Karagozian & Case, Inc.

700 N. Brand Blvd., Suite 700

Glendale, CA 91203-3215

818-240-1919

www.kcse.com

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

Mark K. Weaver, S.E.[1]
Charles Newberry, P.E.[2]
Casey O'Laughlin, P.E.[3]
Lisa Podesto, P.E.[4]

[1] *Karagozian & Case, Inc. – Glendale, CA*
[2] *RTL Networks Inc. – Tyndall AFB, FL*
[3] *Jacobs Technology SLG – Fort Walton Beach, FL*
[4] *WoodWorks – Washington, DC*

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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.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

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

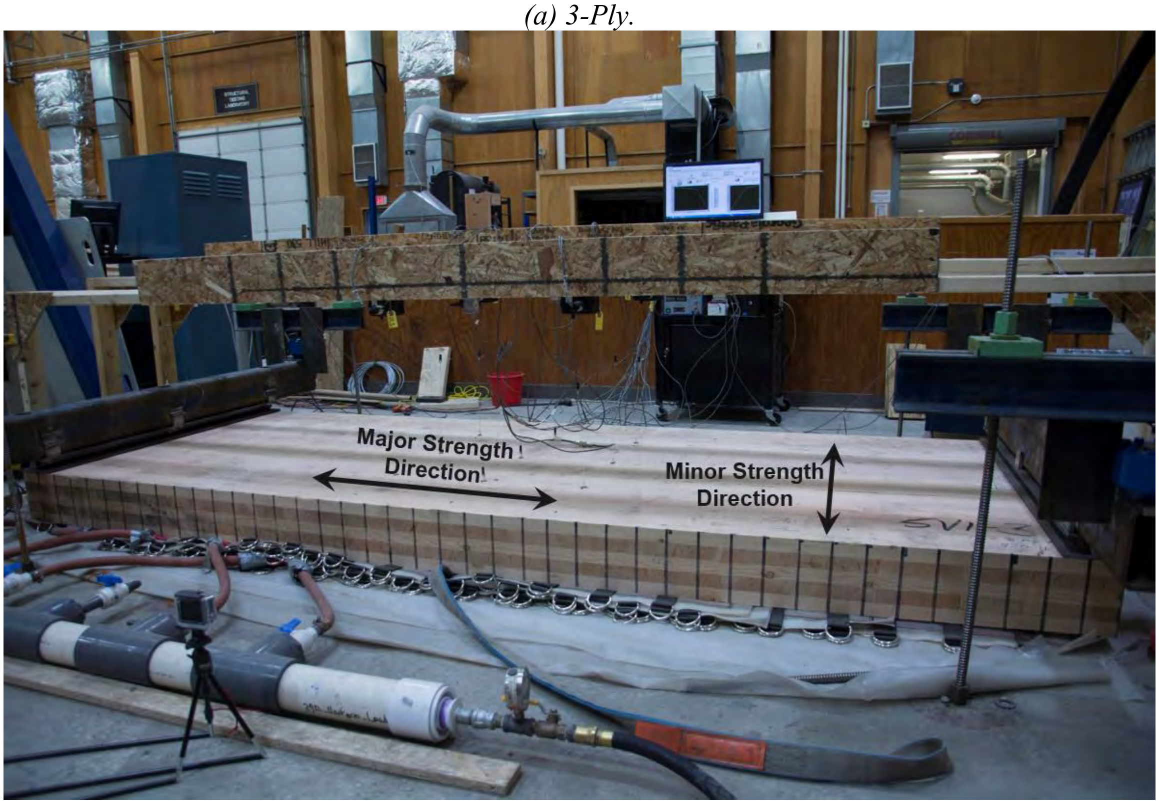
1.1.1 CLT Panel Description

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.

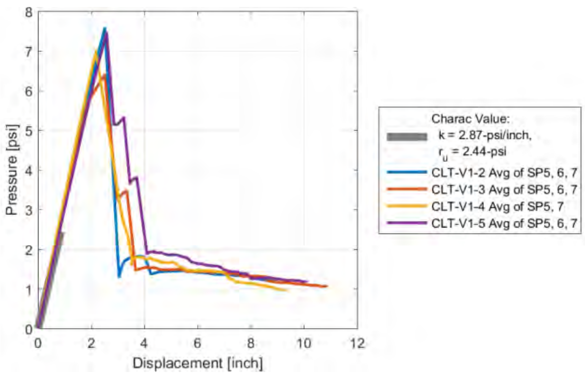
2.1 QUASI-STATIC LABORATORY TESTING

2.1.1 Overview

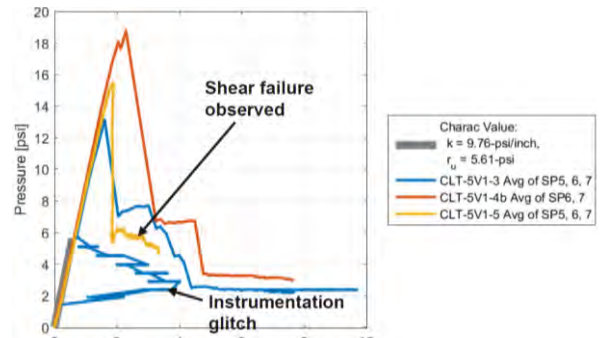
The University of Maine (UMaine) in conjunction with WW and K&C performed a testing program aimed at investigating the bending response of Grade V1 (3-ply and 5-ply), Grade E1, and Grade V4 CLT panels in their major strength direction under a uniformly-applied quasi-static load [3]. The apparatus utilized 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.

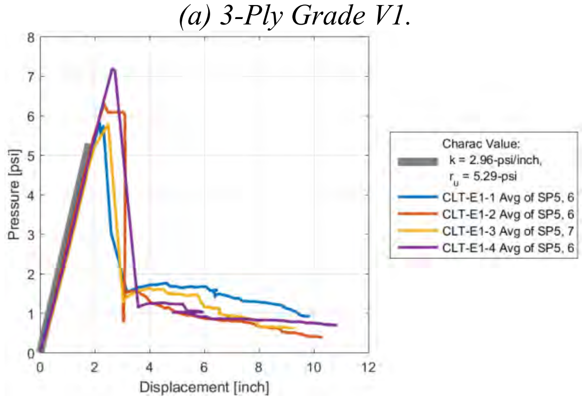
Applied pressure, out-of-plane displacement, and total resisted load were measured and recorded as panels were displaced well beyond the displacement associated with peak panel strength. Load-displacement plots for each CLT grade and ply configuration tested are shown in Figure 2-2.



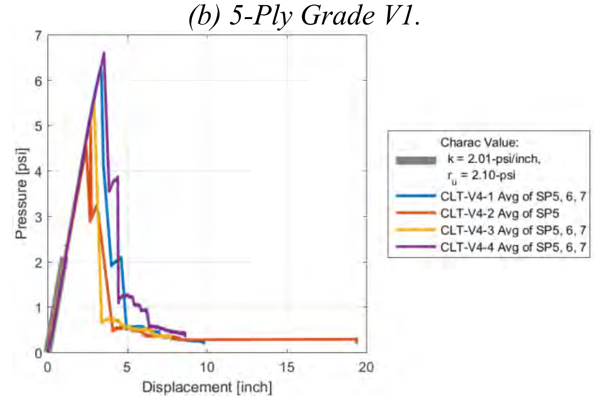
(a) 3-Ply Grade V1.



(b) 5-Ply Grade V1.



(c) 3-Ply Grade E1.

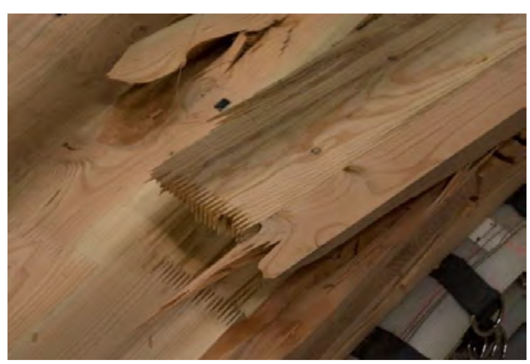


(d) 3-Ply Grade V4.

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.

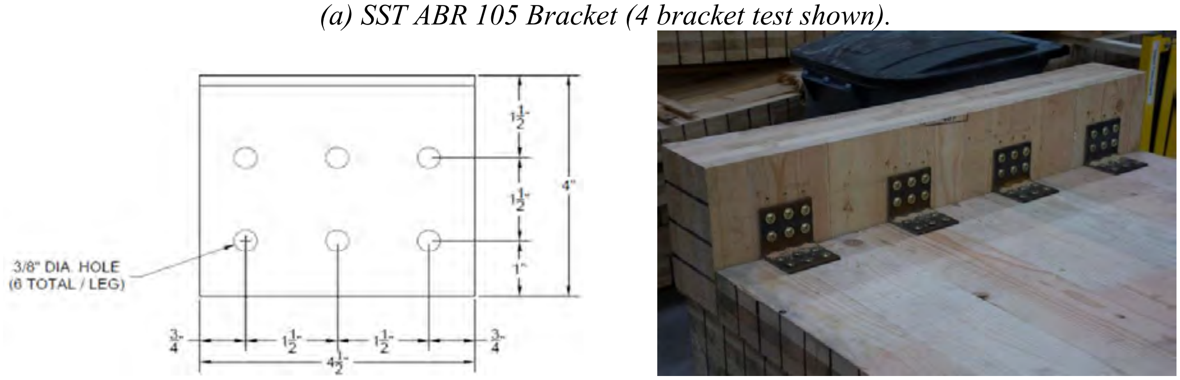
While most of the panels were tested with end conditions that did not restrain panel rotation, six 3-ply Grade V1 CLT panels were tested with connections meant to represent those that might be used to attach a wall to a floor and ceiling in a building designed to resist significant out-of-plane wall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

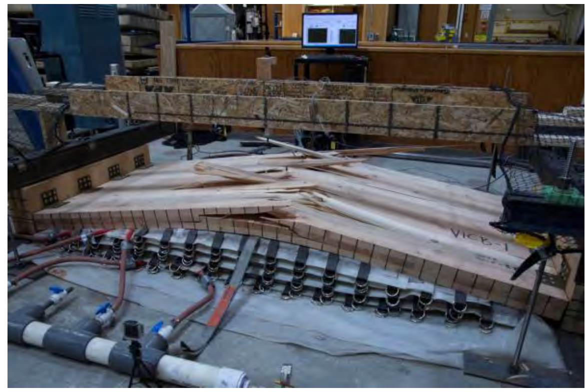


(b) L4x4x1/4 Bracket (4 bracket test shown).

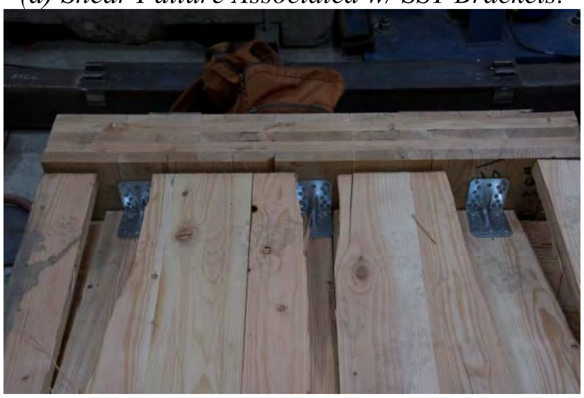
Figure 2-4. Quasi-Static Testing Connection Types.

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.

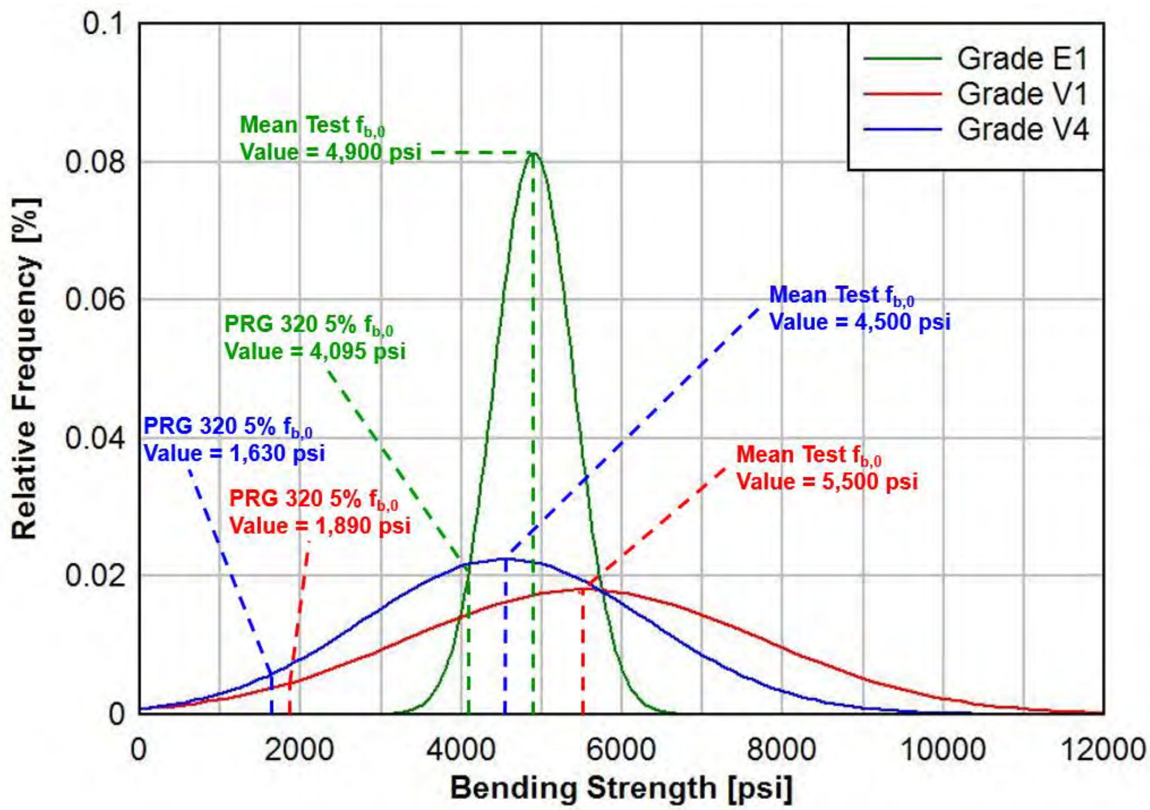


Figure 2-6. Bending Strength Normal Distributions by CLT Grade.

- 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.

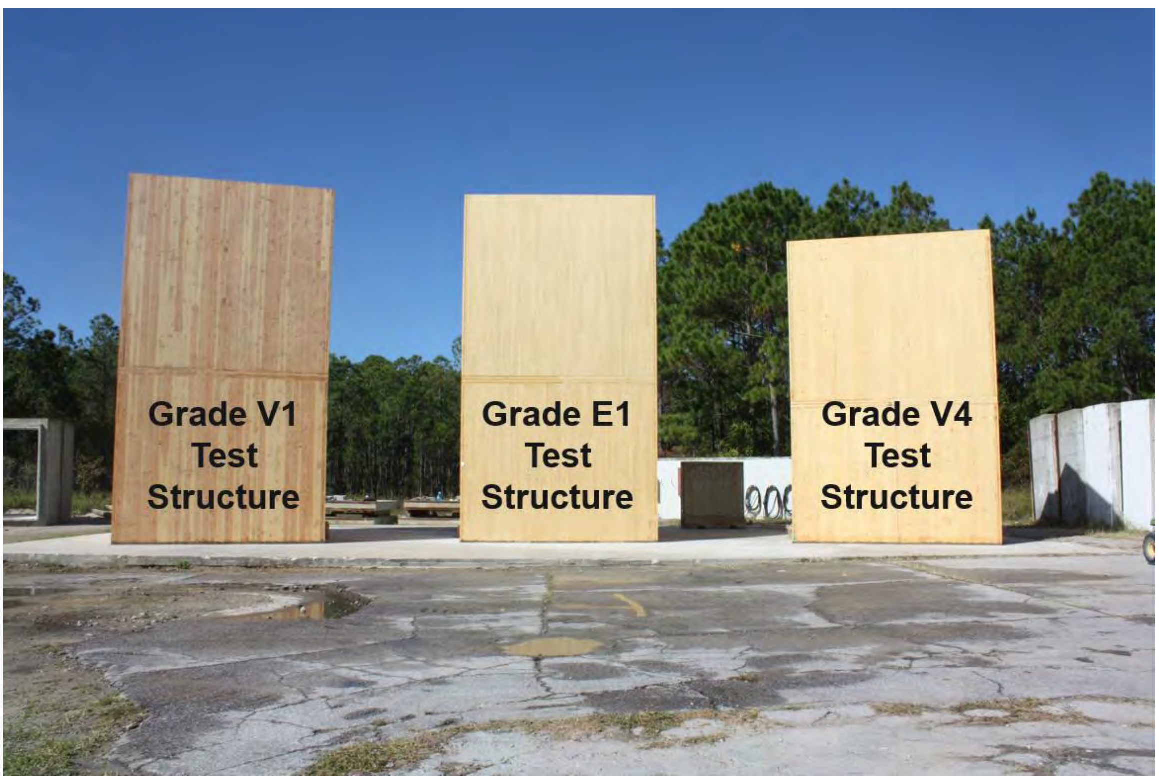


Figure 3-1. Pre-Test Photograph of All Test Structures.

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.

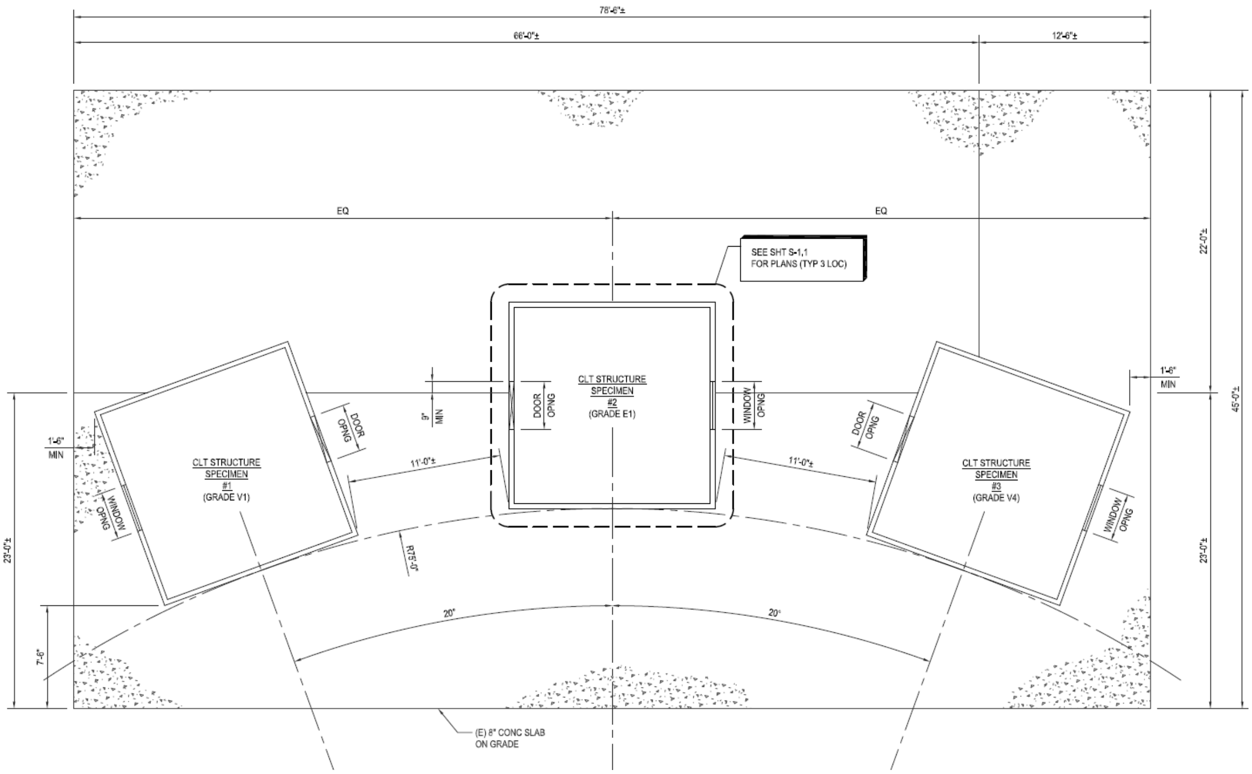


Figure 3-2. Site Plan.

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).

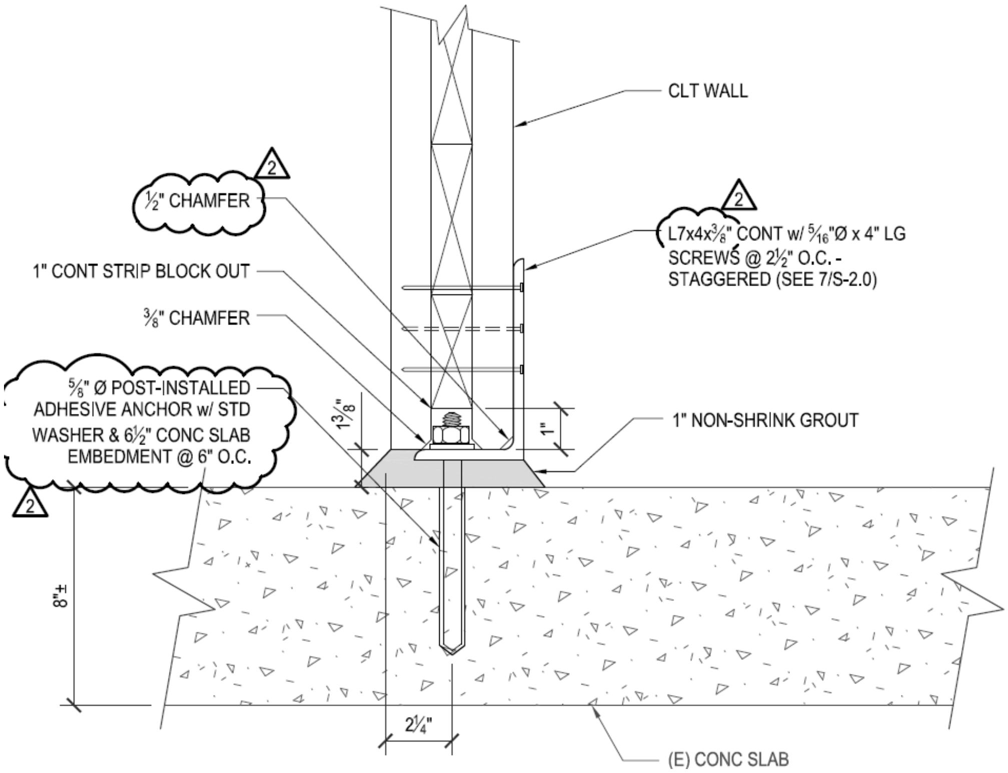
Figure 3-3. Self-Tapping Screws Used in Test Structure Connections.

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.



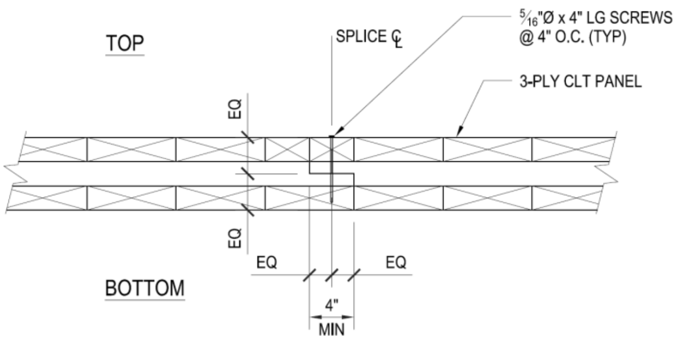
(a) Detail.

(b) Angle Placement Prior to Panel Install.

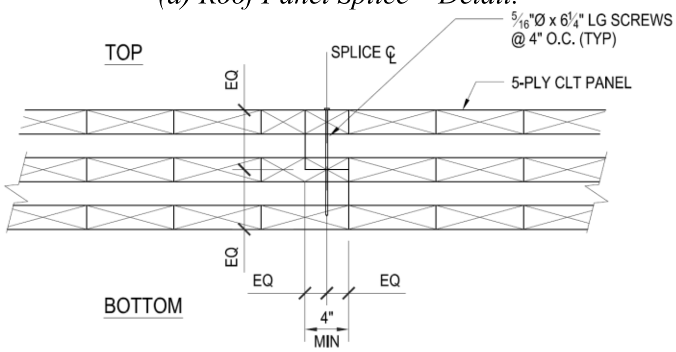
Figure 3-4. Panel-to-Foundation Connection.

3.1.3. 2 *PANEL-TO-PANEL SPLICE CONNECTION*

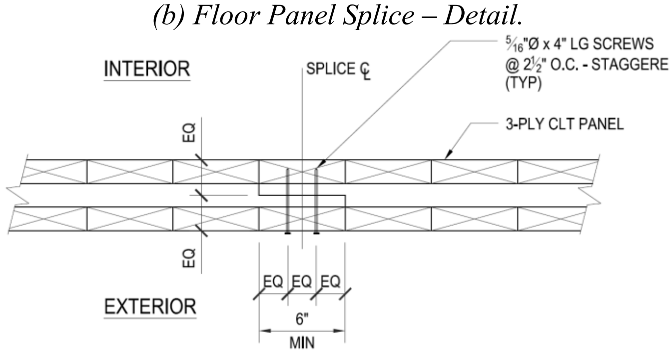
The panel splice connections are shown in Figure 3-5. Half-lapped joints were used to cause adjacent diaphragm and shear wall panels to act together. Self-tapping screw spacing was computed to resist the in-plane shear forces associated with Test 2. In all cases, screw length was sized to engage all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



(a) Roof Panel Splice – Detail.



(b) Floor Panel Splice – Detail.



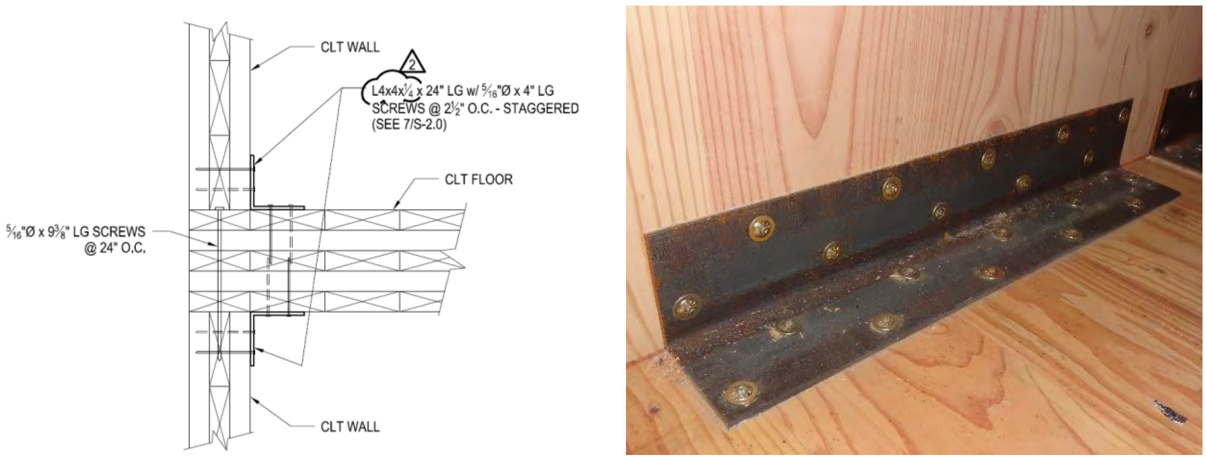
(c) Wall Panel Splice – Detail.

(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.

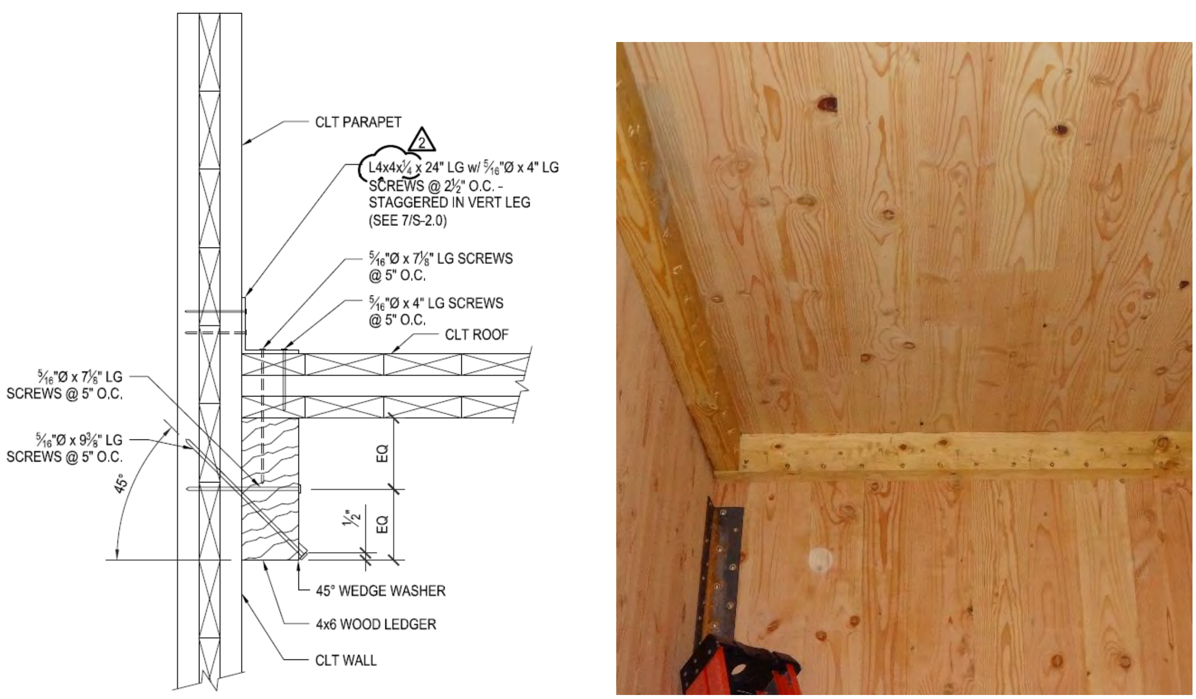


/(a) Detail. (b) As Installed.

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.

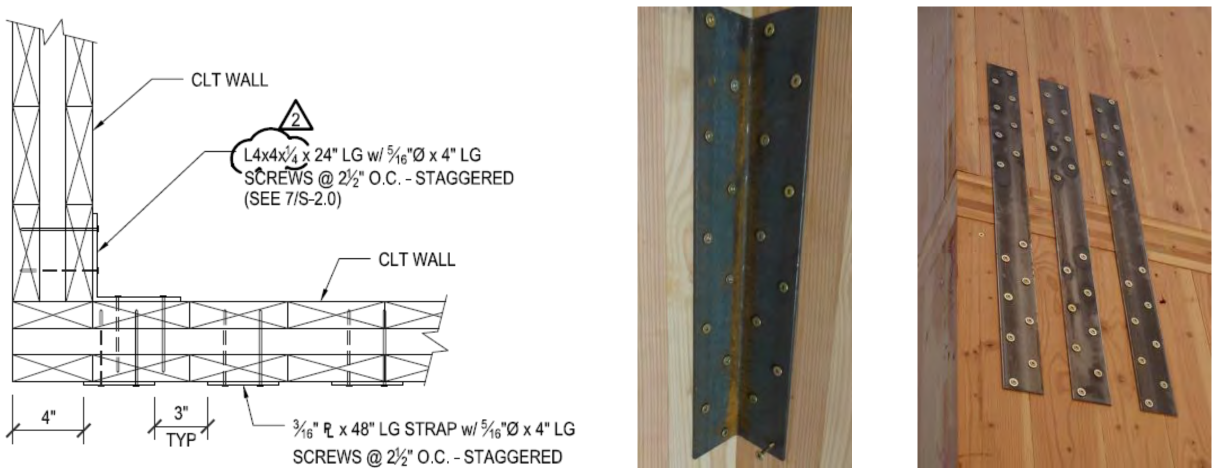


(a) Detail. (b) As Installed.

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.*

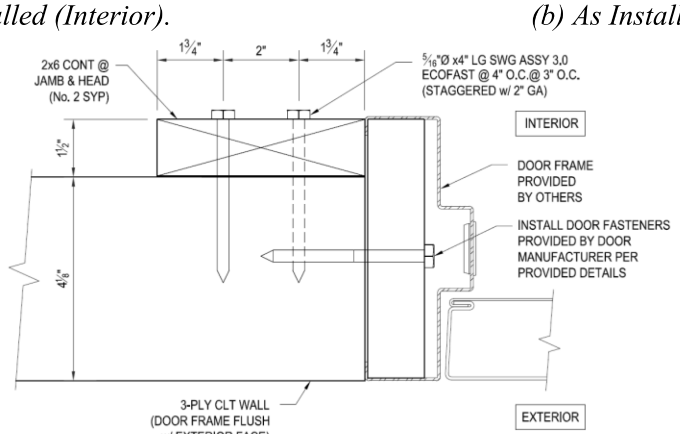
Figure 3-9. Window Opening Connection.

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.

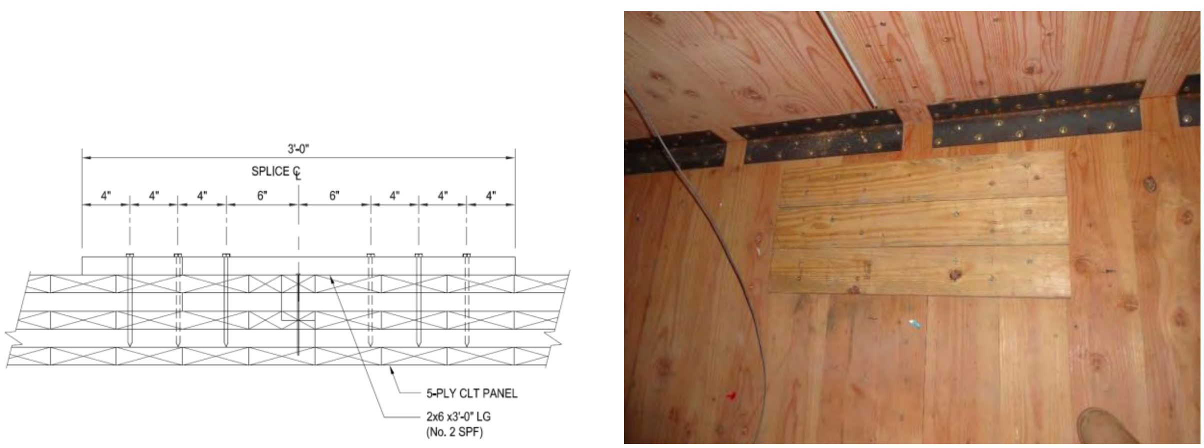
Two types of fasteners were used to secure the door frame to the CLT test structures: (1) ten $^{1}/_{2}$-inch diameter by 5-inch long lag screws and (used at the Grade E1 and V1 structures) and (2) twenty-eight $^{5}/_{16}$-inch diameter by $5^{1}/_{2}$-inch long SWG ASSY® Kombi STS manufactured by 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.

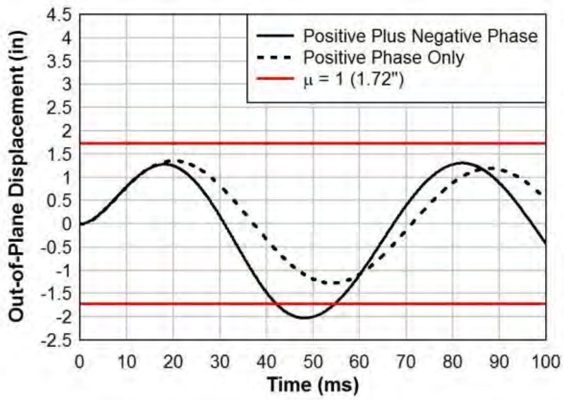
Both positive-phase-only and positive-plus-negative-phase airblast load cases were considered in the SDOF analyses. In all cases, the inclusion of the negative phase led to maximum displacement response; this result is illustrated in Figure 3-12.

Table 3-2. SDOF Dynamic Analysis Parameters.

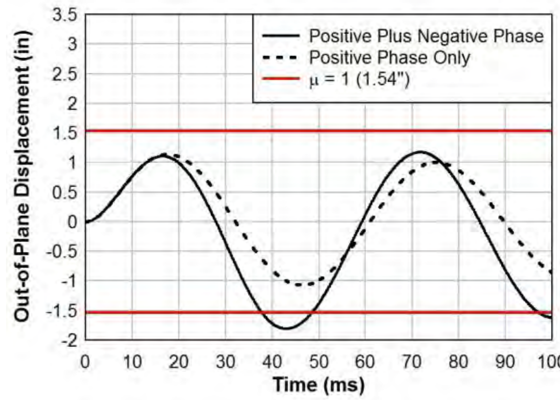
Structure Grade	L [ft]	m [psi-ms^2/in]	k [psi/in]	r [psi]	x_E [in]
V1	12	216.2	1.45	2.49	1.72
V4	10	216.2	2.01	3.09	1.54
E1	12	216.2	1.50	5.39	3.58

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

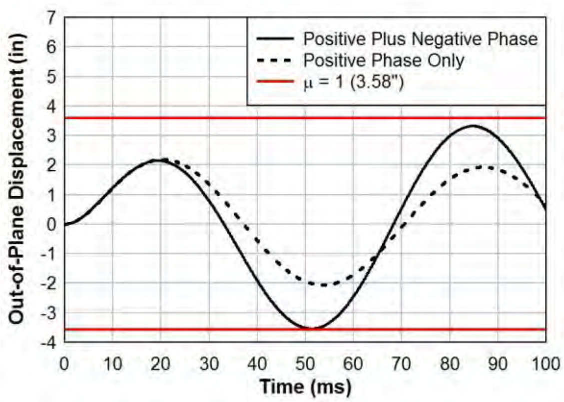
Structure Grade	Test 1		Test 2		Test 3	
	Target	*Computed*	*Target*	*Computed*	*Target*	*Computed*
V1	1.00	1.18	-	-	-	-
V4	1.00	1.18	-	-	-	-
E1	-	-	1.00	0.99	1.50	1.51



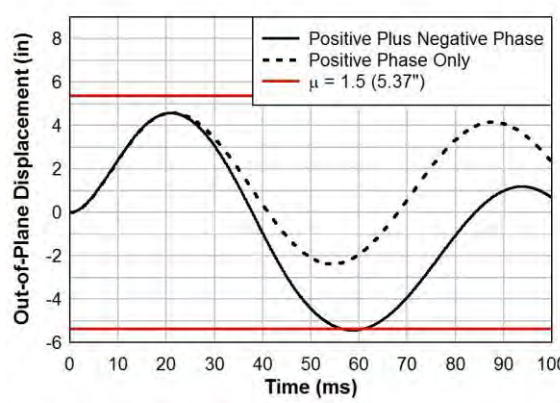
(a) Test 1 – Grade V1.



(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

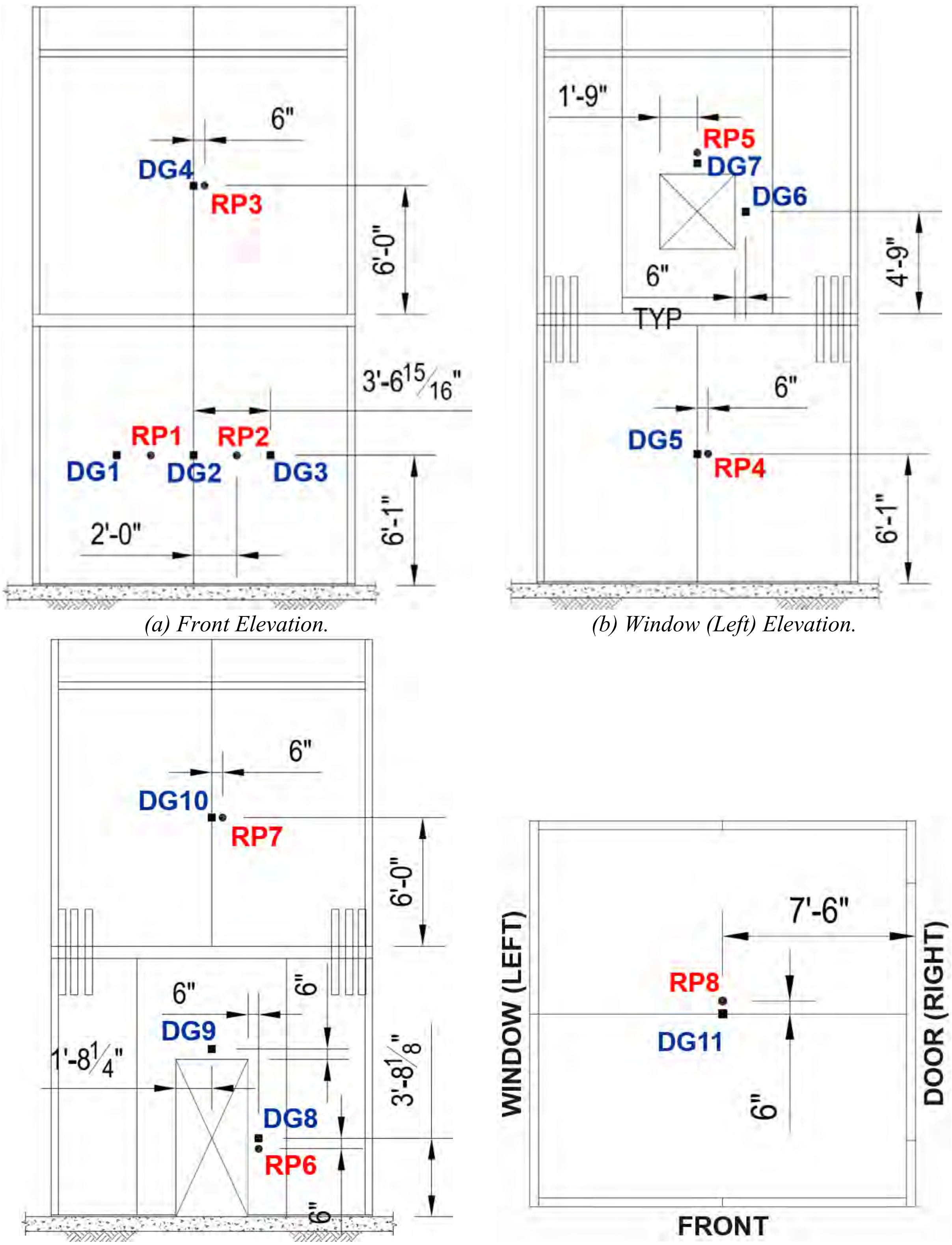
Table 3-4 provides a summary of the twenty-nine Kulite XT-190 pressure gages that were used for each test:

- 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.

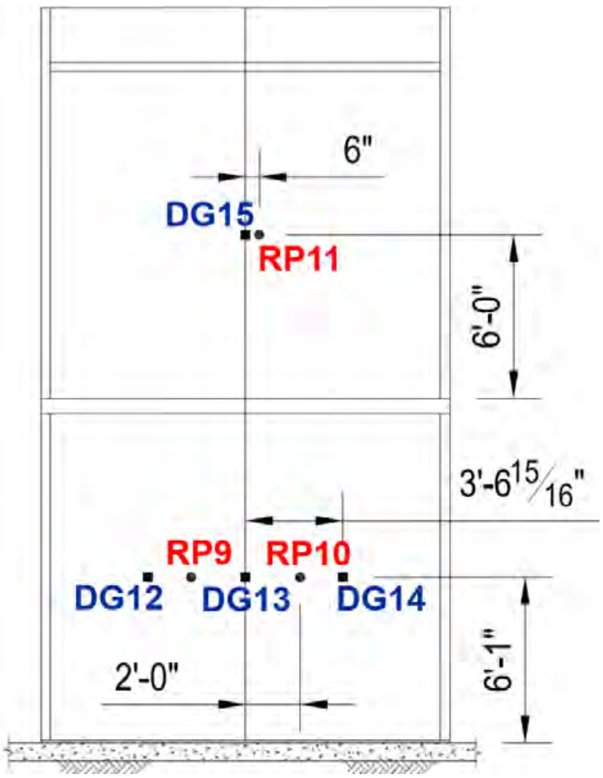
ID	Structure Grade	Measurement	Location	Range
RP1 – RP3	V1	Reflected Pressure	Flush w/ wall (Figure 3-13)	± 25 psi
RP4 – RP8	V1	Reflected Pressure	Flush w/ wall (Figure 3-13)	± 5 psi
RP9 – RP11	E1	Reflected Pressure	Flush w/ wall (Figure 3-14)	± 25 psi
RP12 – RP16	E1	Reflected Pressure	Flush w/ wall (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



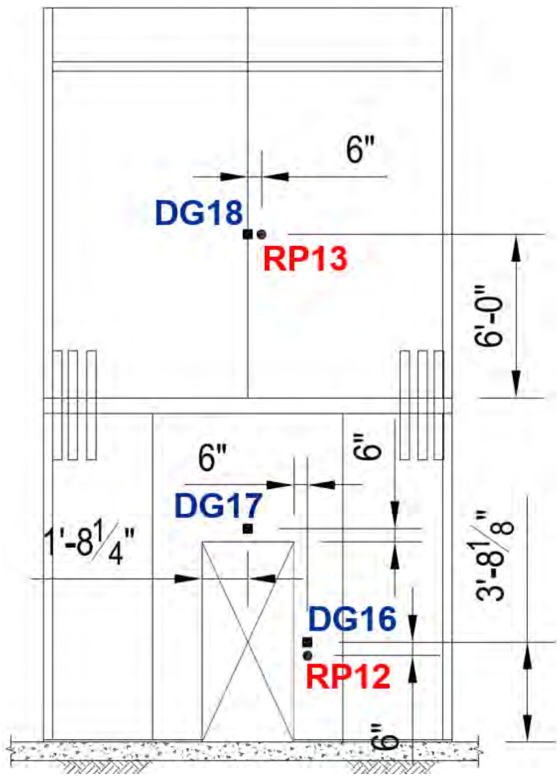
(a) Front Elevation. *(b) Window (Left) Elevation.*

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

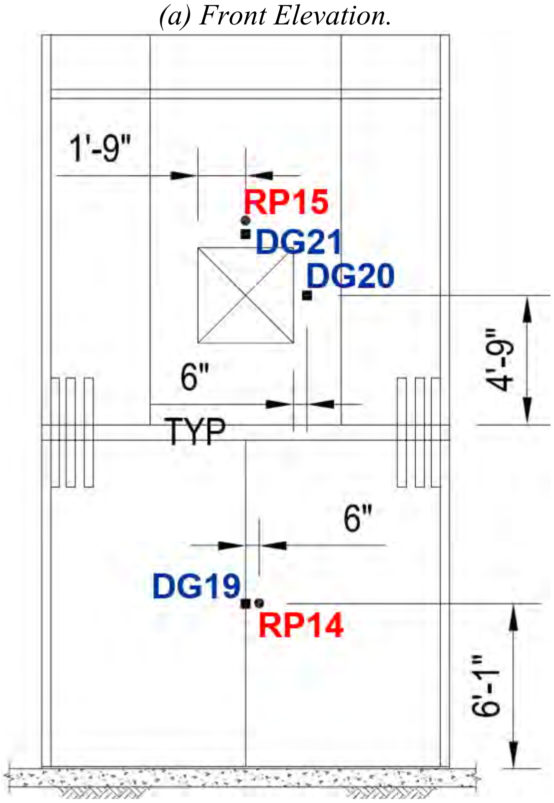
Figure 3-13. Grade V1 Structure Reflected Pressure Gage Key Plan.



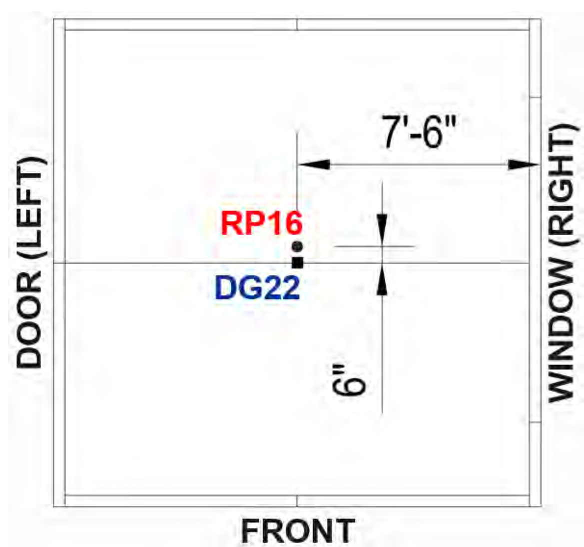
(a) Front Elevation.



(b) Door (Left) Elevation.

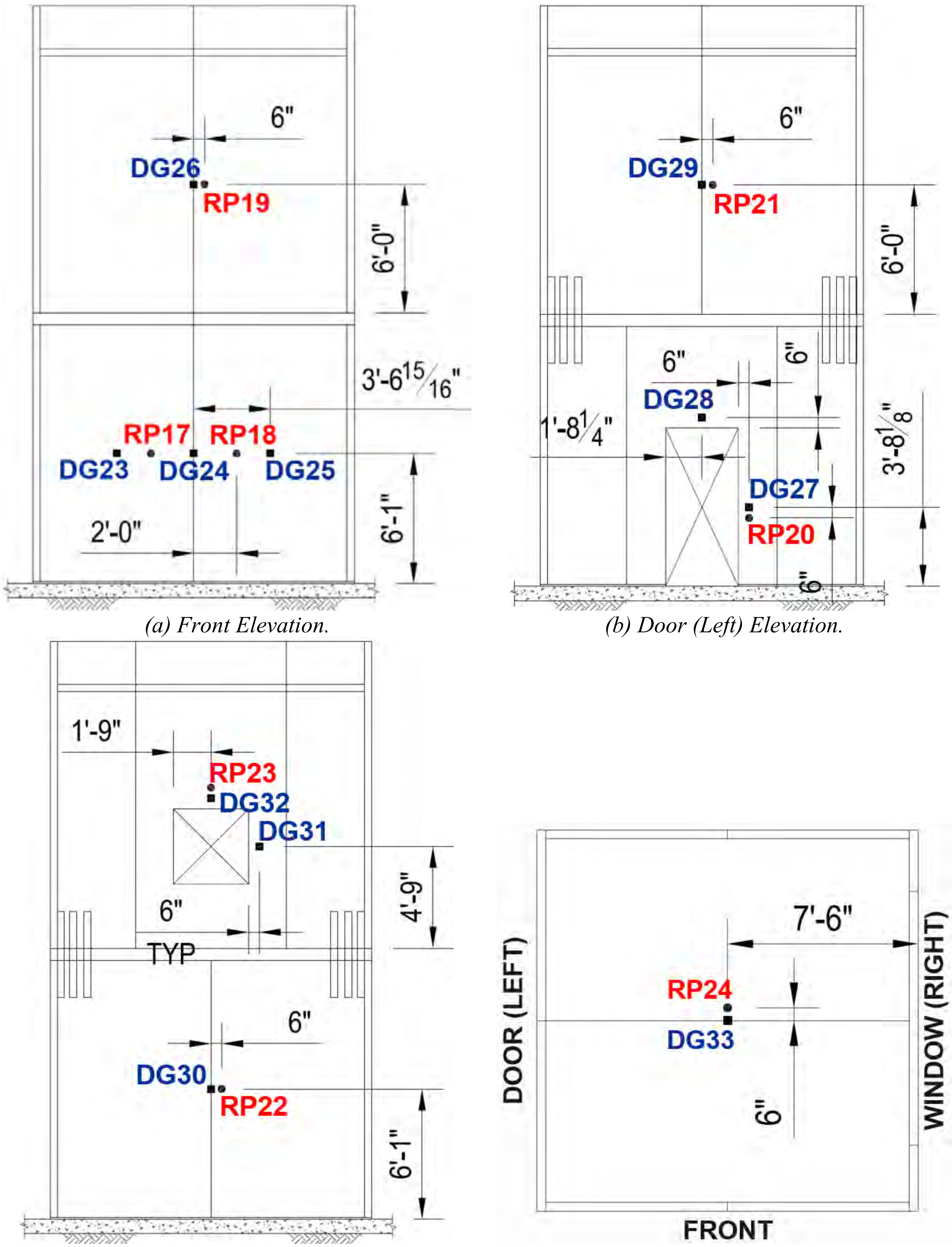


(c) Window (Right) Elevation.



