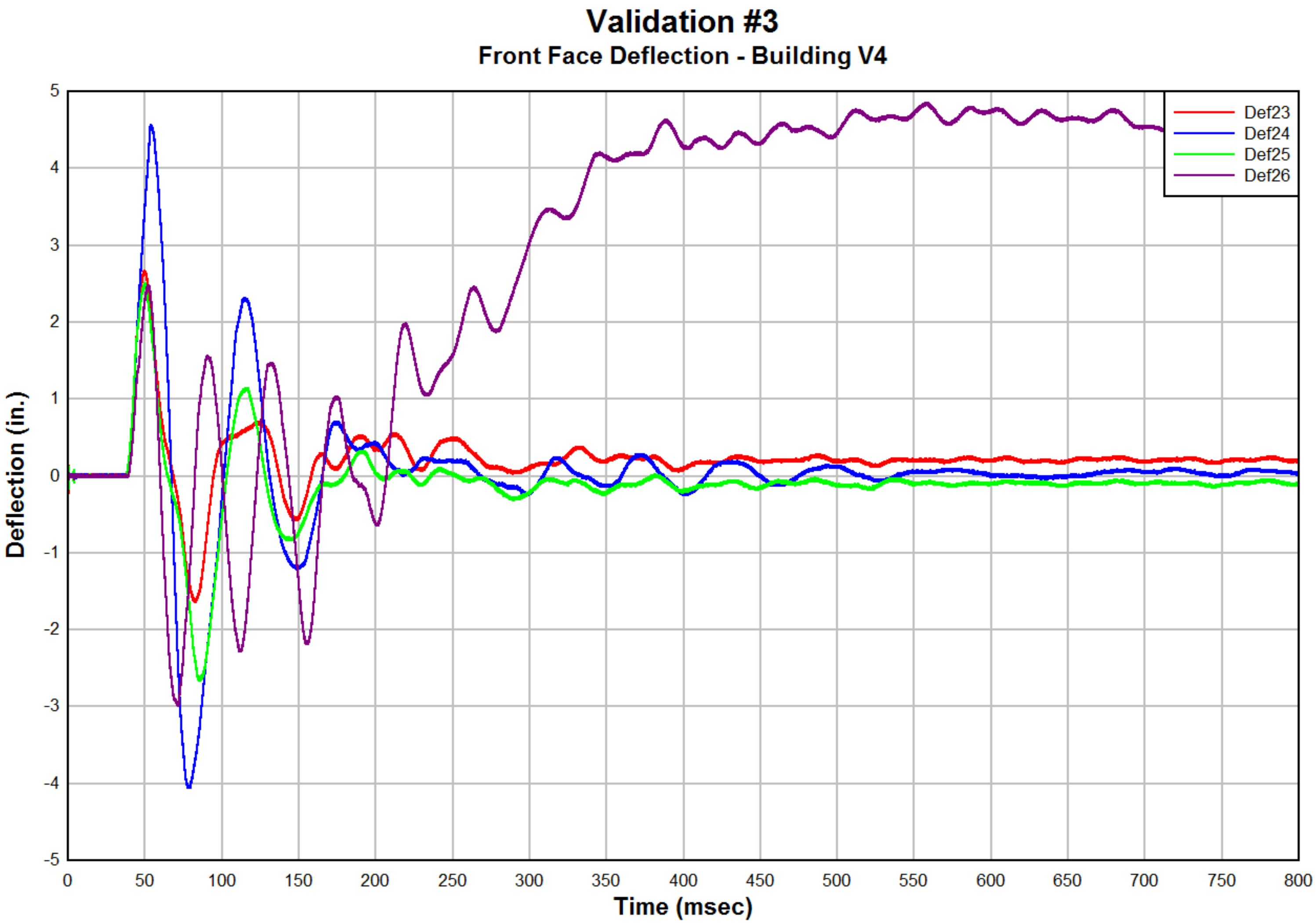
Validation #3 – Building V4: Side Face Incident Pressure Gauges



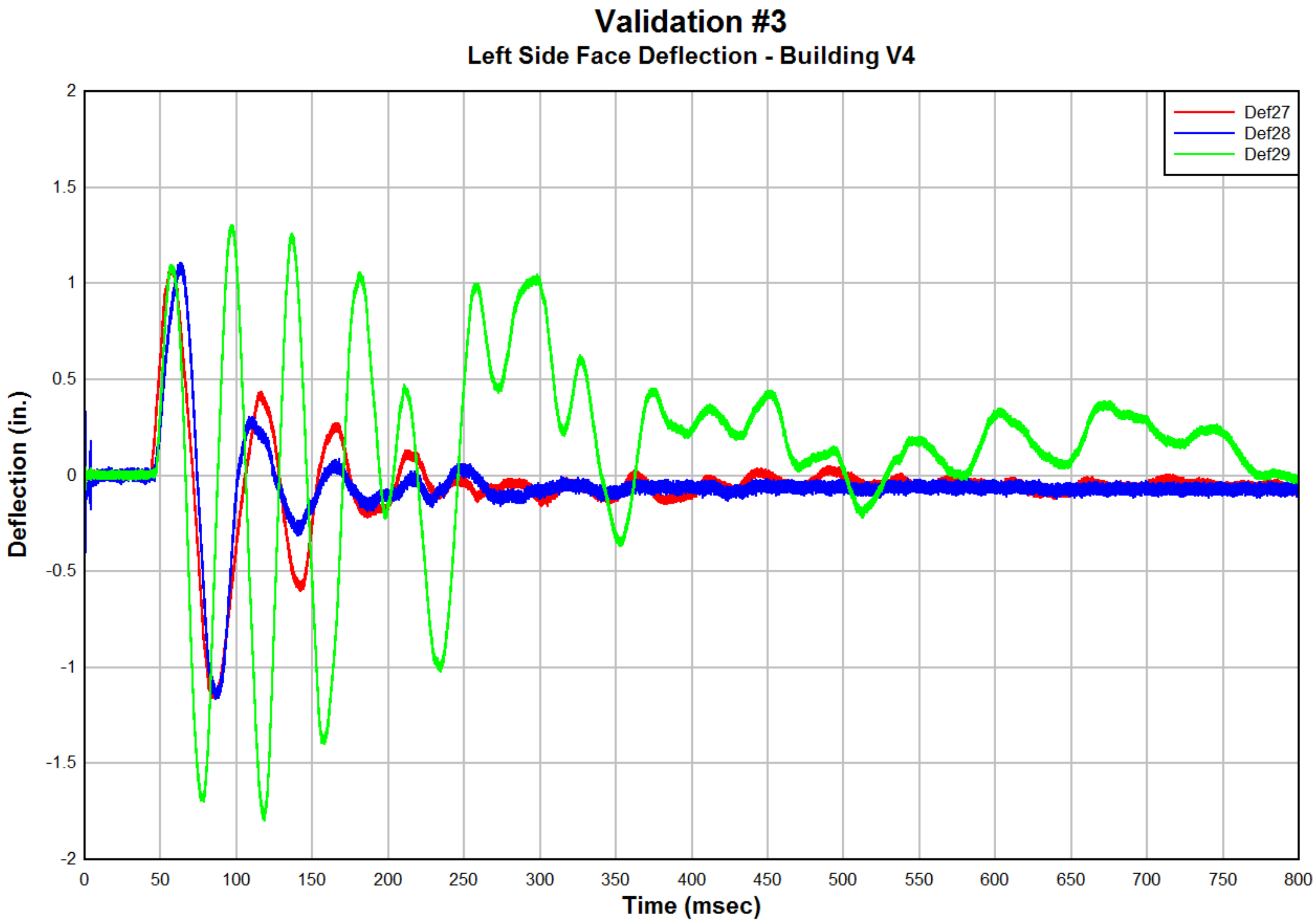
Validation #3 – Building V4: Roof Pressure Gauge



Validation #3 – Building V4: Front Face Deflection Gauges



Validation #3 – Building V4: Left Side Face Deflection Gauges



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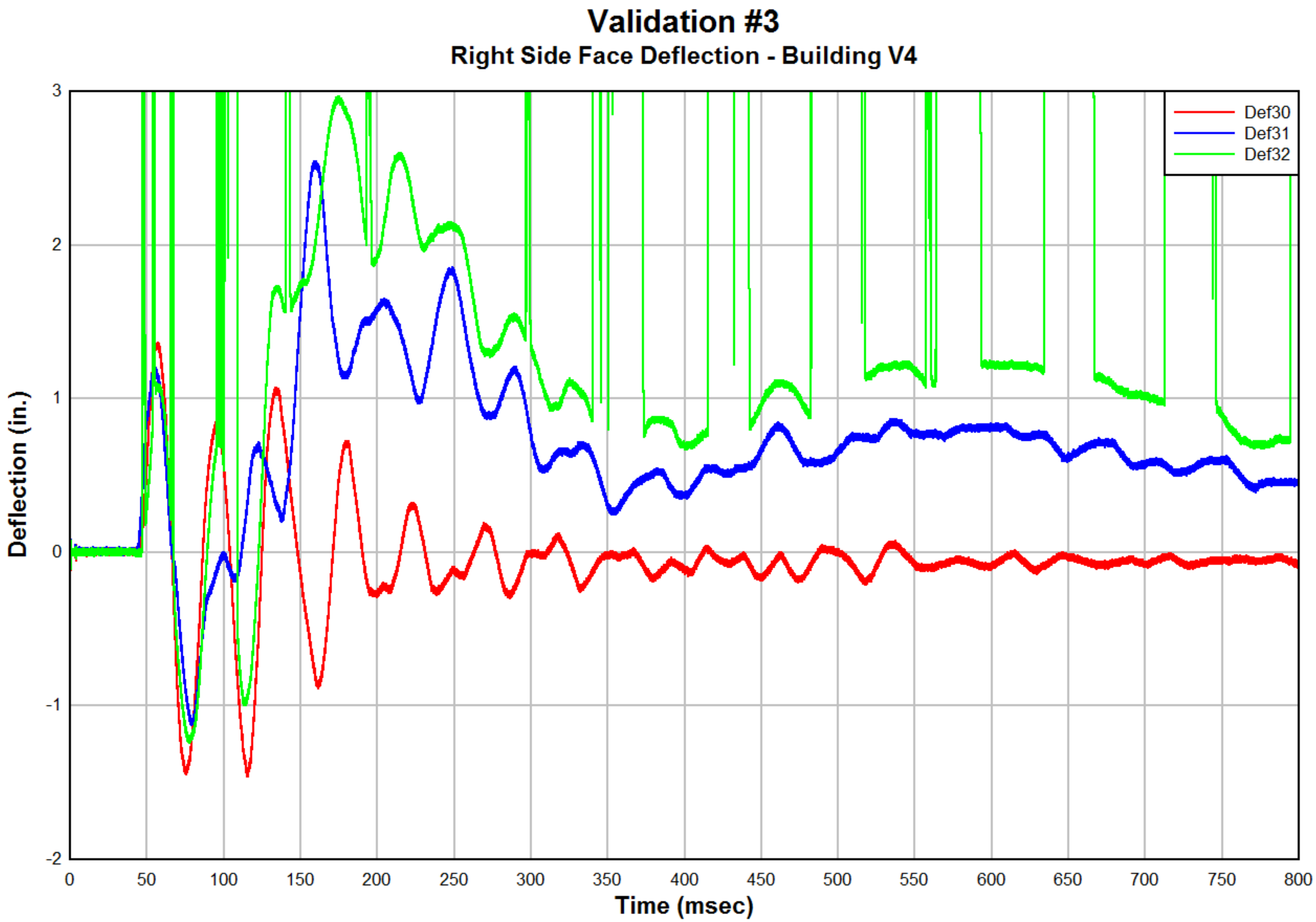