(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

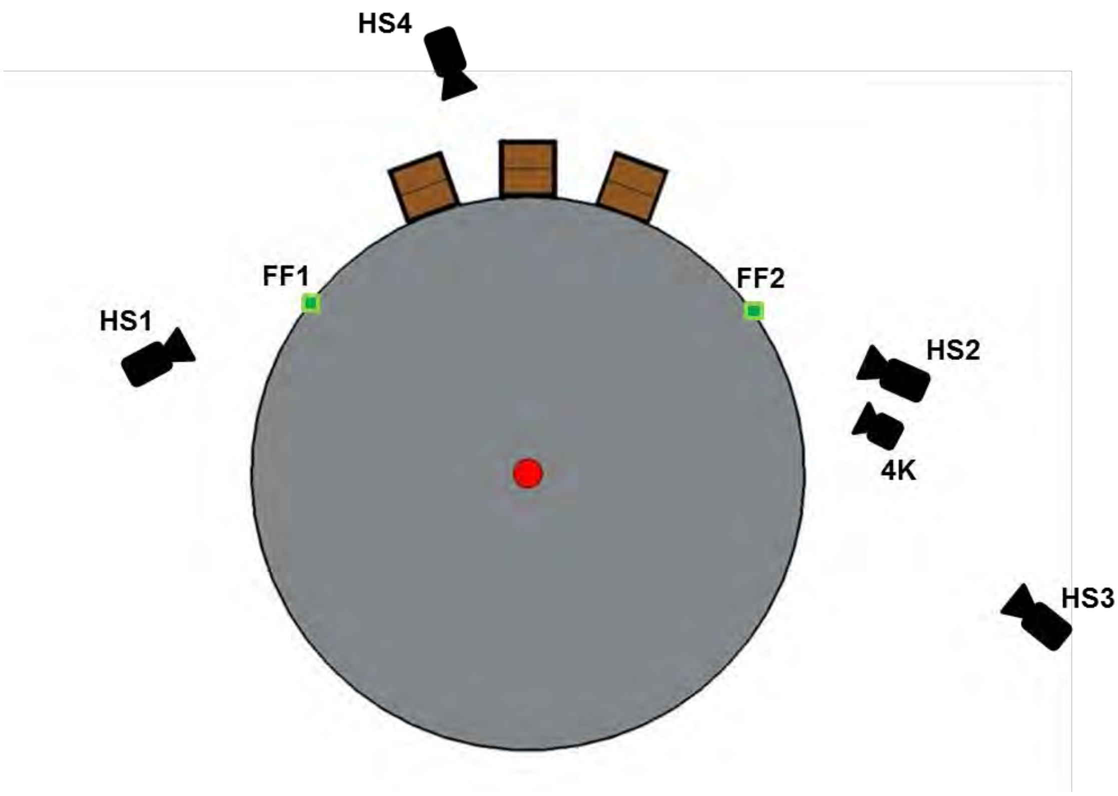


Figure 3-19. Video Camera and Free-Field Pressure Gage Key Plan.

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.

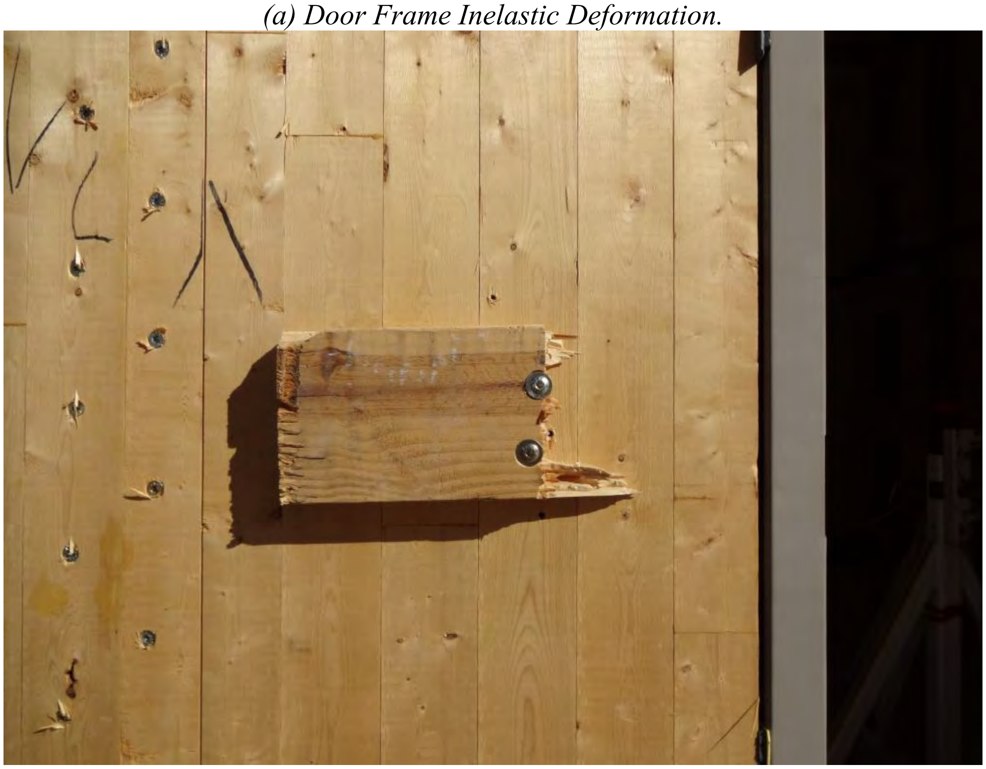
(b) Grade V4 Test Structure – Board Delamination.

(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.

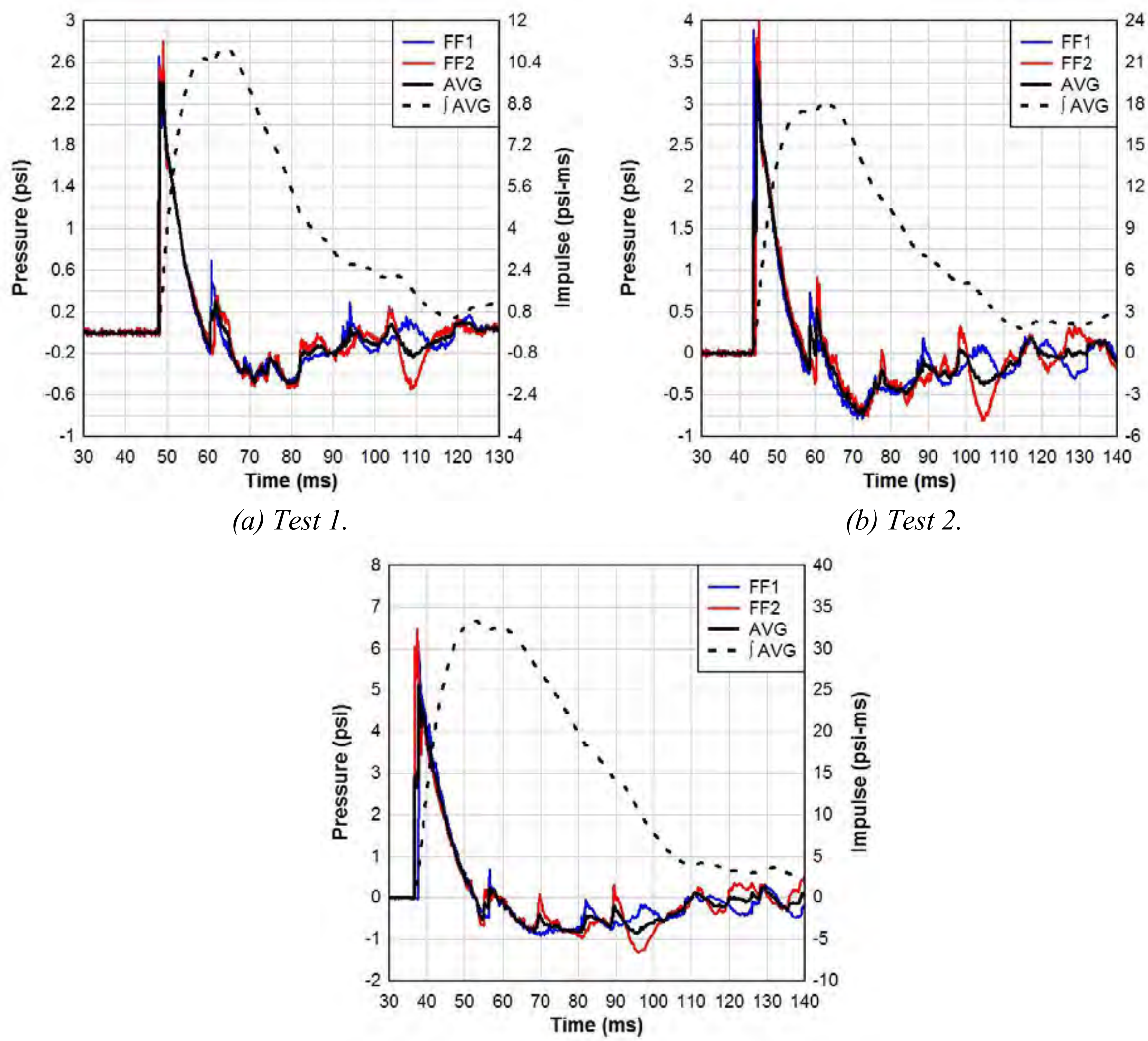
Figure 4-14. Test 3 Post-Test Photograph of Damage Near Door Frame.

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.



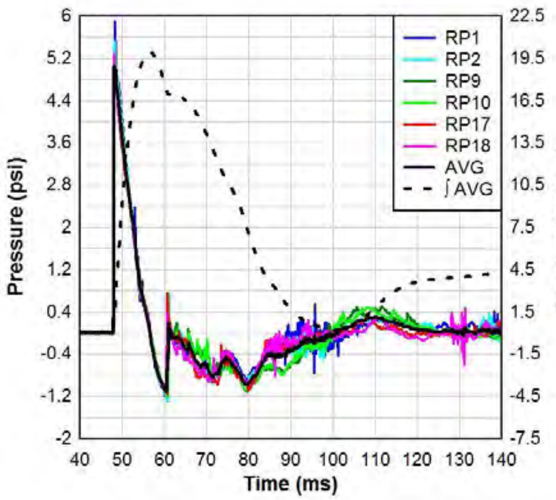
(a) Test 1.

(b) Test 2.

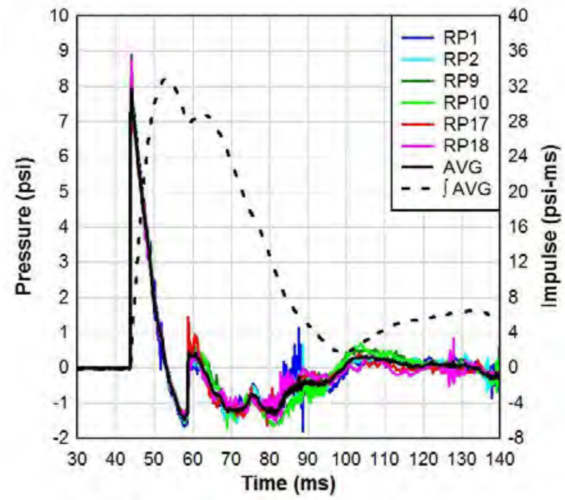
(c) Test 3.

Figure 4-15. Incident Overpressure Data.

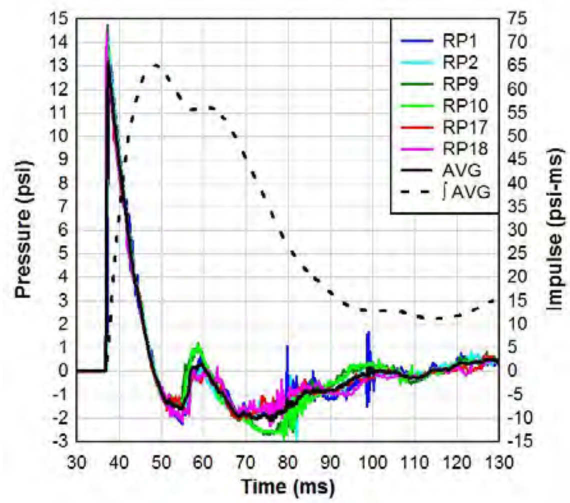
Similarly, Figure 4-16 plots the reflected pressure data recorded at the first-floor front panels (i.e., by gages RP1, RP2, RP9, RP10, RP17, and RP18) and the average of these six gages for each of the three tests. Plots of the remaining pressure histories are included in Appendix D.



(a) Test 1.



(b) Test 2.



(c) Test 3.

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

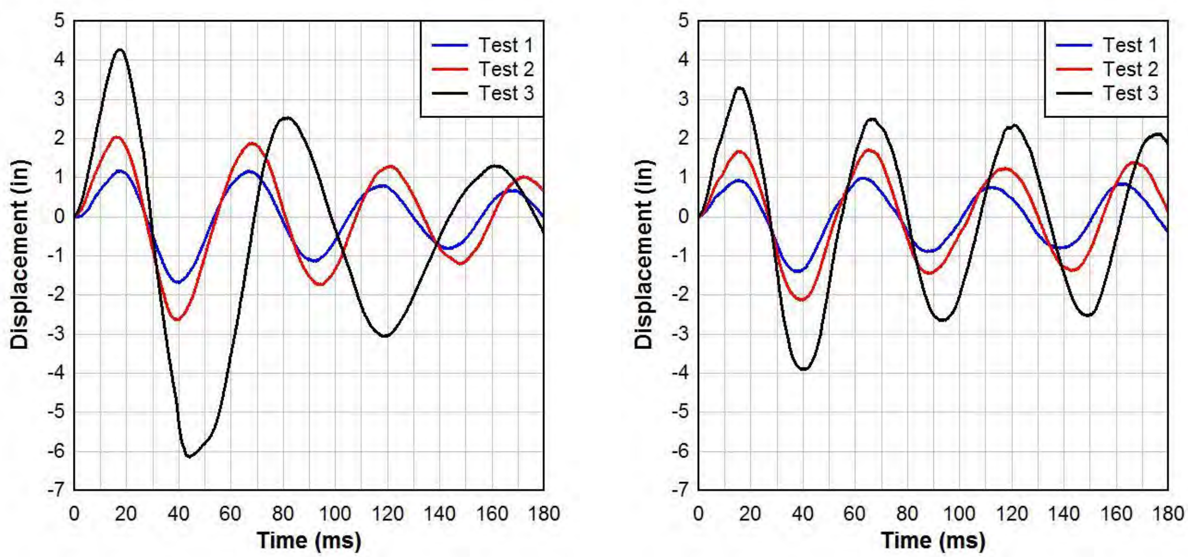
Table 5-1 provides a summary of the incident and peak reflected pressure positive phase data for all three shots. The values shown in Table 5-1 are generated based on the average curves shown in Figure 4-15 and Figure 4-16.

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

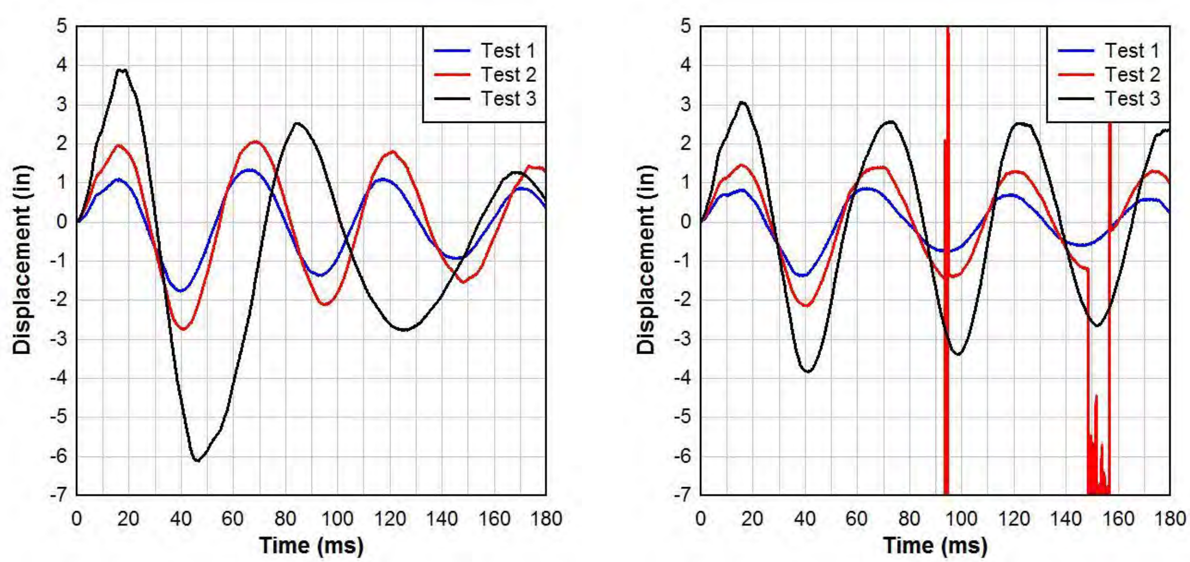
4.2.2 Displacement

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.



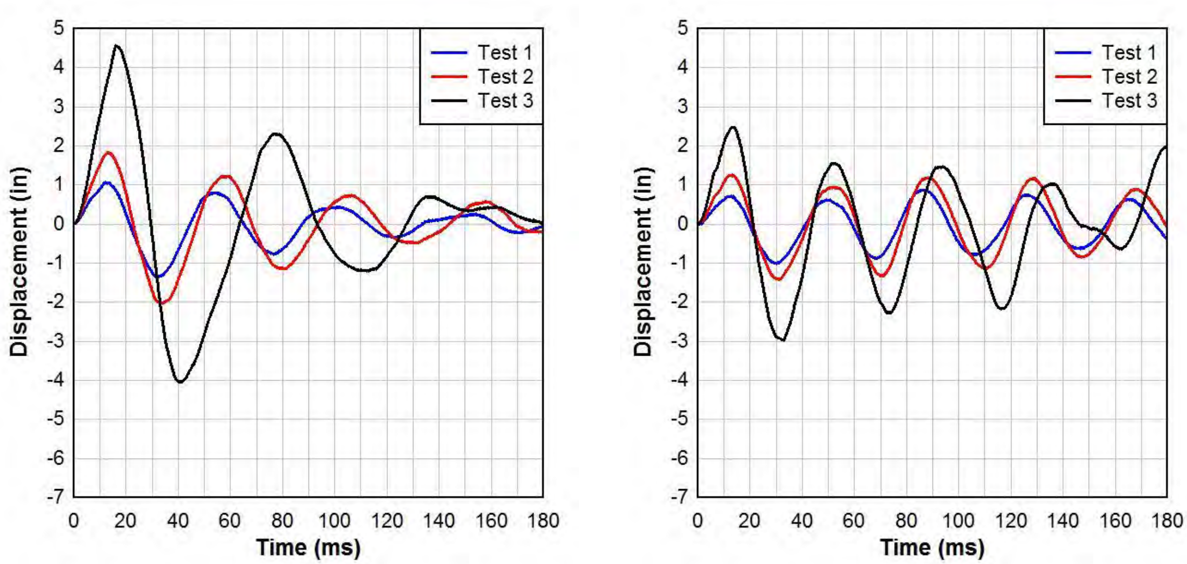
(a) 1st Floor Front Panel (DG2). *(b) 2nd Floor Front Panel (DG4).*

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		E1		V4	
		Inbound [in]	*Rebound [in]*	*Inbound [in]*	*Rebound [in]*	*Inbound [in]*	*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

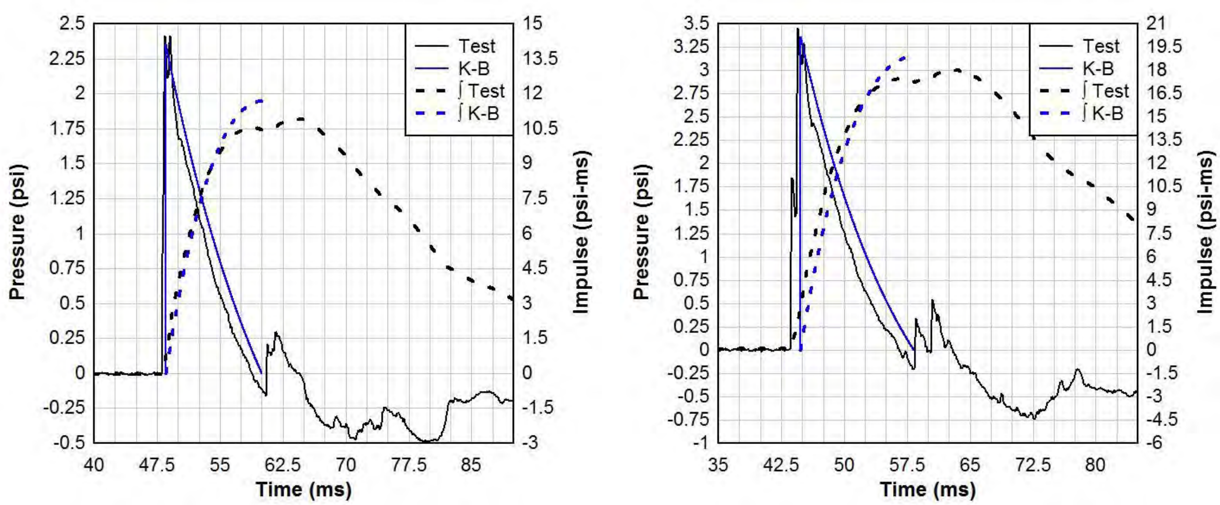
CHAPTER 5

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.

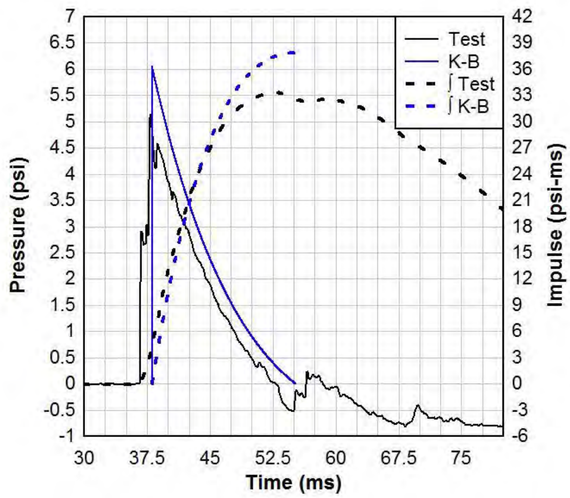
5.1 AIRBLAST LOADING

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.*

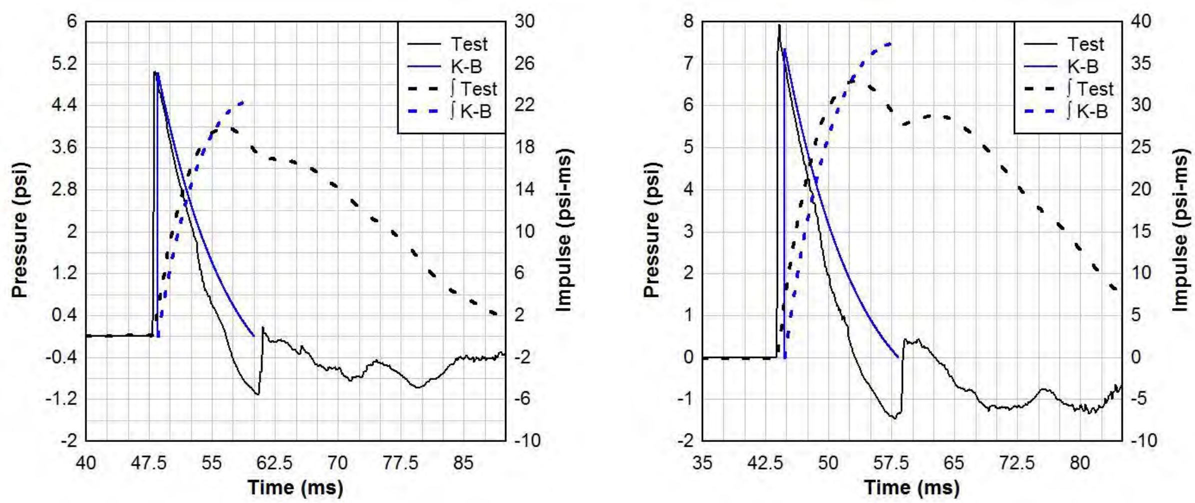
Figure 5-1. Incident Overpressure Data Comparisons.



(c) Test 3.

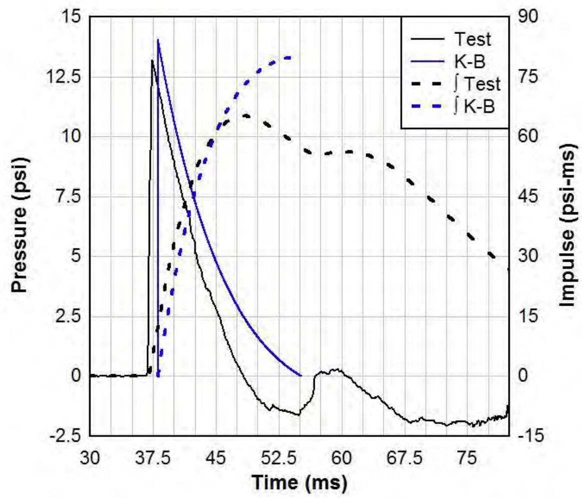
Figure 5-1. Incident Overpressure Data Comparisons. (Cont'd)

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.

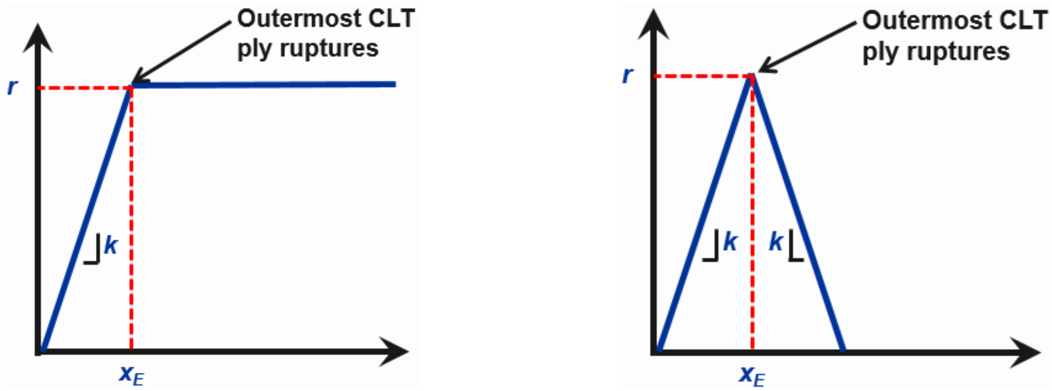
Test	Time of Arrival [ms]		Incident Overpressure [psi]		Incident Impulse [psi-ms]		Peak Reflected Pressure [psi]		Peak Reflected Impulse [psi-ms]	
	Test[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]	*Test*[1]	*K-B*[2]
1	48.1	48.6	2.41	2.35	10.9	11.7	5.05	5.03	19.9	22.5
2	43.6	44.8	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.

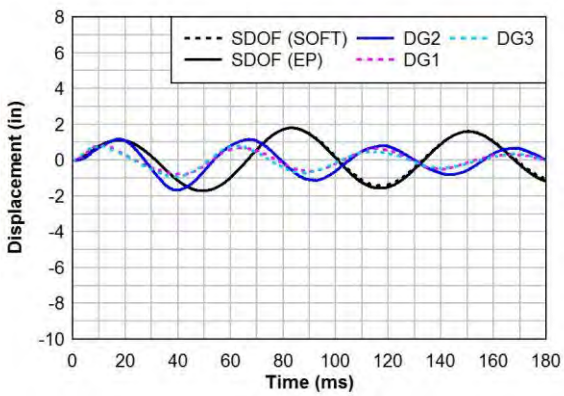
- Viscous damping was applied. The fraction of critical damping was assumed to be 2-percent.

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

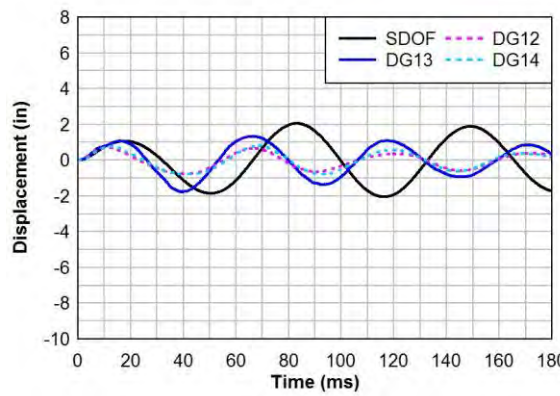
Table 5-2. SDOF Dynamic Analysis Parameters.

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

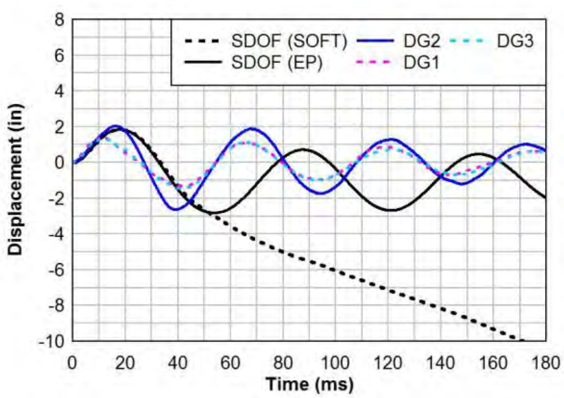
Figure 5-4 and Figure 5-5 show comparisons of how the SDOF dynamic analysis results obtained using these resistance functions compared with the test data for the front panels for a CLT made up of visually graded lamella (i.e., Grade V1) and CLT made up of MSR lamella (i.e., Grade E1).



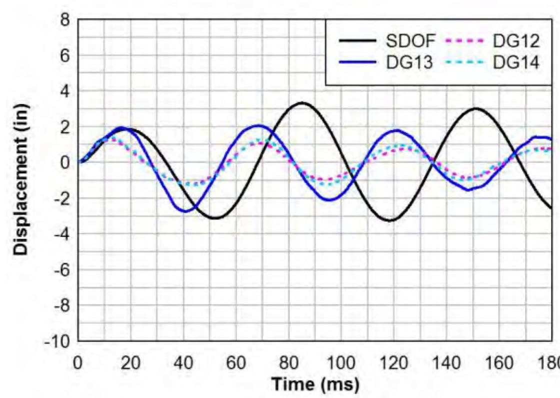
(a) Test 1, Grade V1.



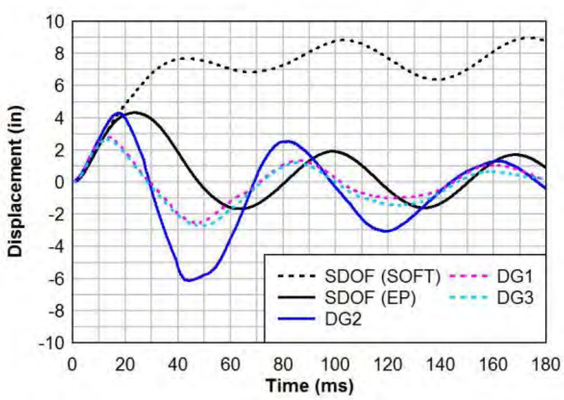
(b) Test 1, Grade E1.



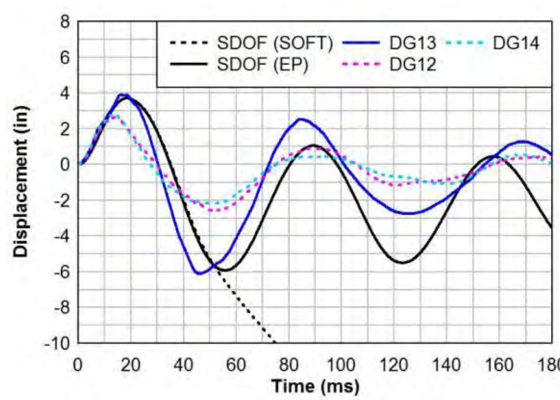
(c) Test 2, Grade V1.



(d) Test 2, Grade E1.

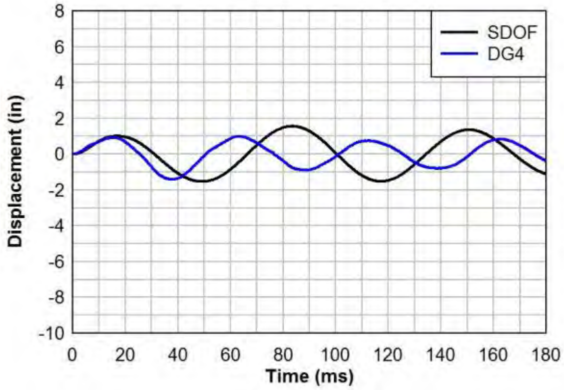


(e) Test 3, Grade V1.

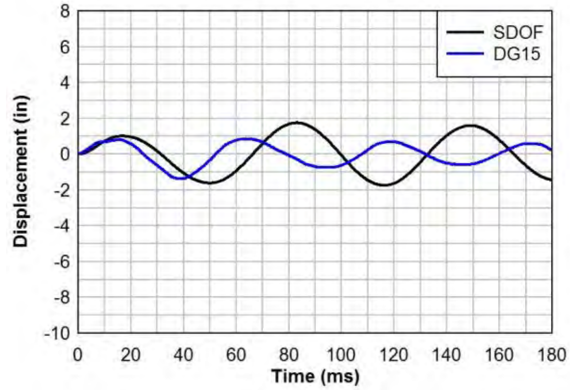


(f) Test 3, Grade E1.

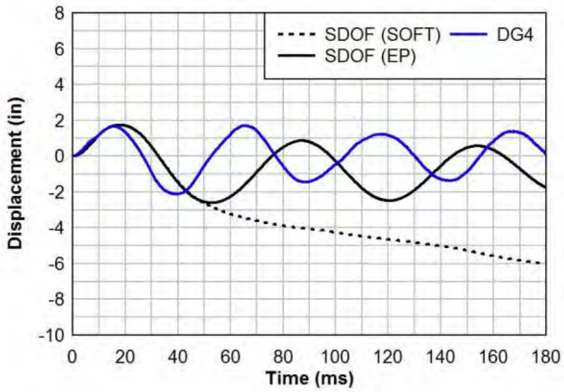
Figure 5-4. First-Floor Front Panel Displacement Comparisons.



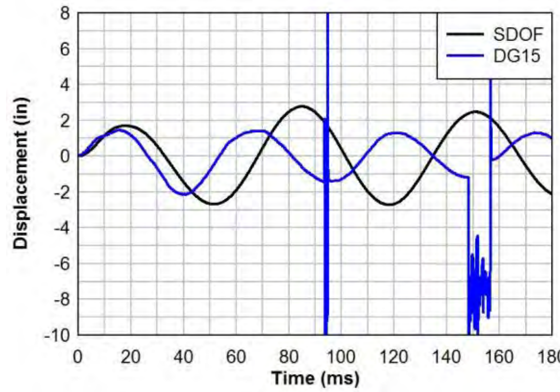
(a) Test 1, Grade V1.



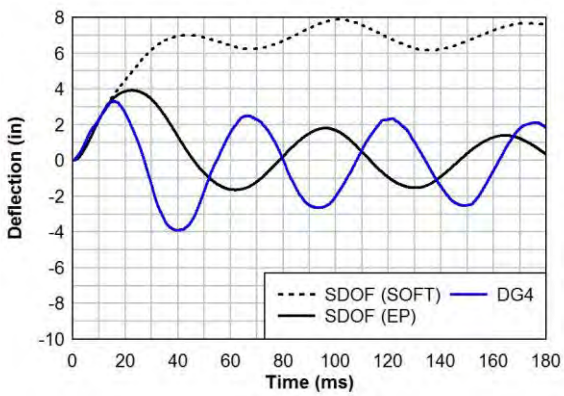
(b) Test 1, Grade E1.



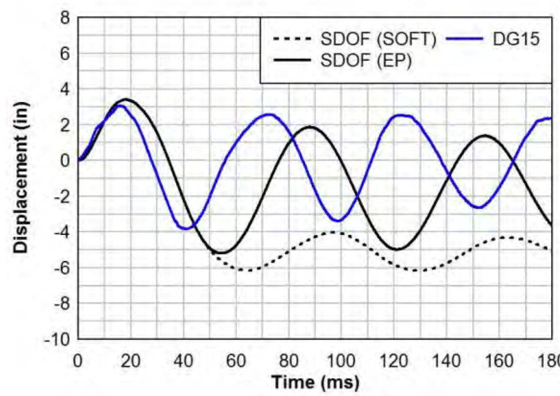
(c) Test 2, Grade V1.



(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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SHEET TITLE

GENERAL NOTES

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CJ	MW	LT	09/19/2016

JOB NO. 2271_2015

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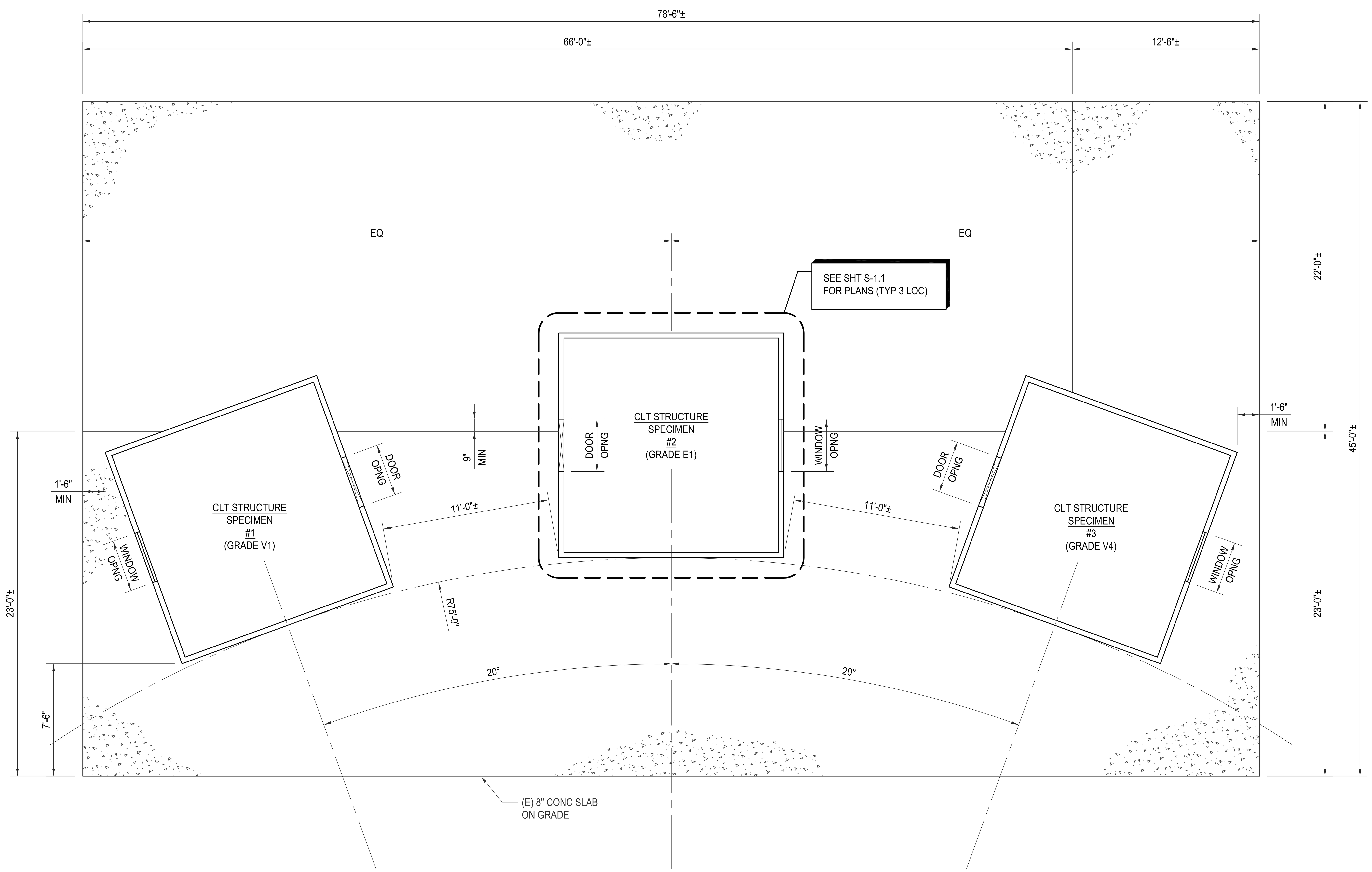

A
S-1.0

SITE PLAN

SCALE: 1/4"=1'-0"

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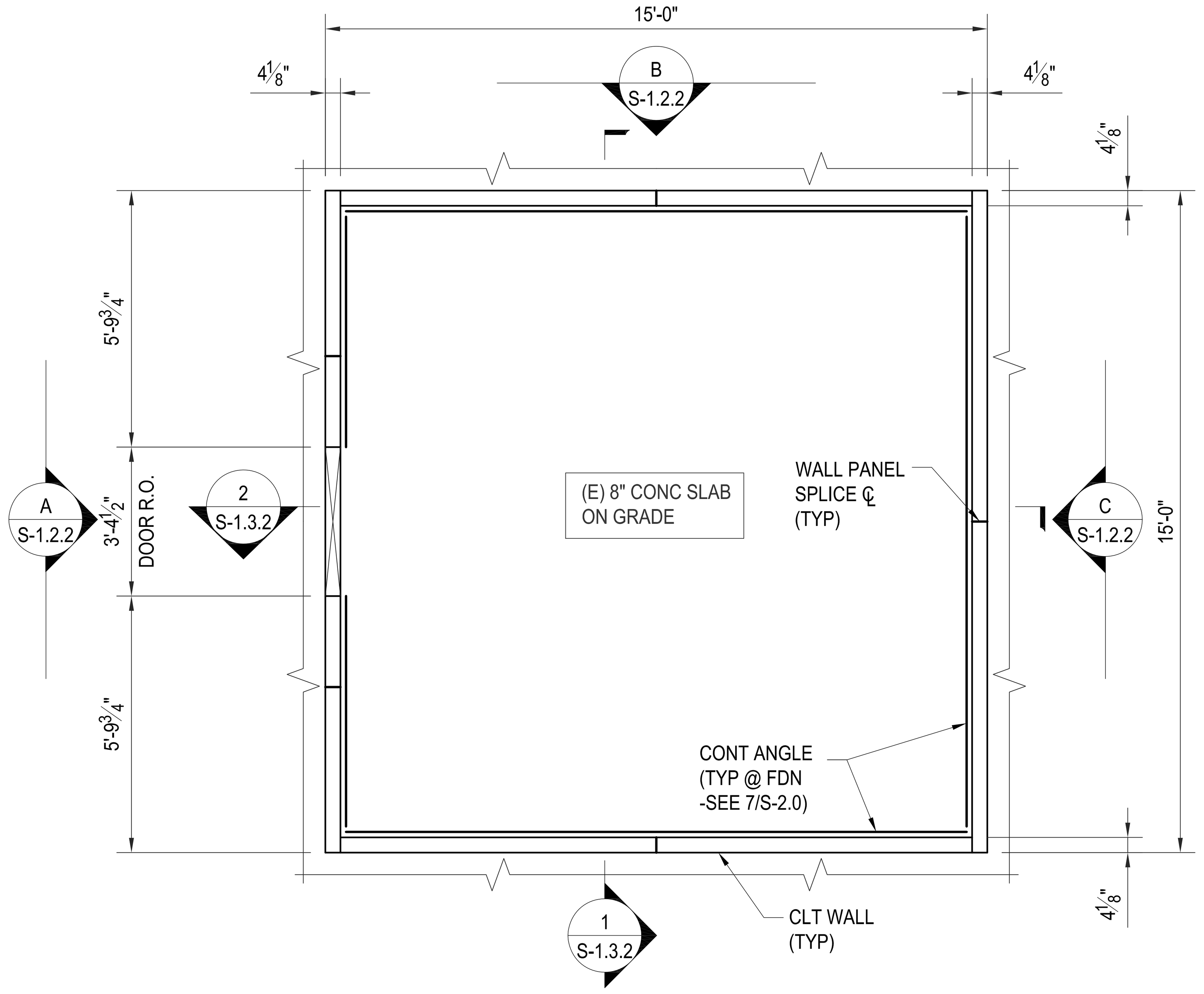

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GROUND FLOOR PLAN

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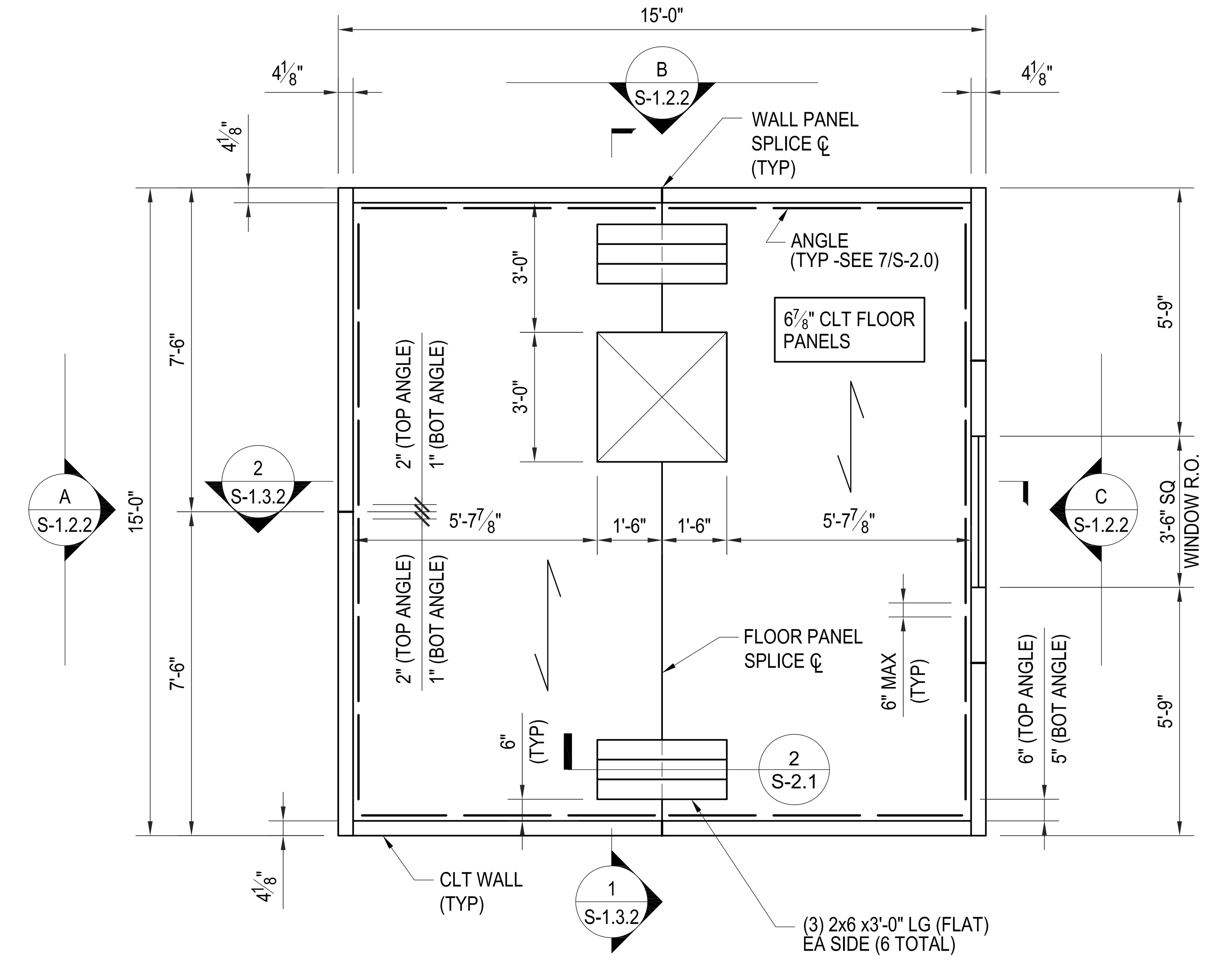

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SECOND FLOOR PLAN

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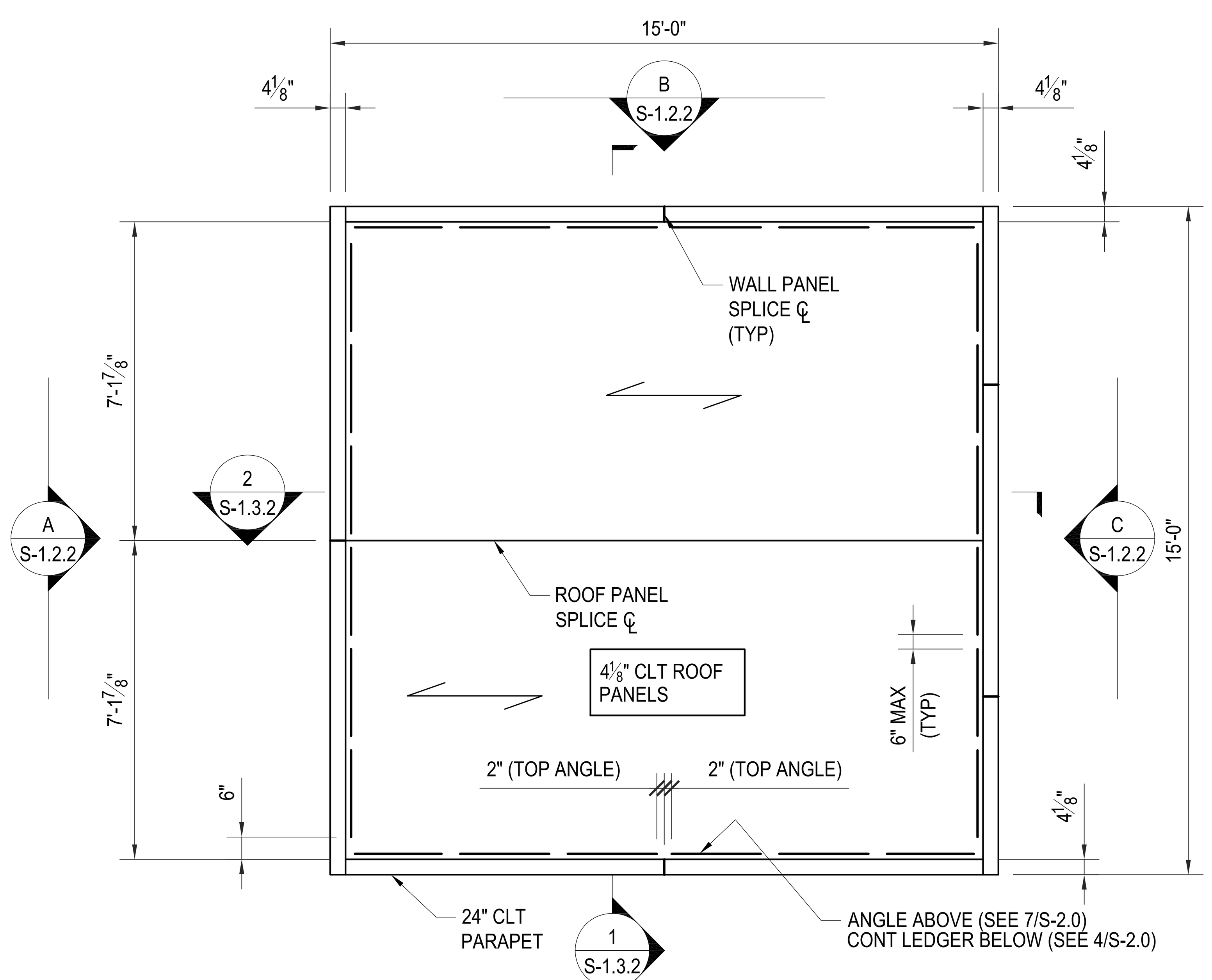

C
S-1.1

ROOF PLAN

SCALE: 3/8"=1'-0"