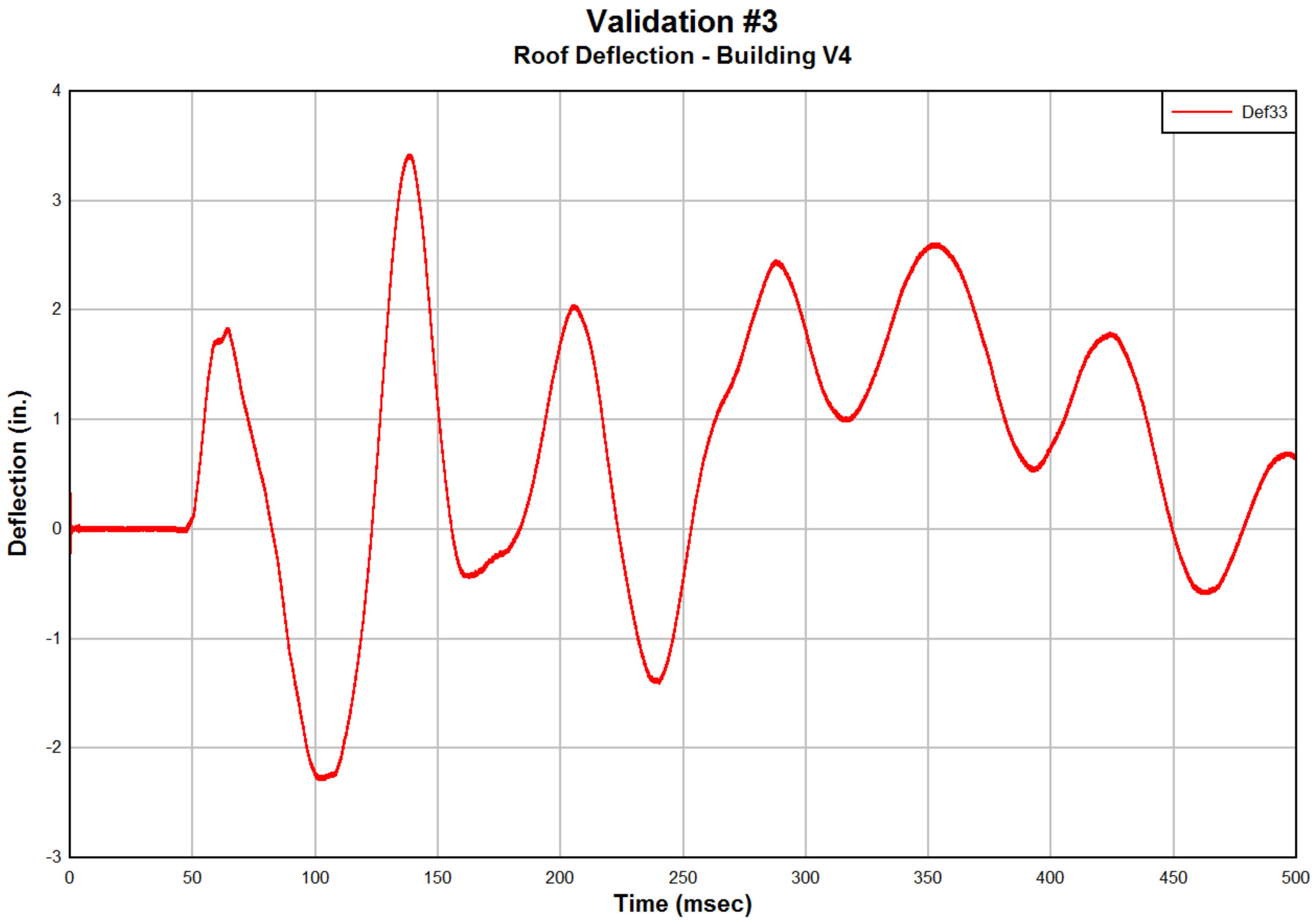
Validation #3 – Building V4: Right Side Face Deflection Gauges



Validation #3 – Building V4: Roof Deflection Gauge



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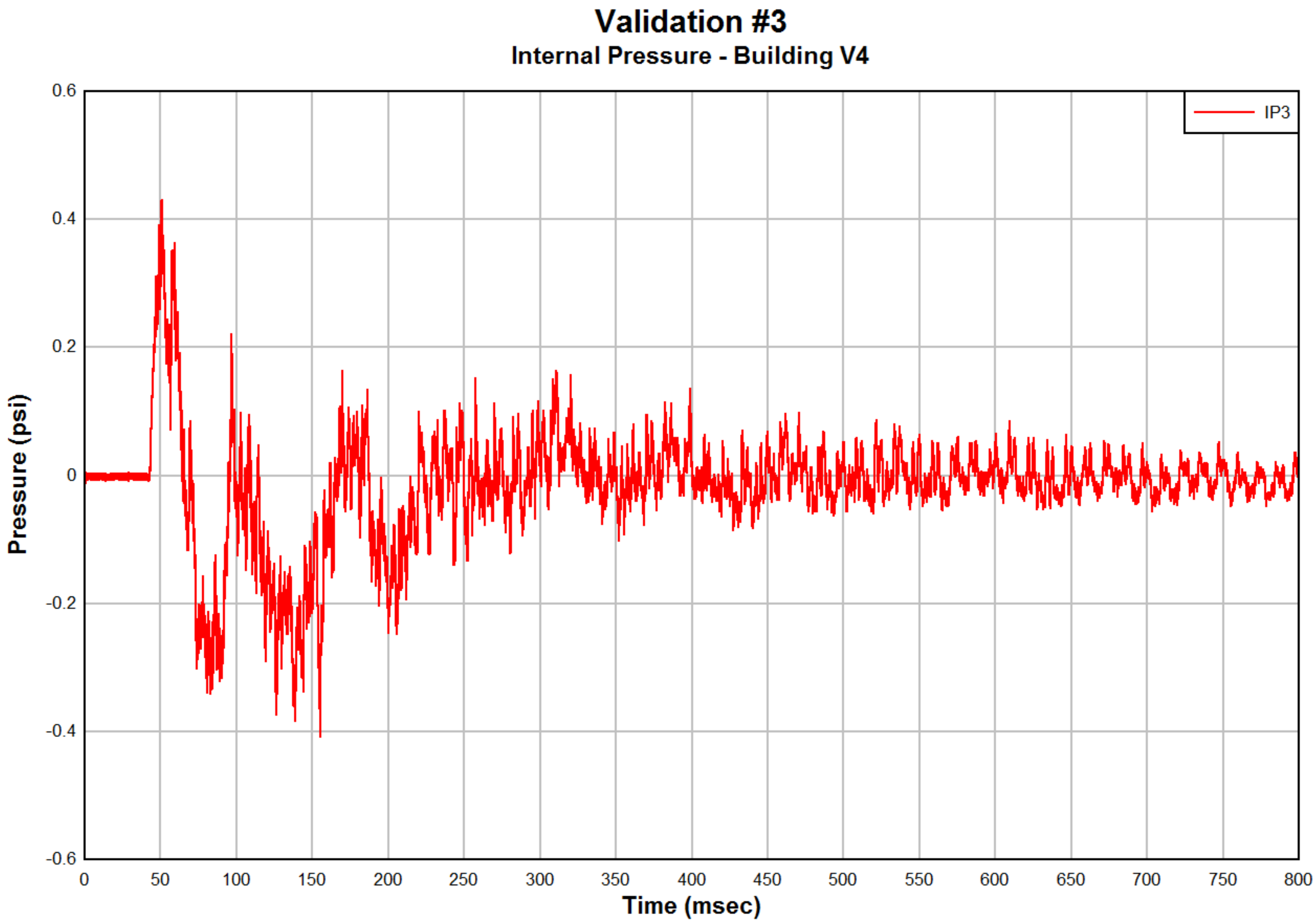
Validation #3 – Building V4: Internal Pressure Gauge



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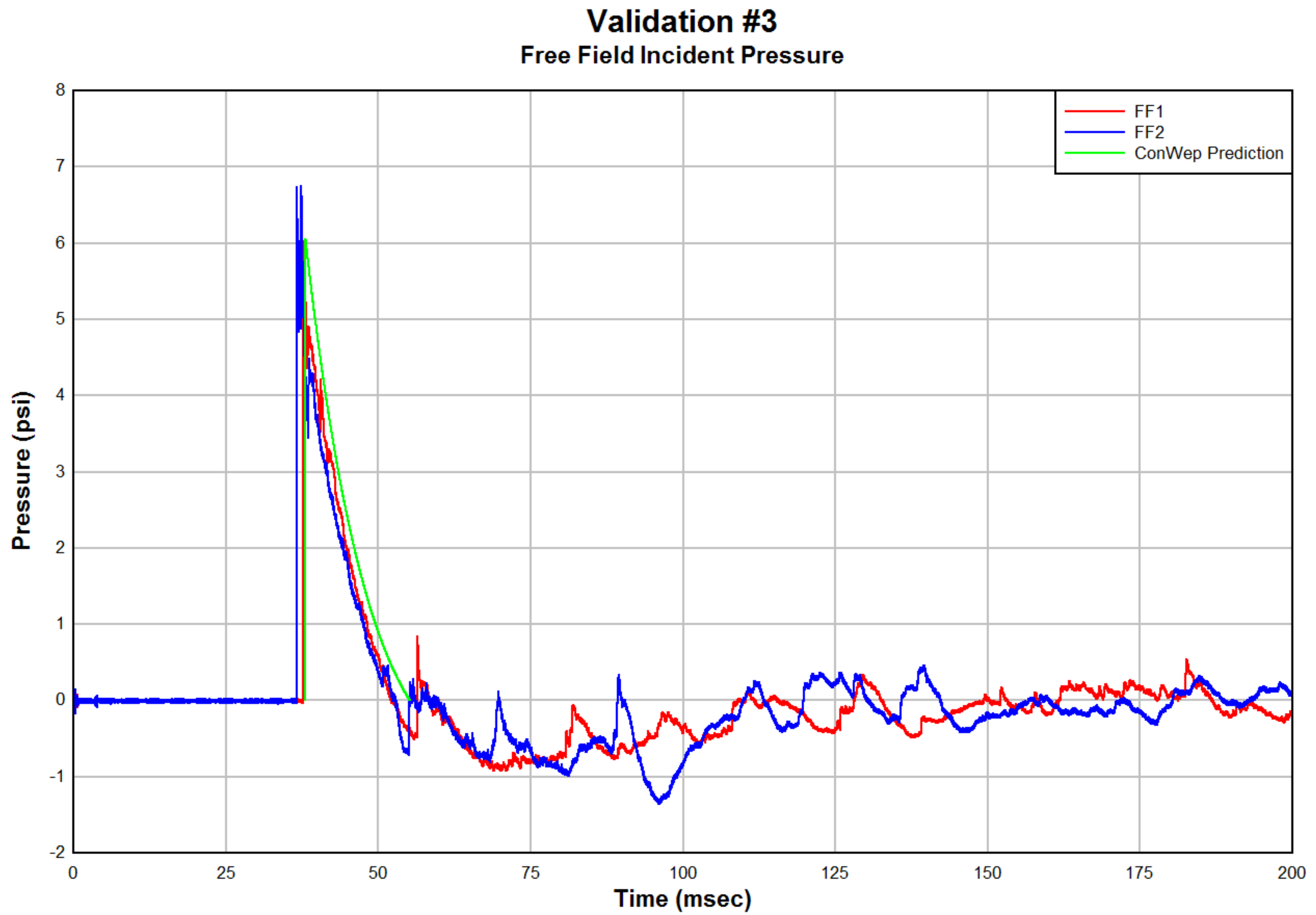
Validation #3: Free Field Incident Pressure Gauges



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Conclusions

- Measured responses for all structures and validations matched K&C developed predictions
- Structures responded elastically during Validations #1 and #2.
- All structures suffered predicted damage to bottom story front faces - both interior and exterior wythes.
- Post test discussions focused on options for subsequent testing – including load bearing or fenestrations.