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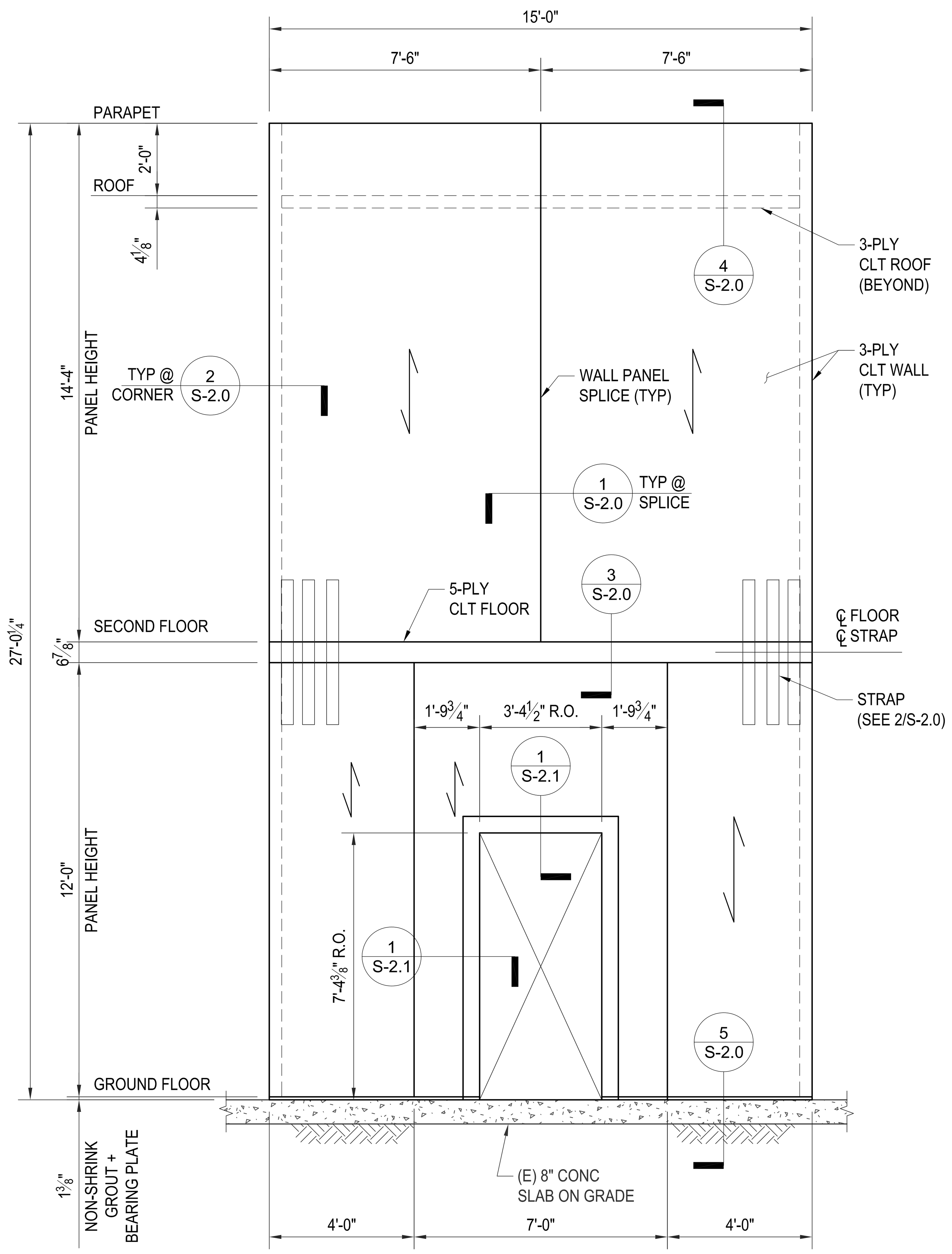

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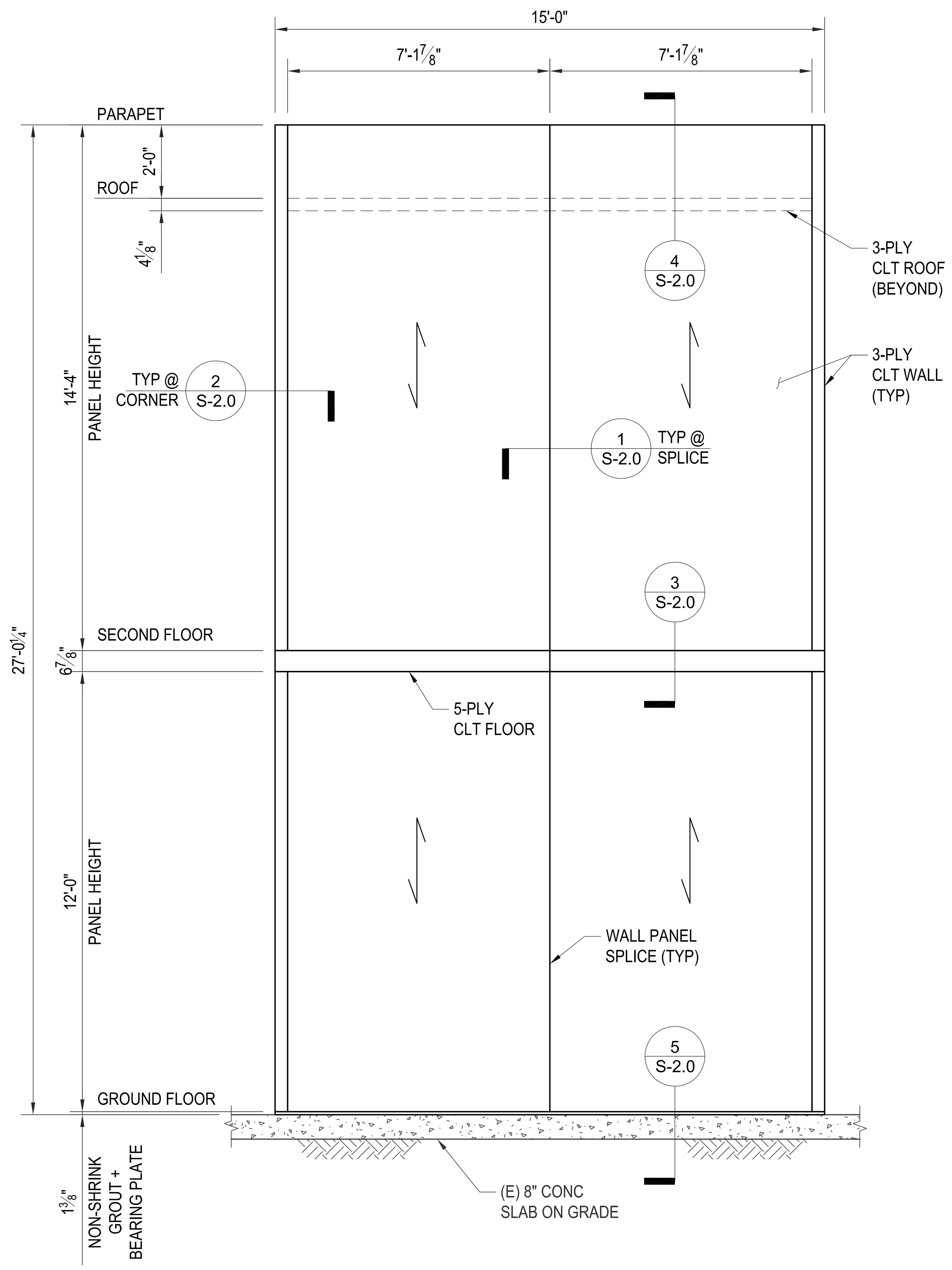

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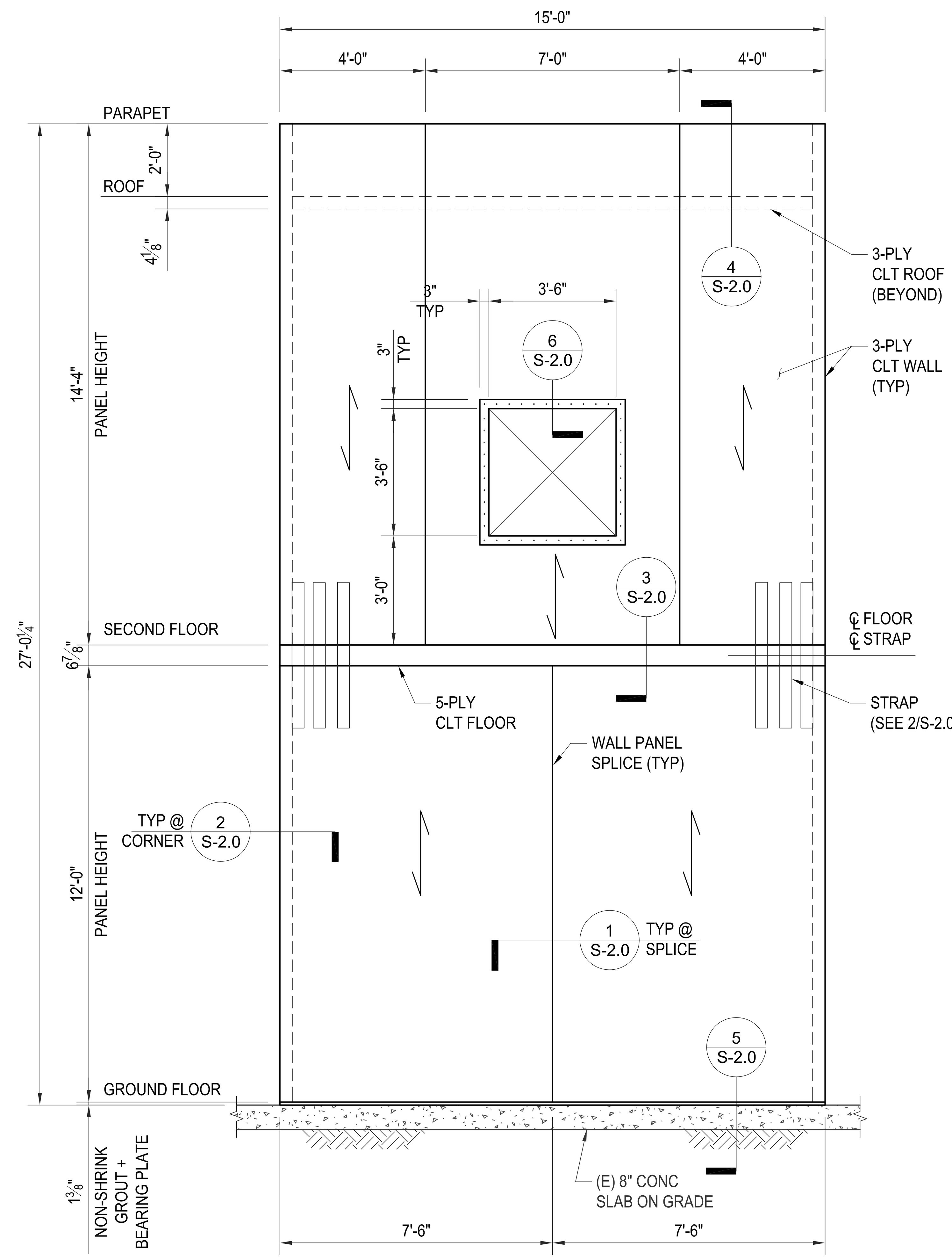

C S-1.2.1 SIDE ELEVATION (WITH WINDOW)
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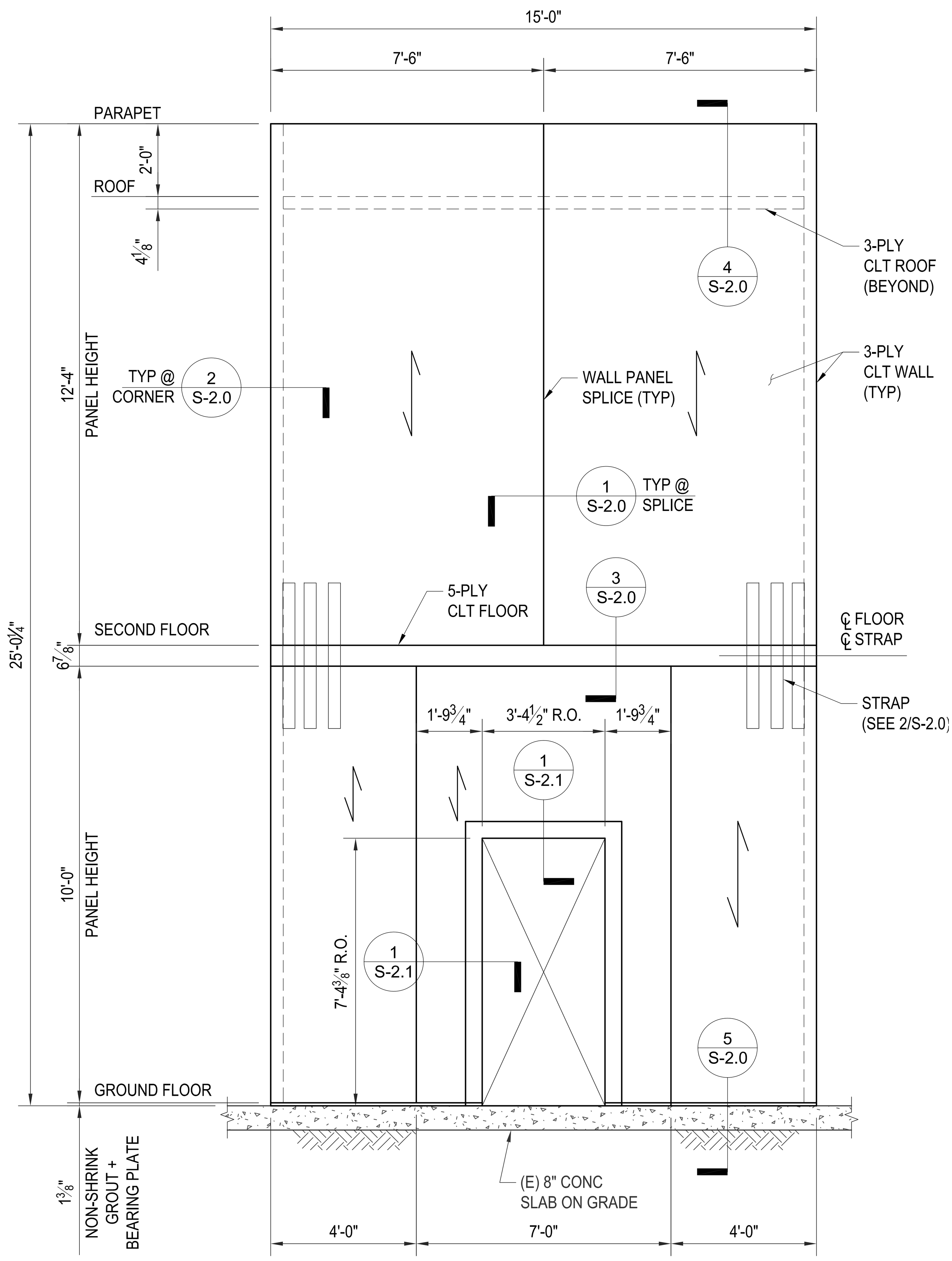

A S-1.2.2 SIDE ELEVATION (WITH DOOR)
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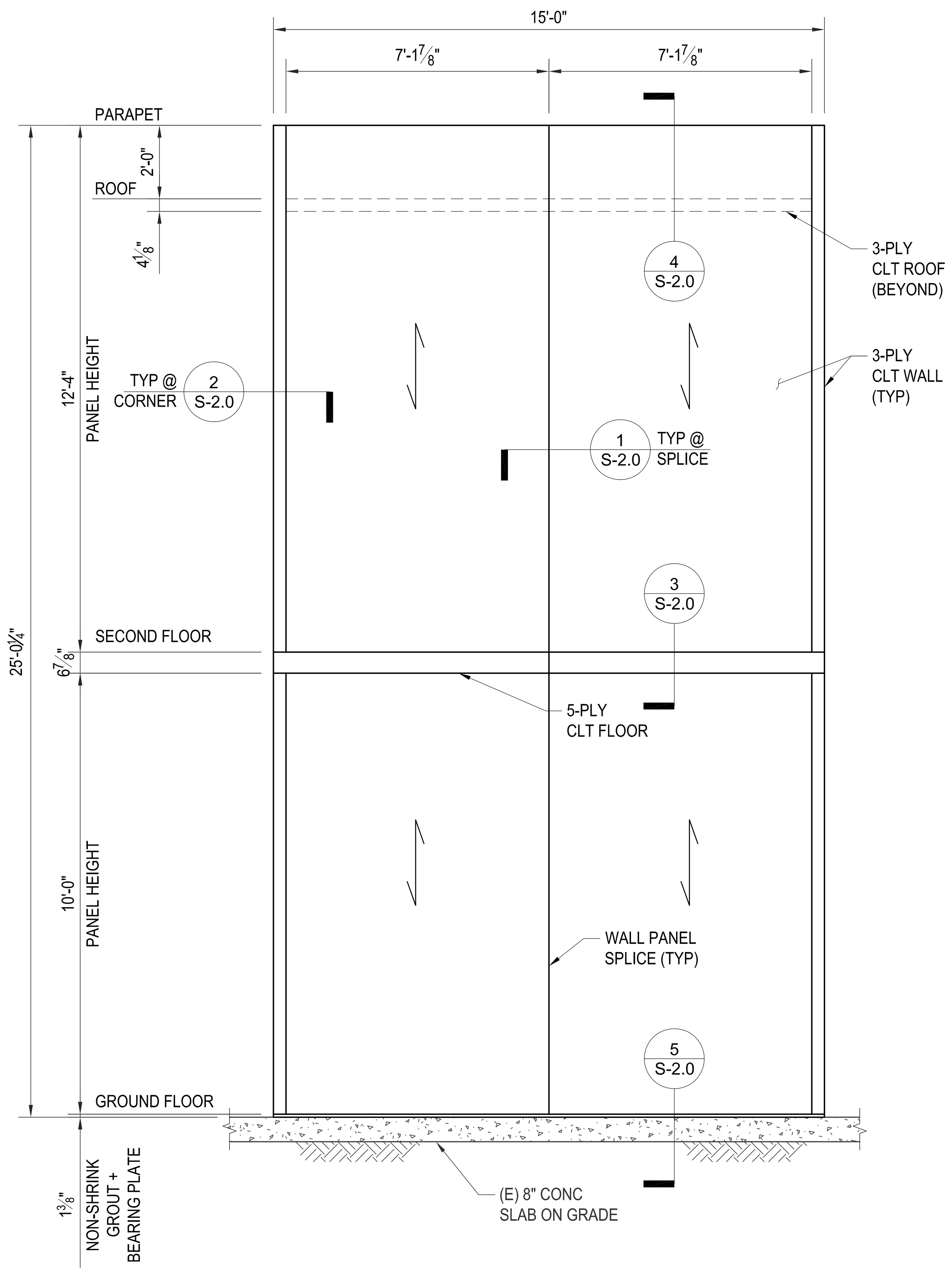

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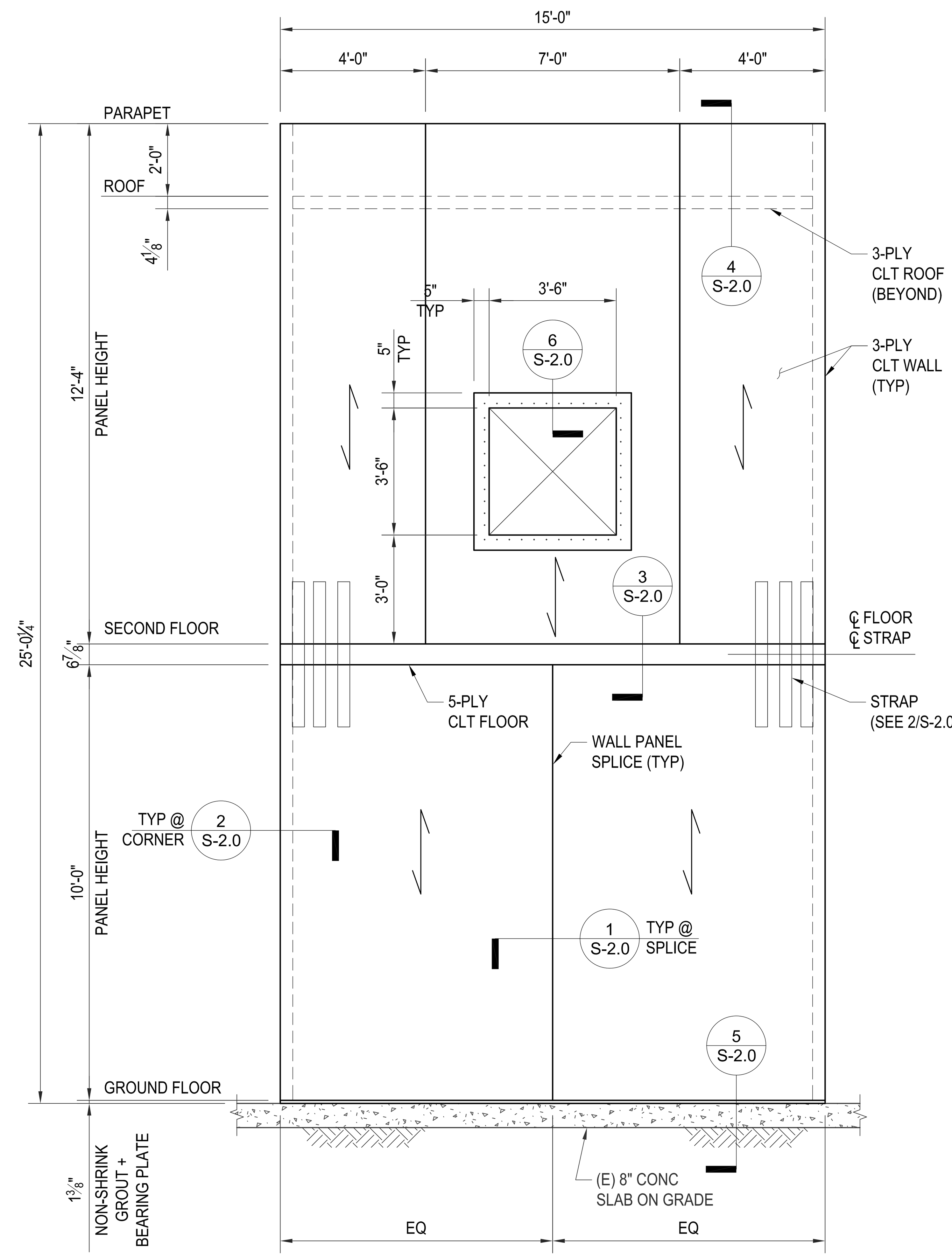

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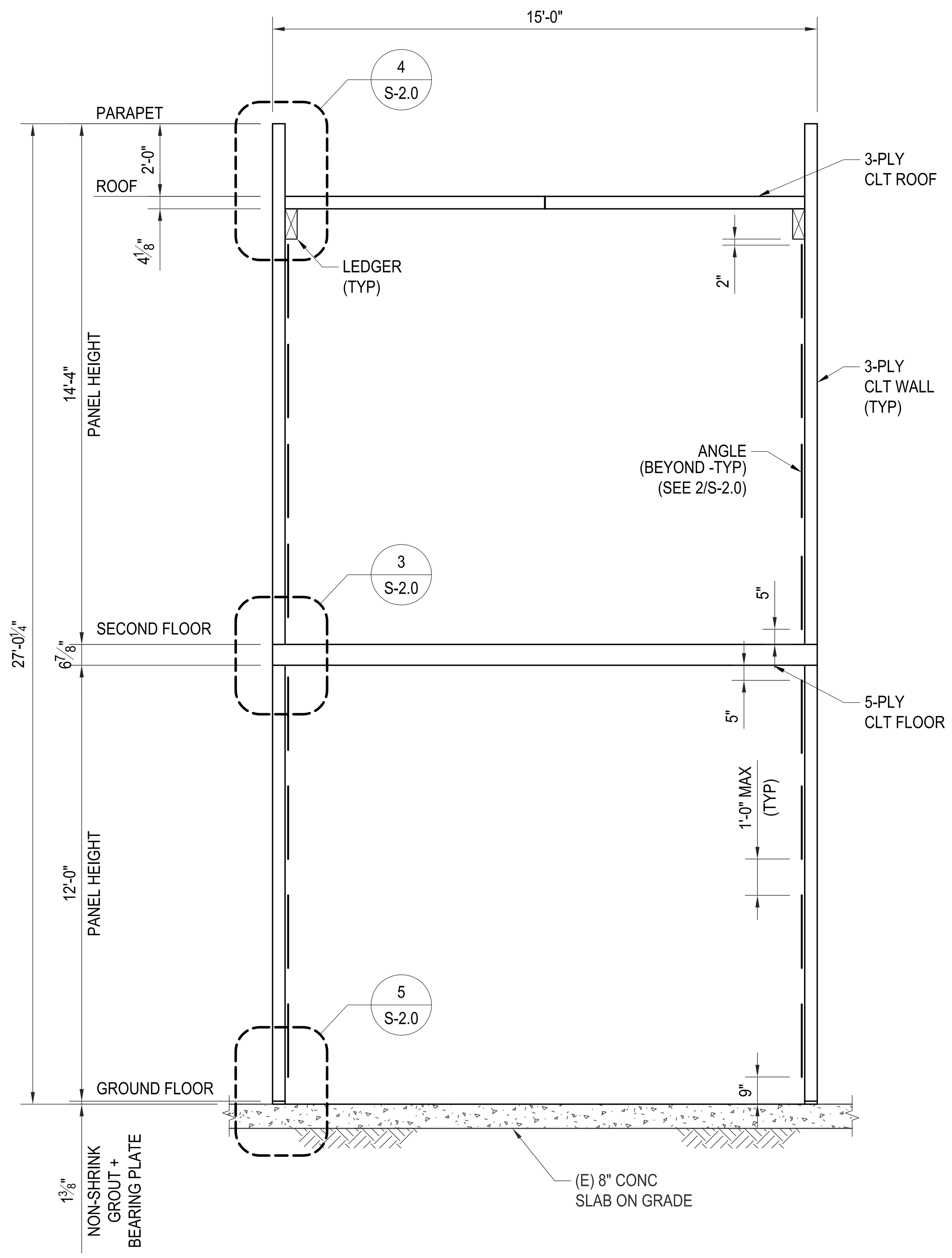

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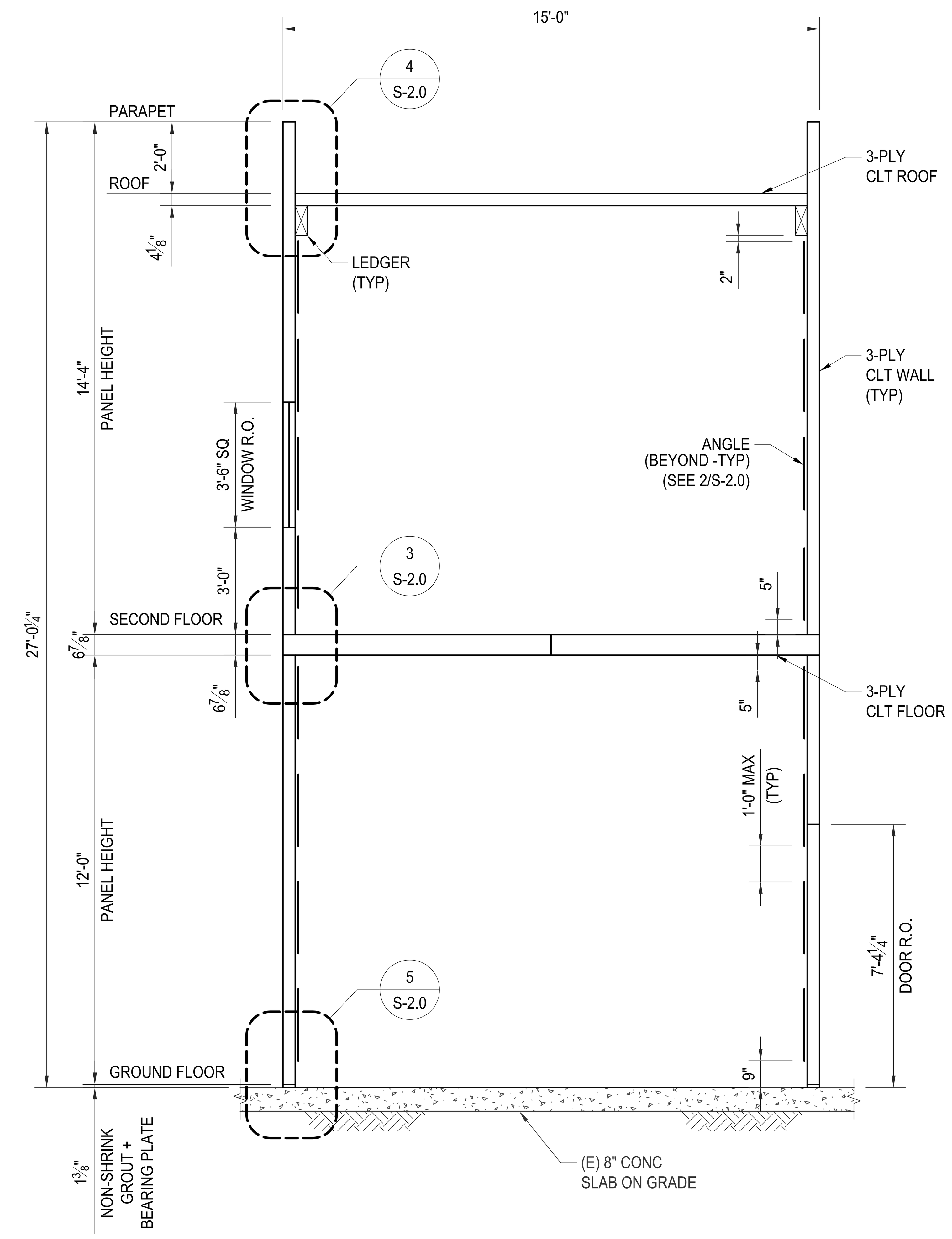

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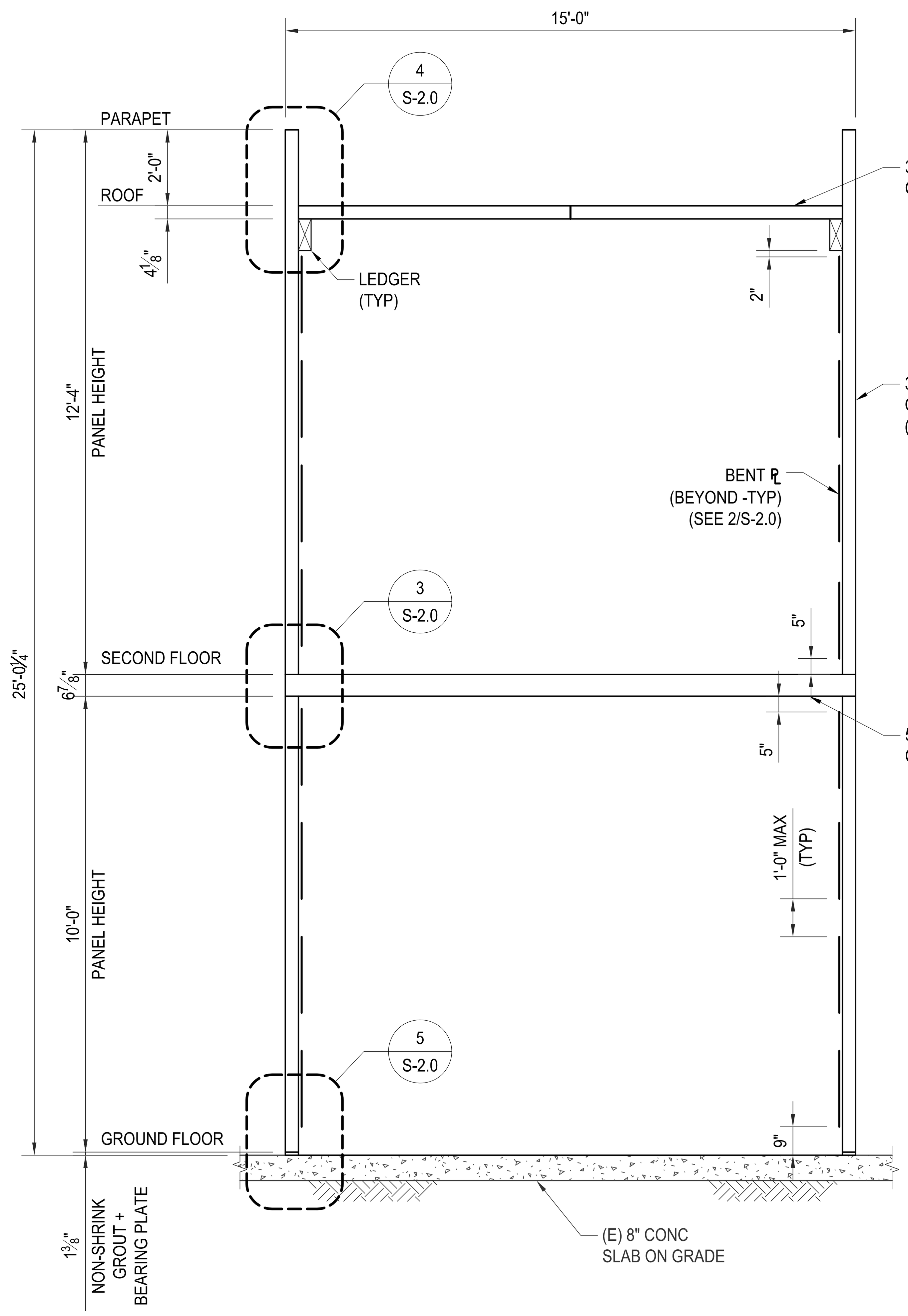

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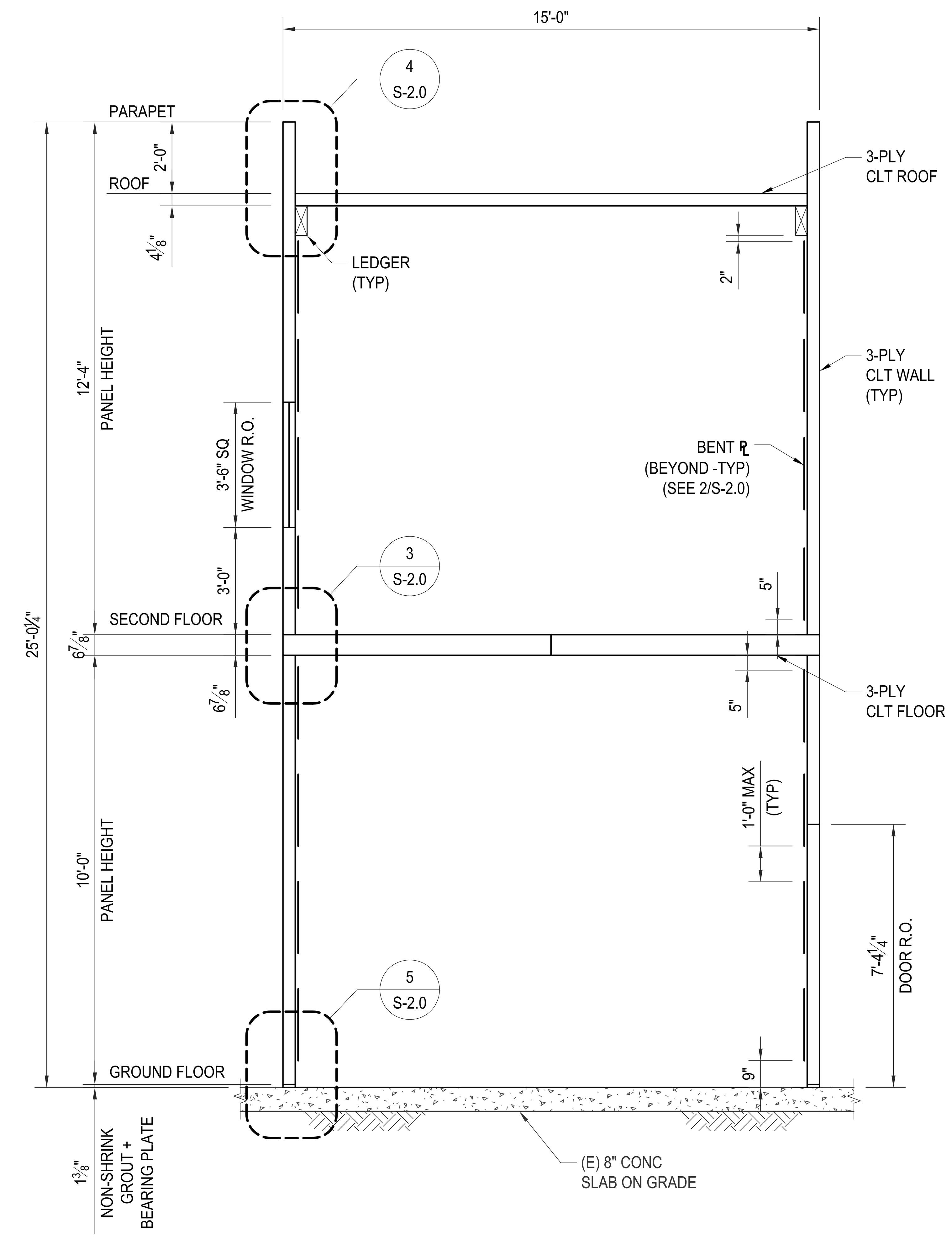

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SHEET NUMBER

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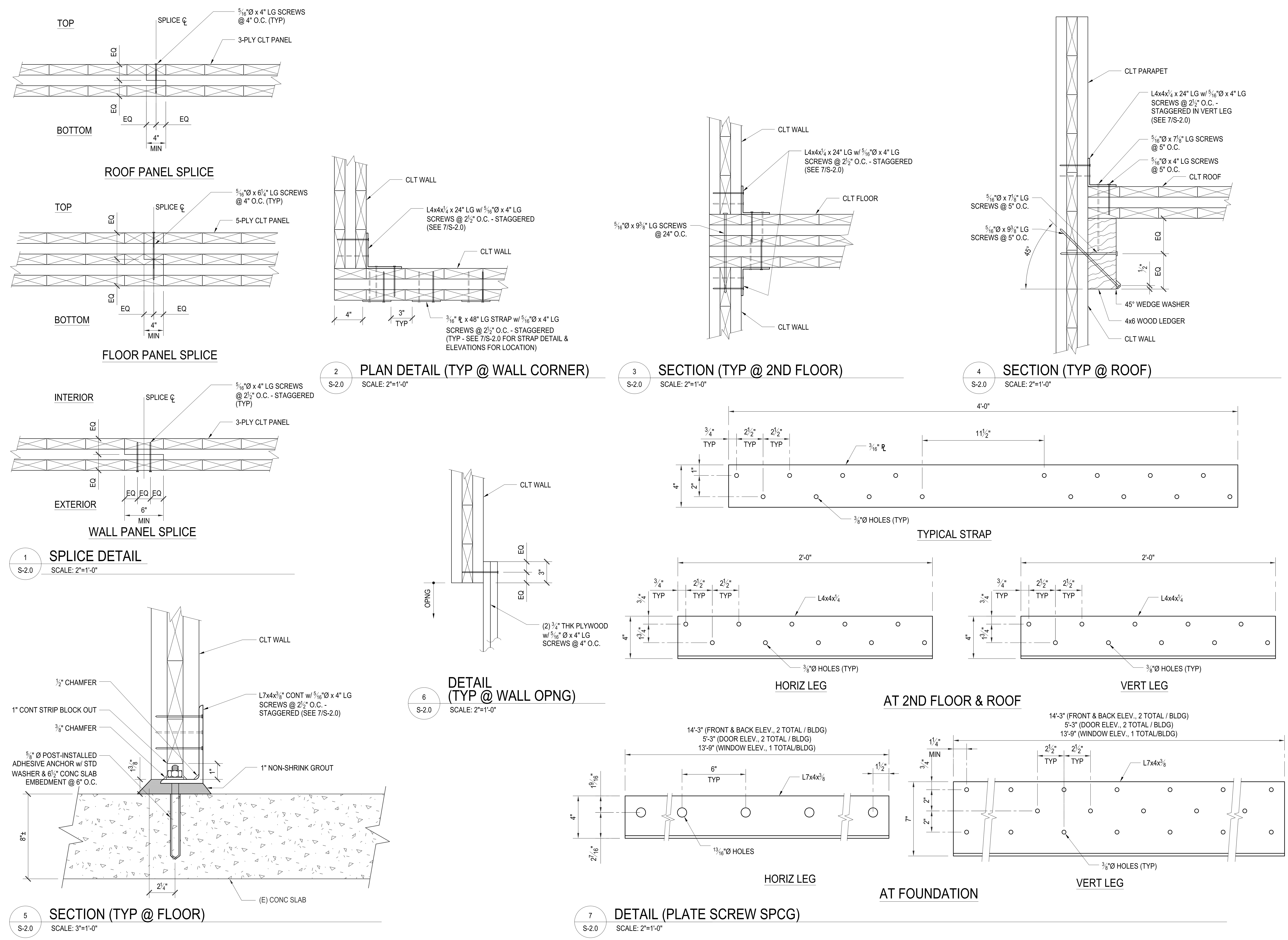
May 23, 2017



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FILE NAME: S200_2271.DWG

SHEET NUMBER

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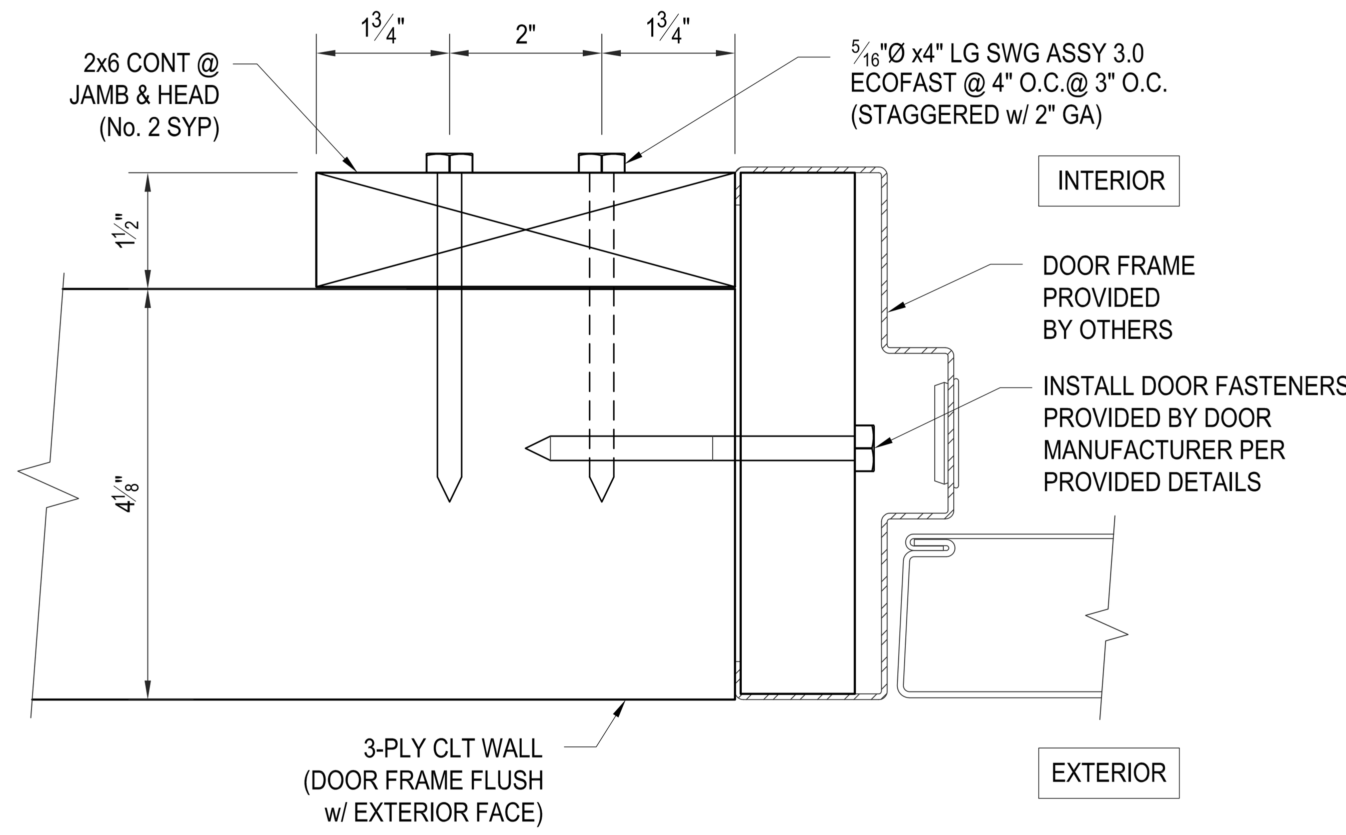

1 S-2.1 **PLAN DETAIL @ DOOR JAMB**
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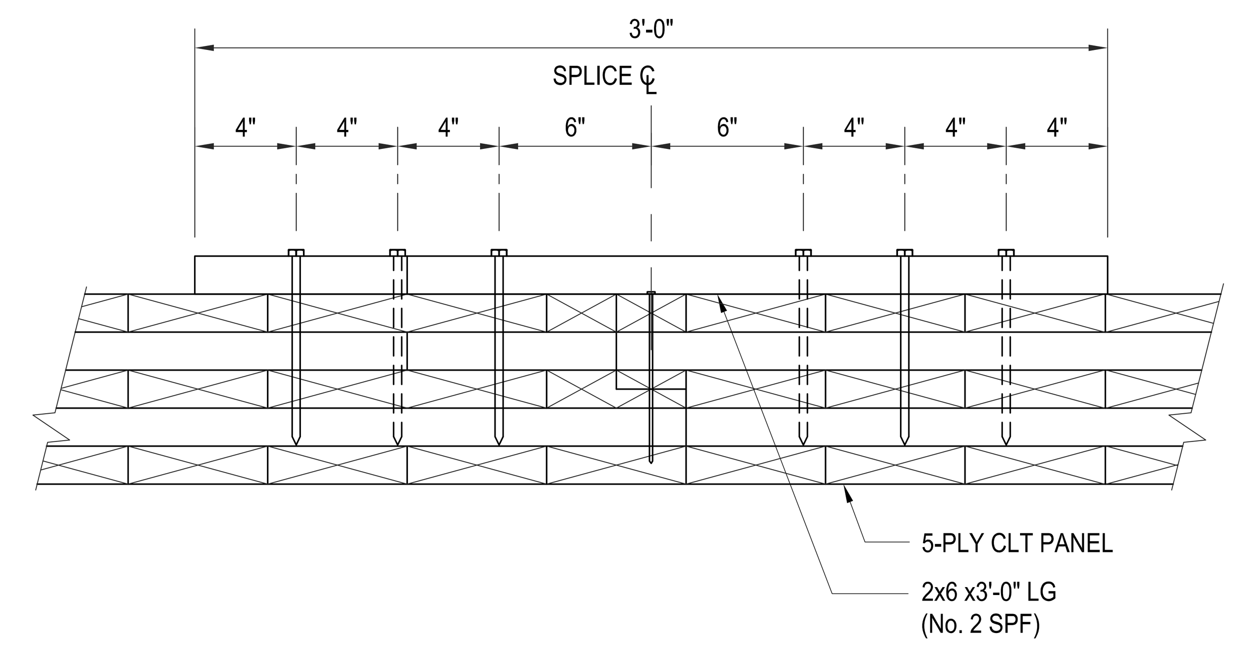

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Revisions

PROJECT

MASSIVE TIMBER LIVE BLAST TEST SPECIMEN

SHEET TITLE

DETAILS

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

JOB NO.	FILE NAME:
2271_2015	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:

AMERICAN DIRECT
11000 LAKEVIEW AVENUE,
LENEXA,, KS 66219
USA

Ship To:

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

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

AMBICO
SPECIALIZED STEEL DOORS & FRAMES
SPECIALIZED WOOD DOORS

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

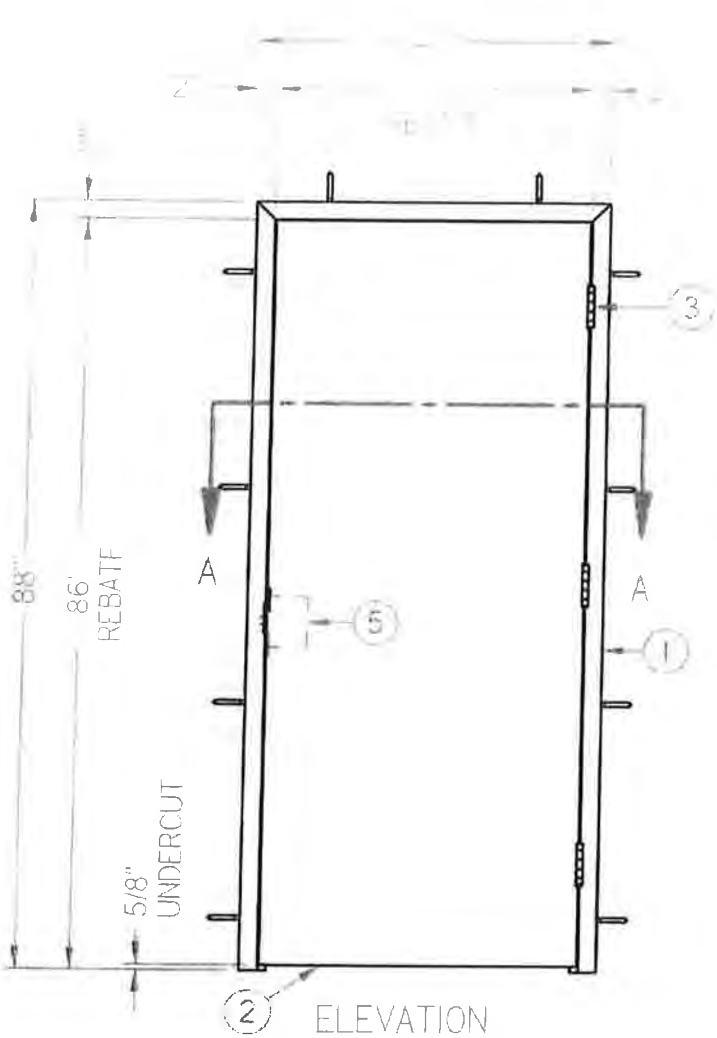
HARDWARE

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

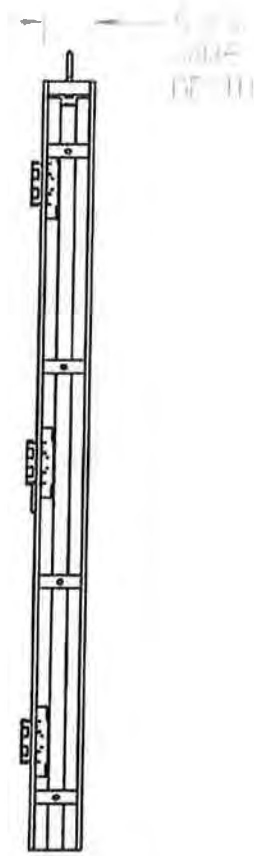
NOTES
ALL HARDWARE SUPPLIED AND INSTALLED BY OTHERS.

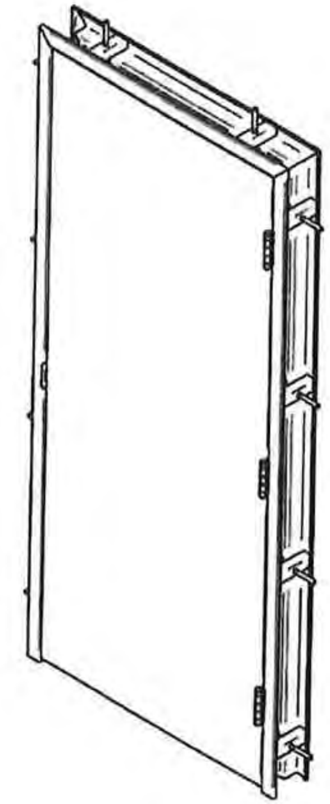
2/6

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.



ELEVATION





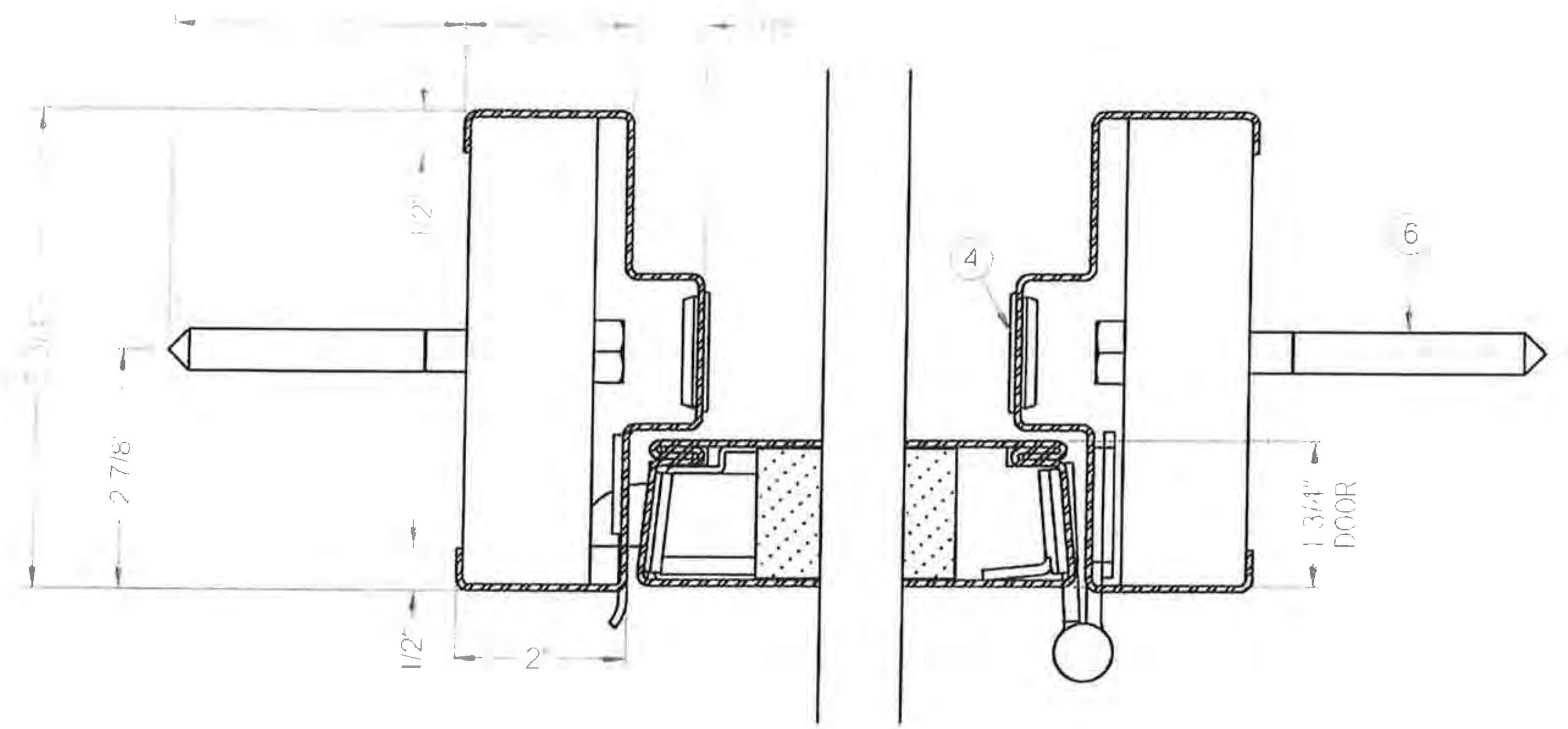
ISOMETRIC VIEW

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	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

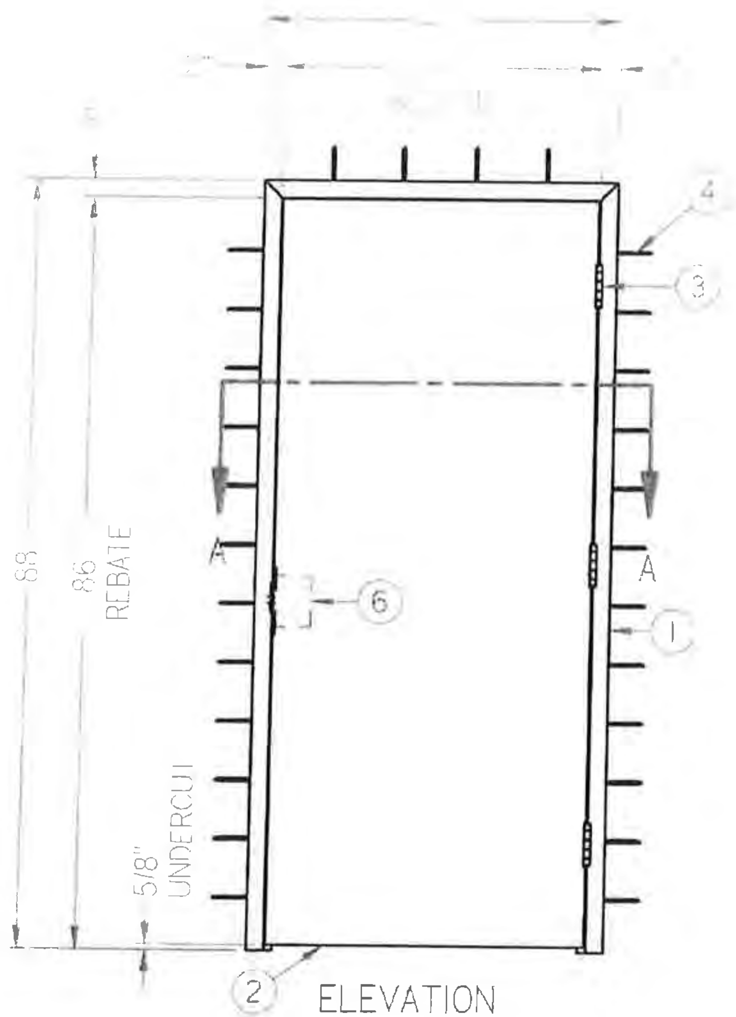
DATE:	03/02/2015	ORDER NO	
DRAWN BY	tberry	CUSTOMER NO:	A18898
REVISION	0	SHEET	3 OF 6



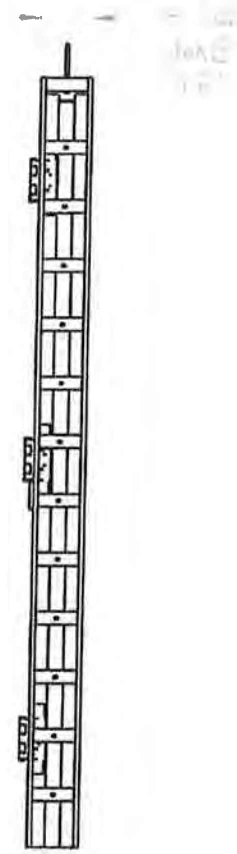
SECTION A-A

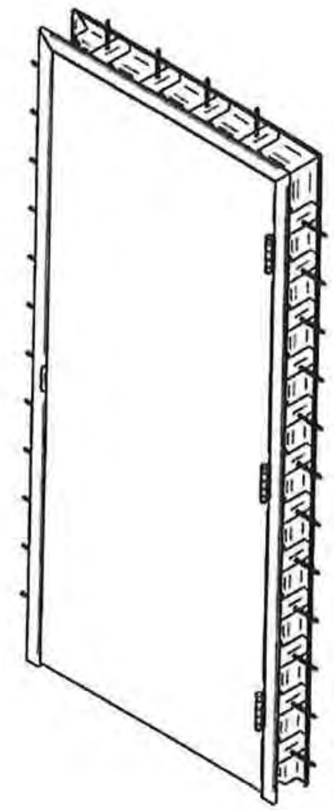
AMBICO SPECIALIZED STEEL DOORS & FRAMES	TITLE: CHARGE WEIGHT II, 105' STANDOFF, LOW LEVEL OF PRTOECTION BLAST RESISTANT FRAME AND DOOR - TAGS 501, 502	DATE 03/02/2015	ORDER NO
		DRAWN BY: tberry	CUSTOMER NO A18898
		REVISION 0	SHEET 4 OF 6

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





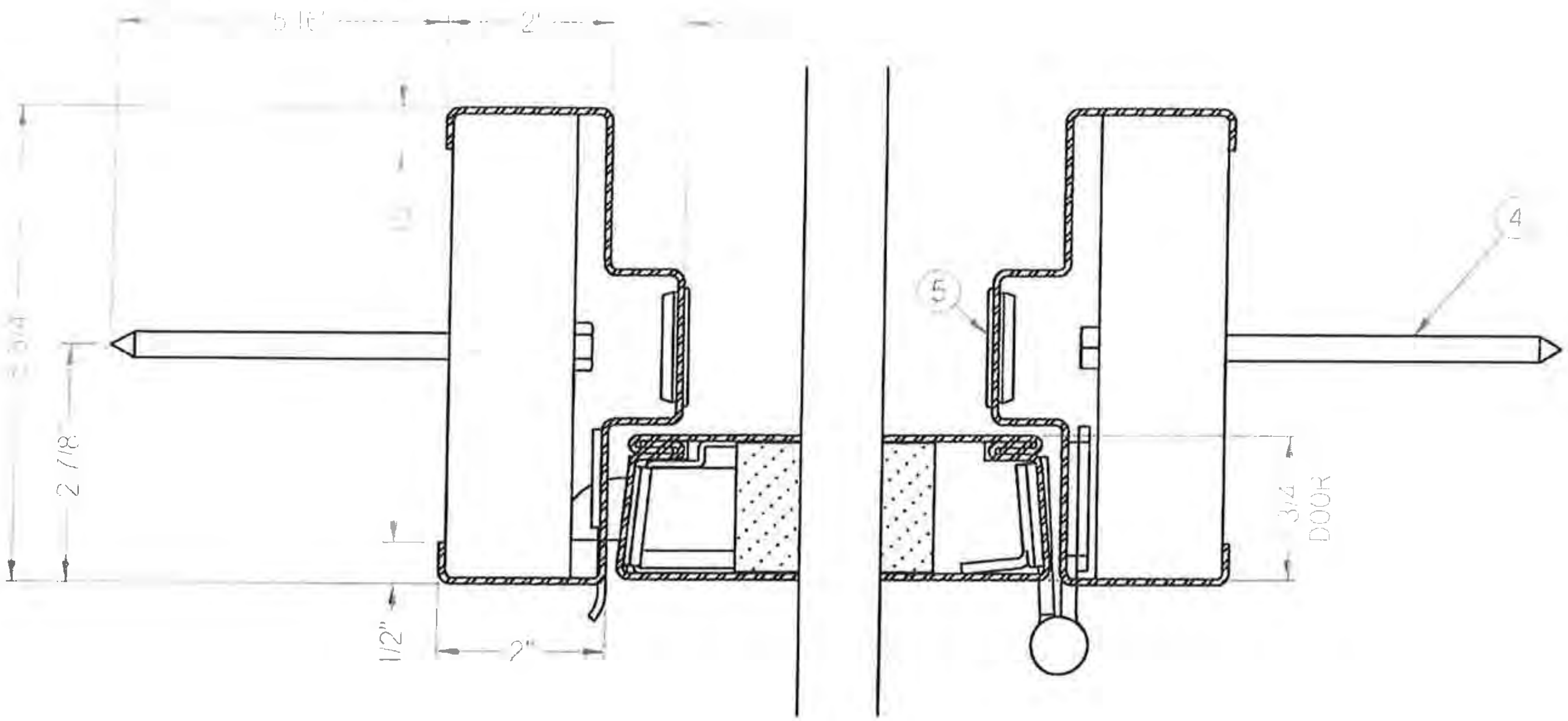
ISOMETRIC VIEW

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

Integrity - Service - Excellence

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

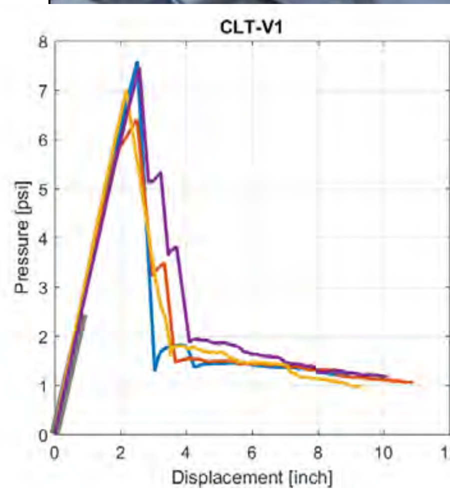
http://www.woodskyscrapers.com/cross-laminated-timber.html

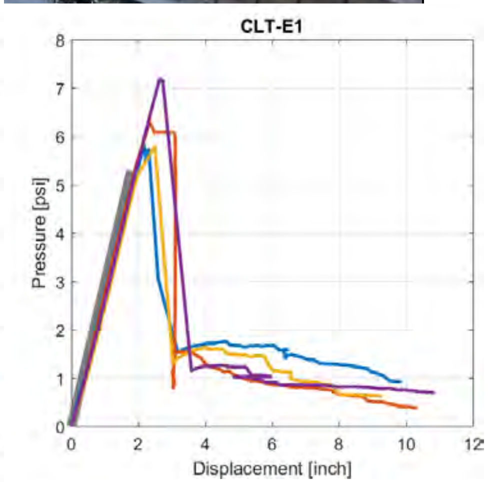
Resistance Function Development

- 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)



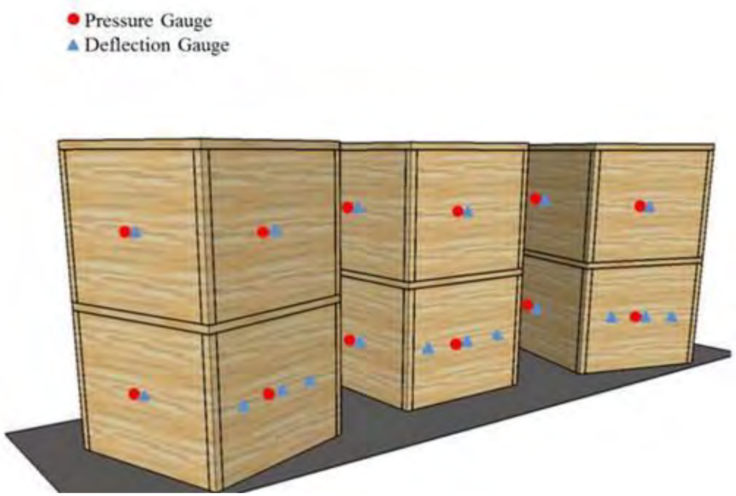


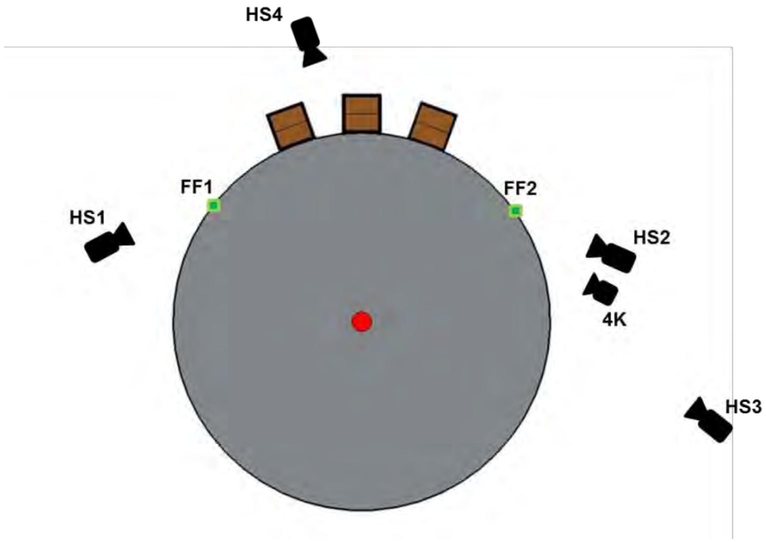
K&C

Full-Scale Blast Validations

Buildings labeled according to grade of CLT panels

Full-Scale Validation Setup





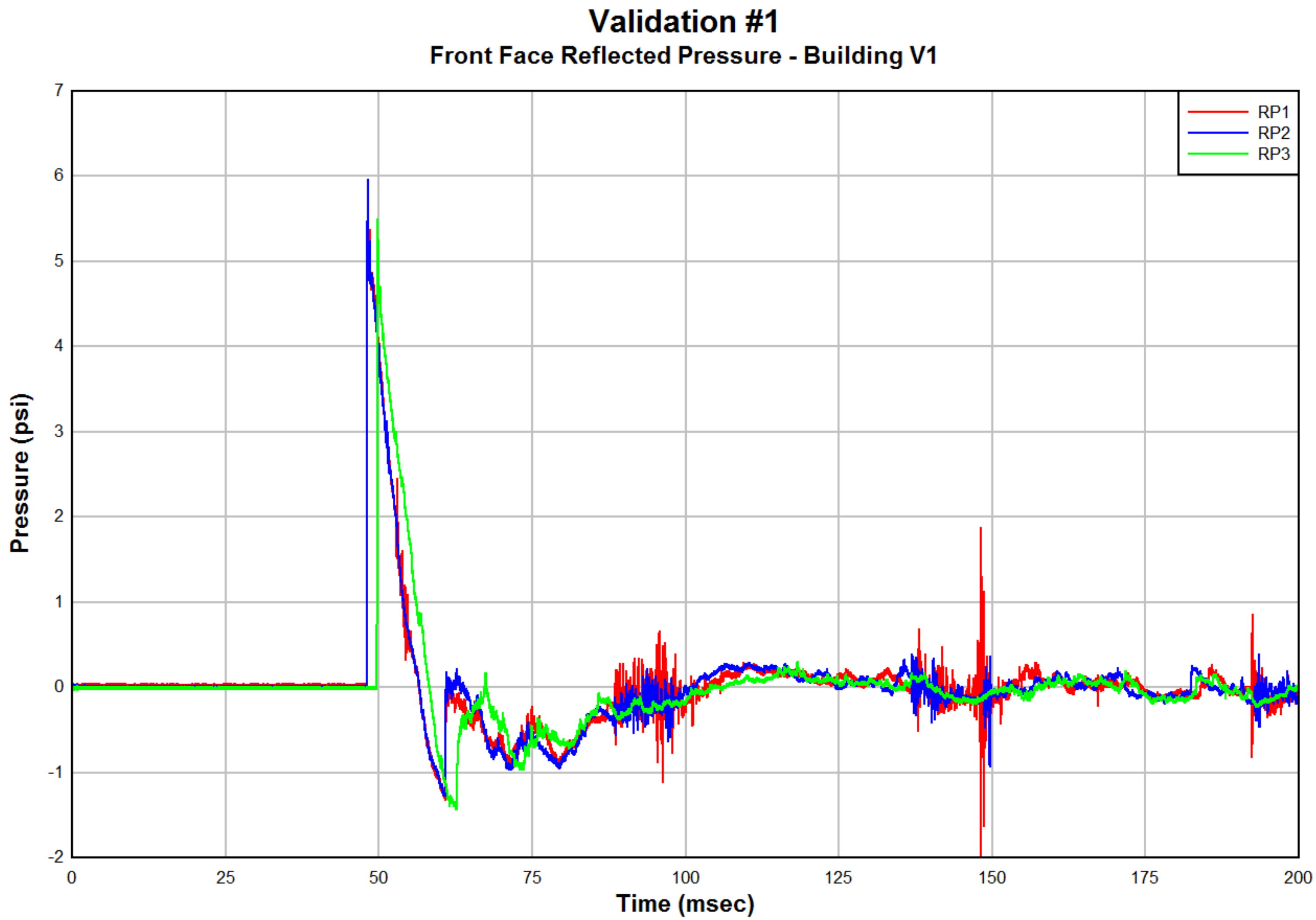
- 62 total gauges
- 24 reflected/incident pressure gauges (8 per building)
- 33 deflection gauges (11 per building)
- 3 internal pressure gauges (1 per building)
- 2 free field incident pressure gauges
- 4 high speed cameras
- 1 4k real-time camera

Full-Scale Validation #1

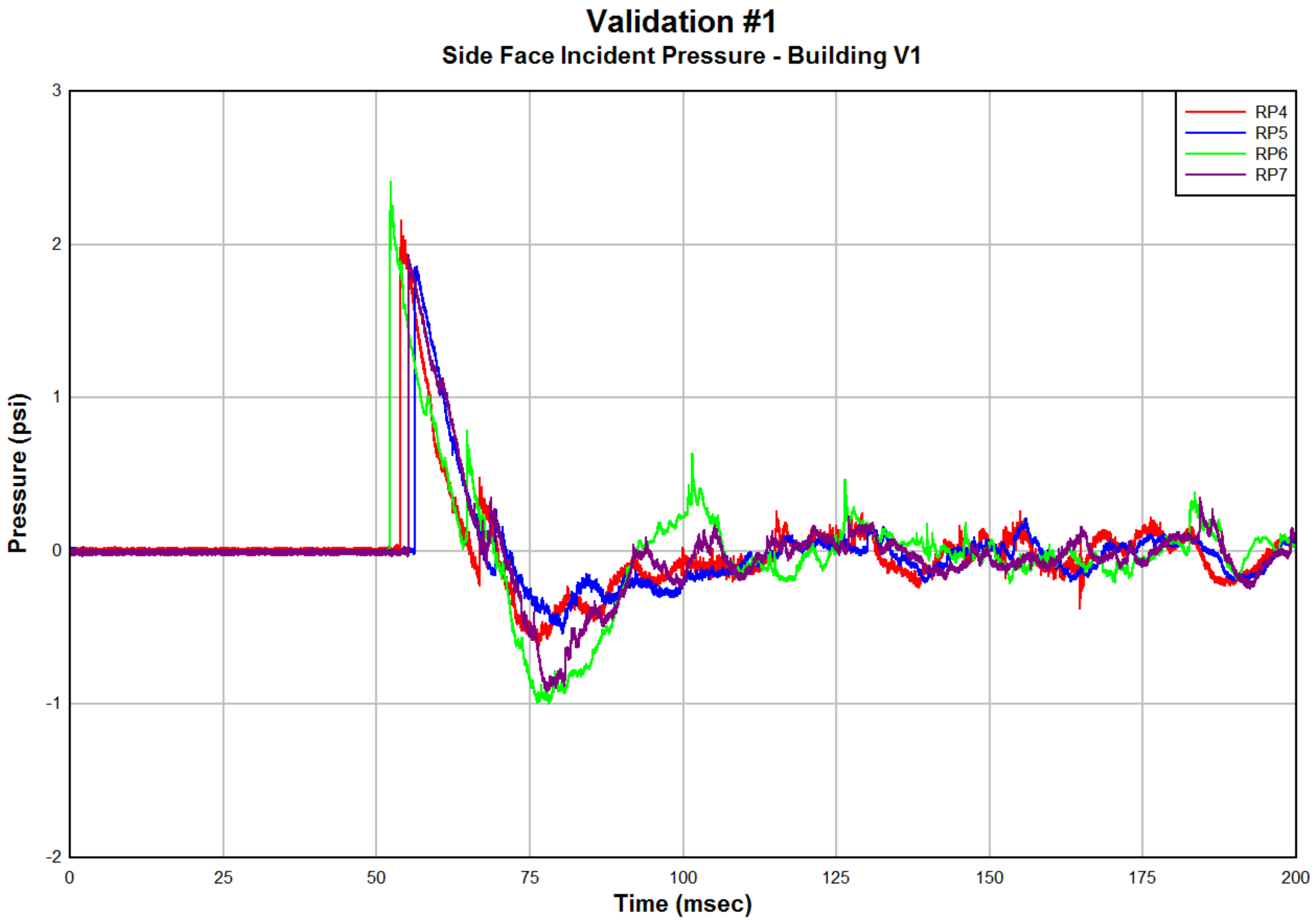
Pre-test

Post-test

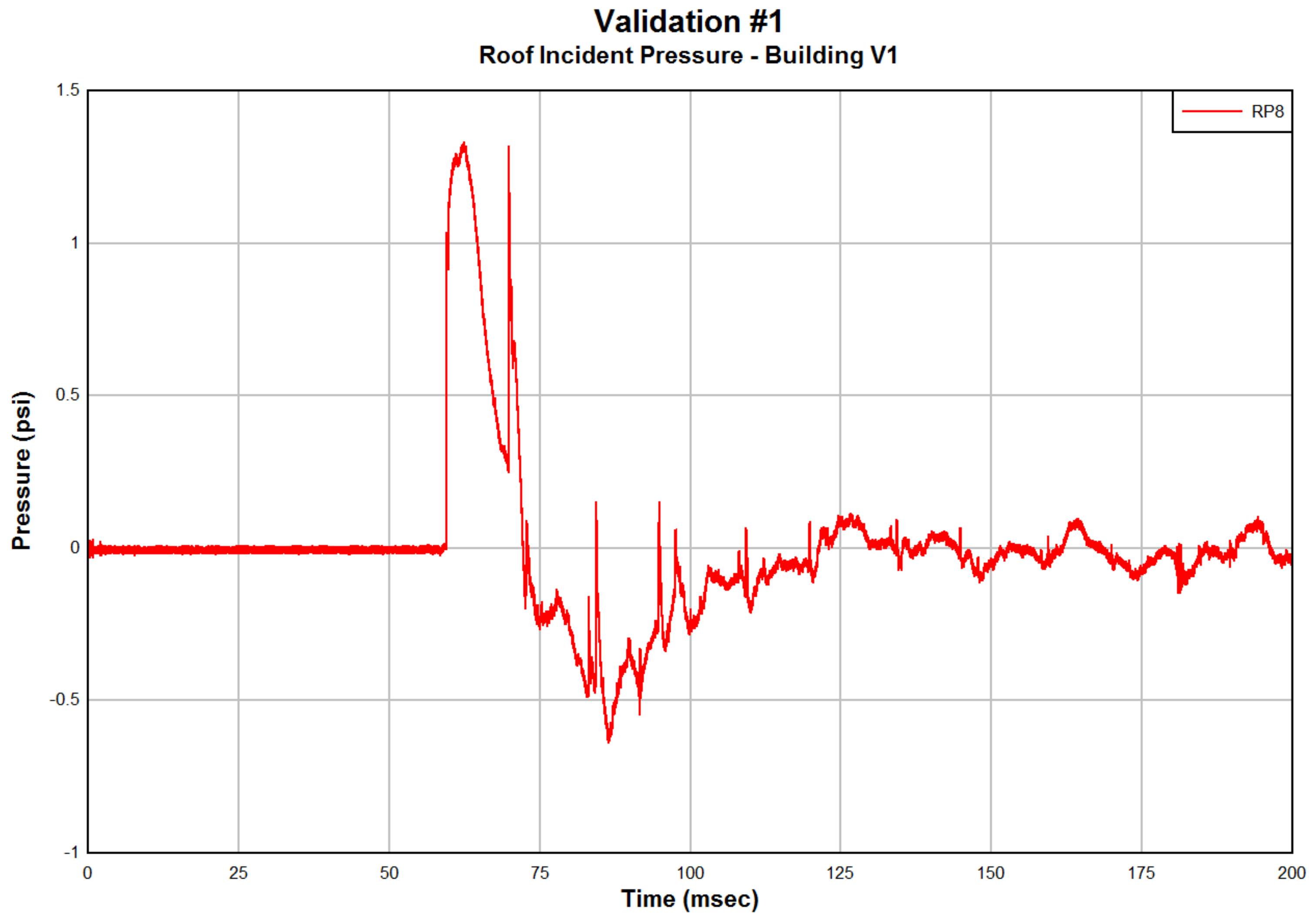
Validation #1 – Building V1: Front Face Reflected Pressure Gauges



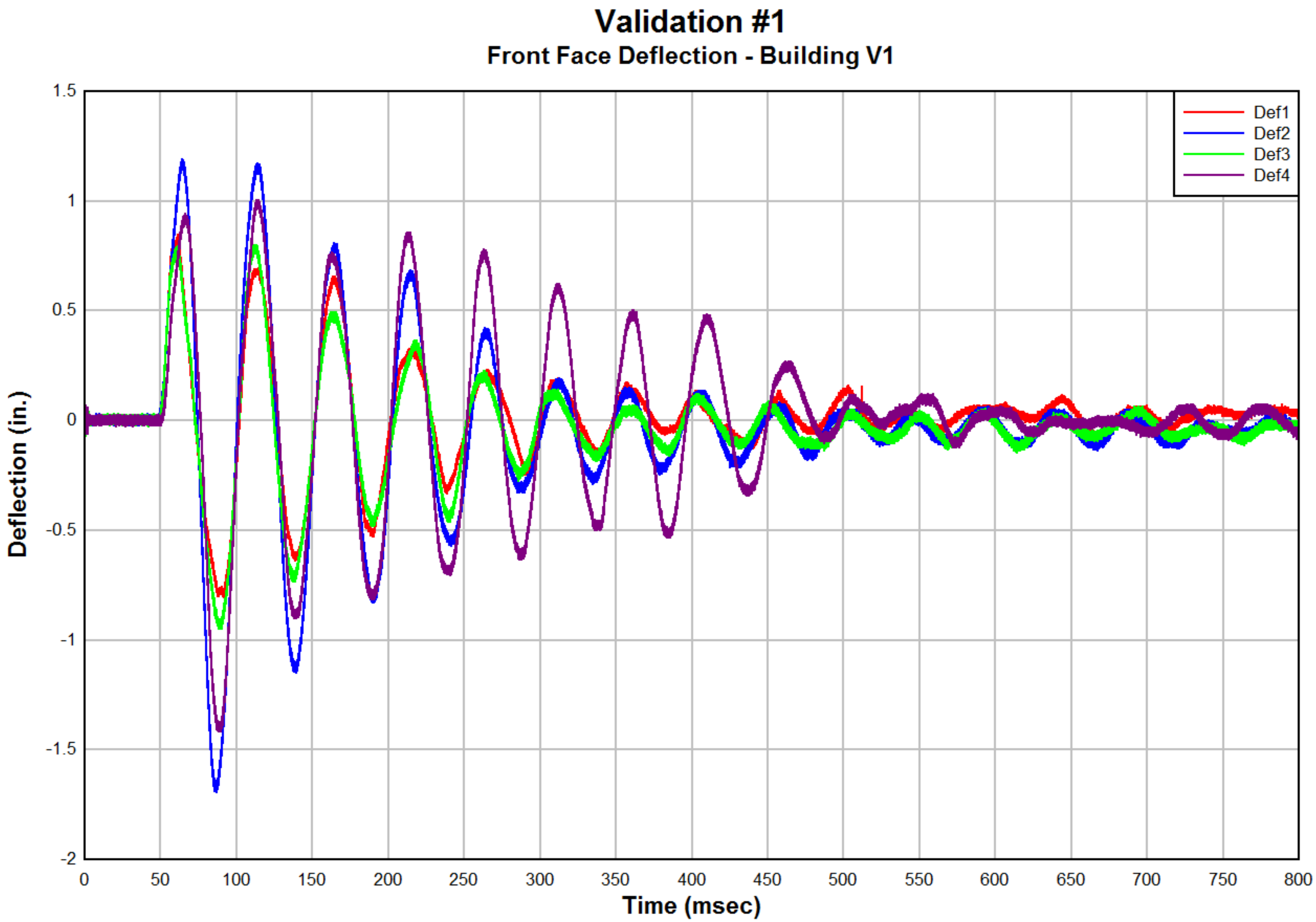
Validation #1 – Building V1: Side Face Incident Pressure Gauges



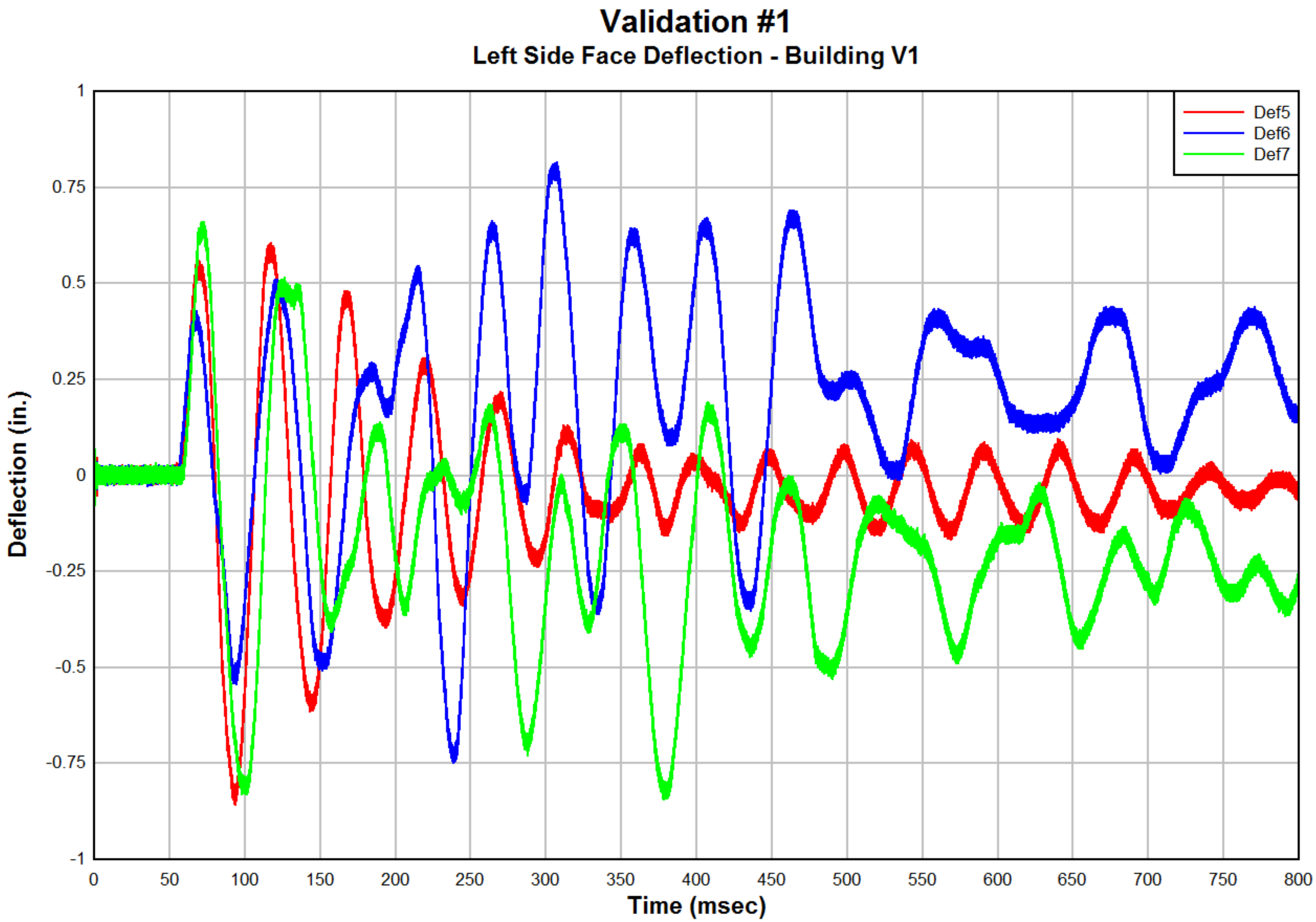
Validation #1 – Building V1: Roof Incident Pressure Gauges



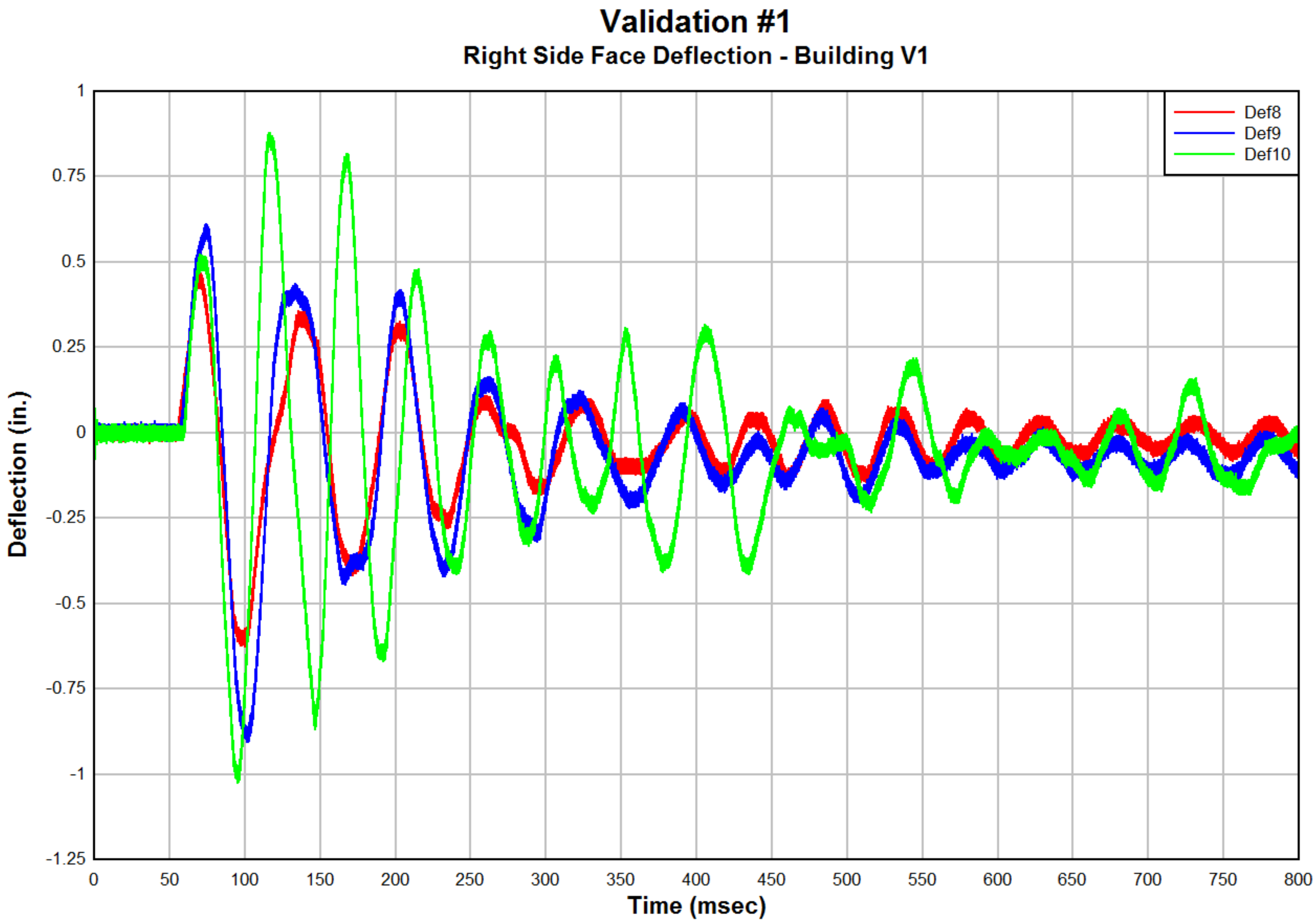
Validation #1 – Building V1: Front Face Deflection Gauges



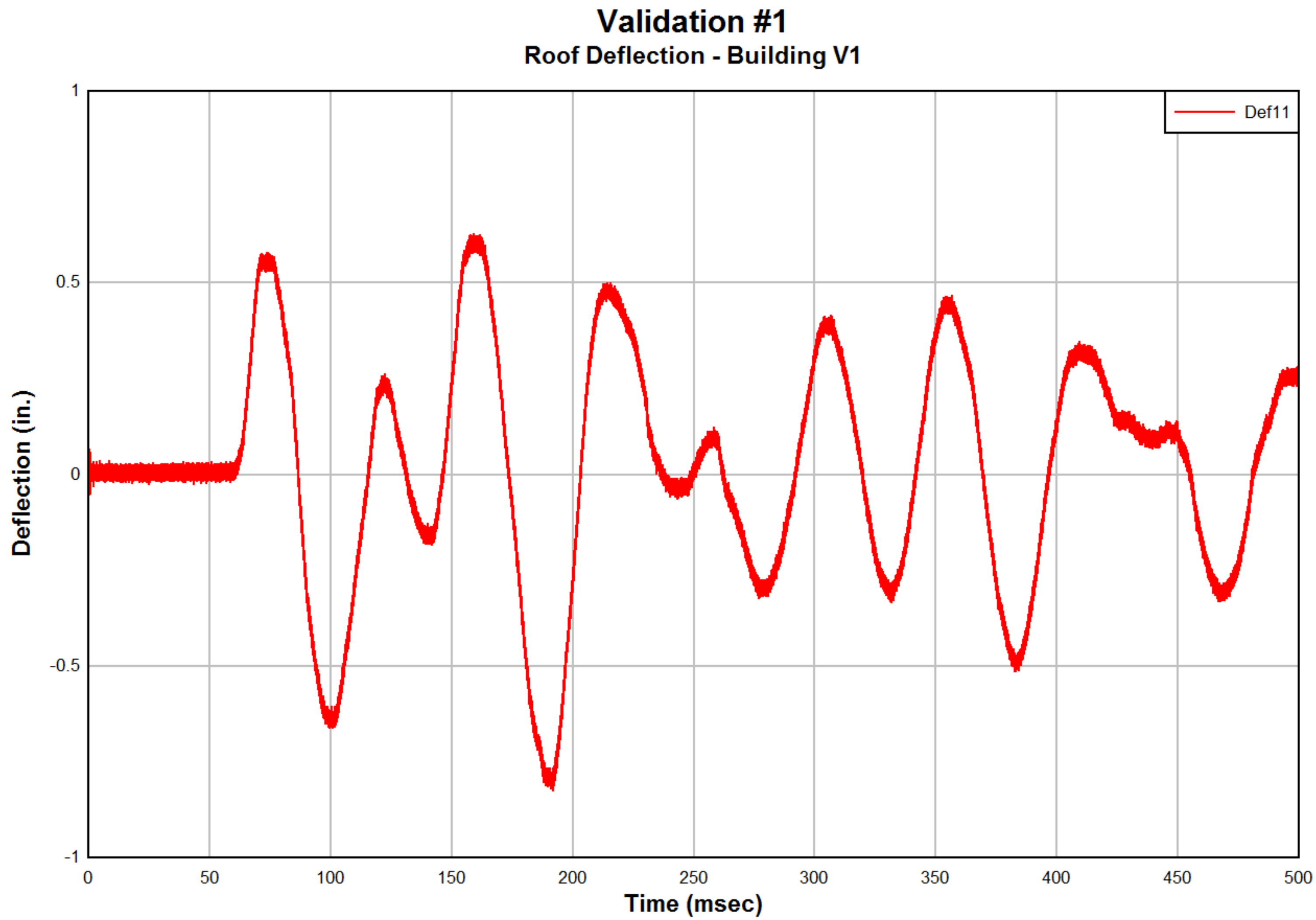
Validation #1 – Building V1: Left Side Face Deflection Gauges



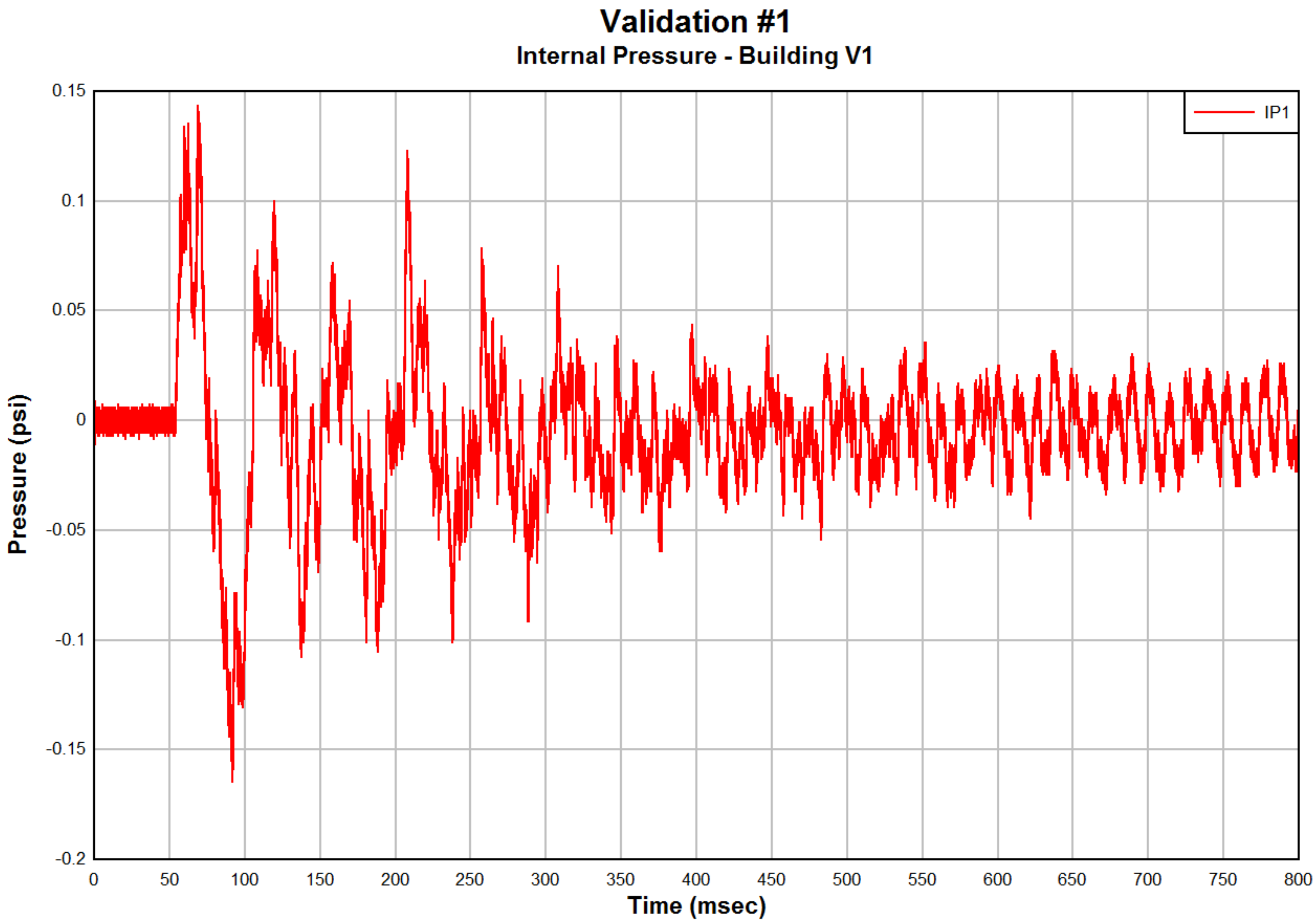
Validation #1 – Building V1: Right Side Face Deflection Gauges



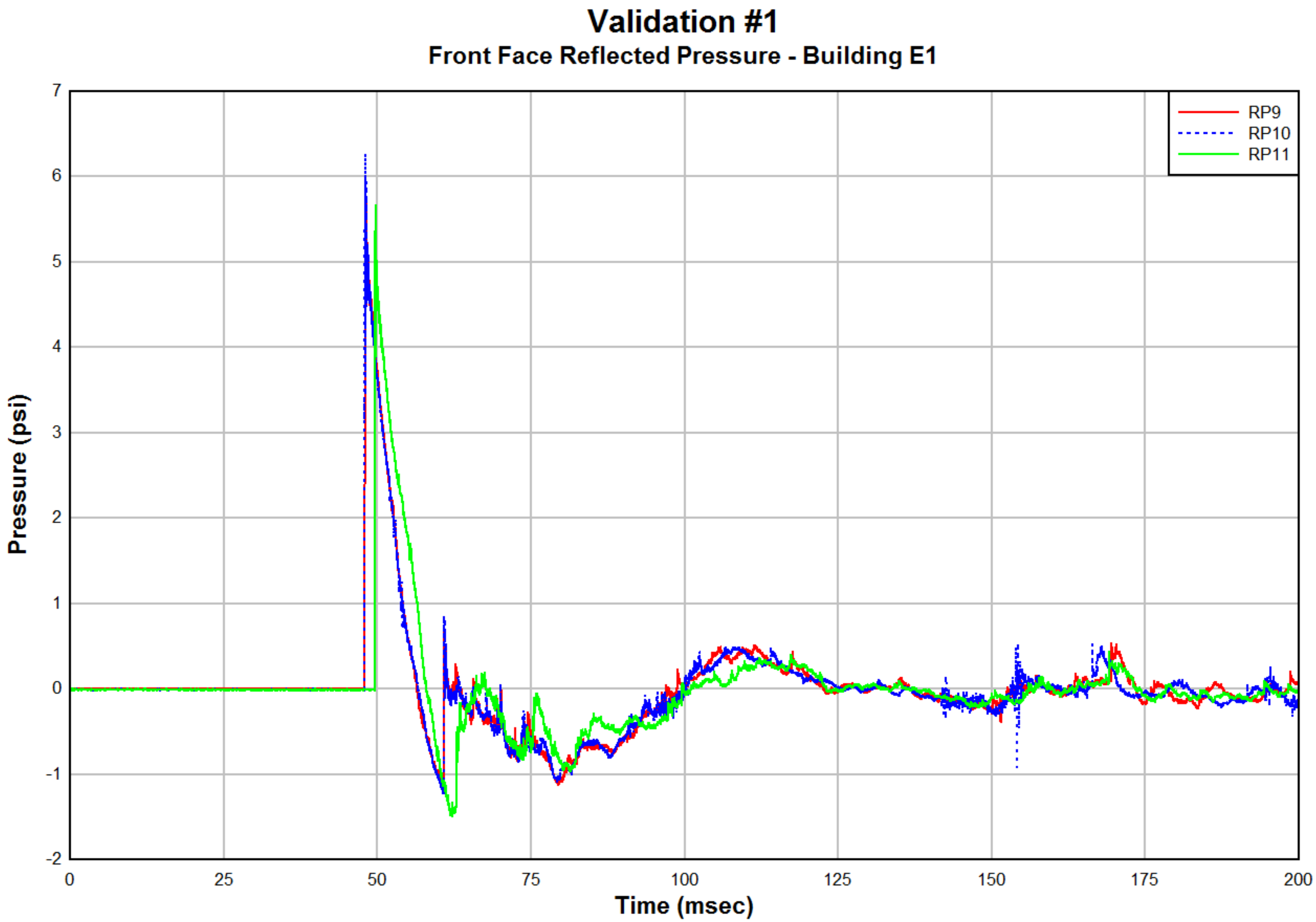
Validation #1 – Building V1: Roof Deflection Gauge



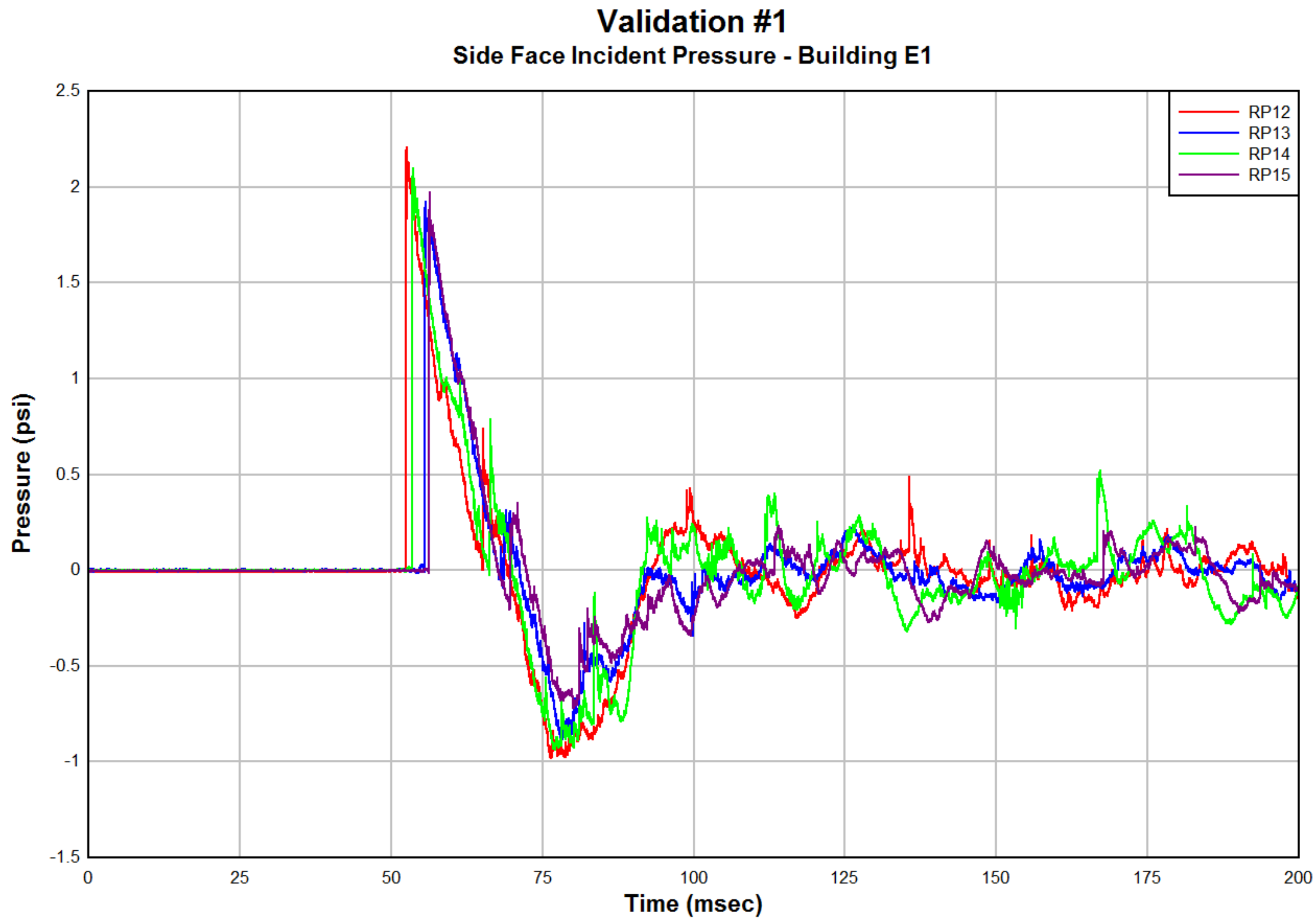
Validation #1 – Building V1: Internal Pressure Gauge



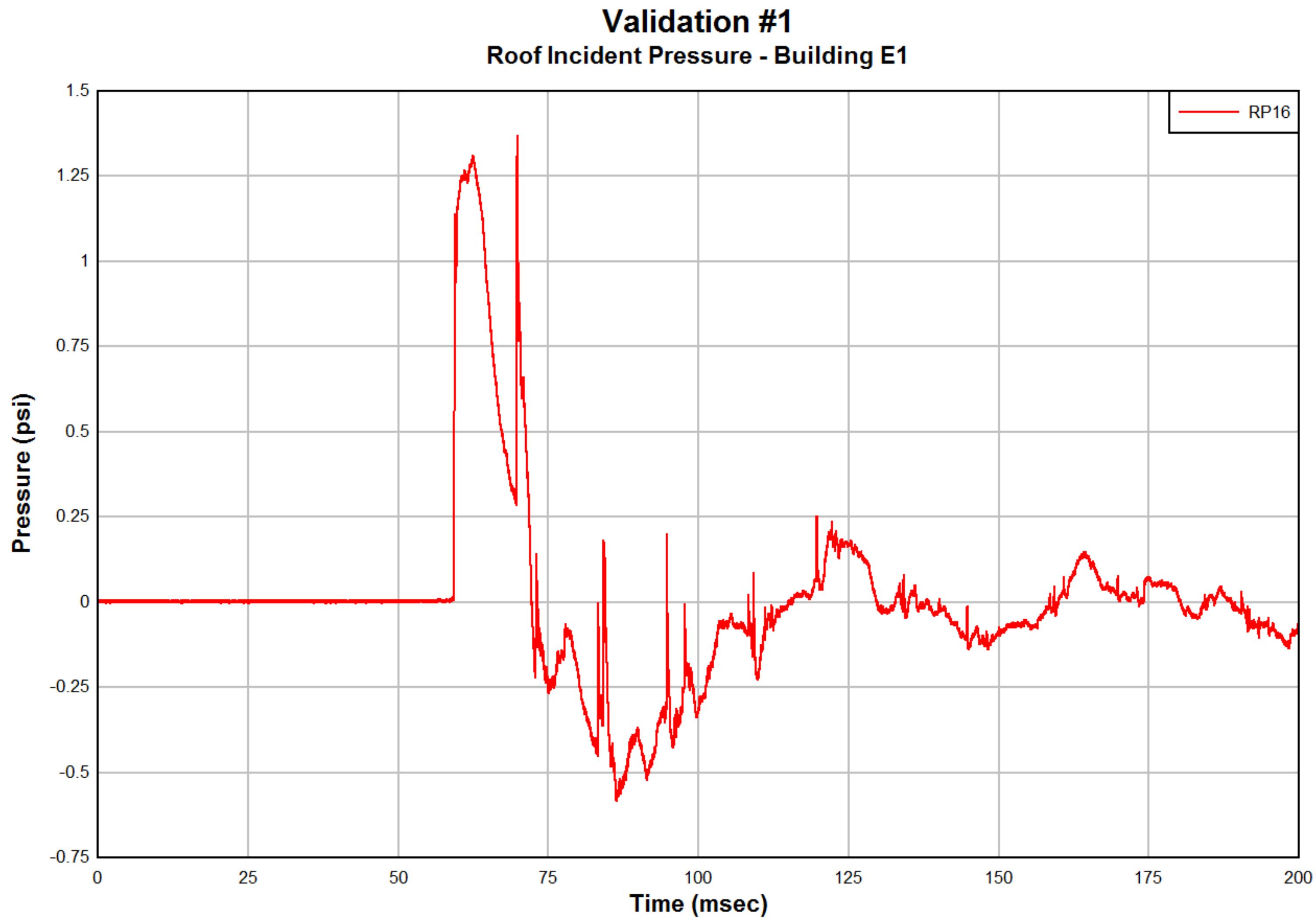
Validation #1 – Building E1: Front Face Reflected Pressure Gauges



Validation #1 – Building E1: Side Face Incident Pressure Gauges



Validation #1 – Building E1: Roof Pressure Gauge



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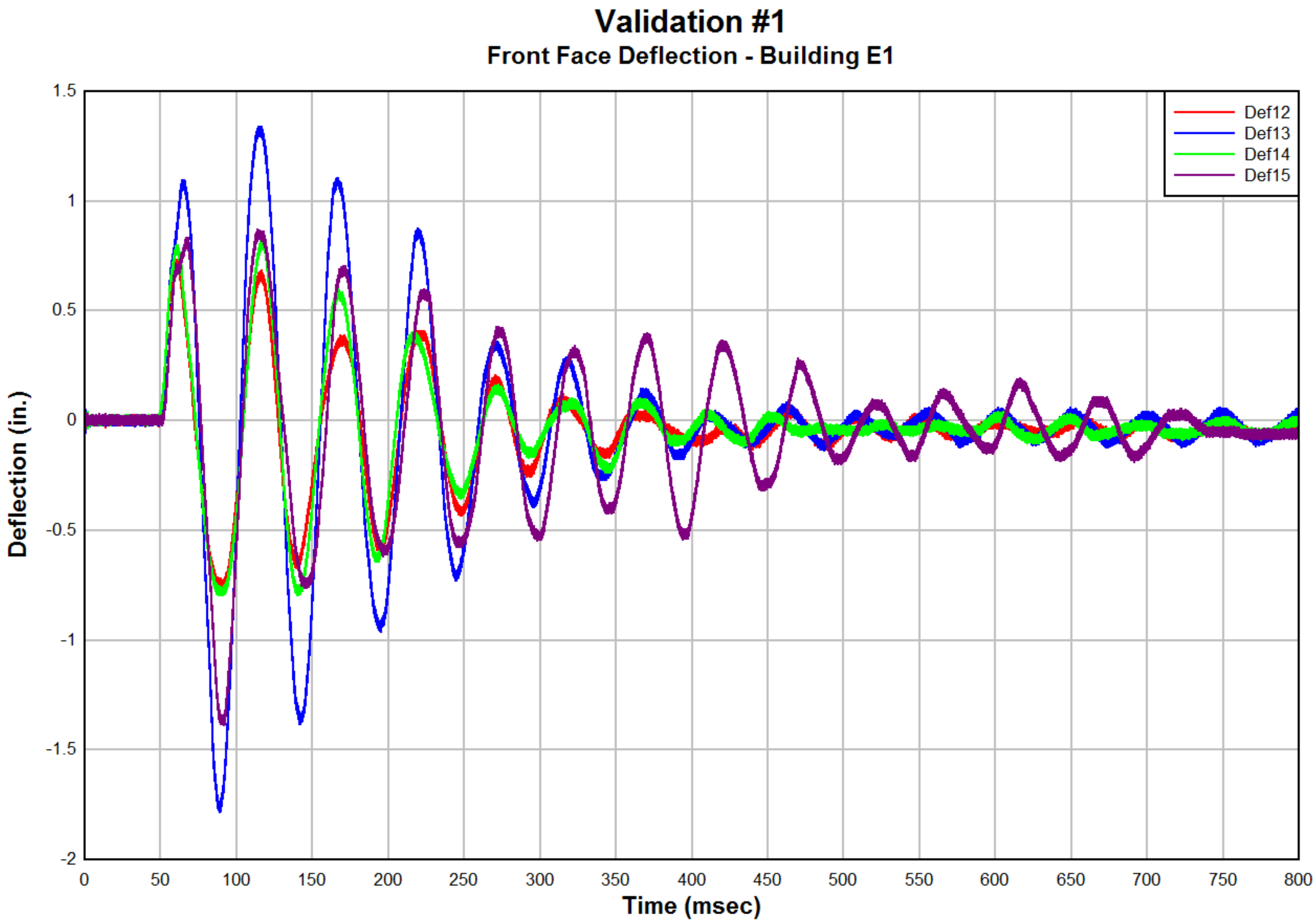
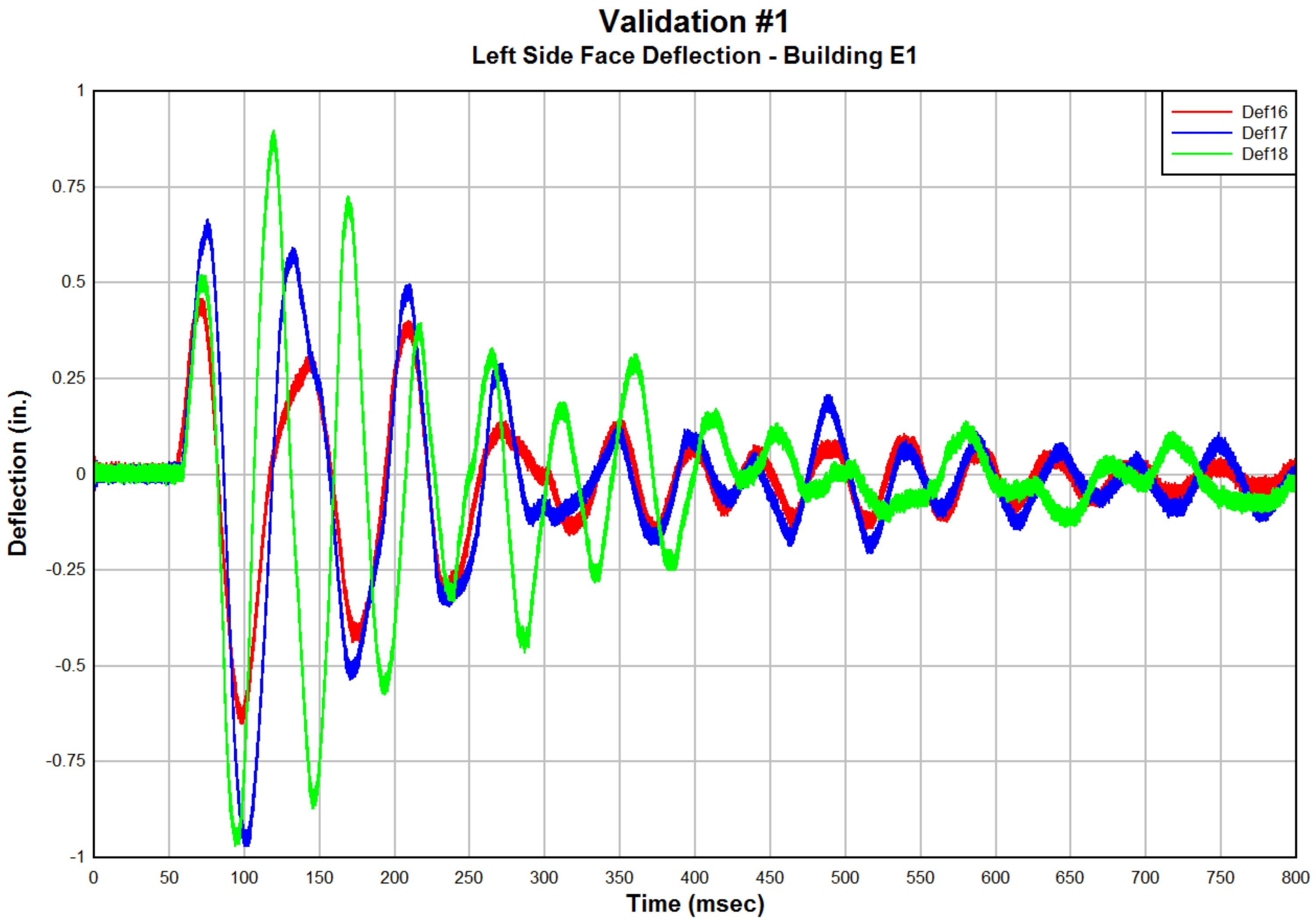
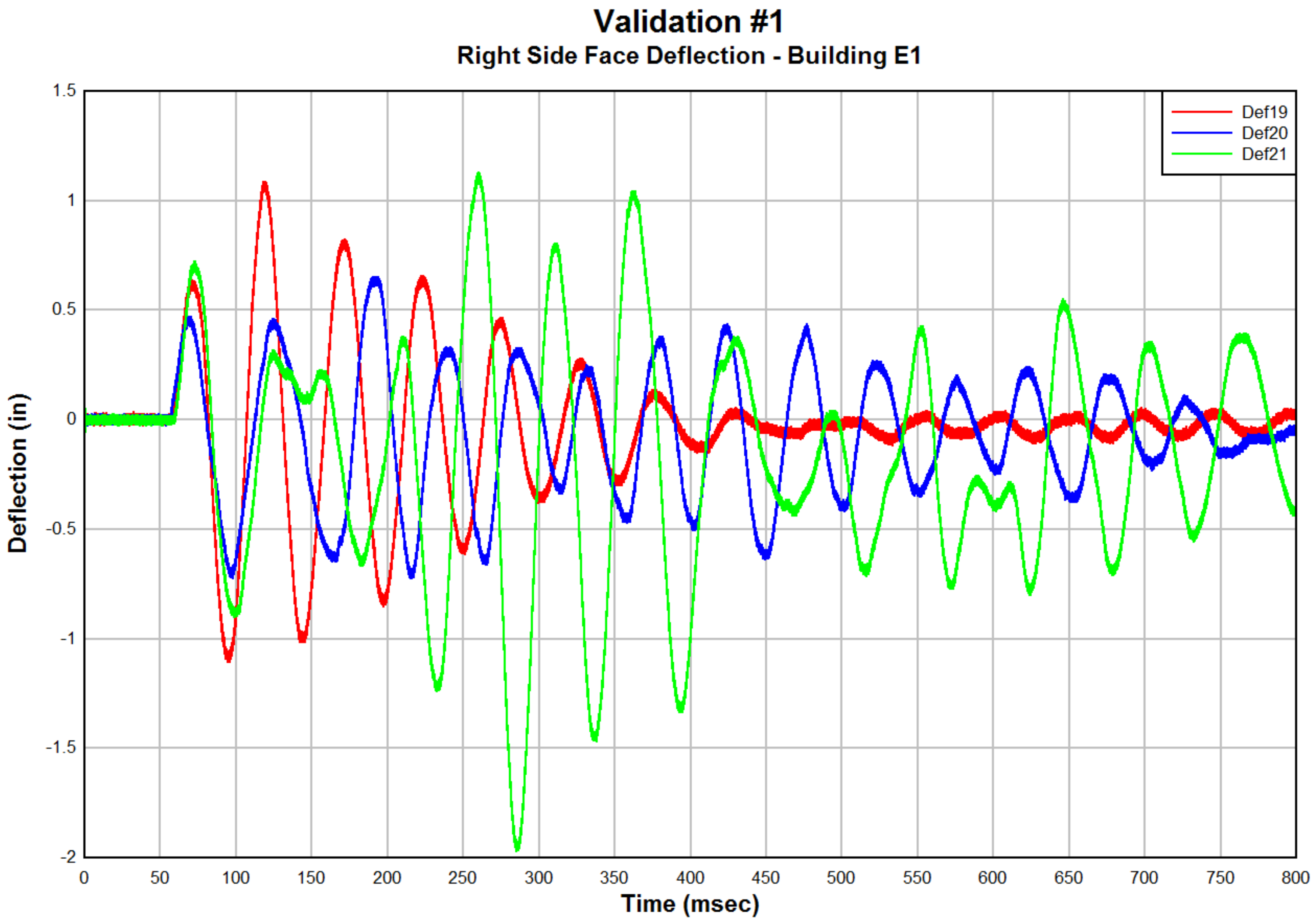


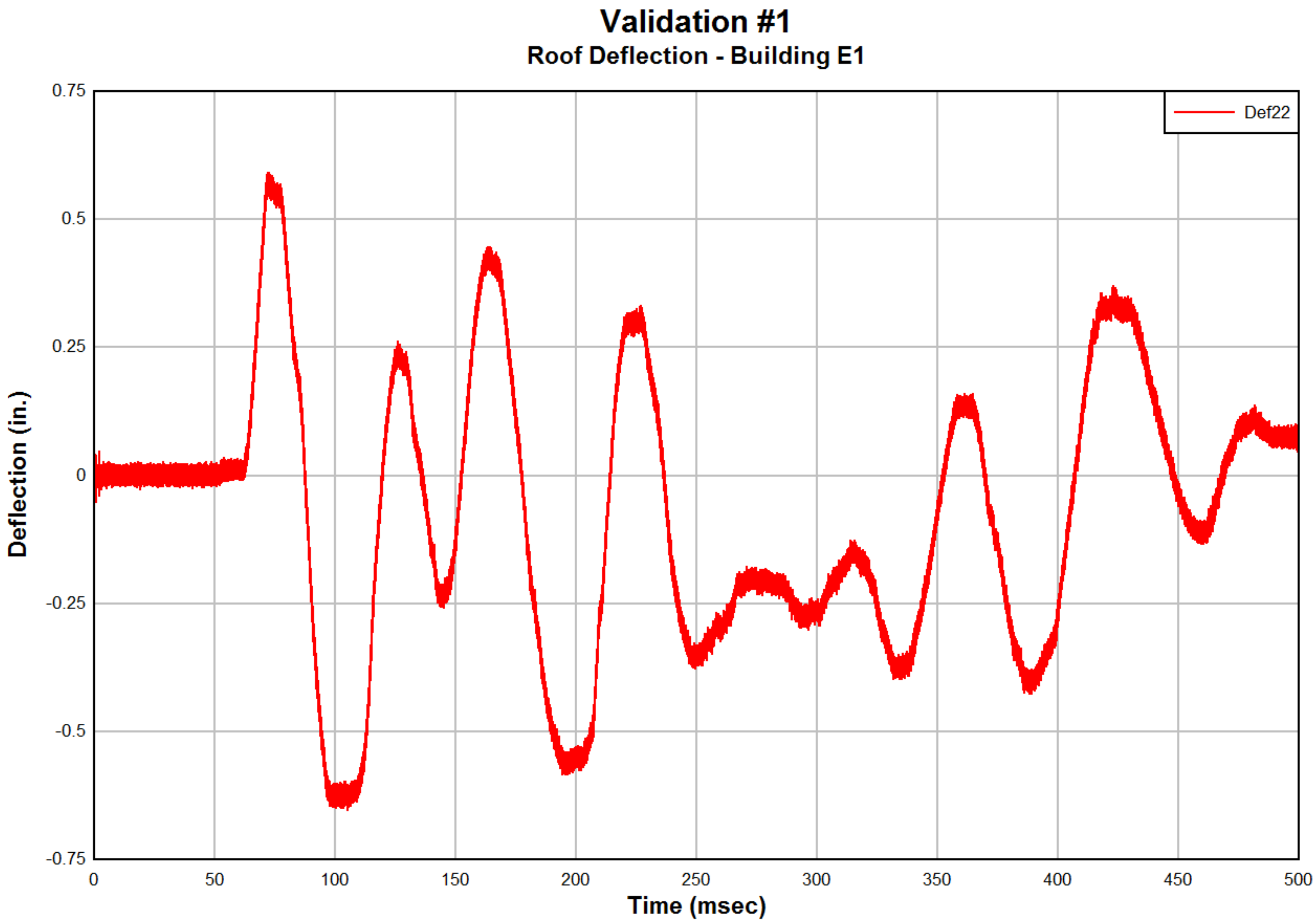
Validation #1 – Building E1: Left Side Face Deflection Gauges



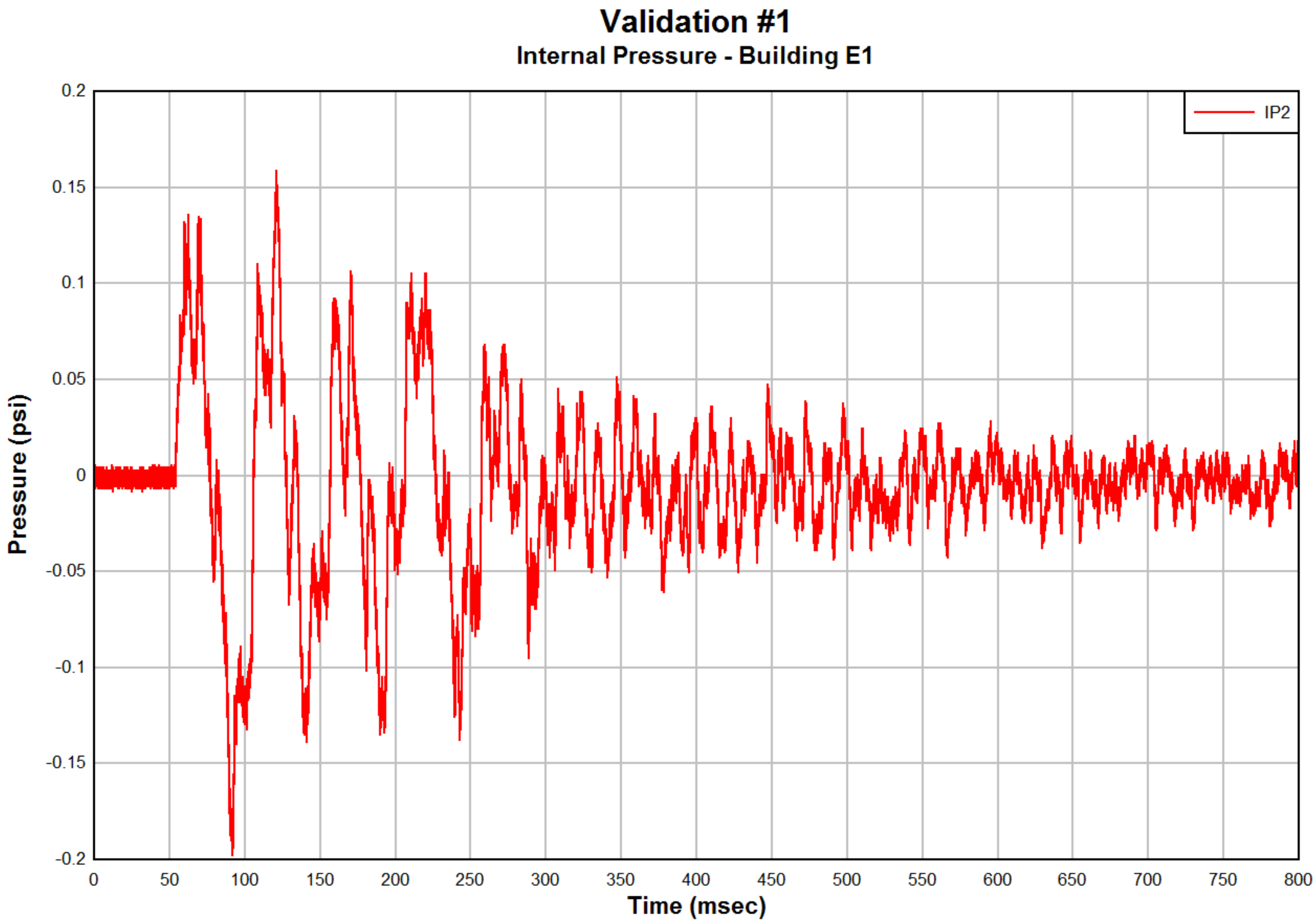
Validation #1 – Building E1: Right Side Face Deflection Gauges



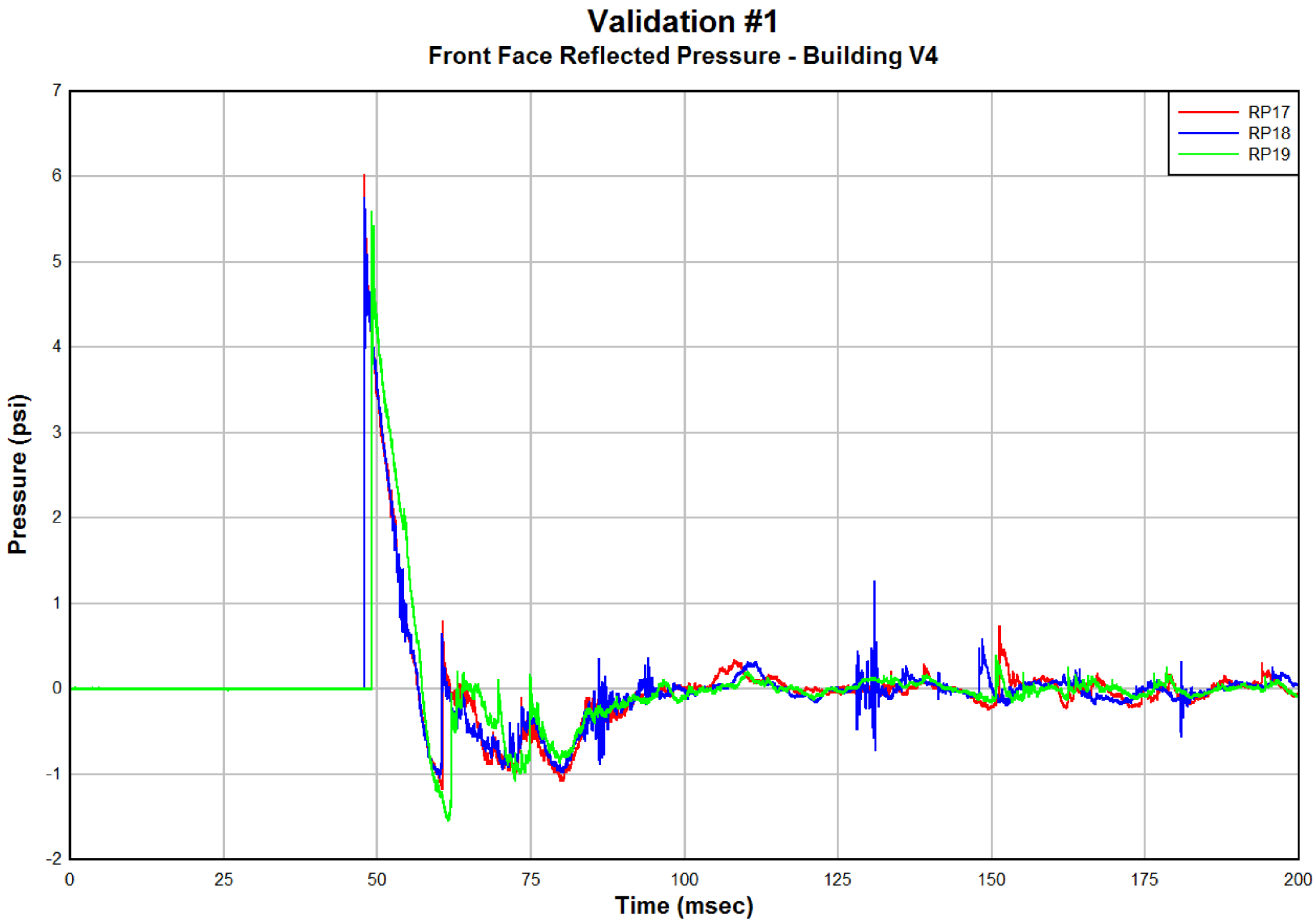
Validation #1 – Building E1: Roof Deflection Gauge



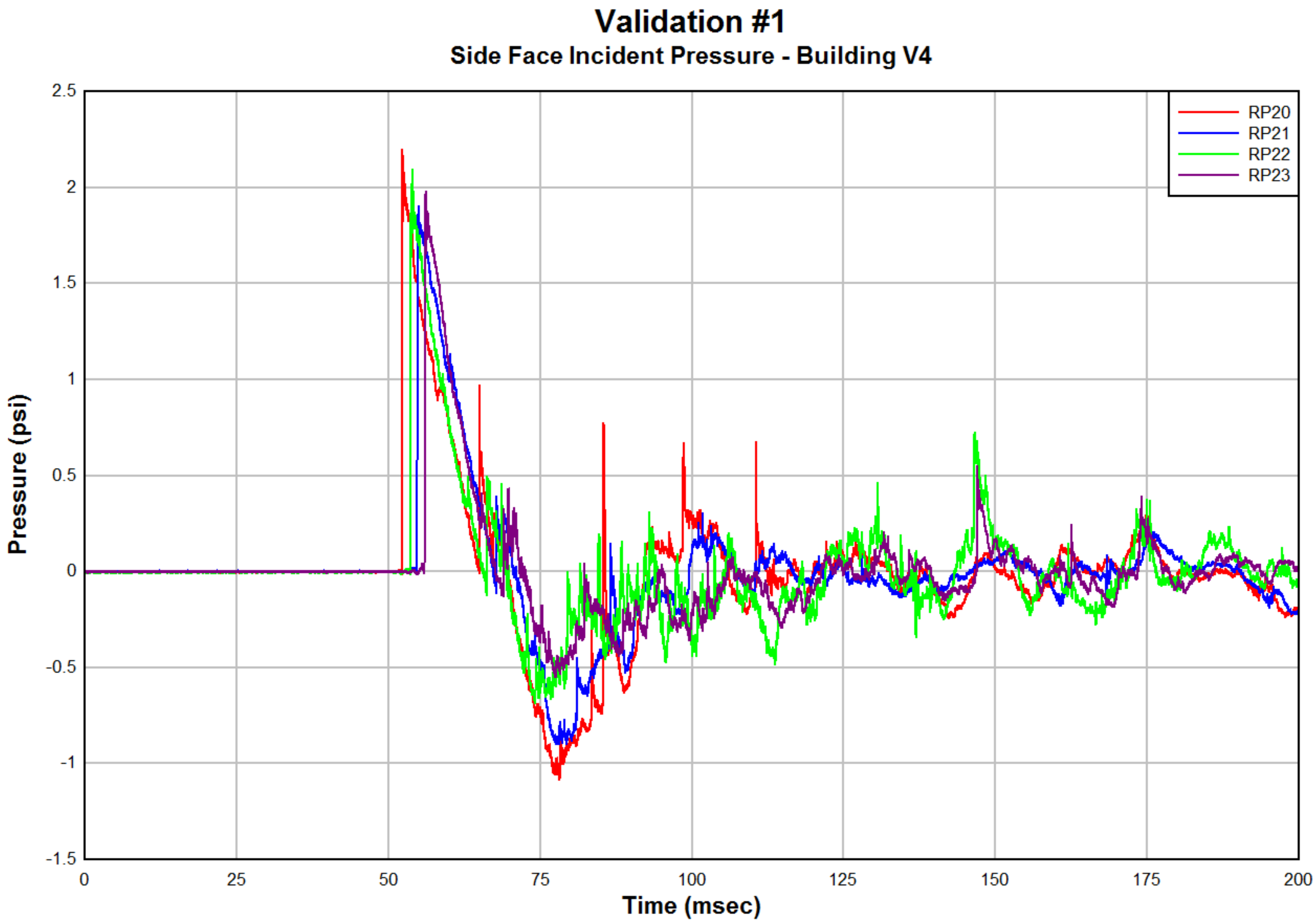
Validation #1 – Building E1: Internal Pressure Gauge



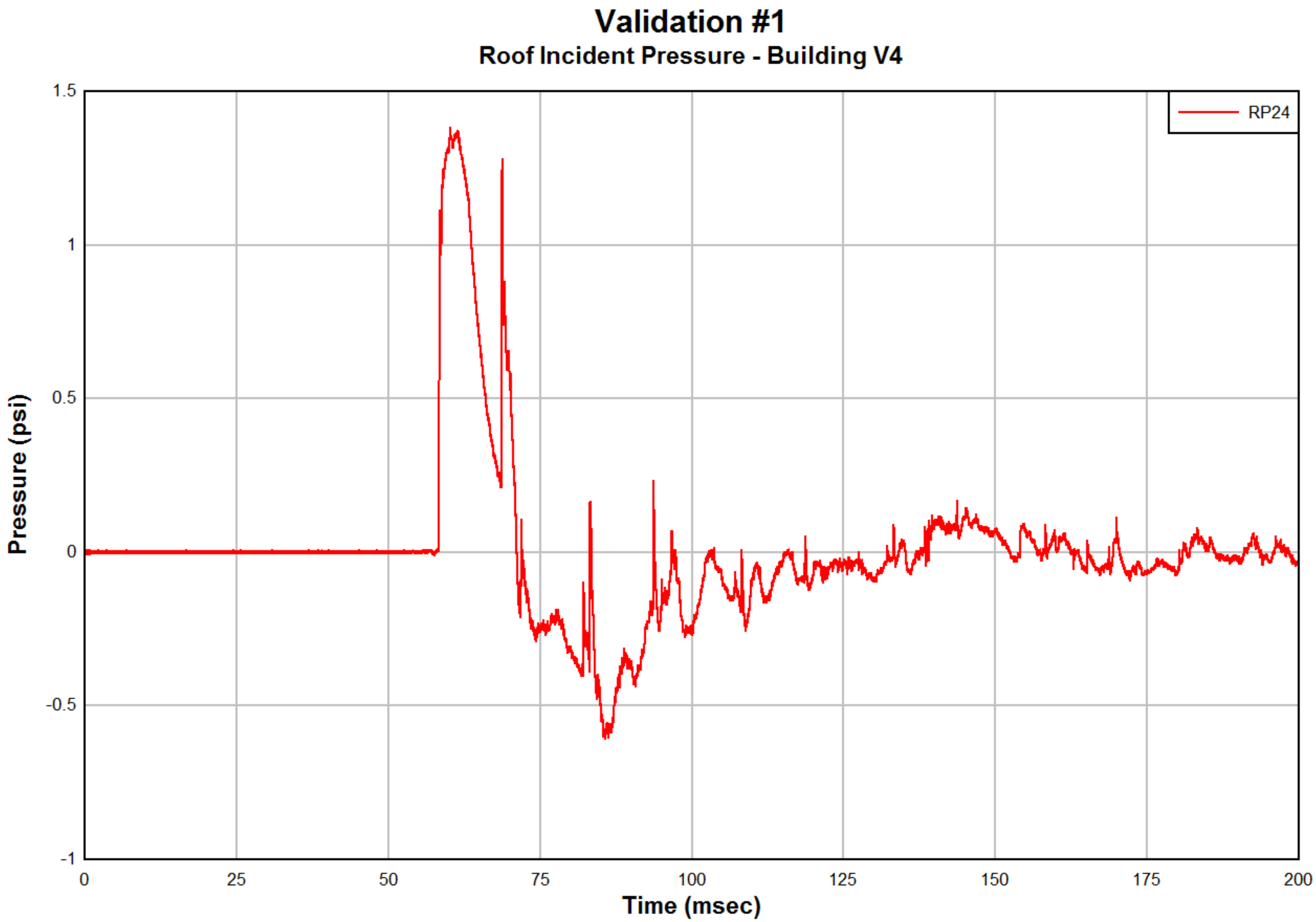
Validation #1 – Building V4: Front Face Reflected Pressure Gauges



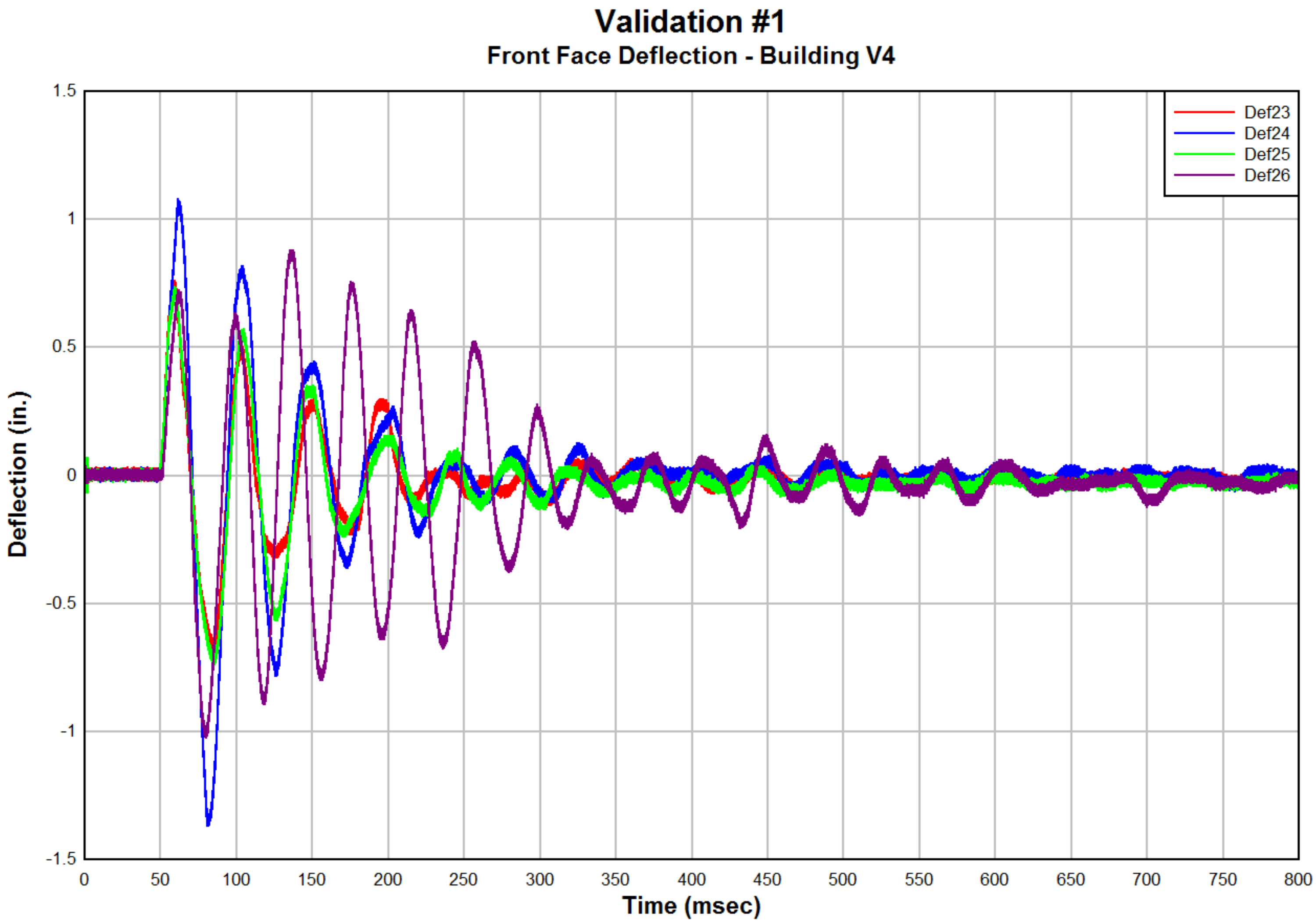
Validation #1 – Building V4: Side Face Incident Pressure Gauges



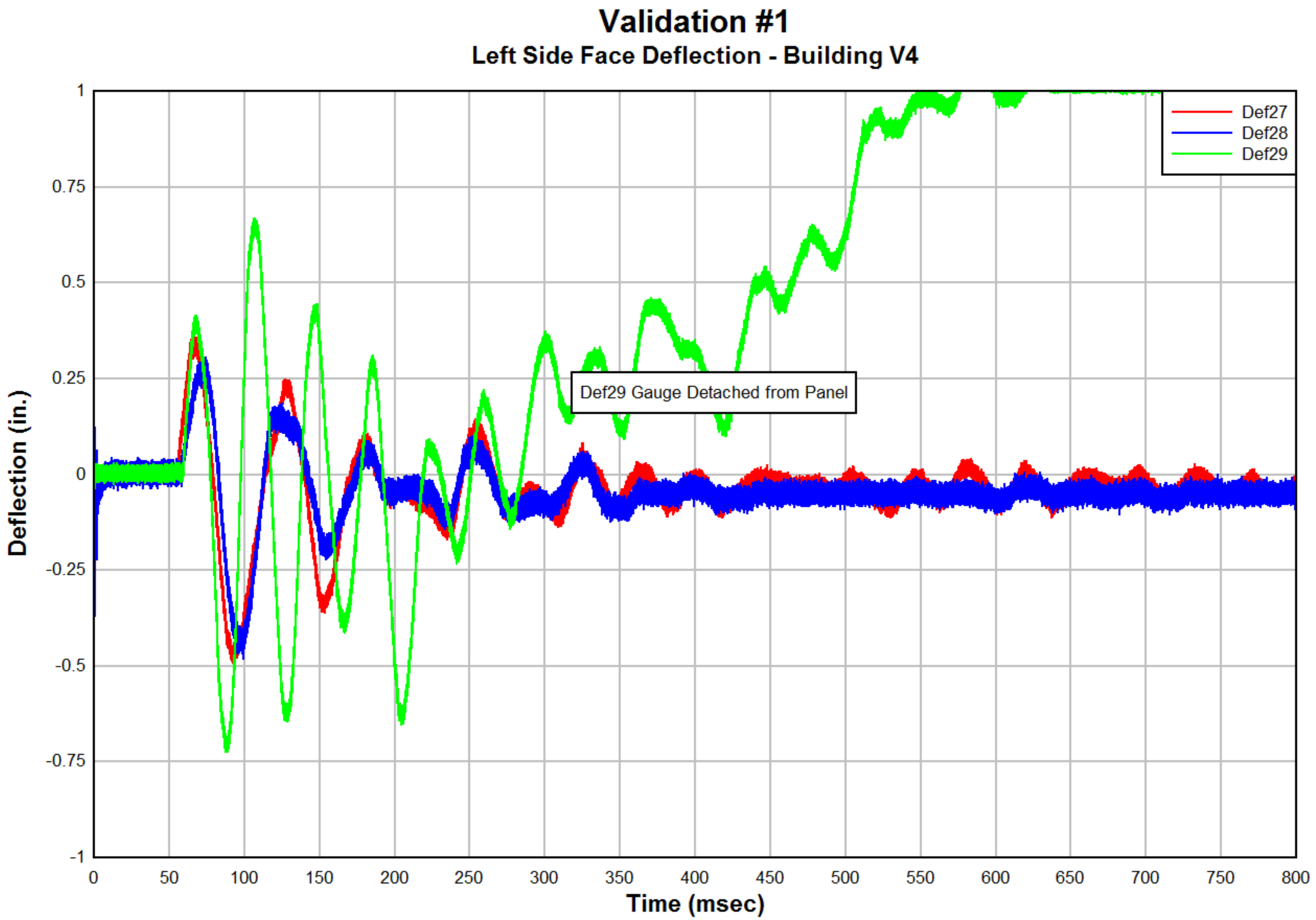
Validation #1 – Building V4: Roof Pressure Gauge



Validation #1 – Building V4: Front Face Deflection Gauges



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



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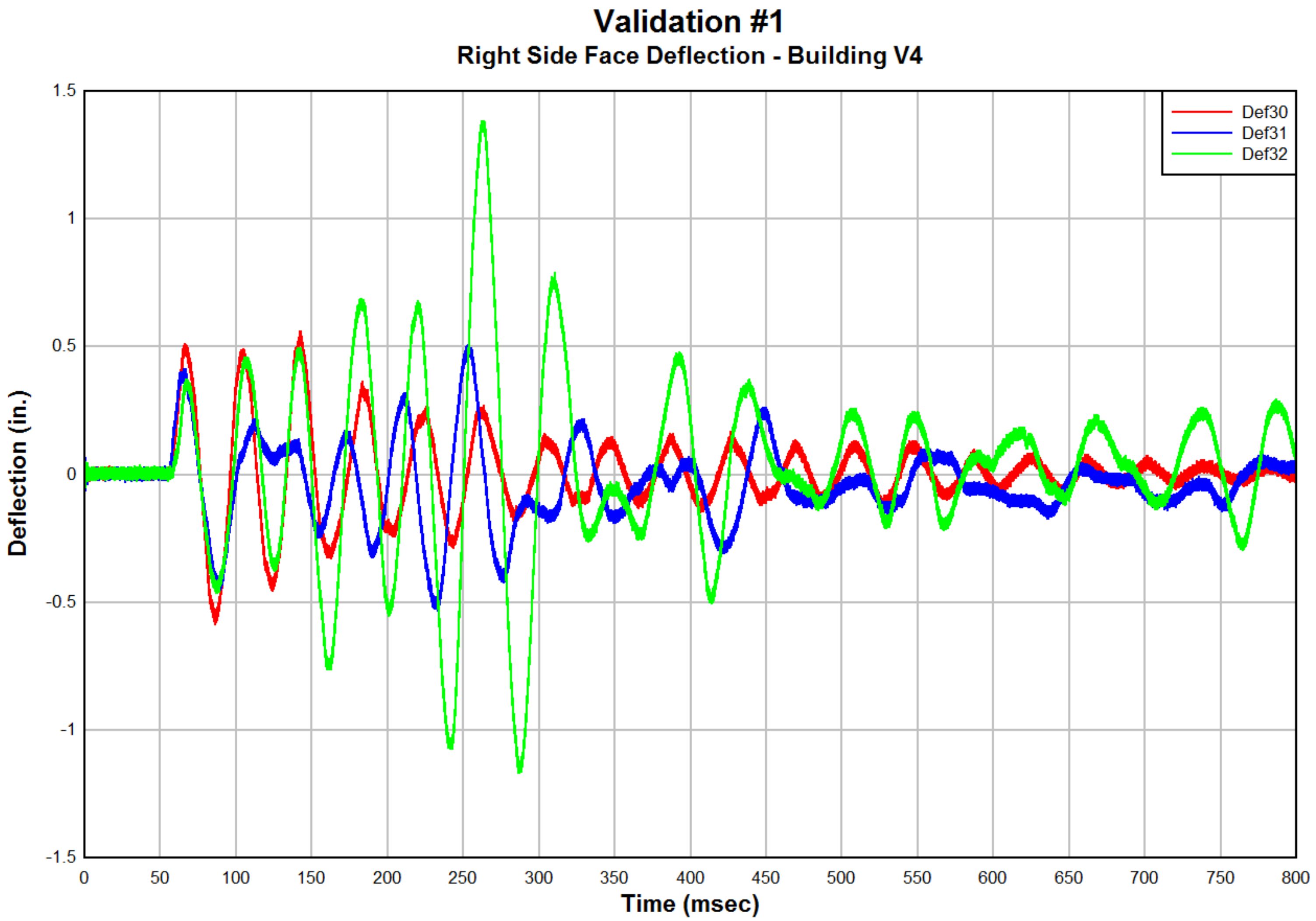

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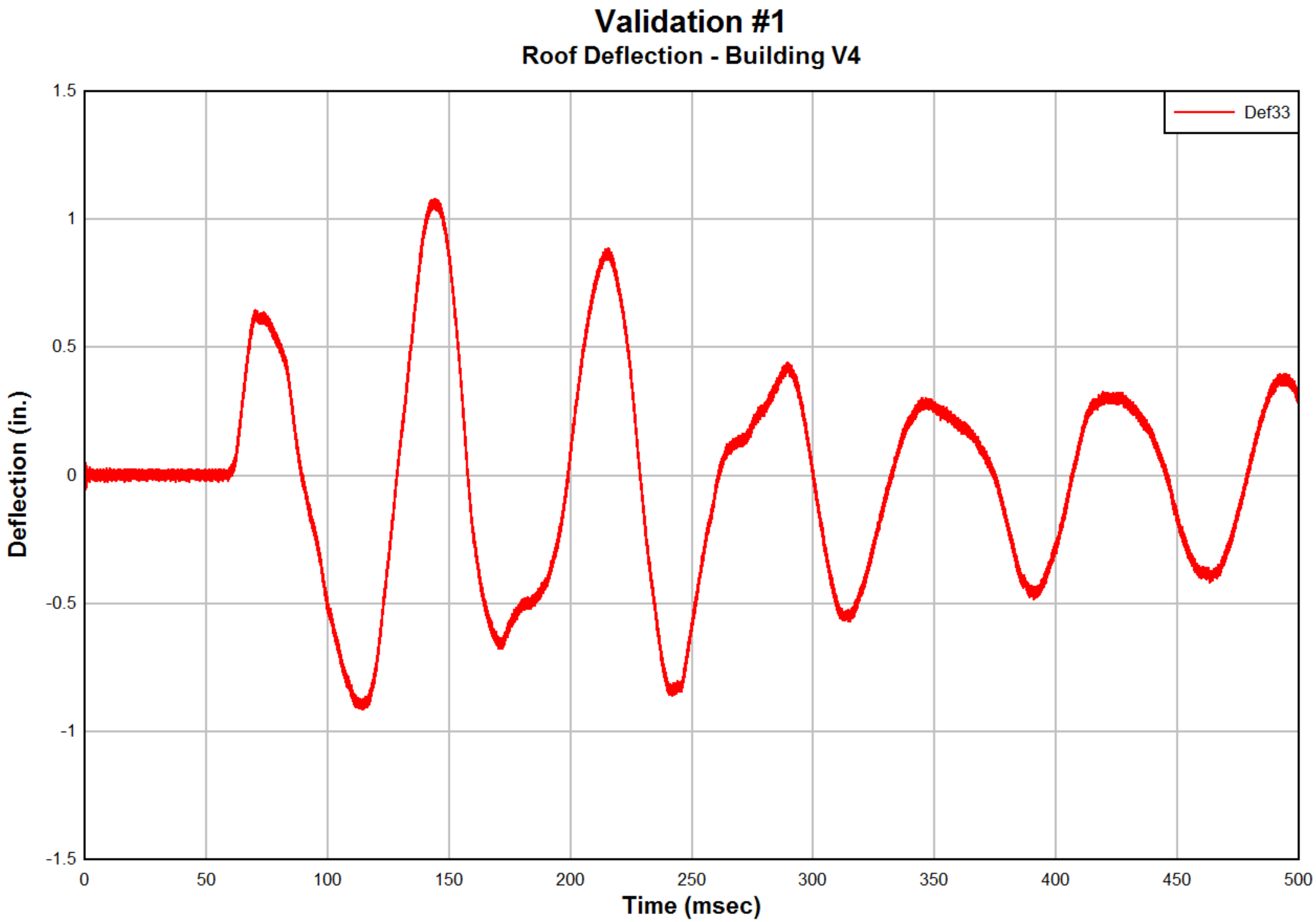

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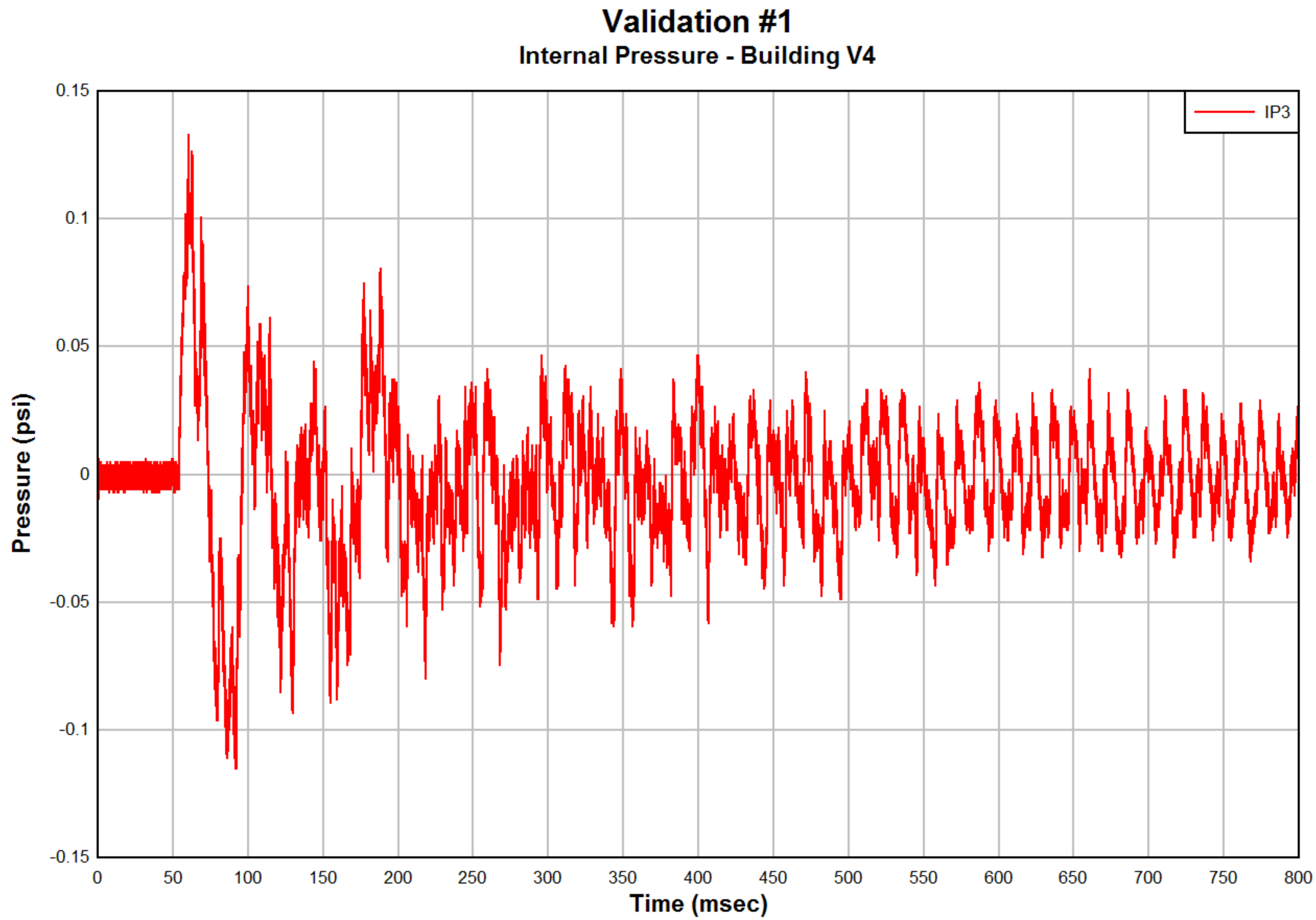
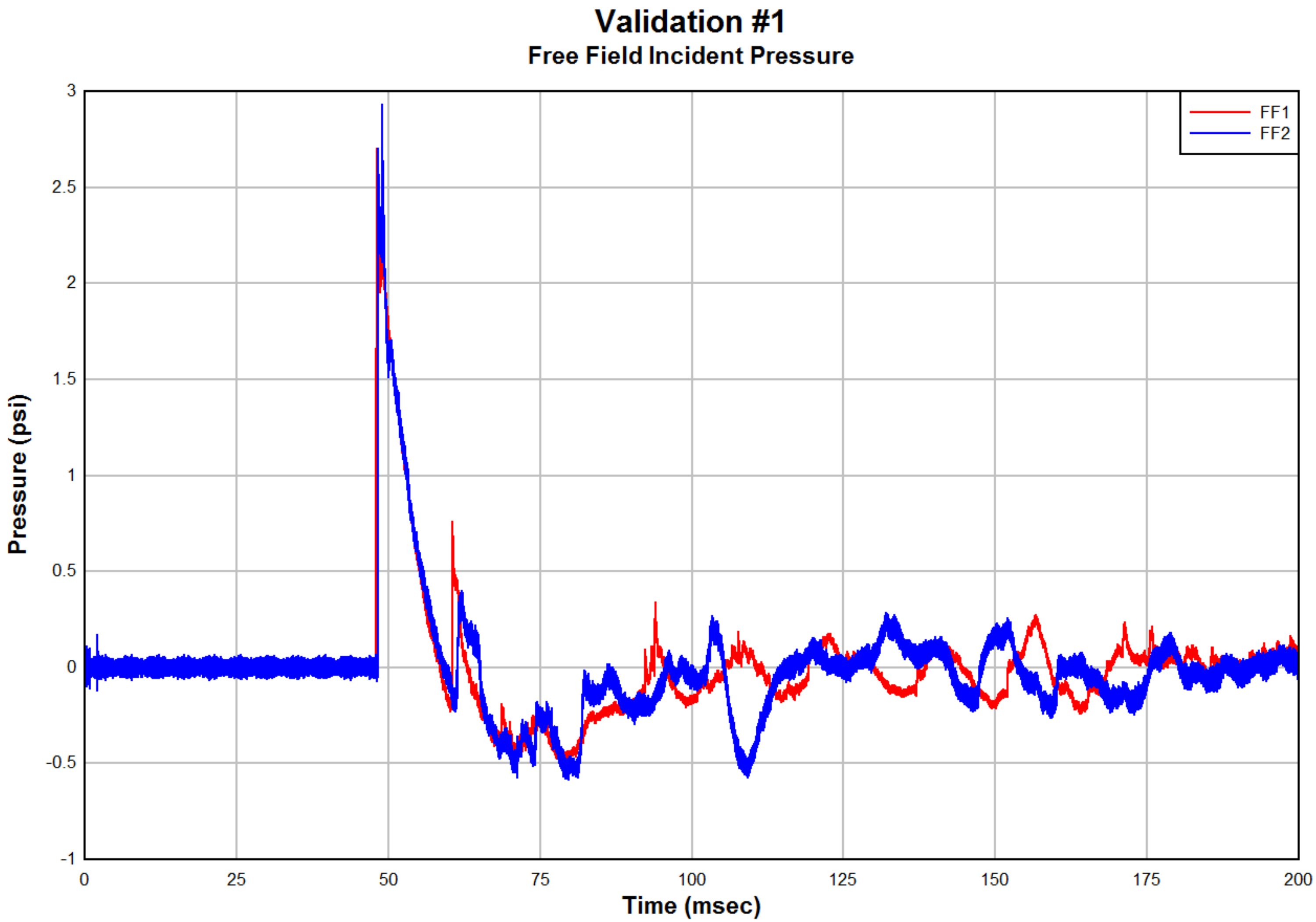


Validation #1:
Free Field Incident Pressure Gauges

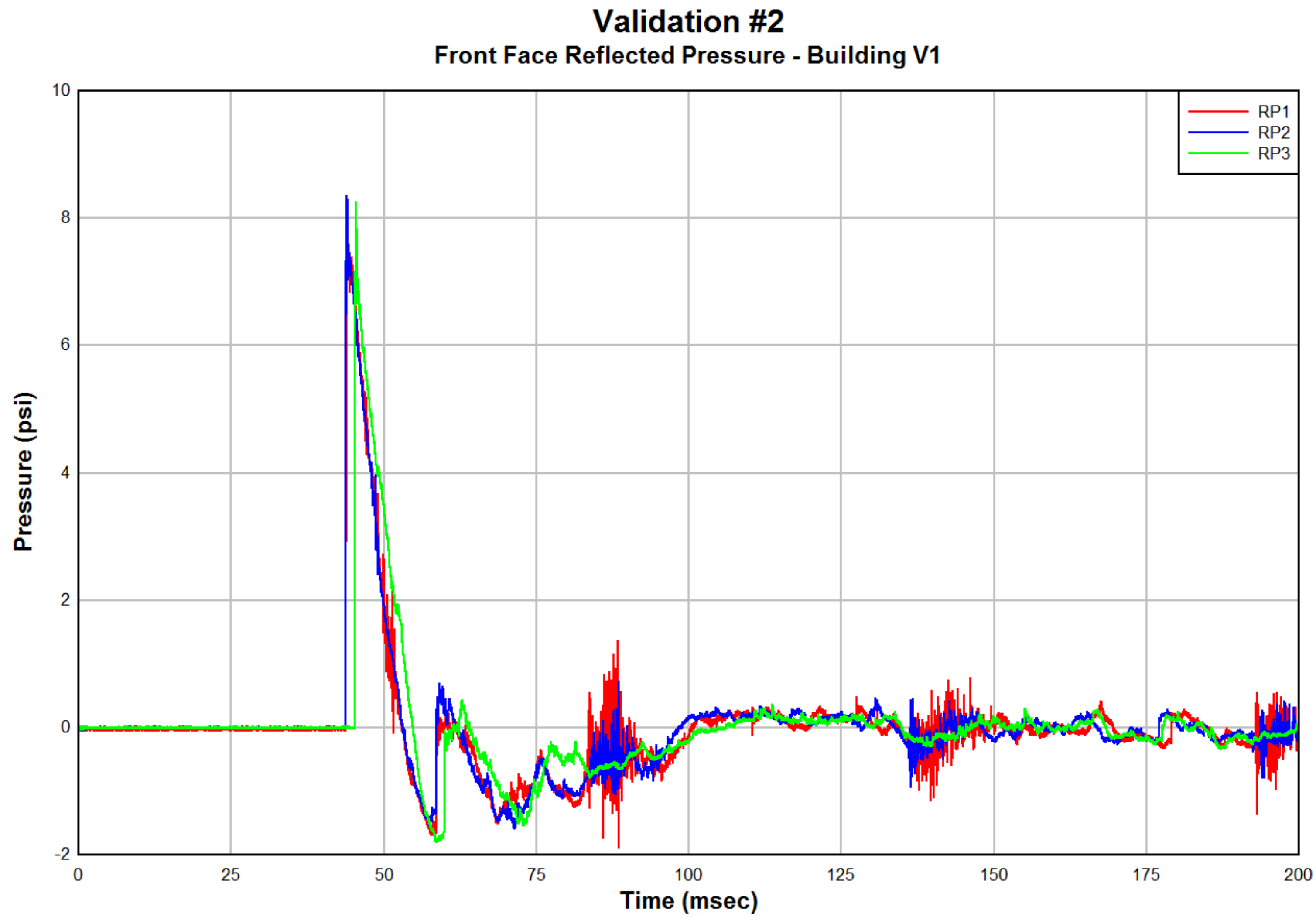


Full-Scale Validation #2

Pre-test

Post-test

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



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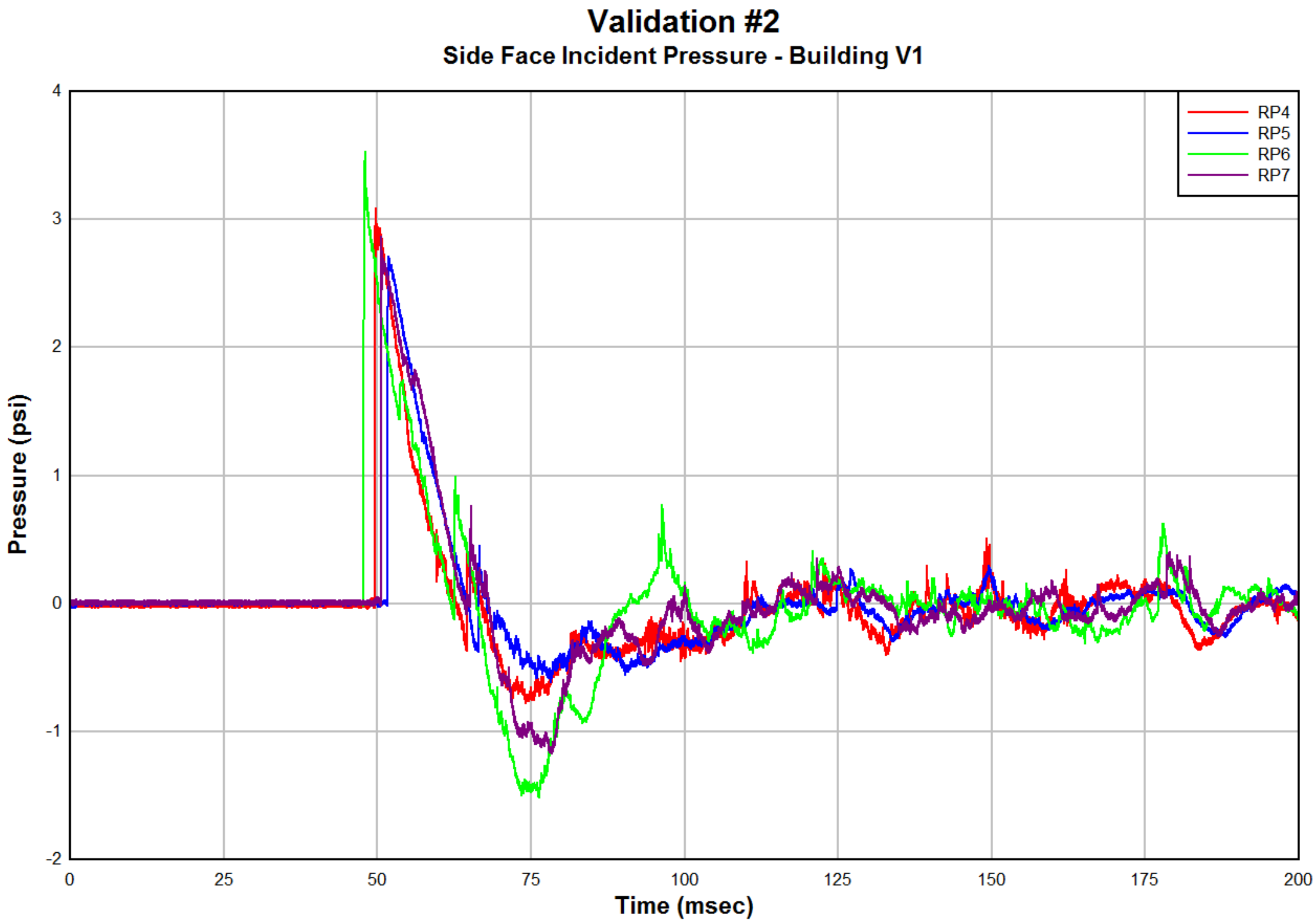
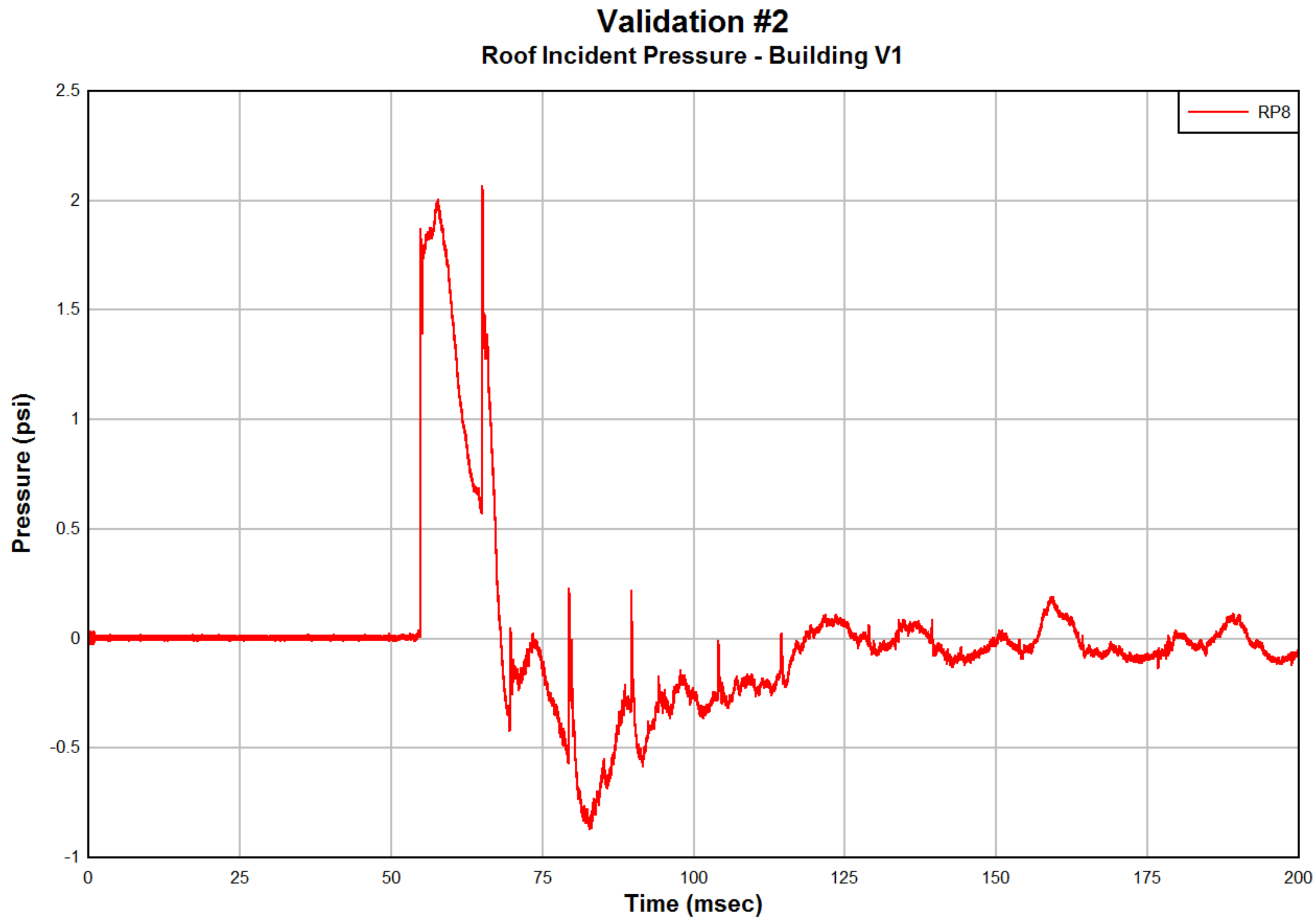
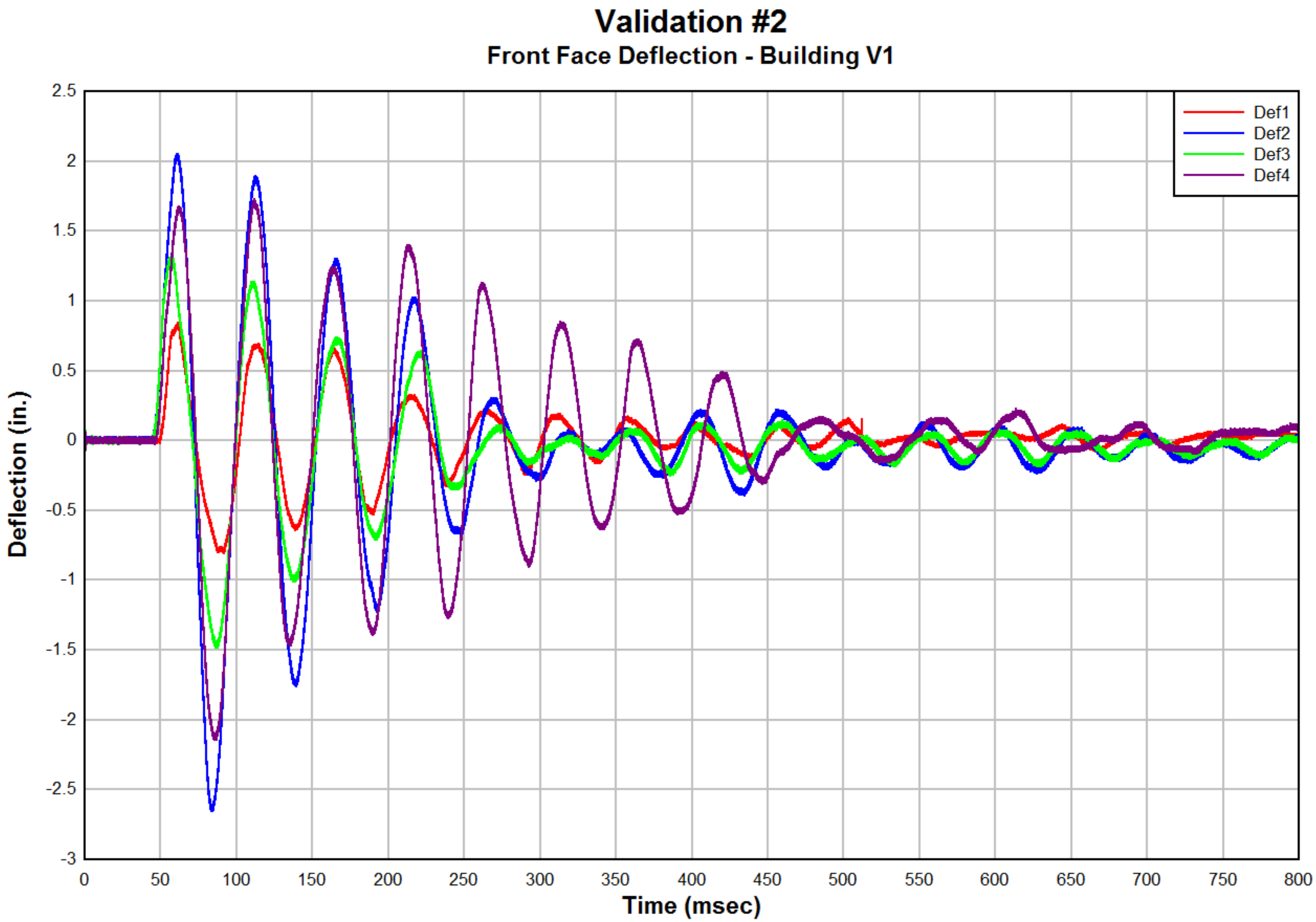


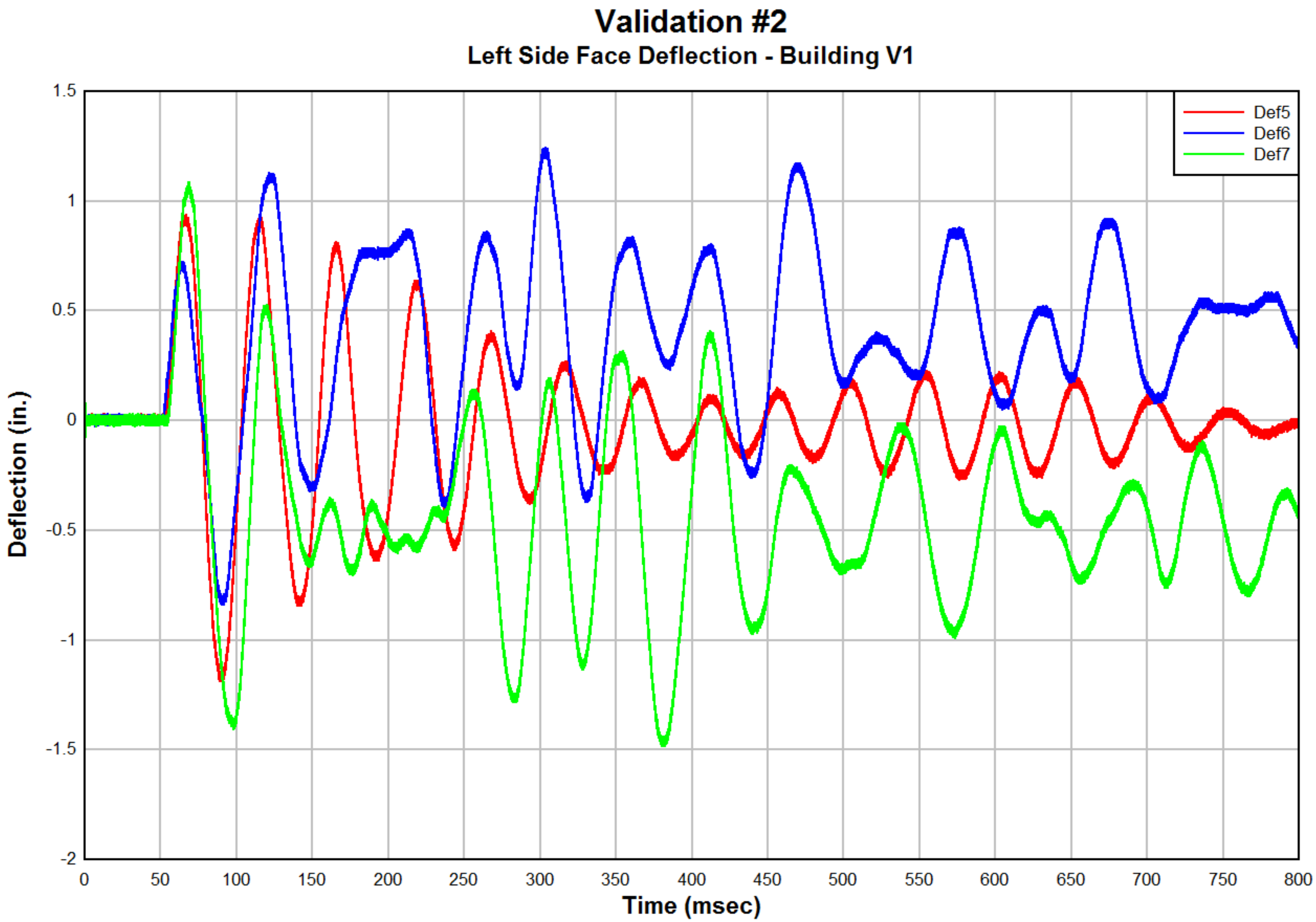
Validation #2 – Building V1: Roof Incident Pressure Gauges



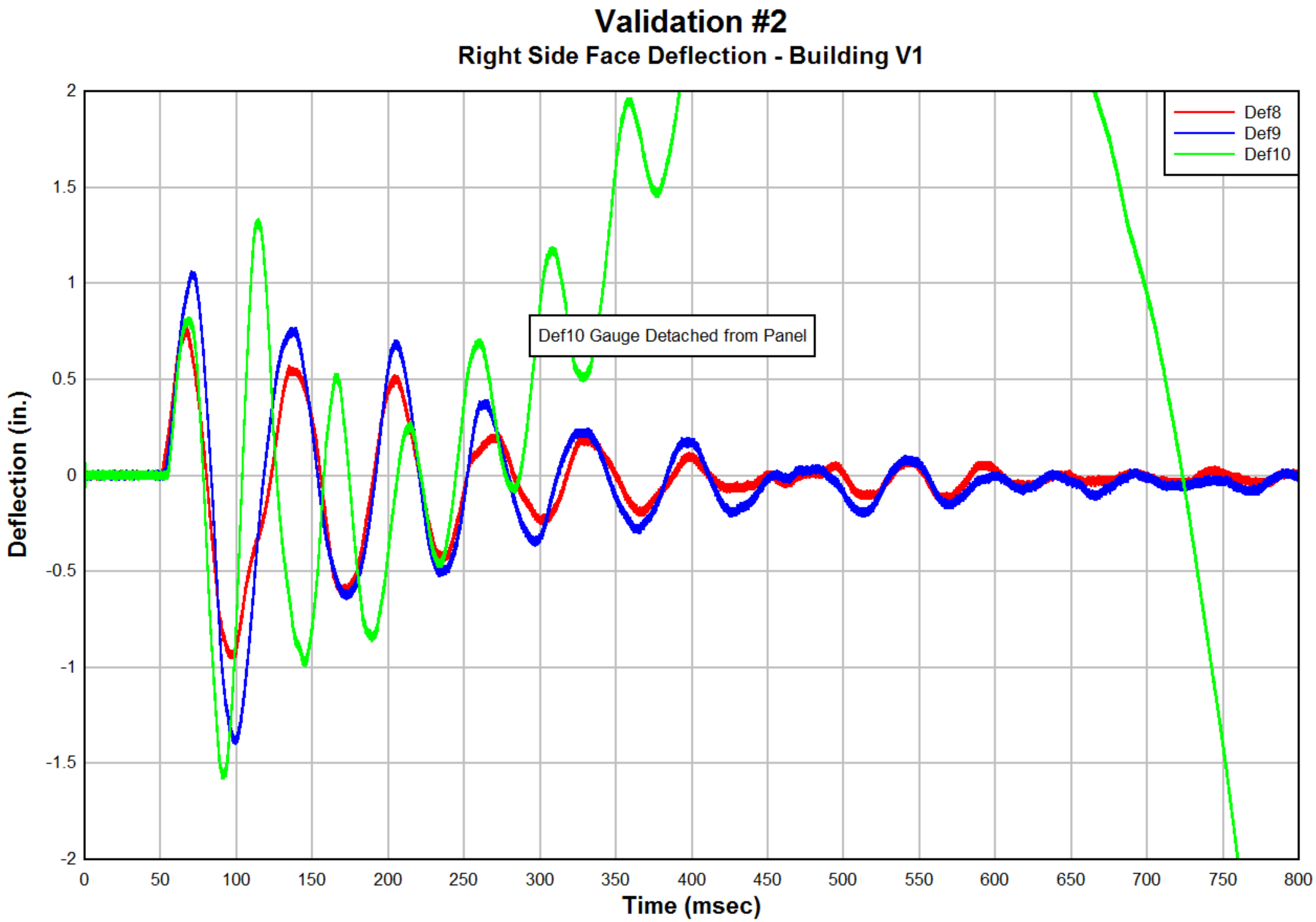
Validation #2 – Building V1: Front Face Deflection Gauges



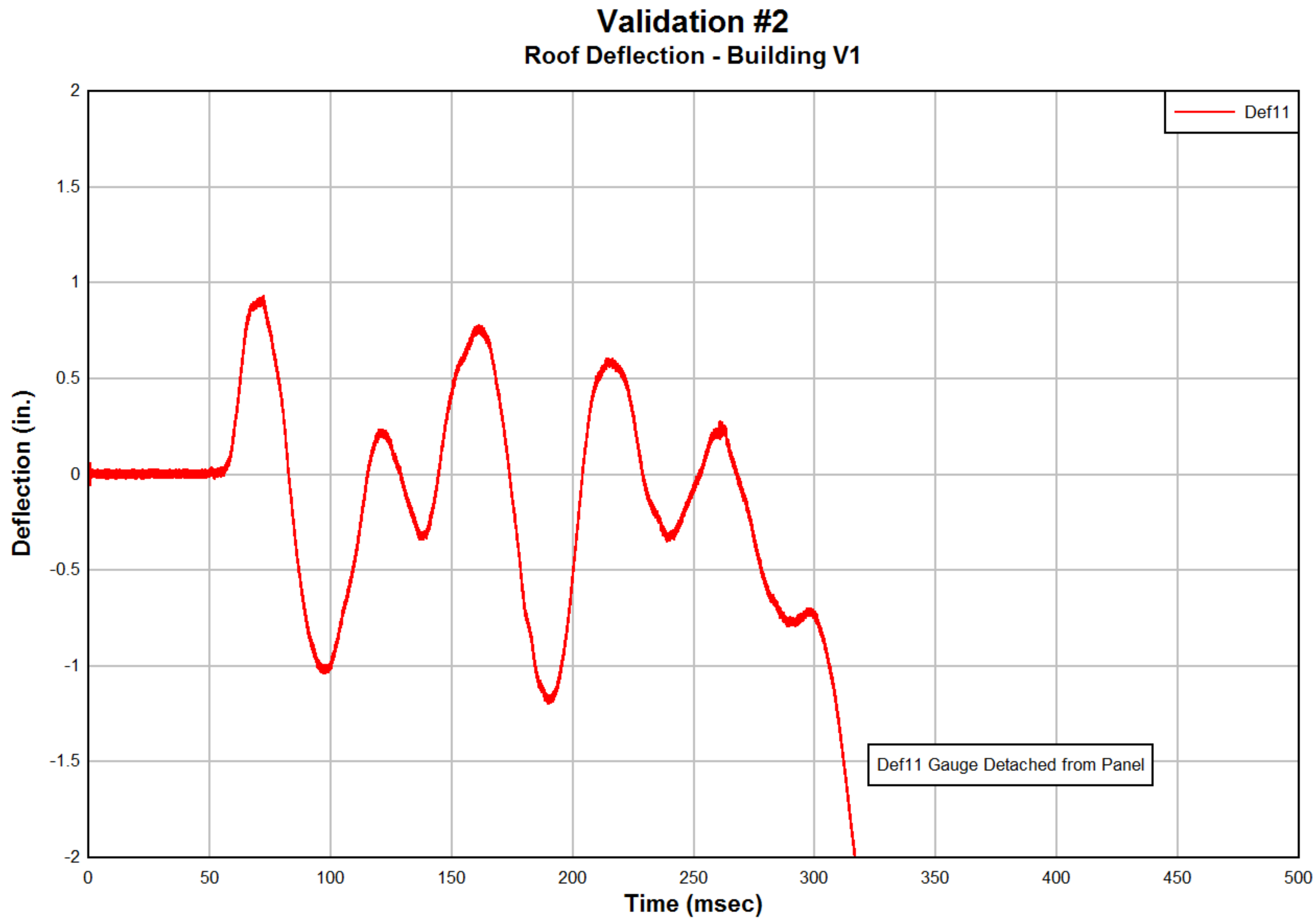
Validation #2 – Building V1: Left Side Face Deflection Gauges



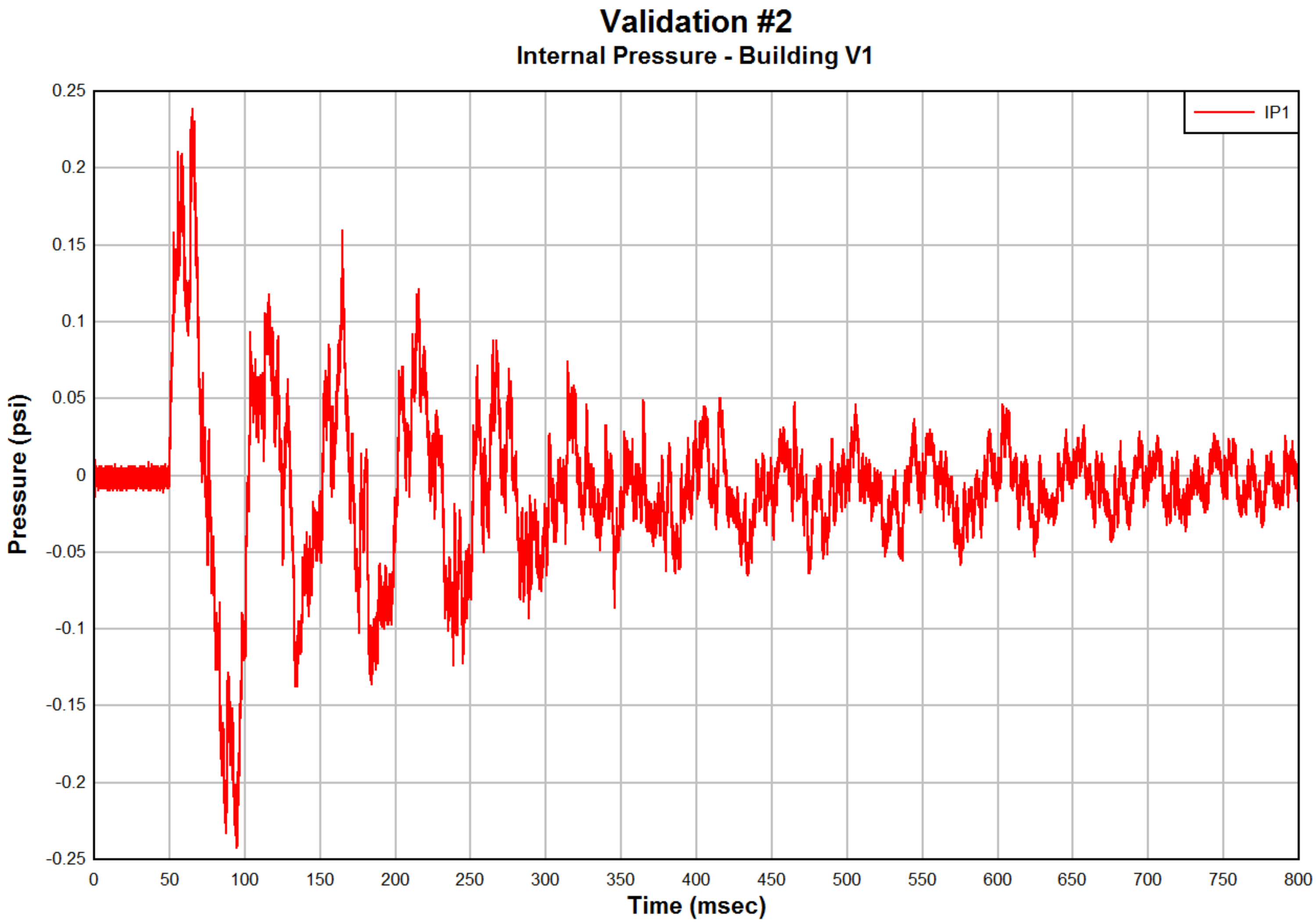
Validation #2 – Building V1: Right Side Face Deflection Gauges



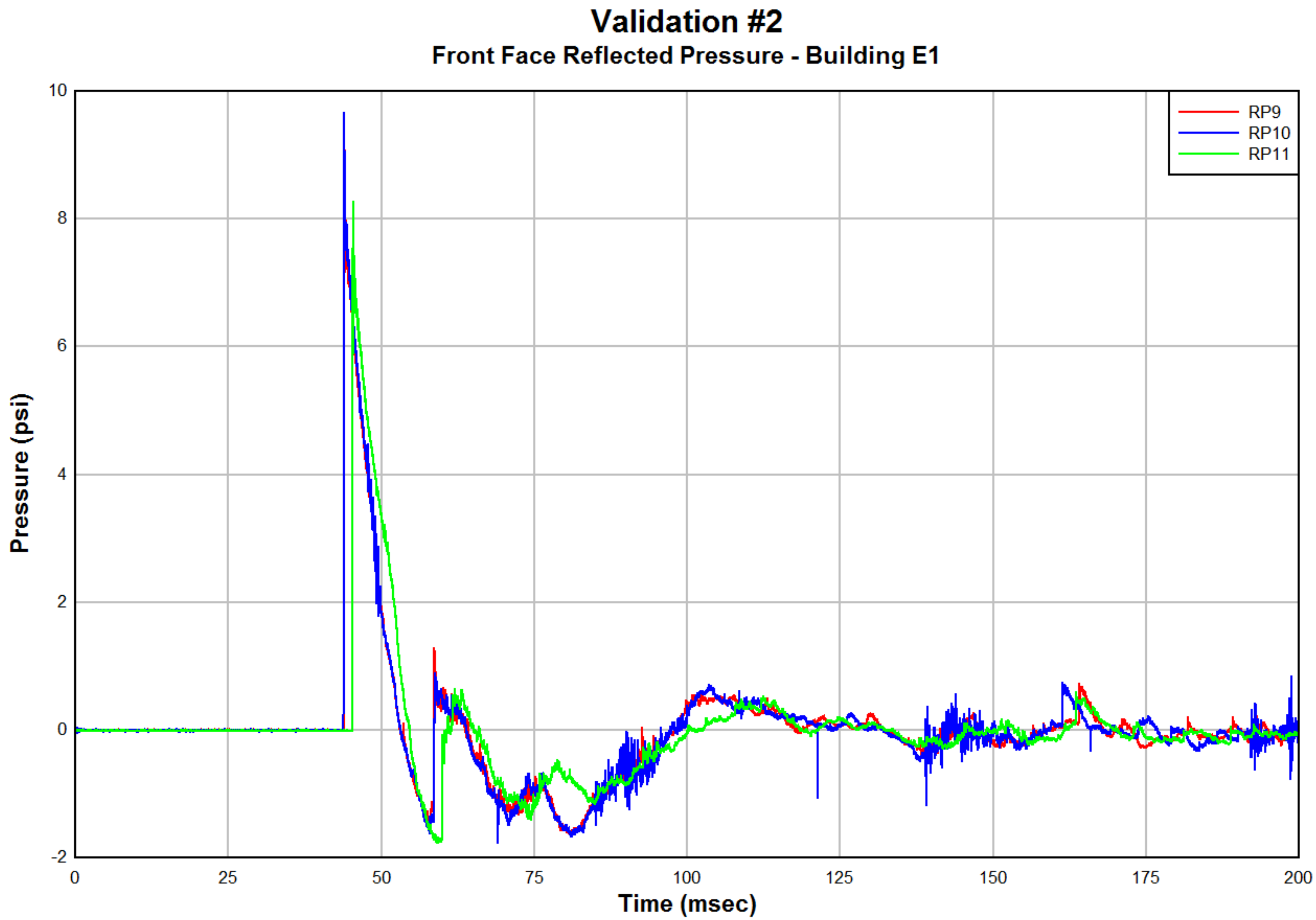
Validation #2 – Building V1: Roof Deflection Gauge



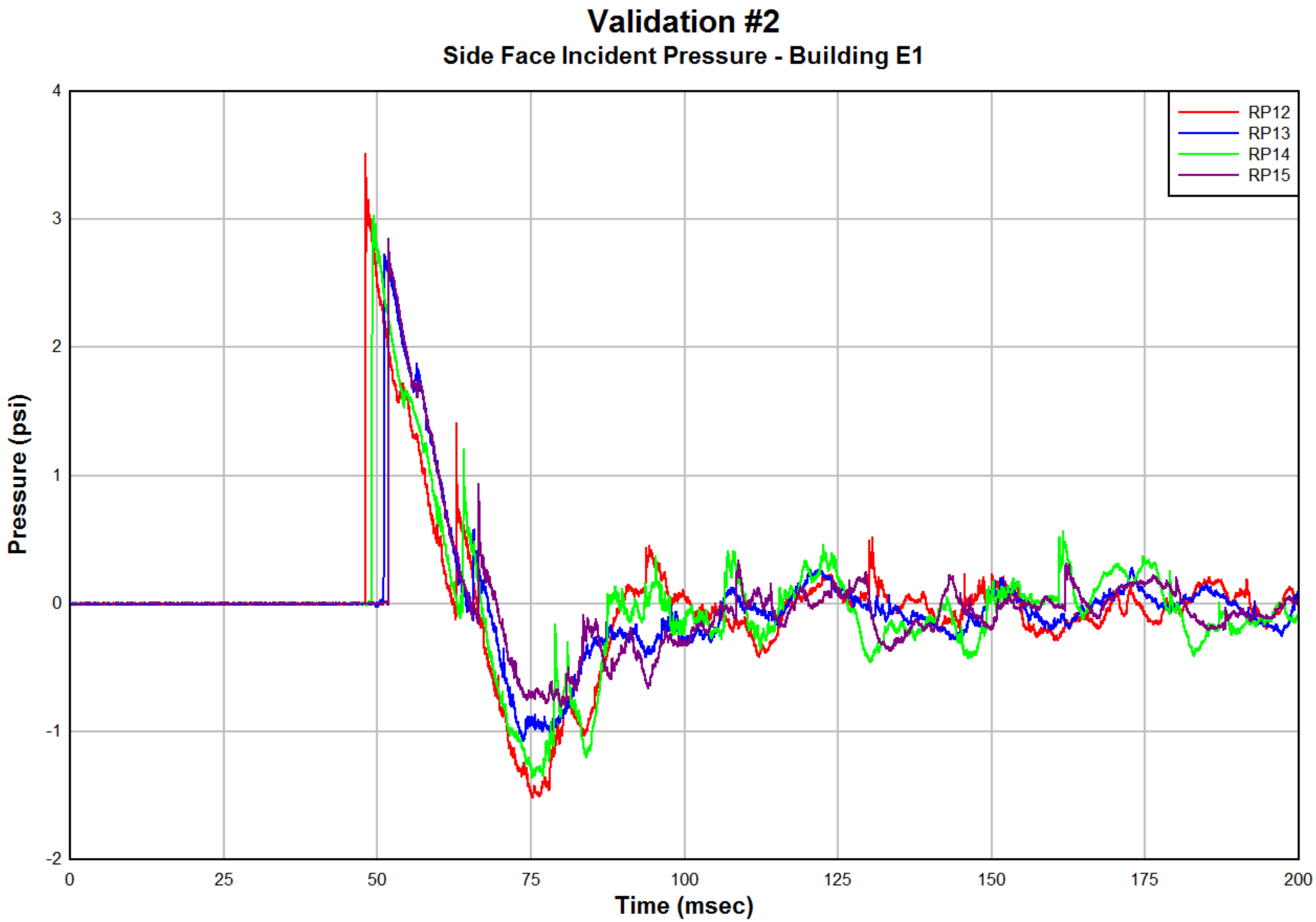
Validation #2 – Building V1: Internal Pressure Gauge



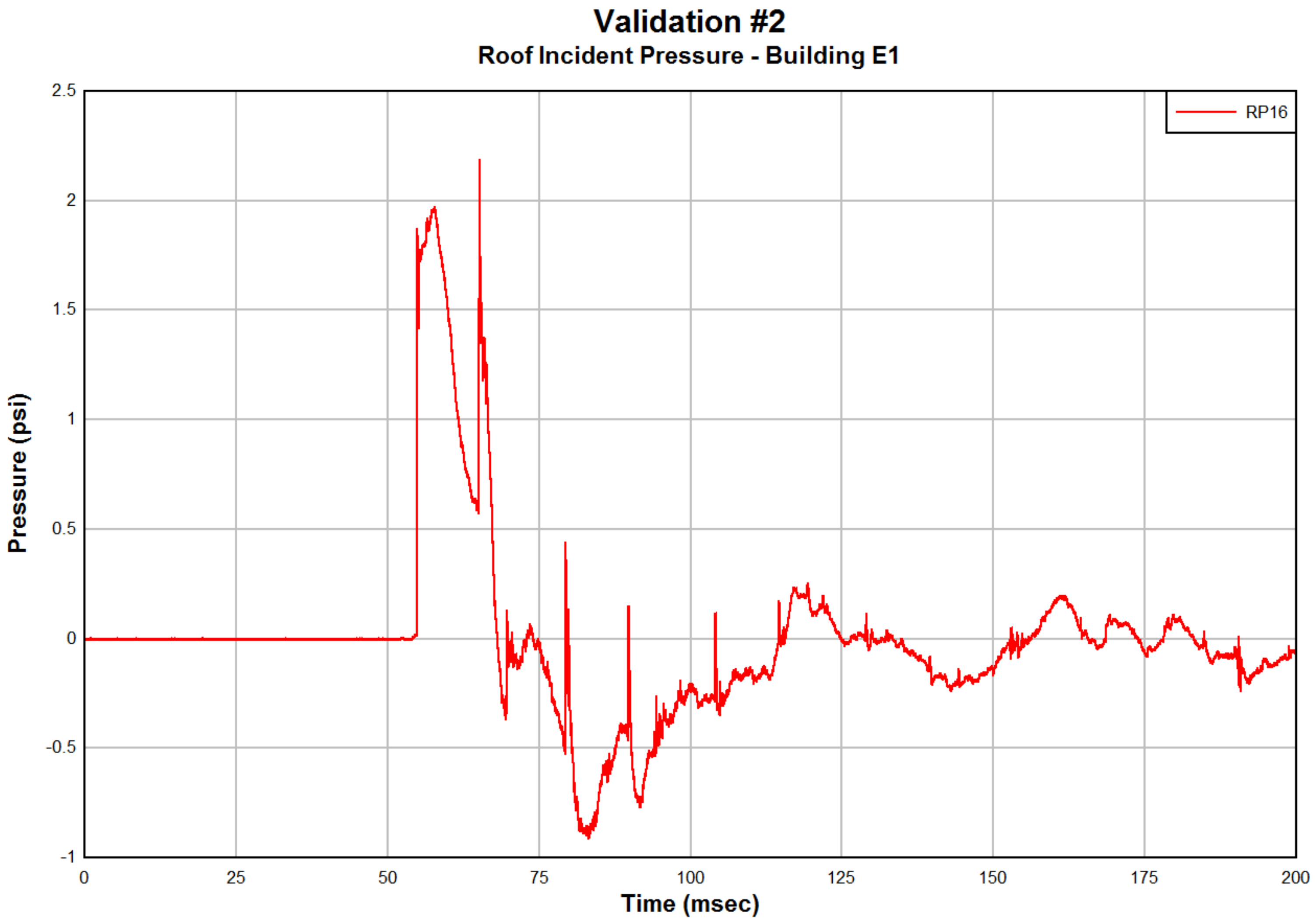
Validation #2 – Building E1: Front Face Reflected Pressure Gauges



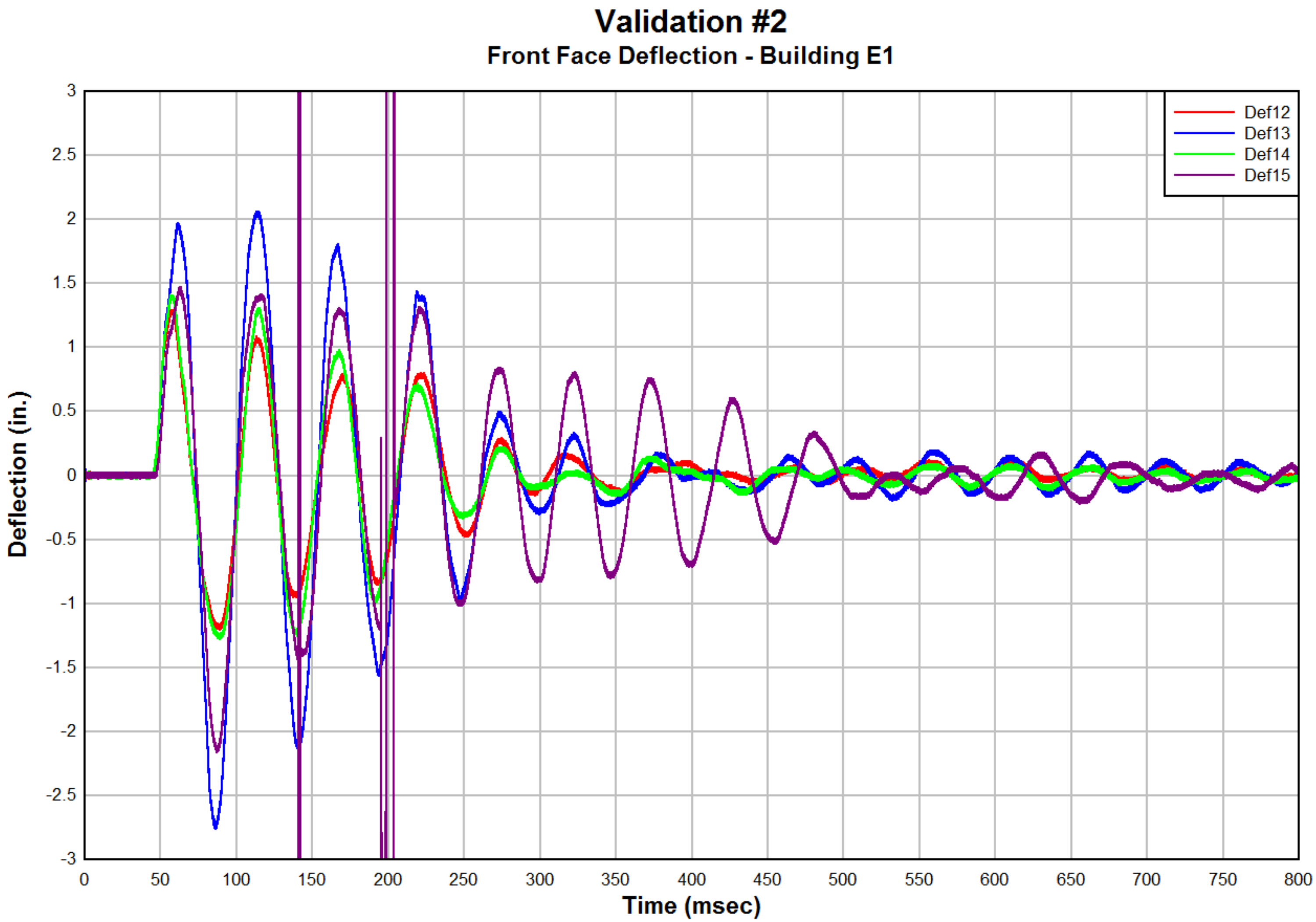
Validation #2 – Building E1: Side Face Incident Pressure Gauges



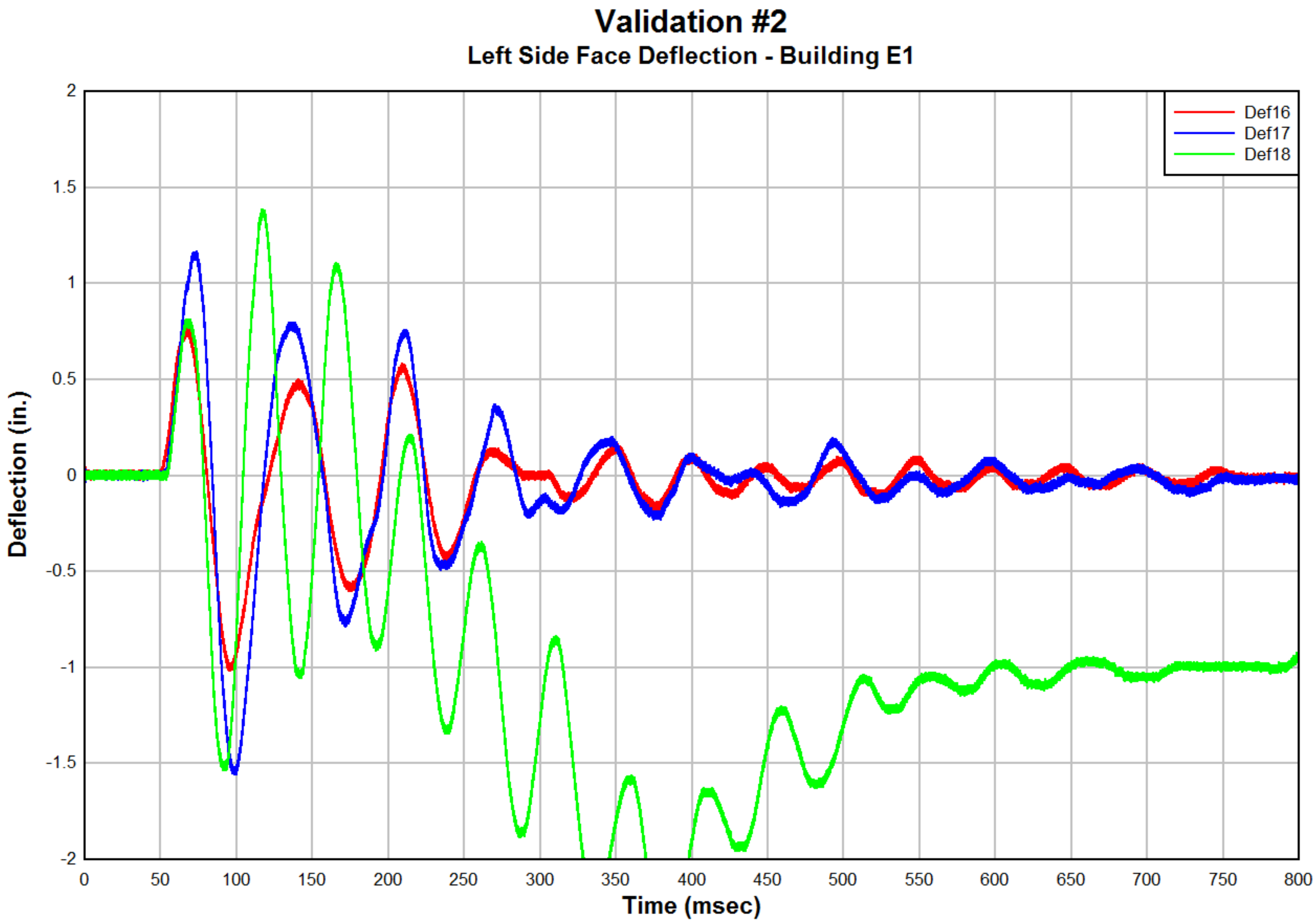
Validation #2 – Building E1: Roof Pressure Gauge



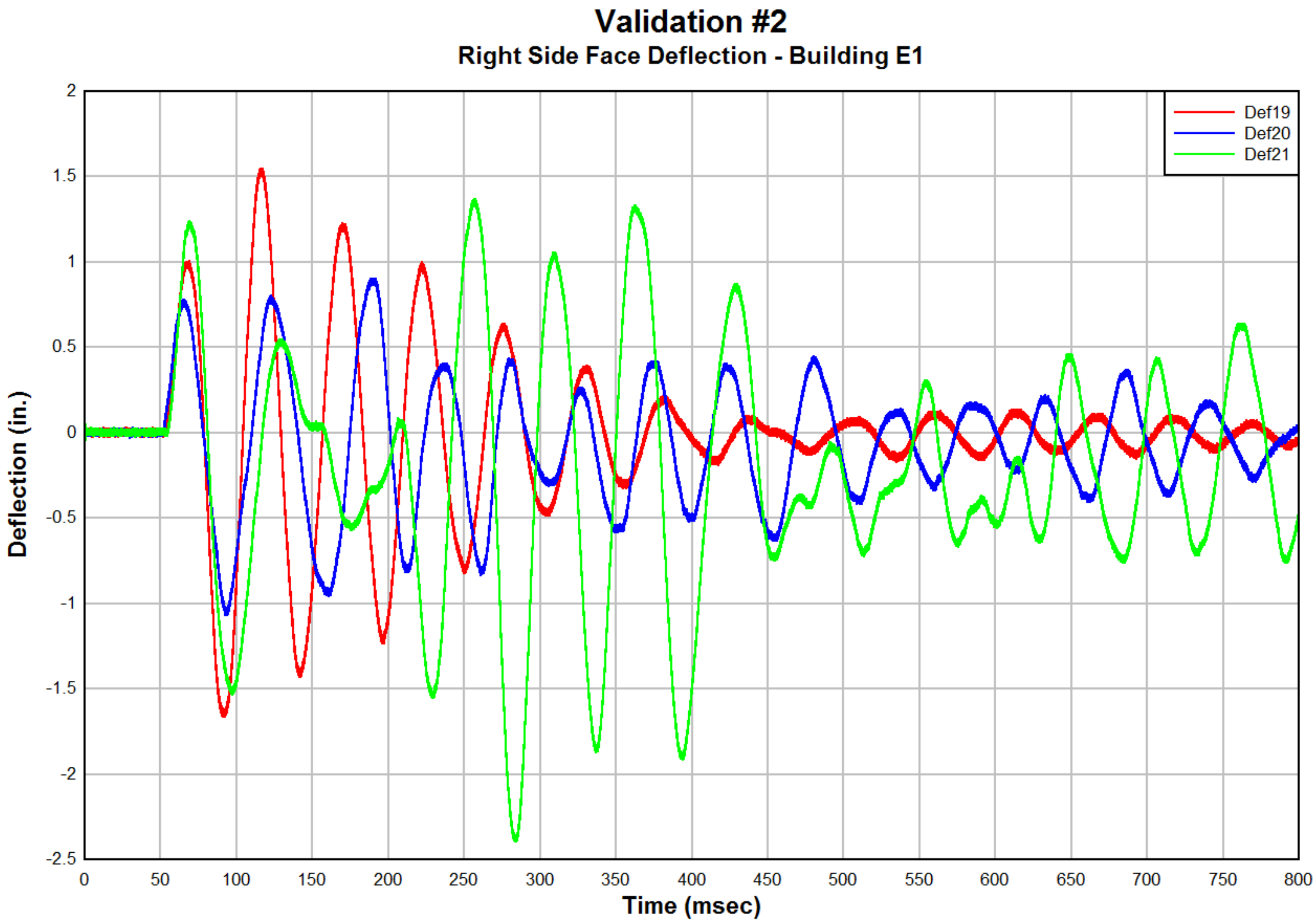
Validation #2 – Building E1: Front Face Deflection Gauges



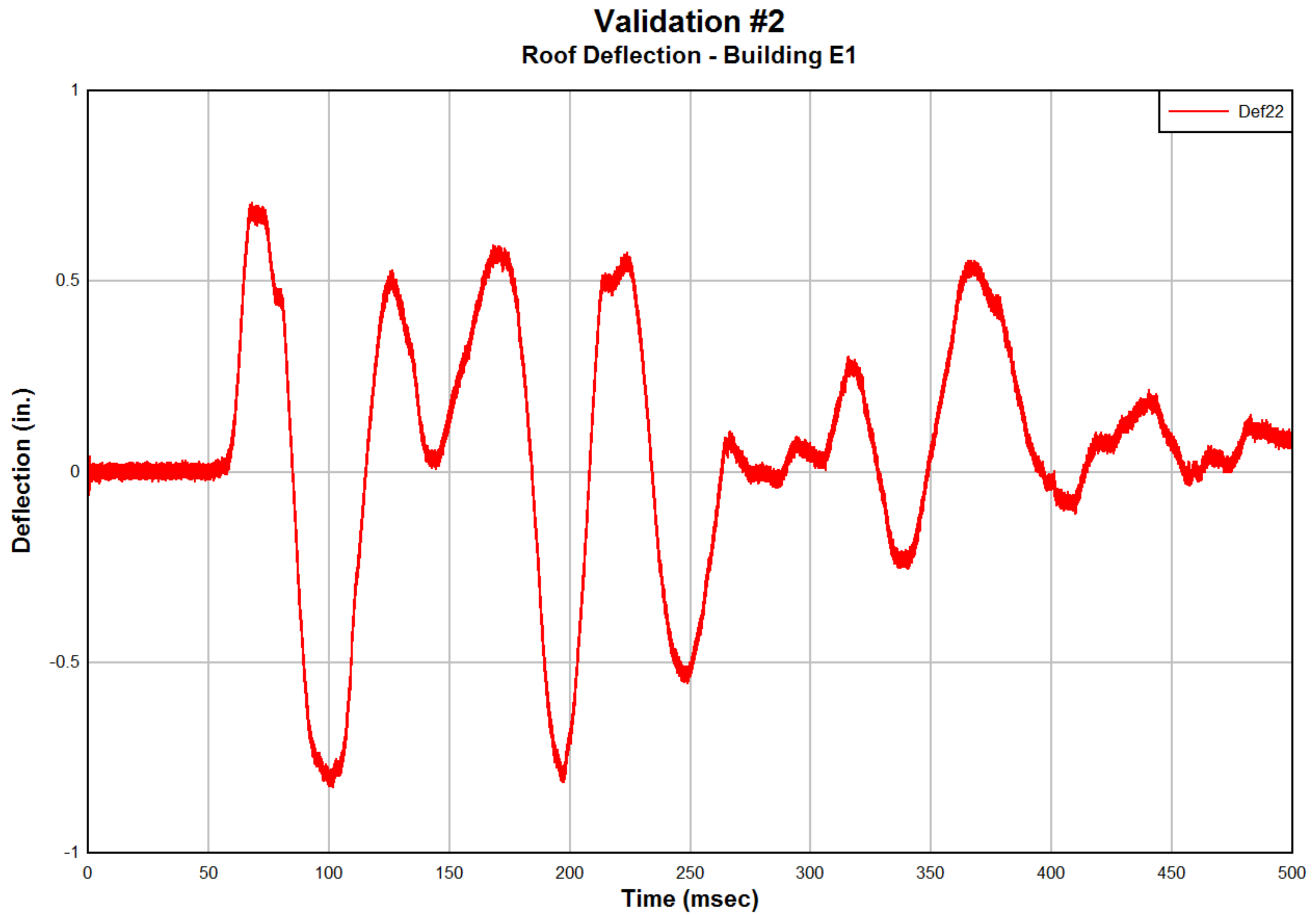
Validation #2 – Building E1: Left Side Face Deflection Gauges



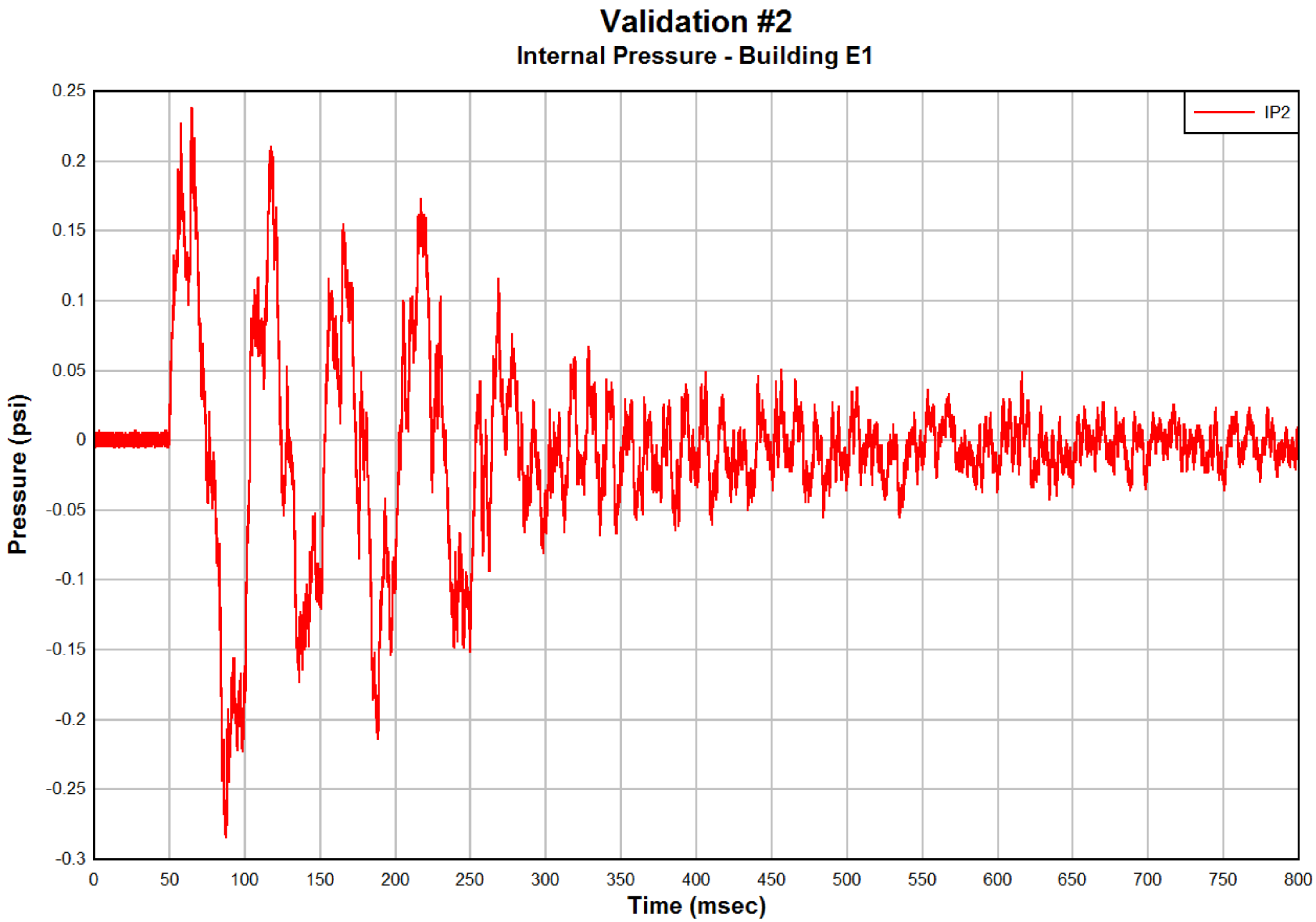
Validation #2 – Building E1: Right Side Face Deflection Gauges



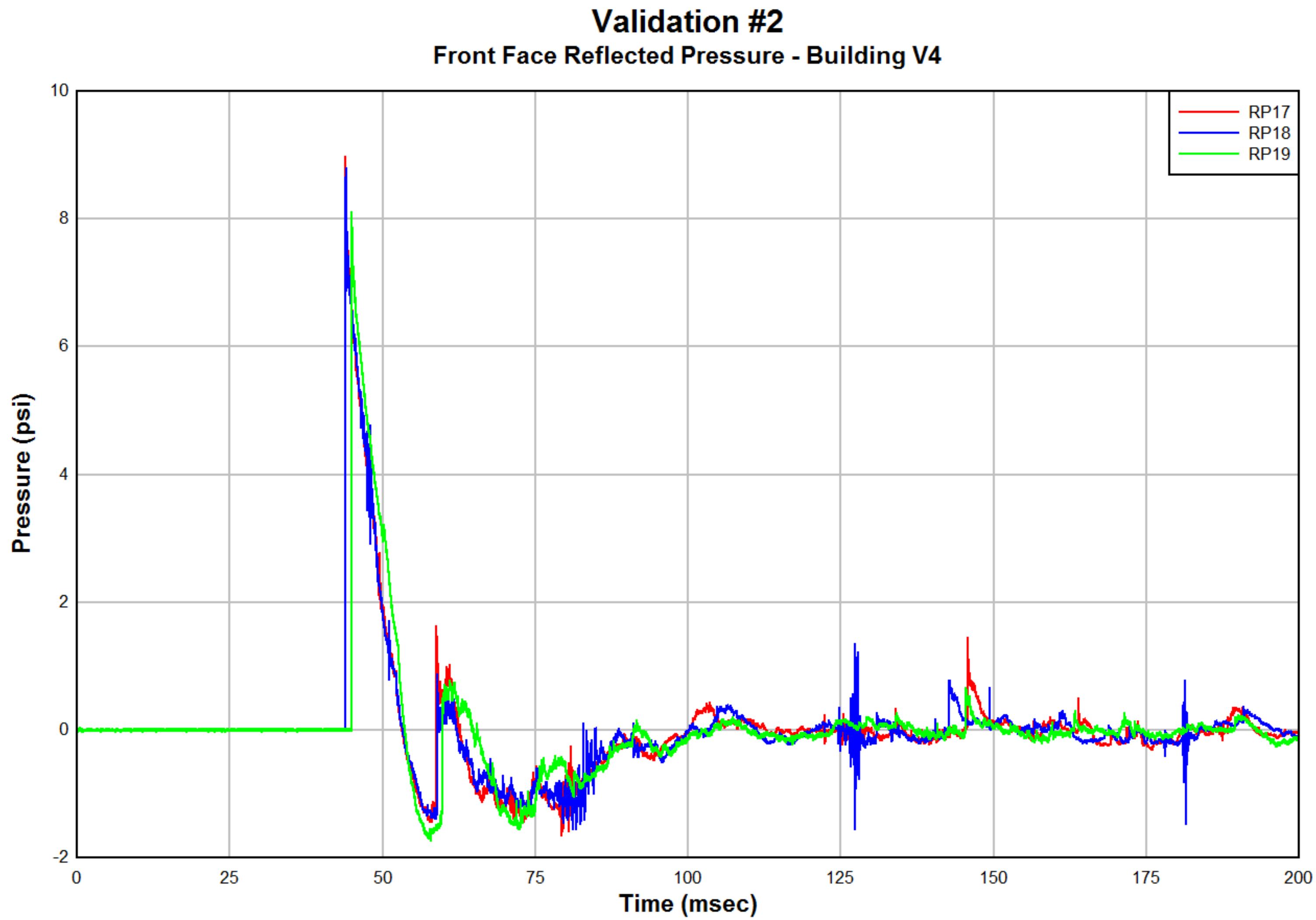
Validation #2 – Building E1: Roof Deflection Gauge



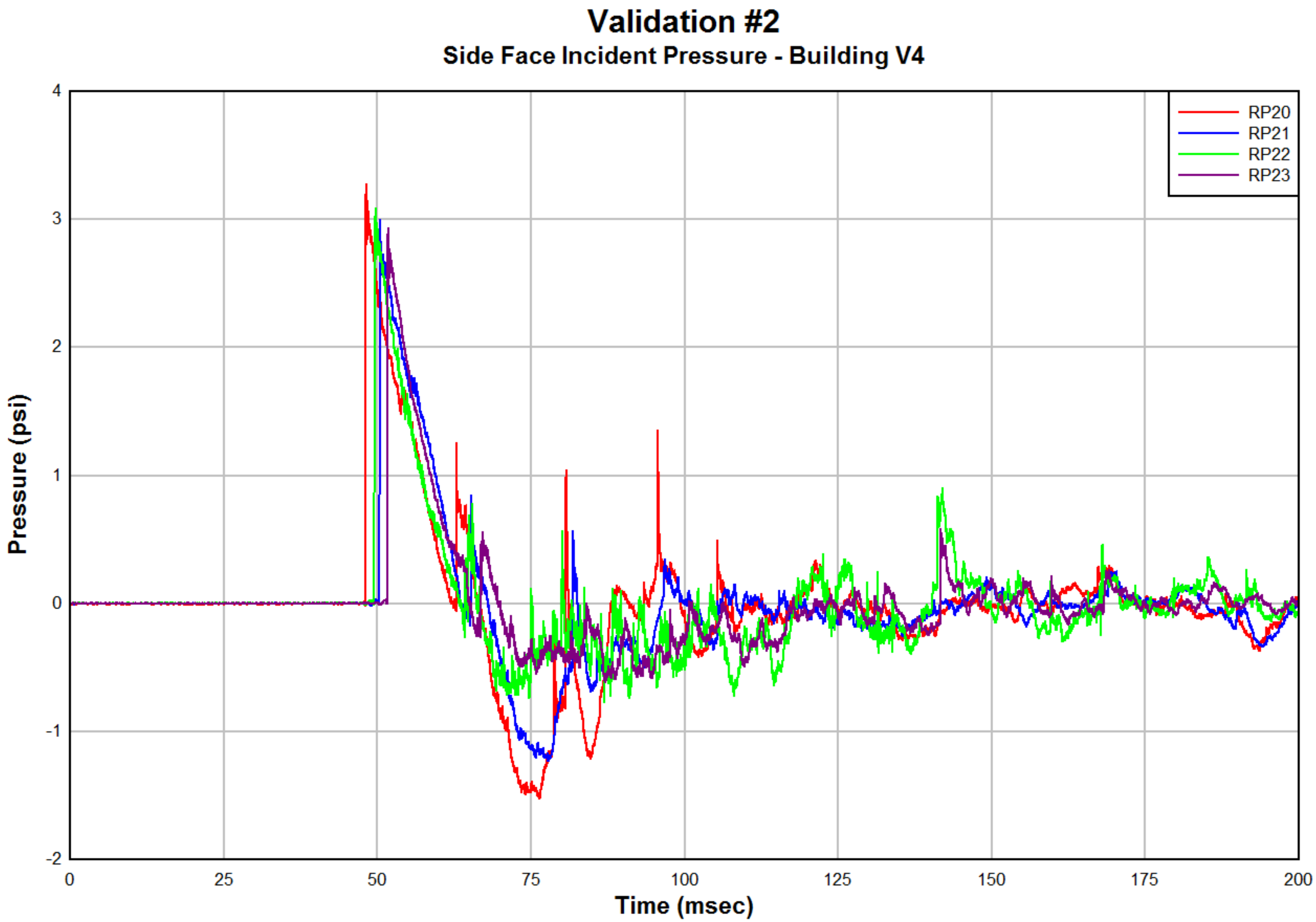
Validation #2– Building E1: Internal Pressure Gauge



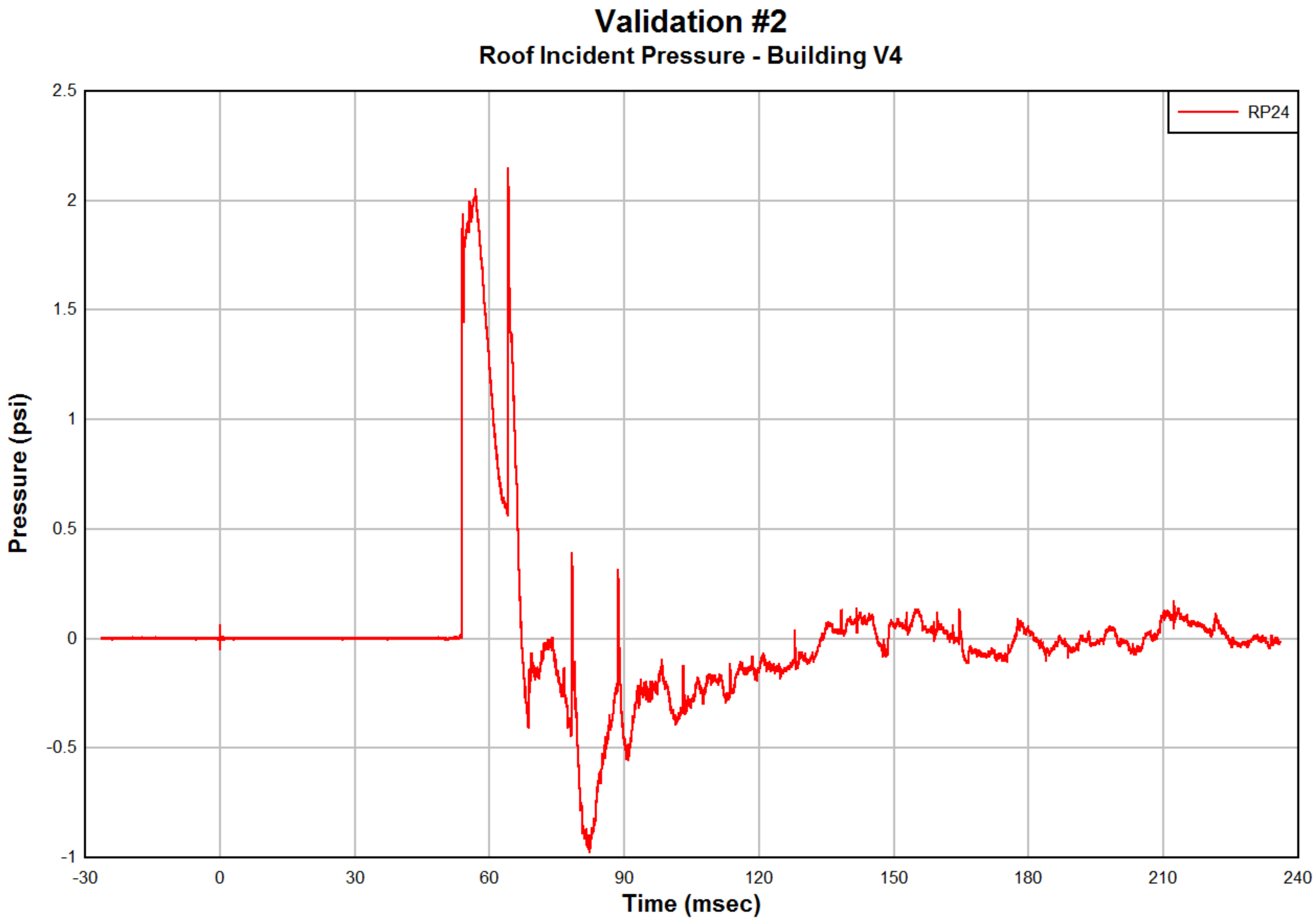
Validation #2 – Building V4: Front Face Reflected Pressure Gauges



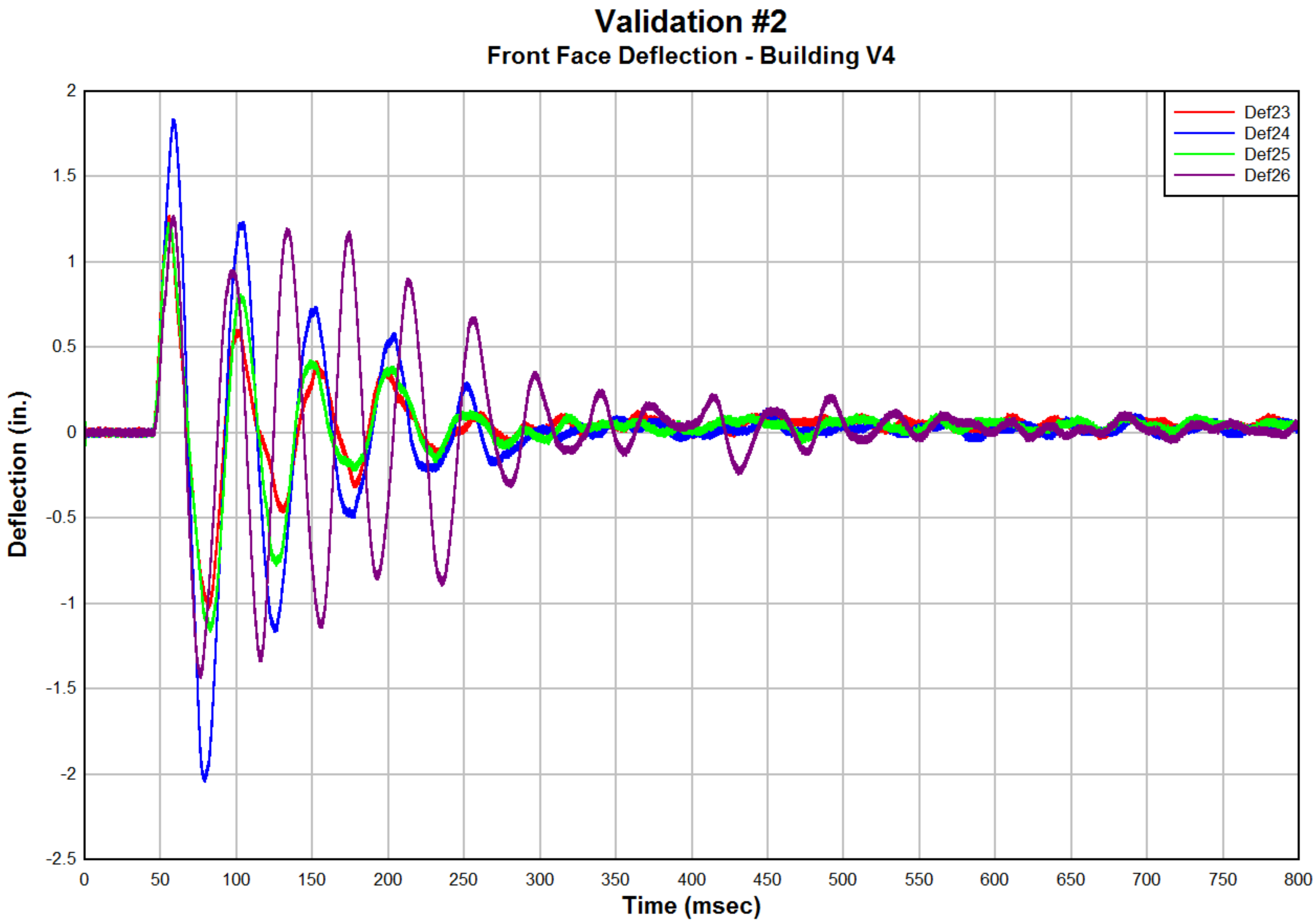
Validation #2 – Building V4: Side Face Incident Pressure Gauges



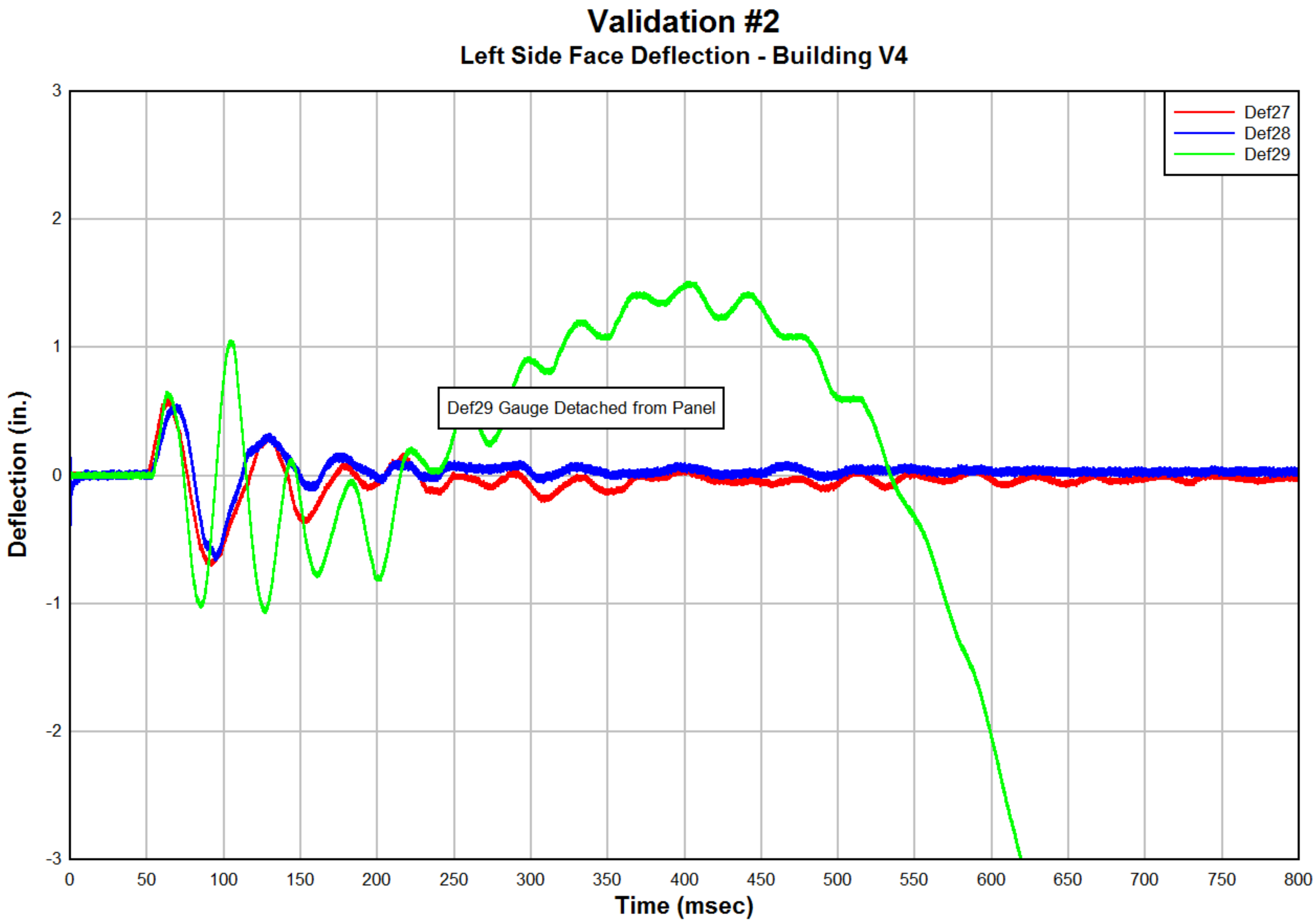
Validation #2 – Building V4: Roof Pressure Gauge



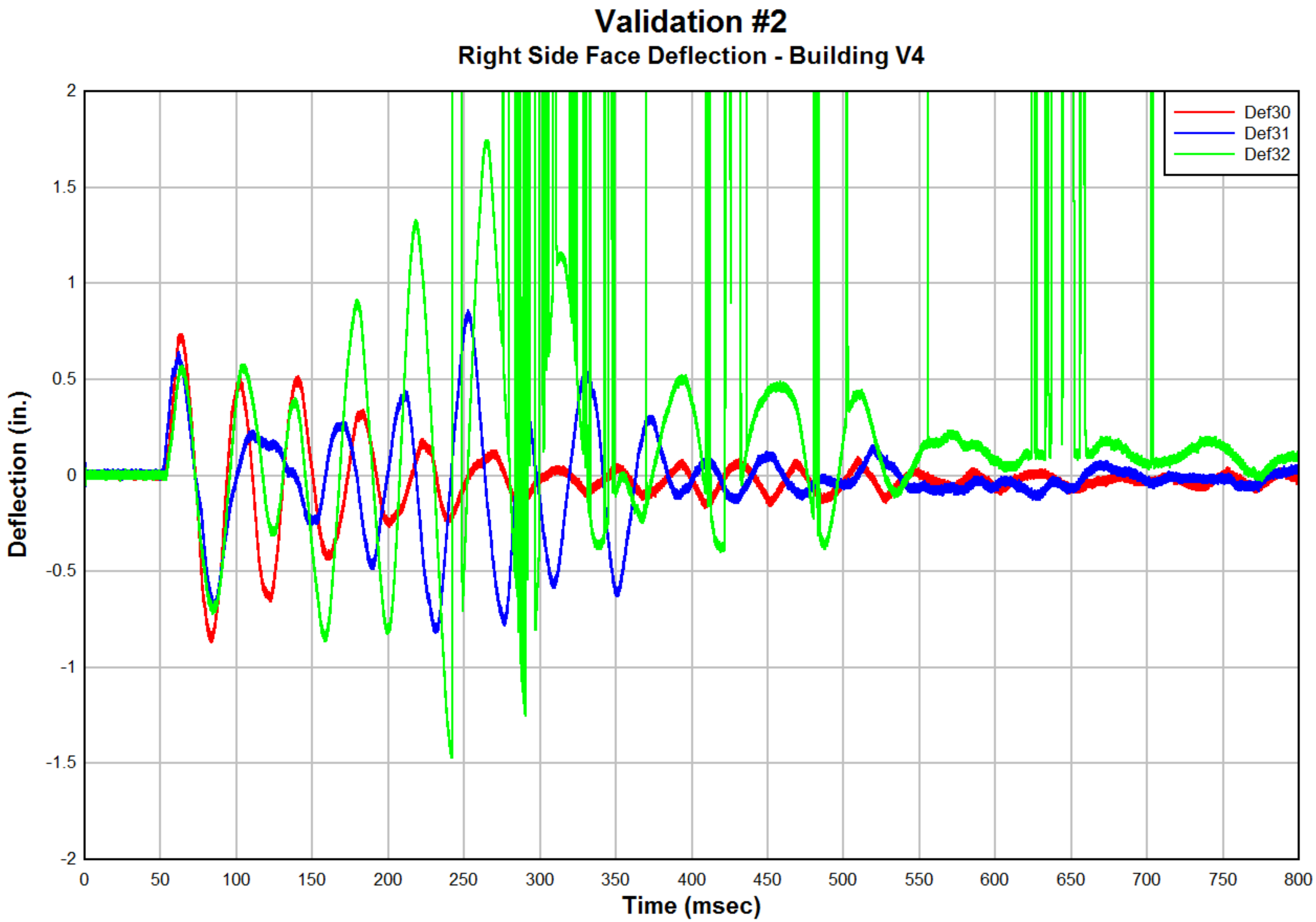
Validation #2 – Building V4: Front Face Deflection Gauges



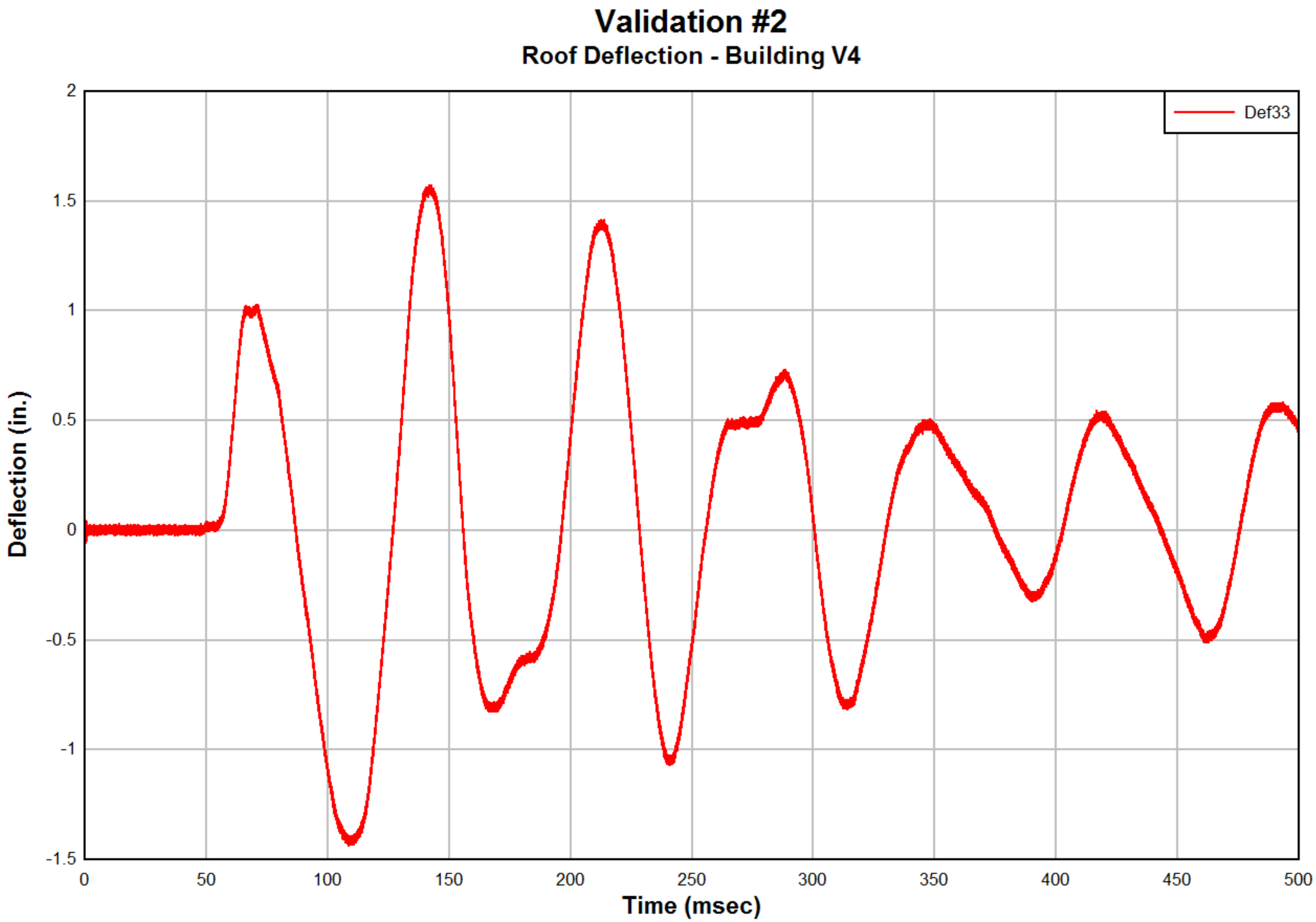
Validation #2 – Building V4: Left Side Face Deflection Gauges



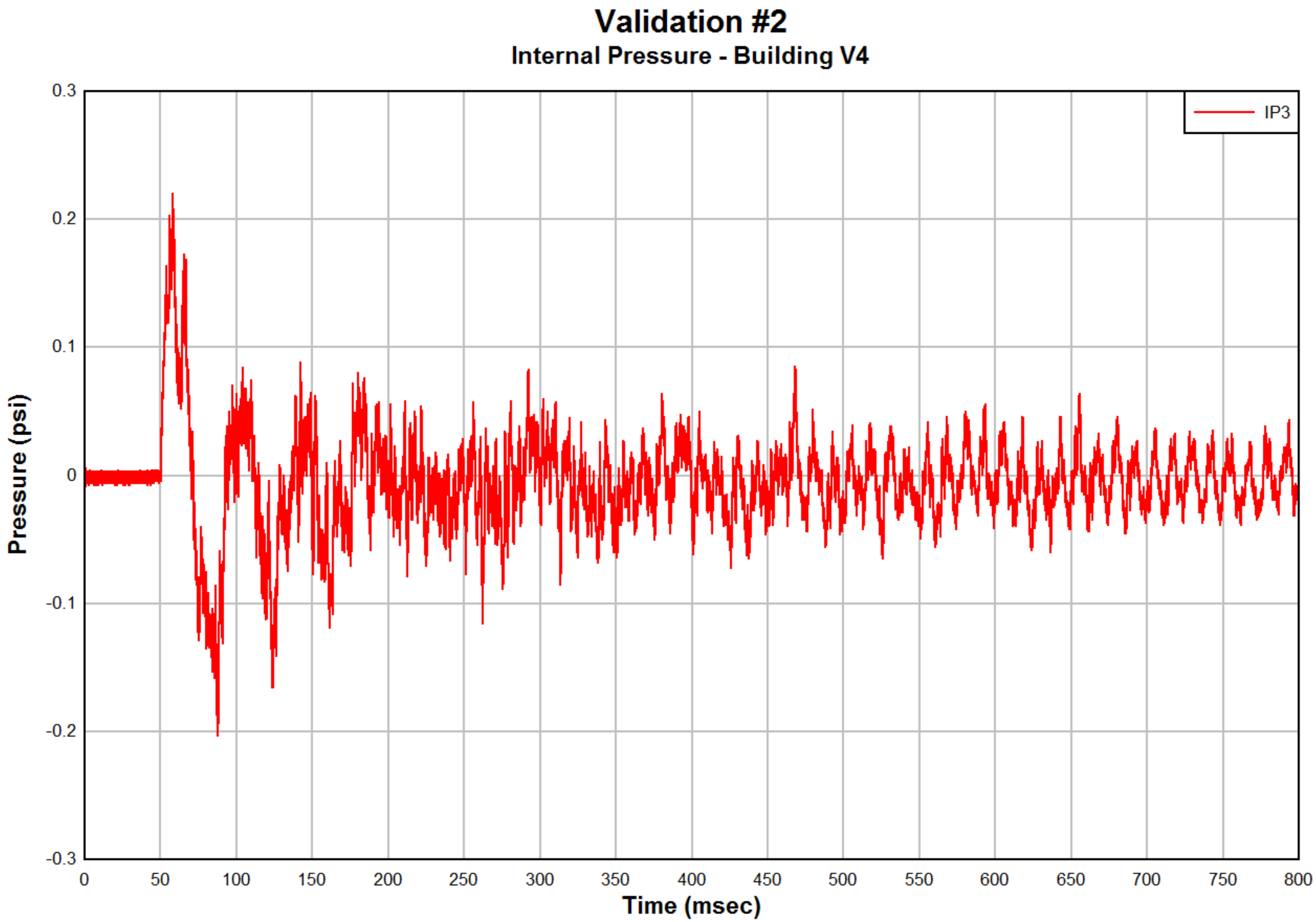
Validation #2 – Building V4: Right Side Face Deflection Gauges



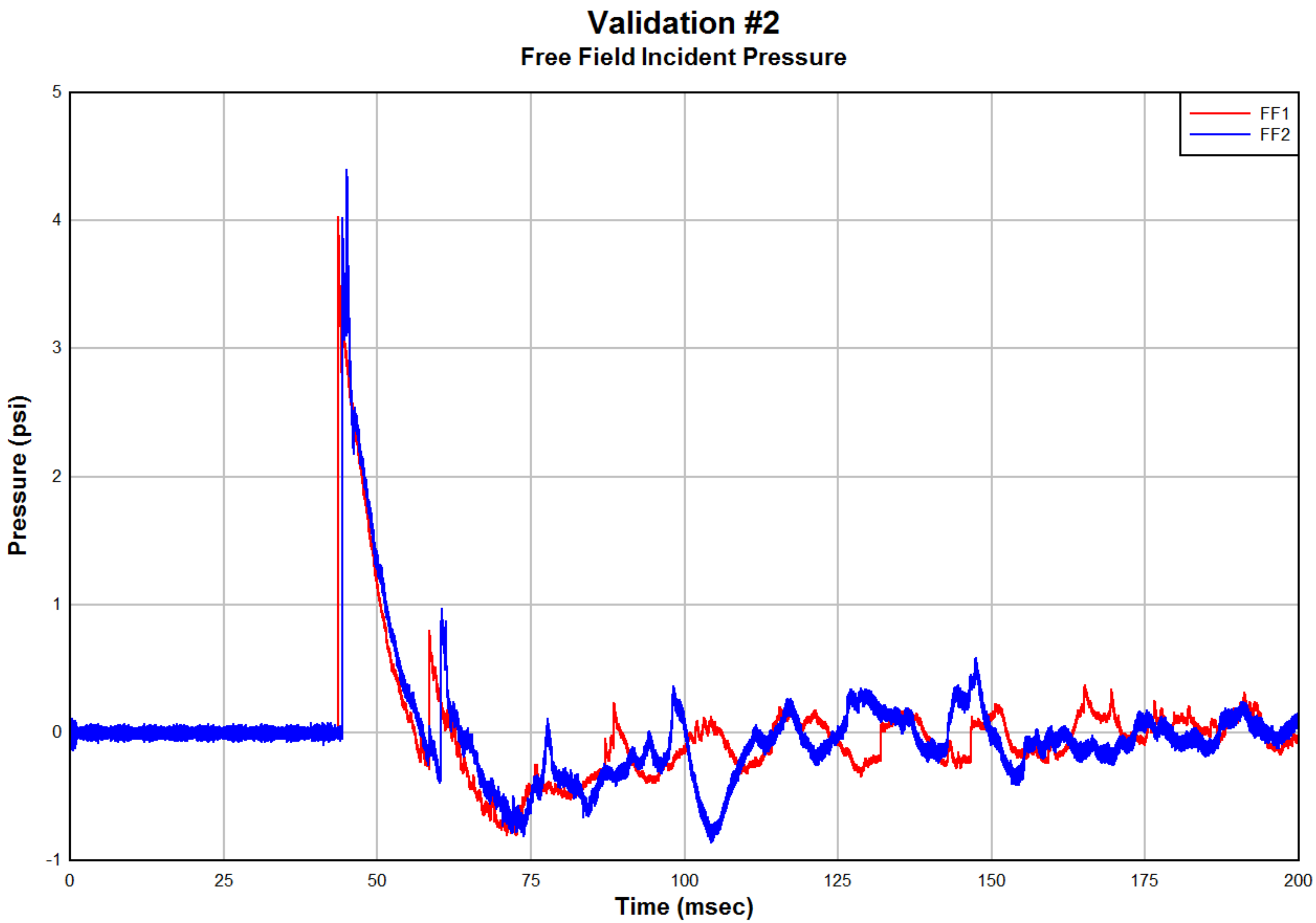
Validation #2 – Building V4: Roof Deflection Gauge



Validation #2 – Building V4: Internal Pressure Gauge



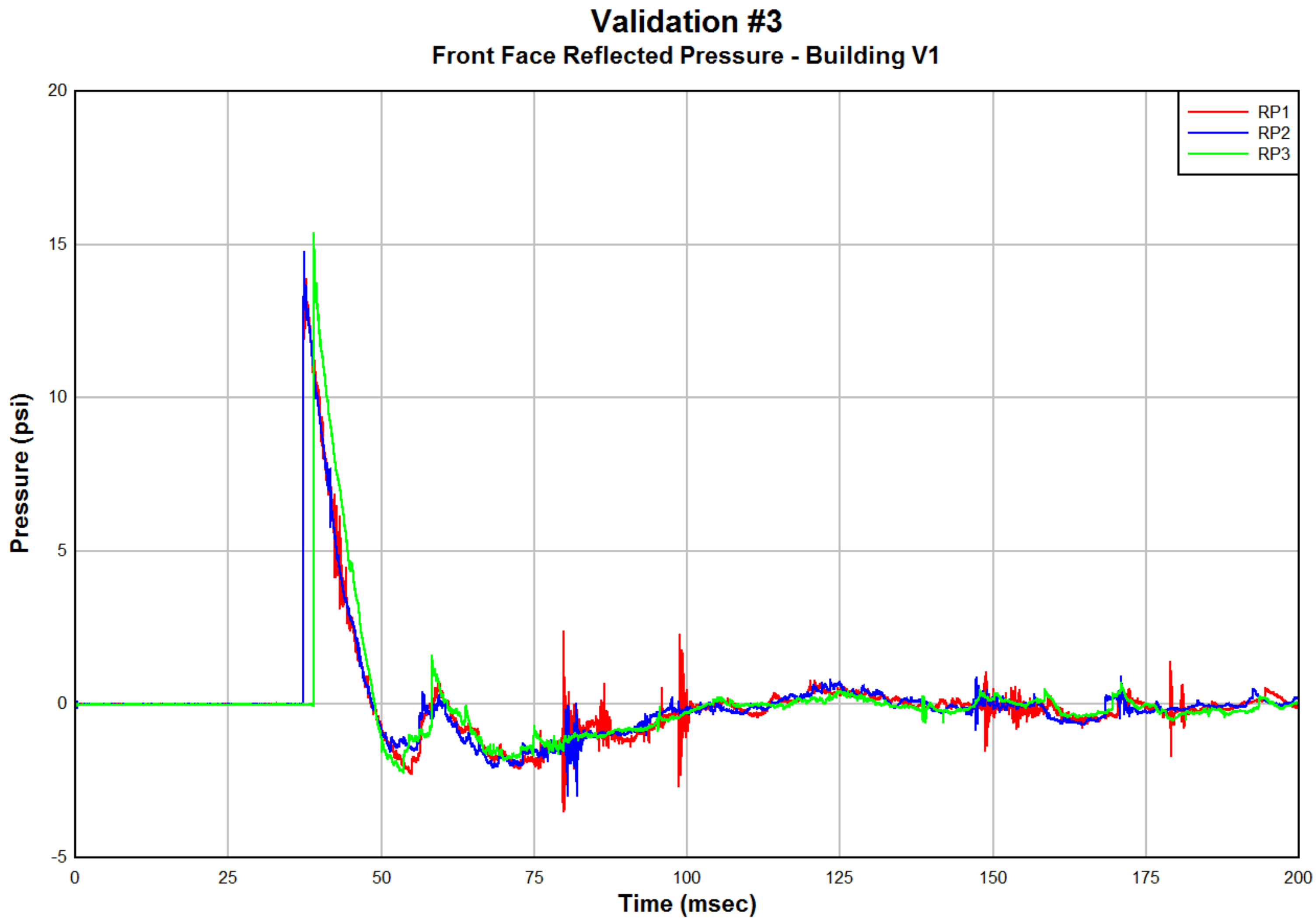
Validation #2: Free Field Incident Pressure Gauges



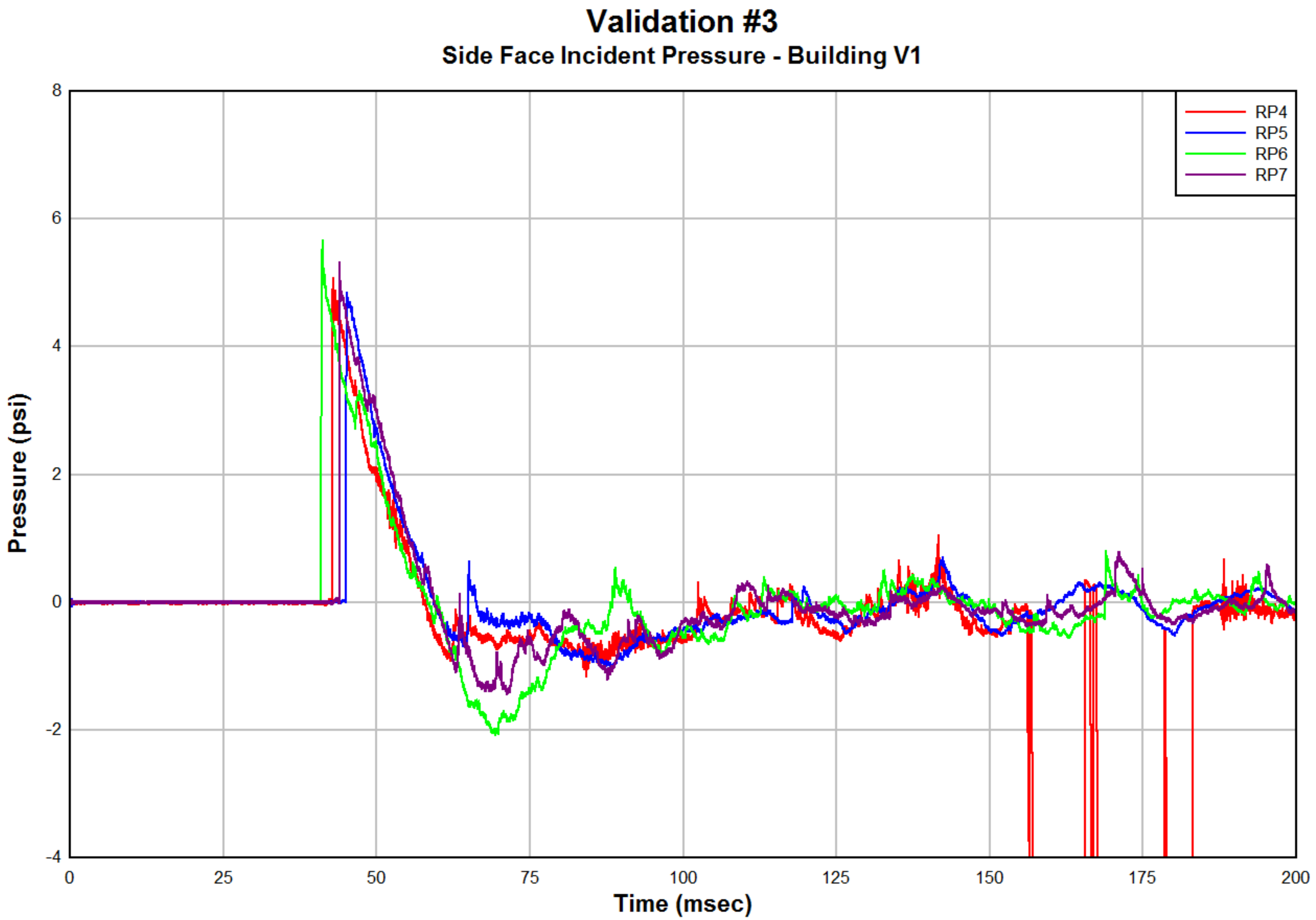
Full-Scale Validation #3

Post-test

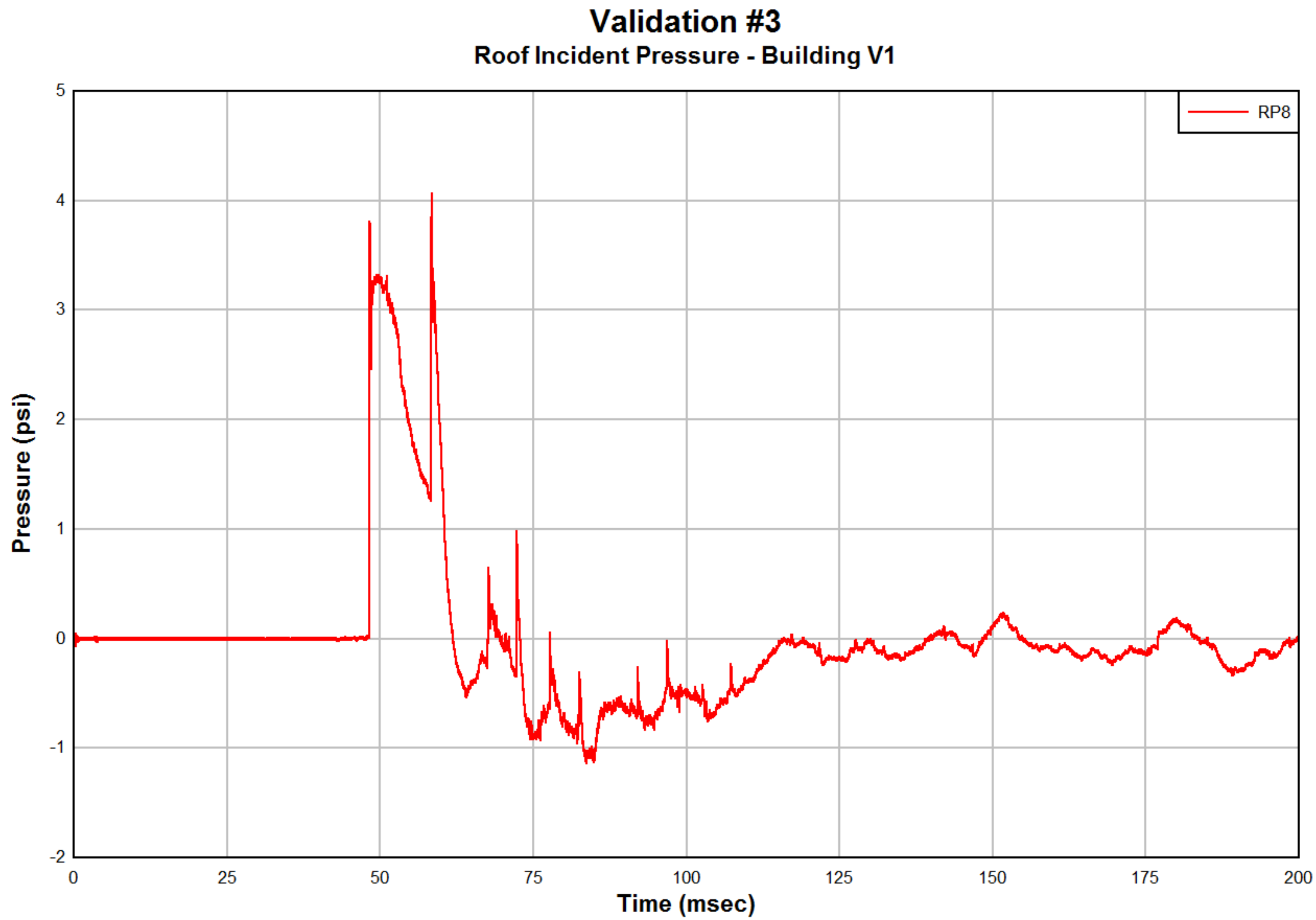
Validation #3 – Building V1: Front Face Reflected Pressure Gauges



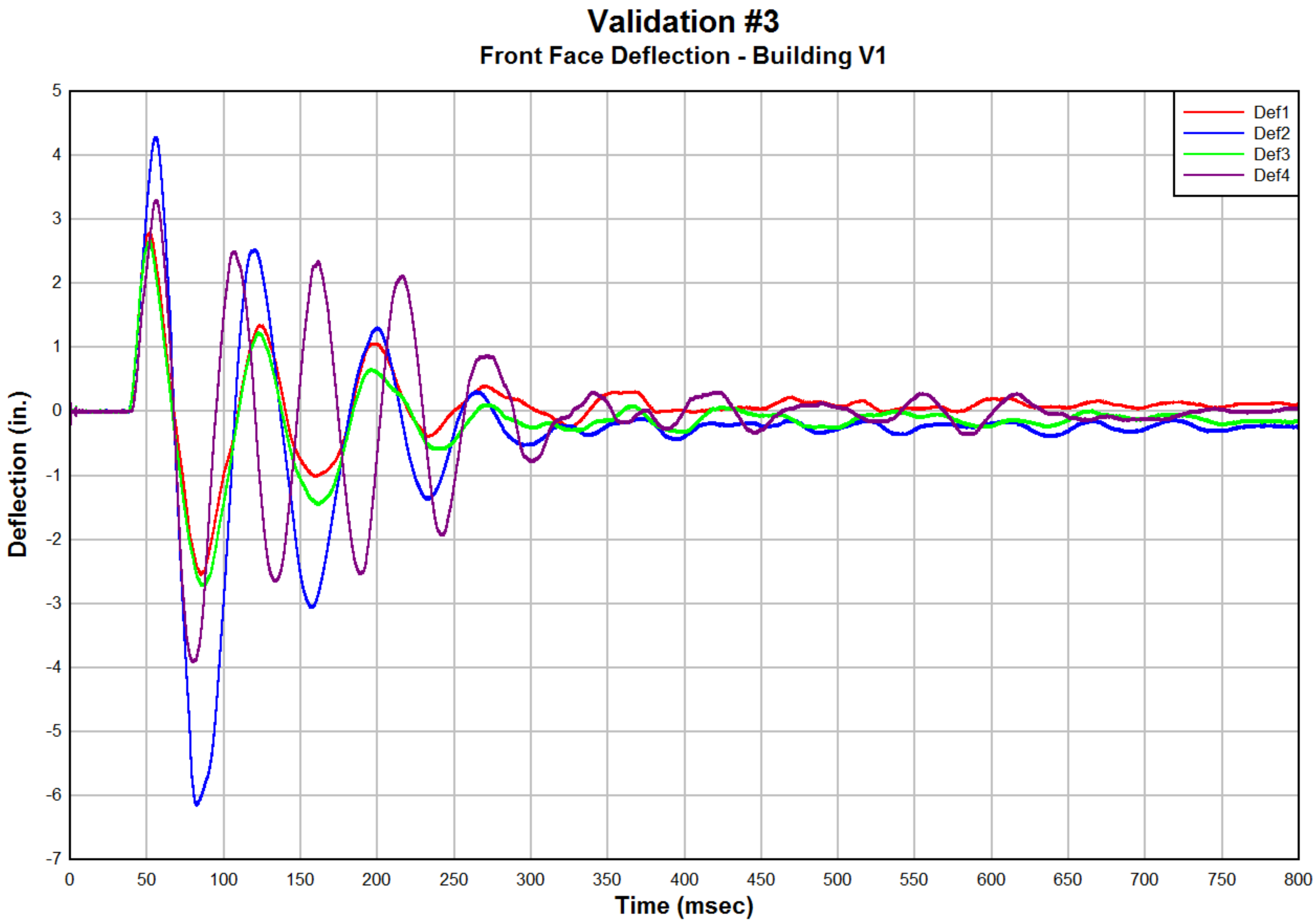
Validation #3 – Building V1: Side Face Incident Pressure Gauges



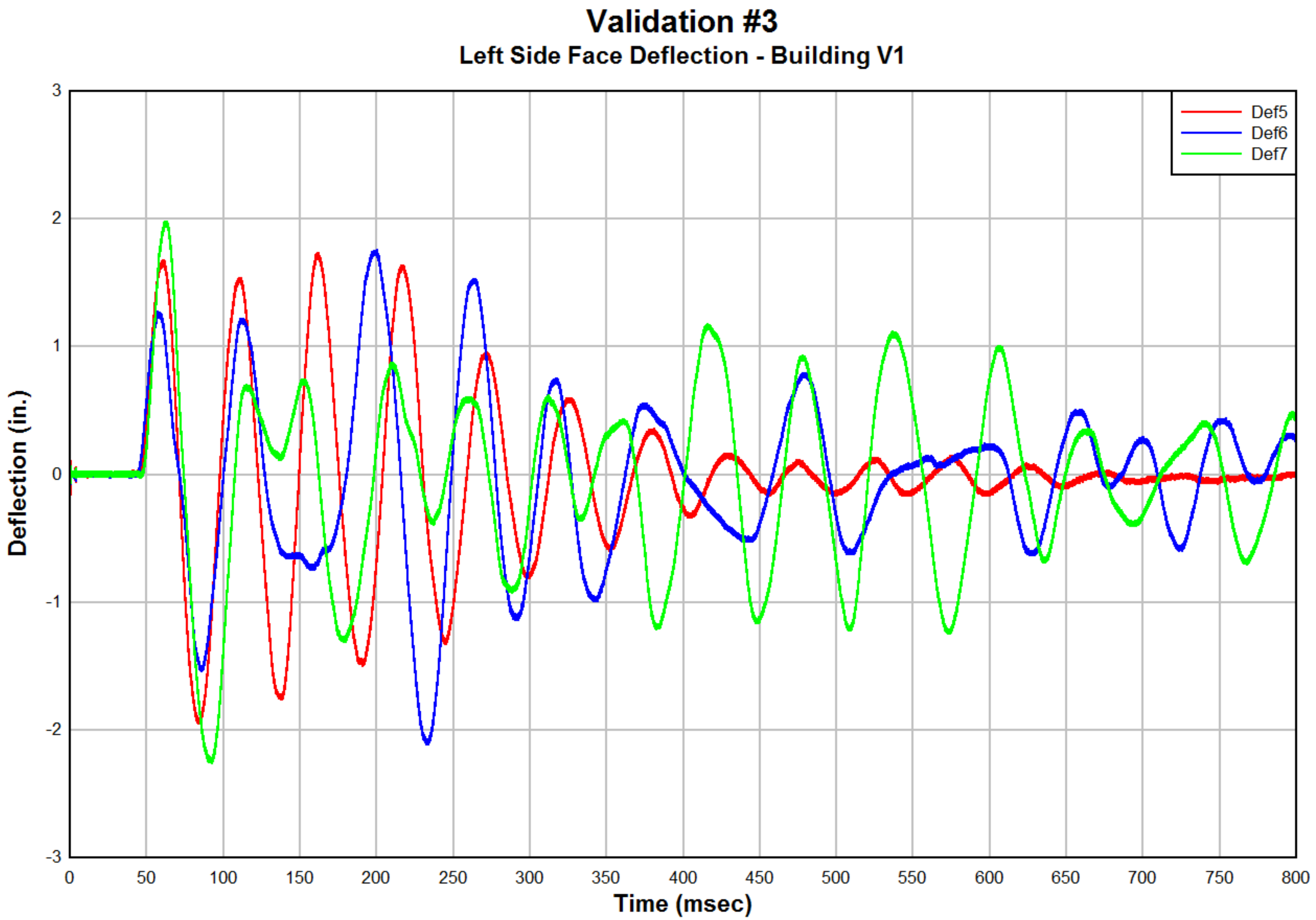
Validation #3 – Building V1: Roof Incident Pressure Gauges



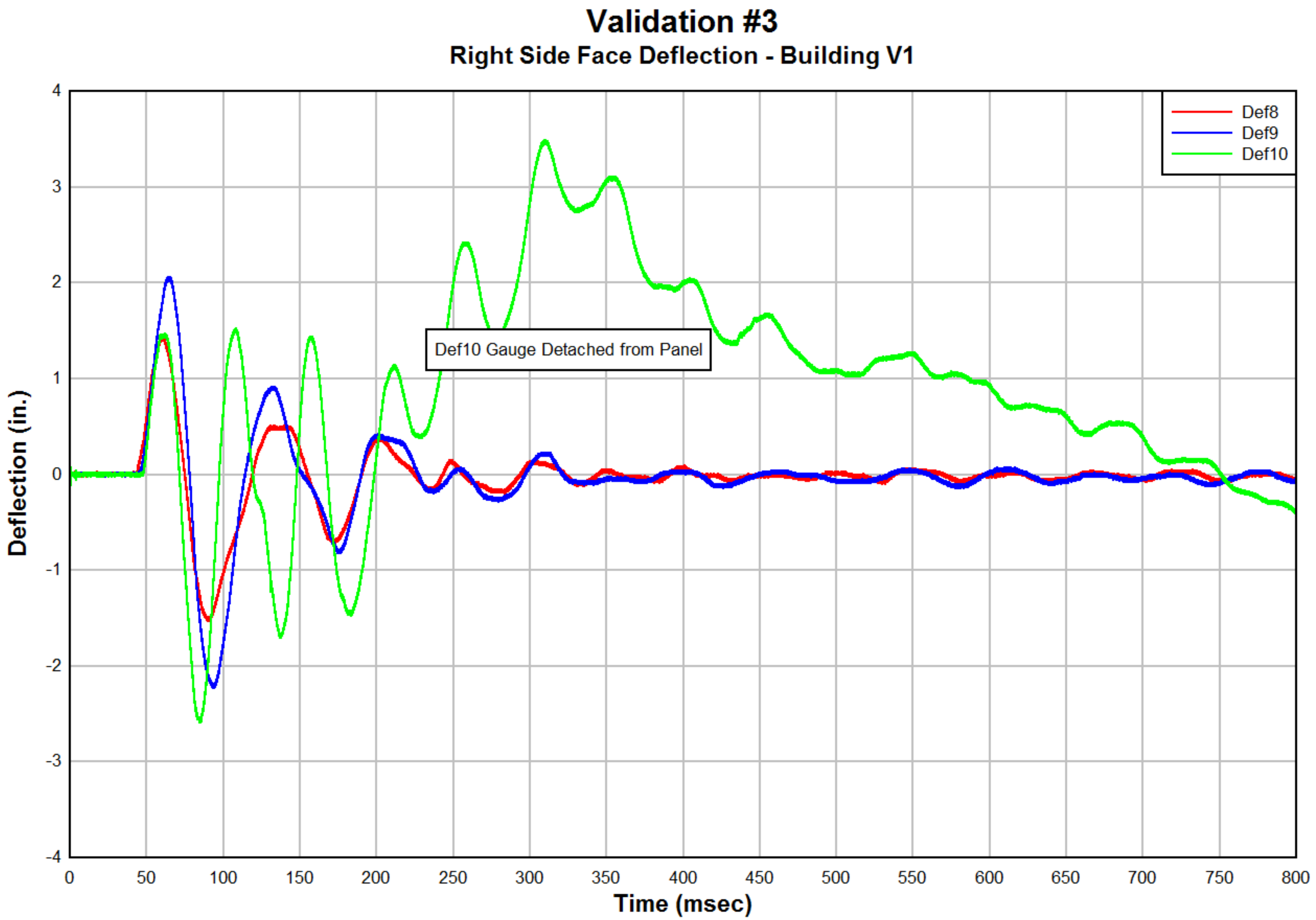
Validation #3 – Building V1: Front Face Deflection Gauges



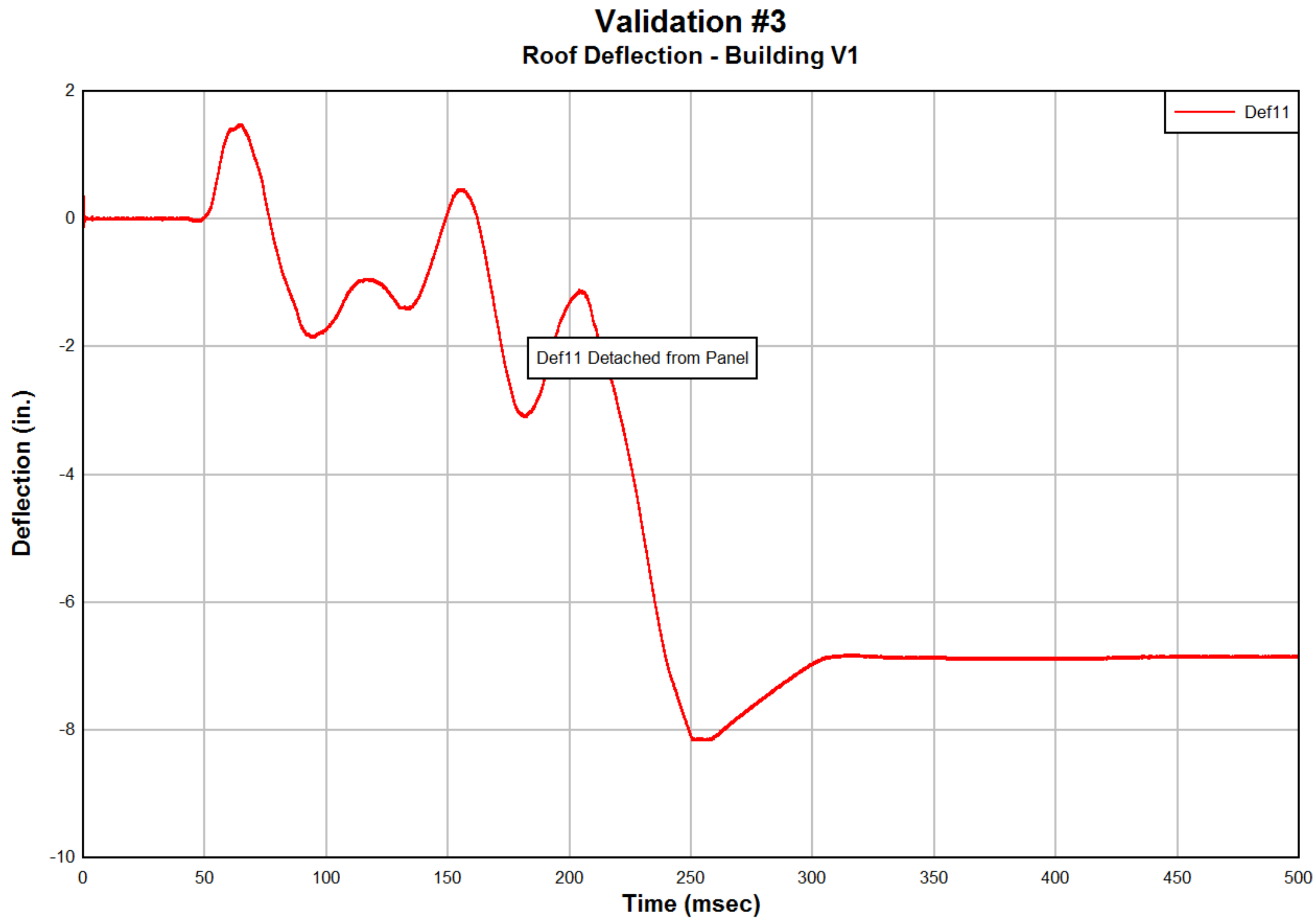
Validation #3 – Building V1: Left Side Face Deflection Gauges



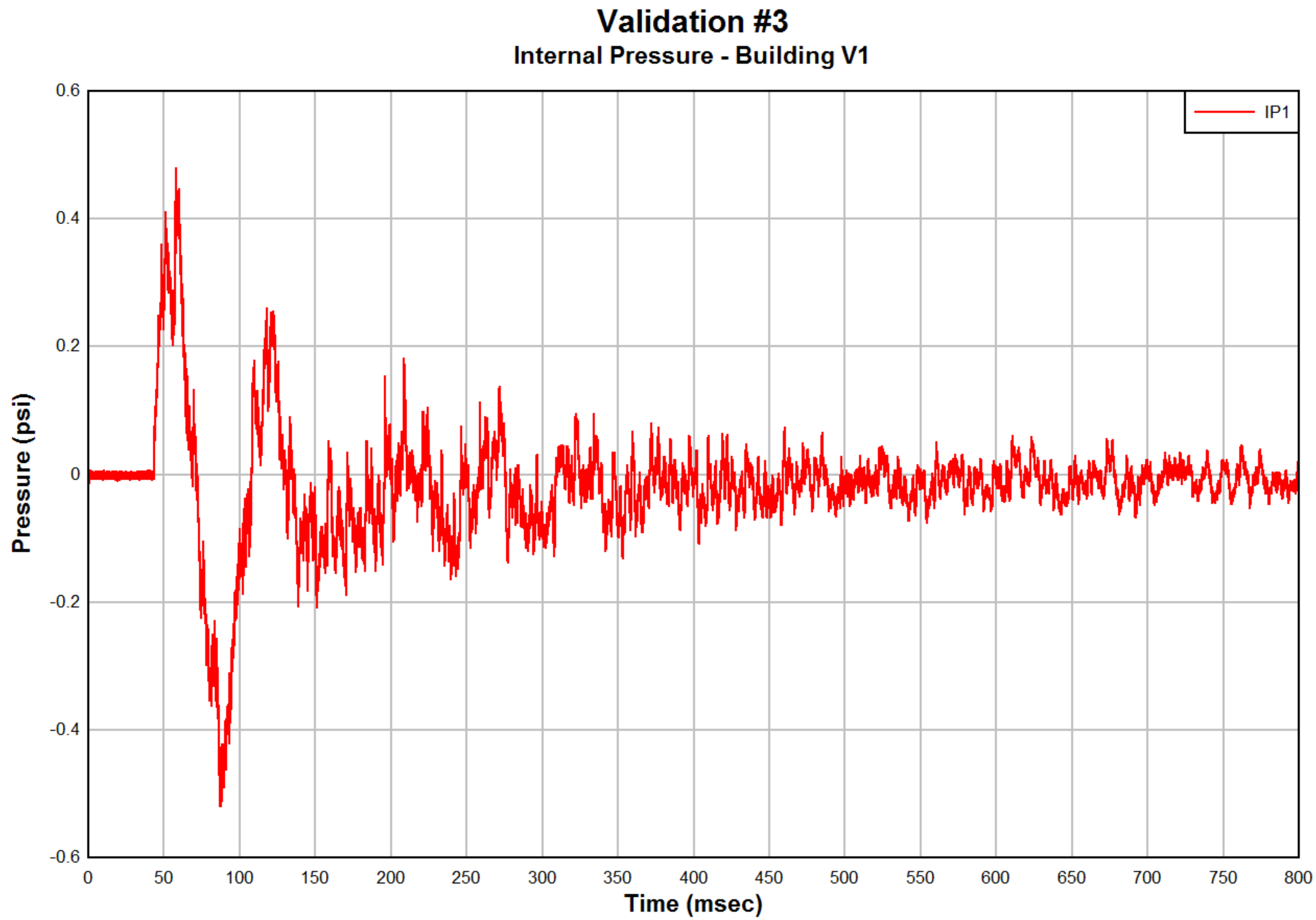
Validation #3 – Building V1: Right Side Face Deflection Gauges



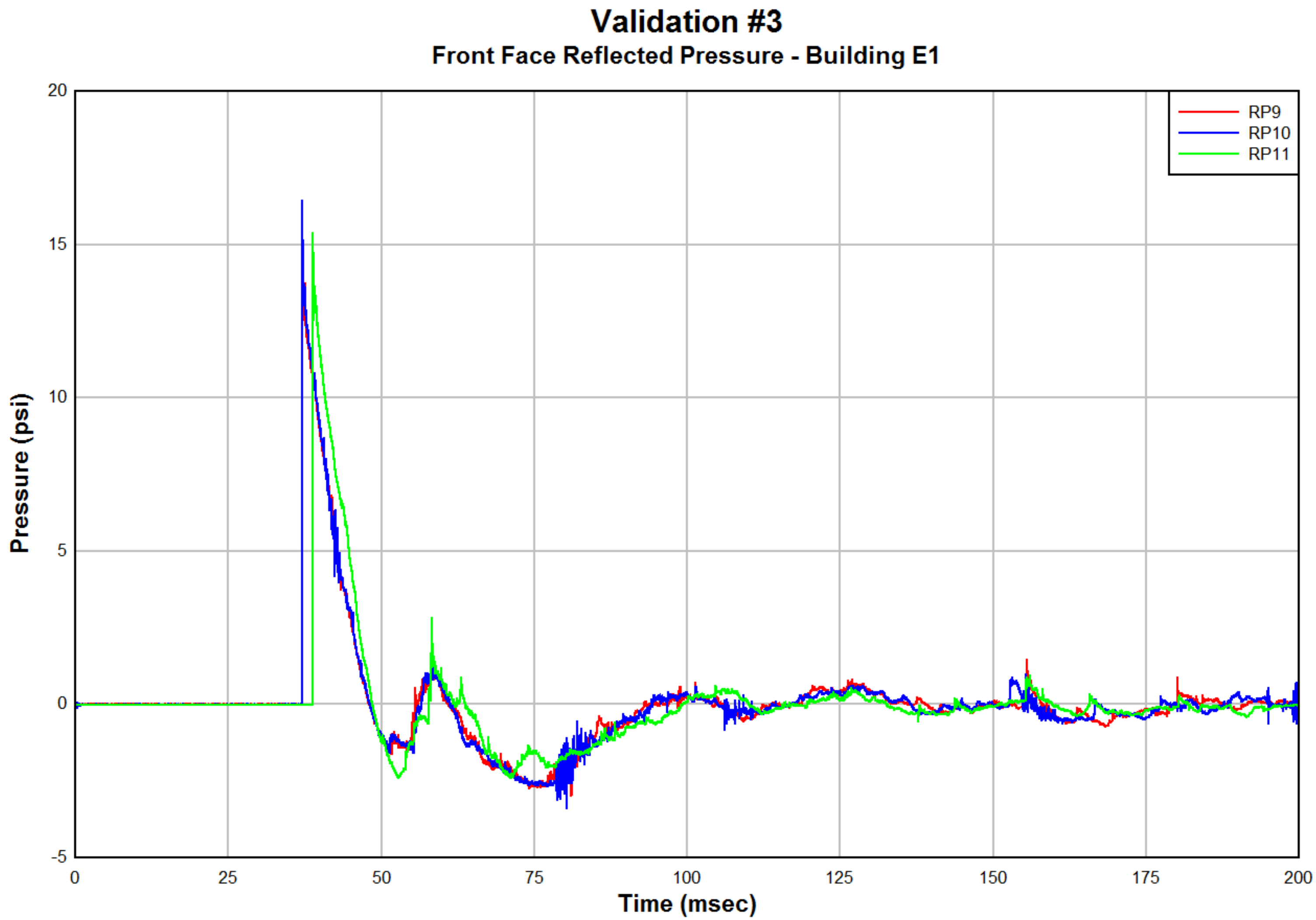
Validation #3 – Building V1: Roof Deflection Gauge



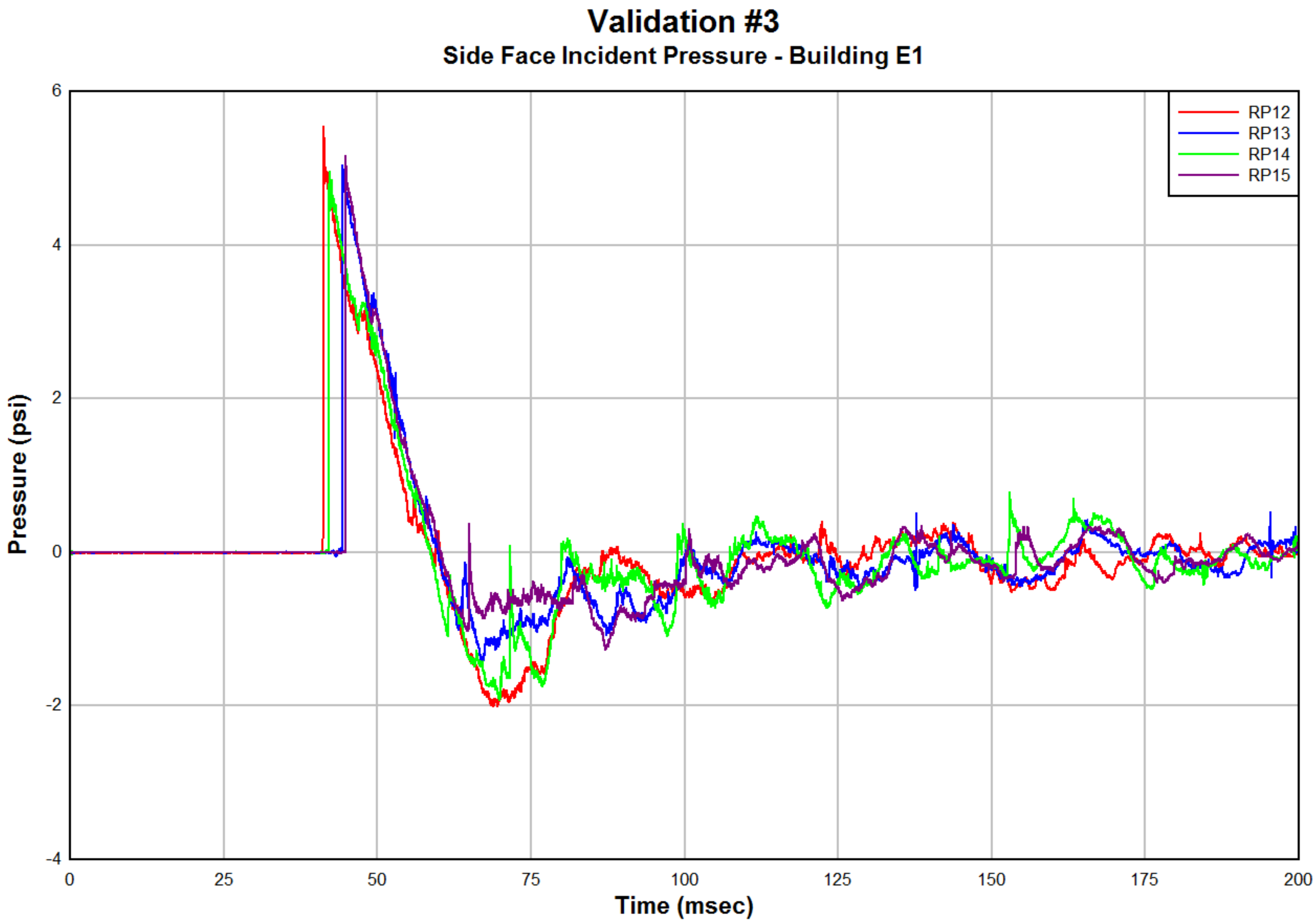
Validation #3 – Building V1: Internal Pressure Gauge



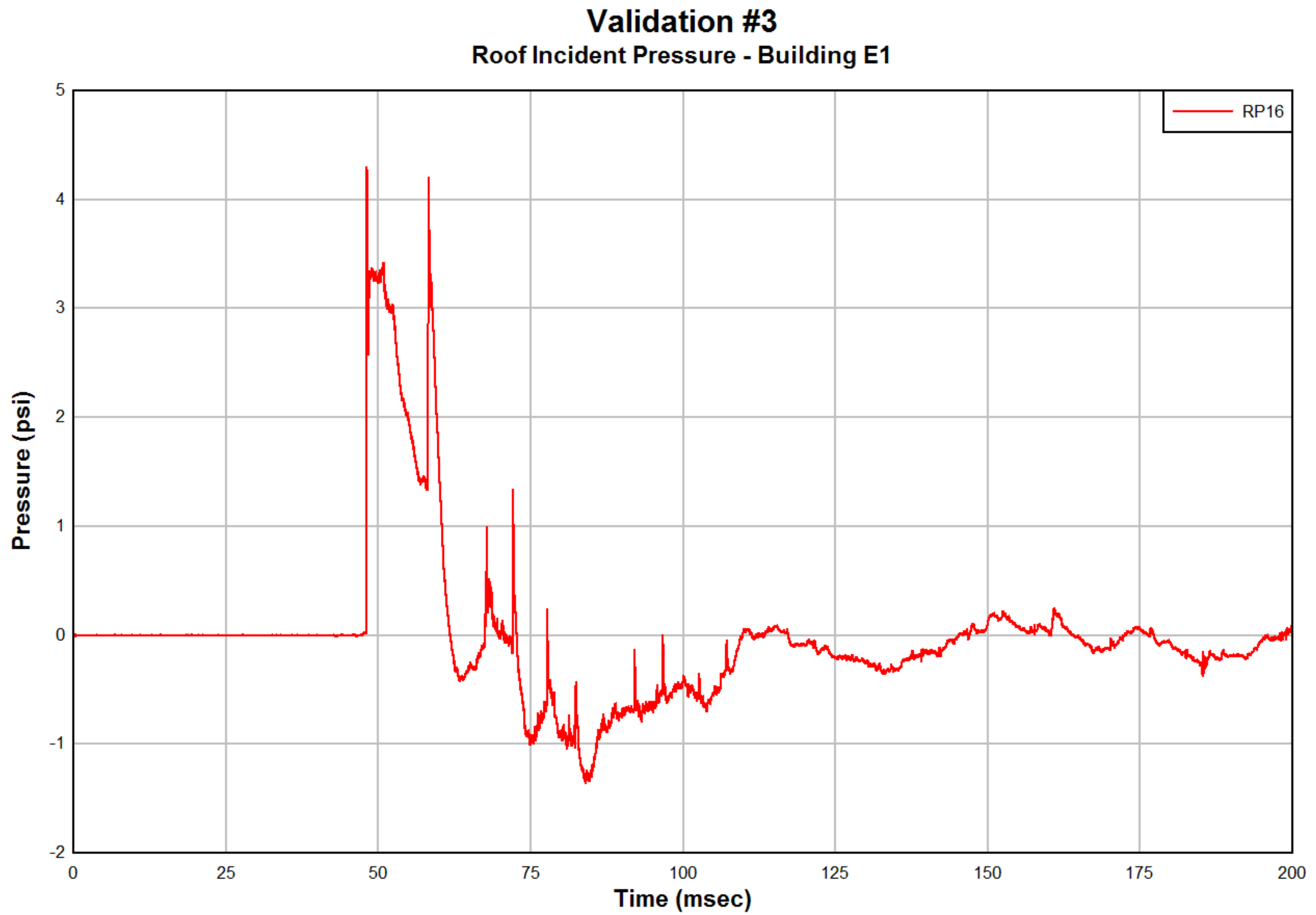
Validation #3 – Building E1: Front Face Reflected Pressure Gauges



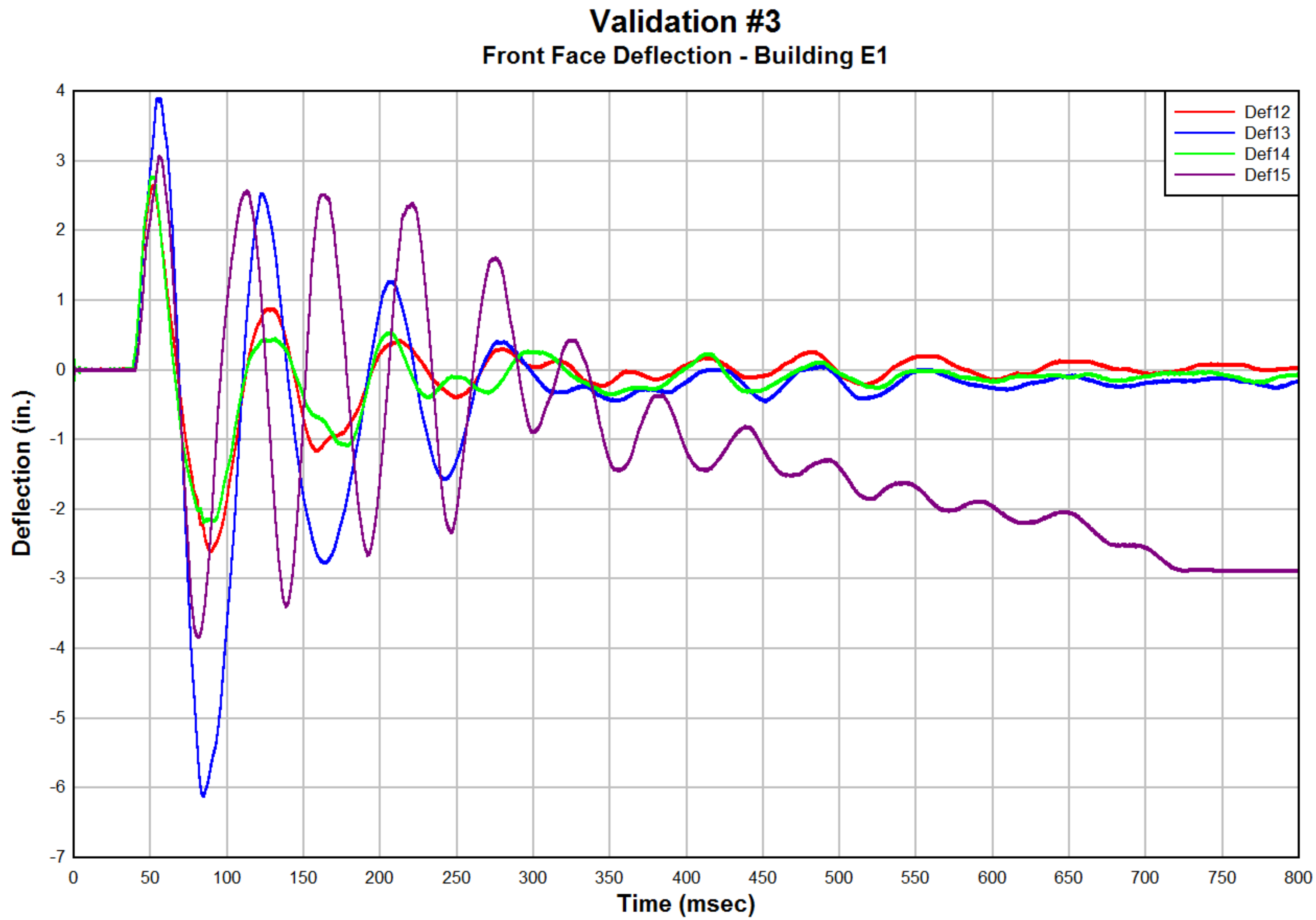
Validation #3 – Building E1: Side Face Incident Pressure Gauges



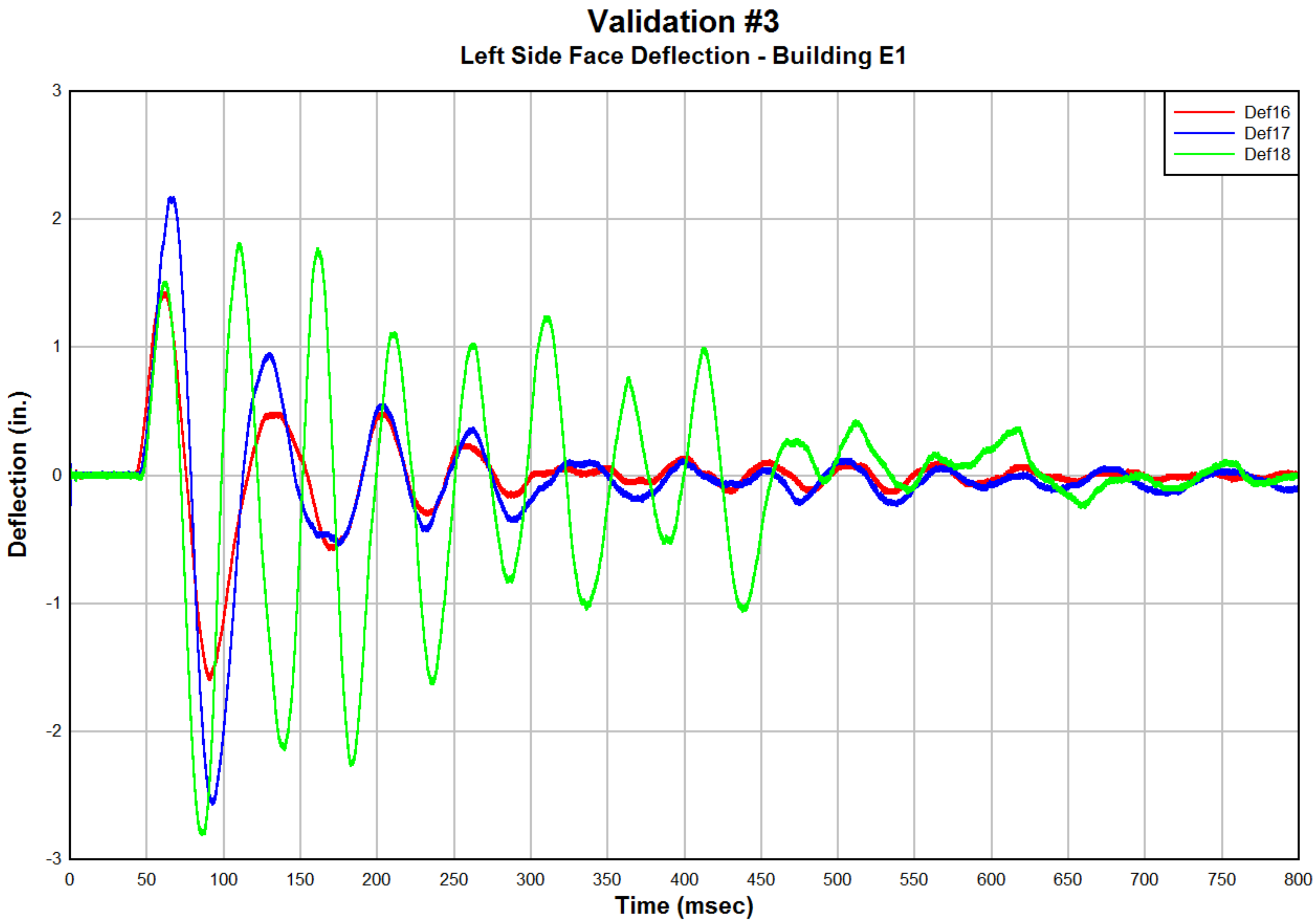
Validation #3 – Building E1: Roof Pressure Gauge



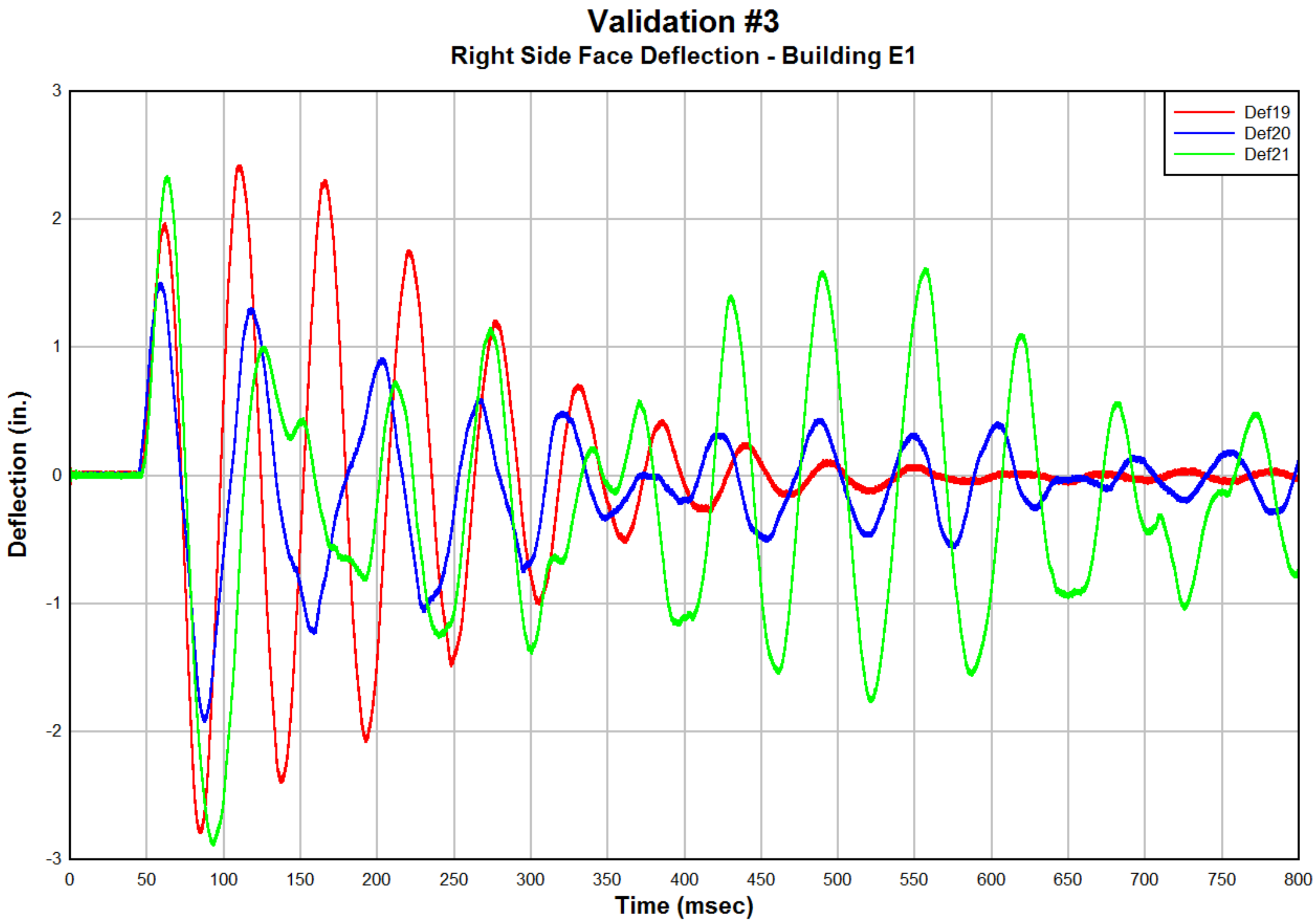
Validation #3 – Building E1: Front Face Deflection Gauges



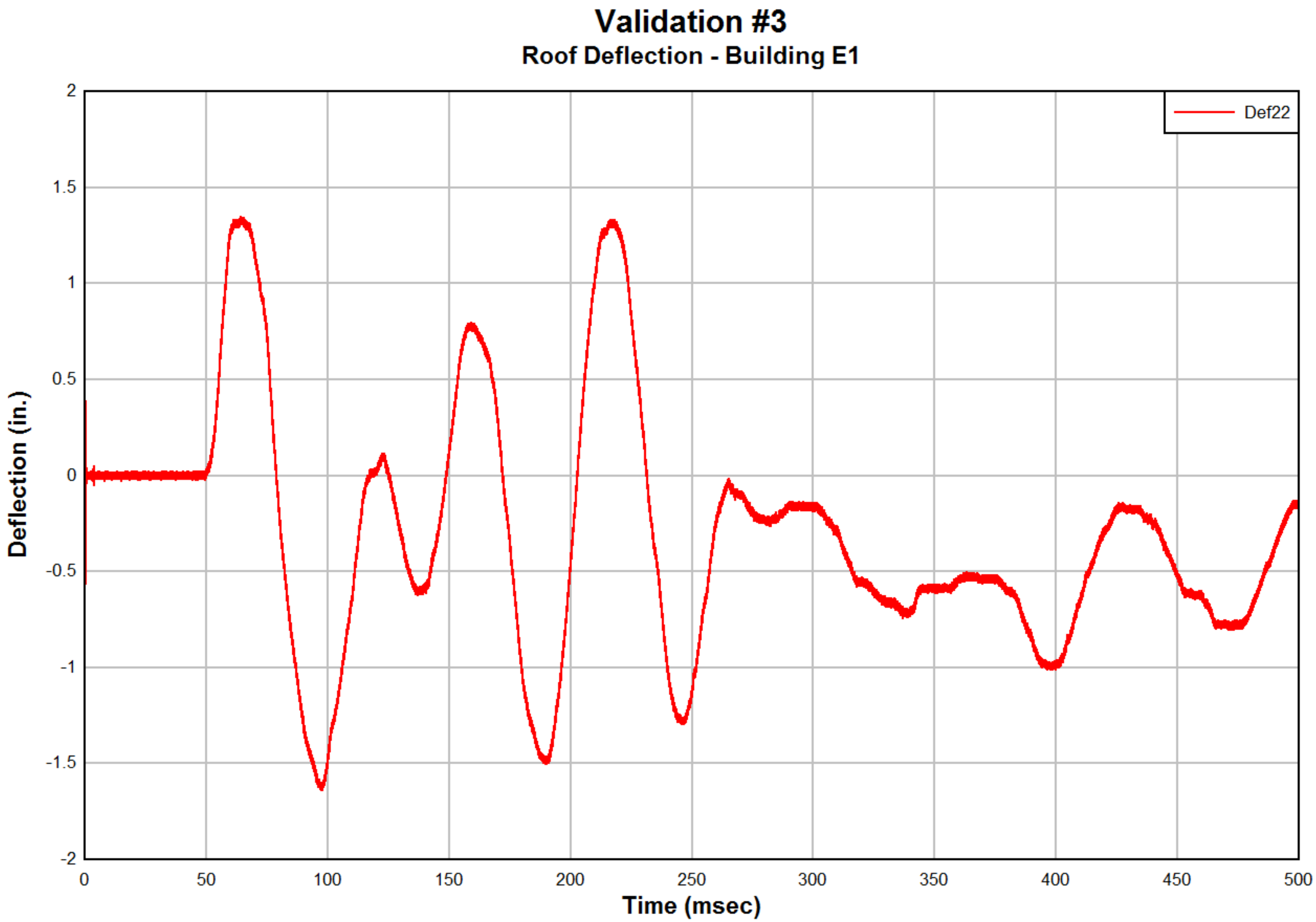
Validation #3 – Building E1: Left Side Face Deflection Gauges



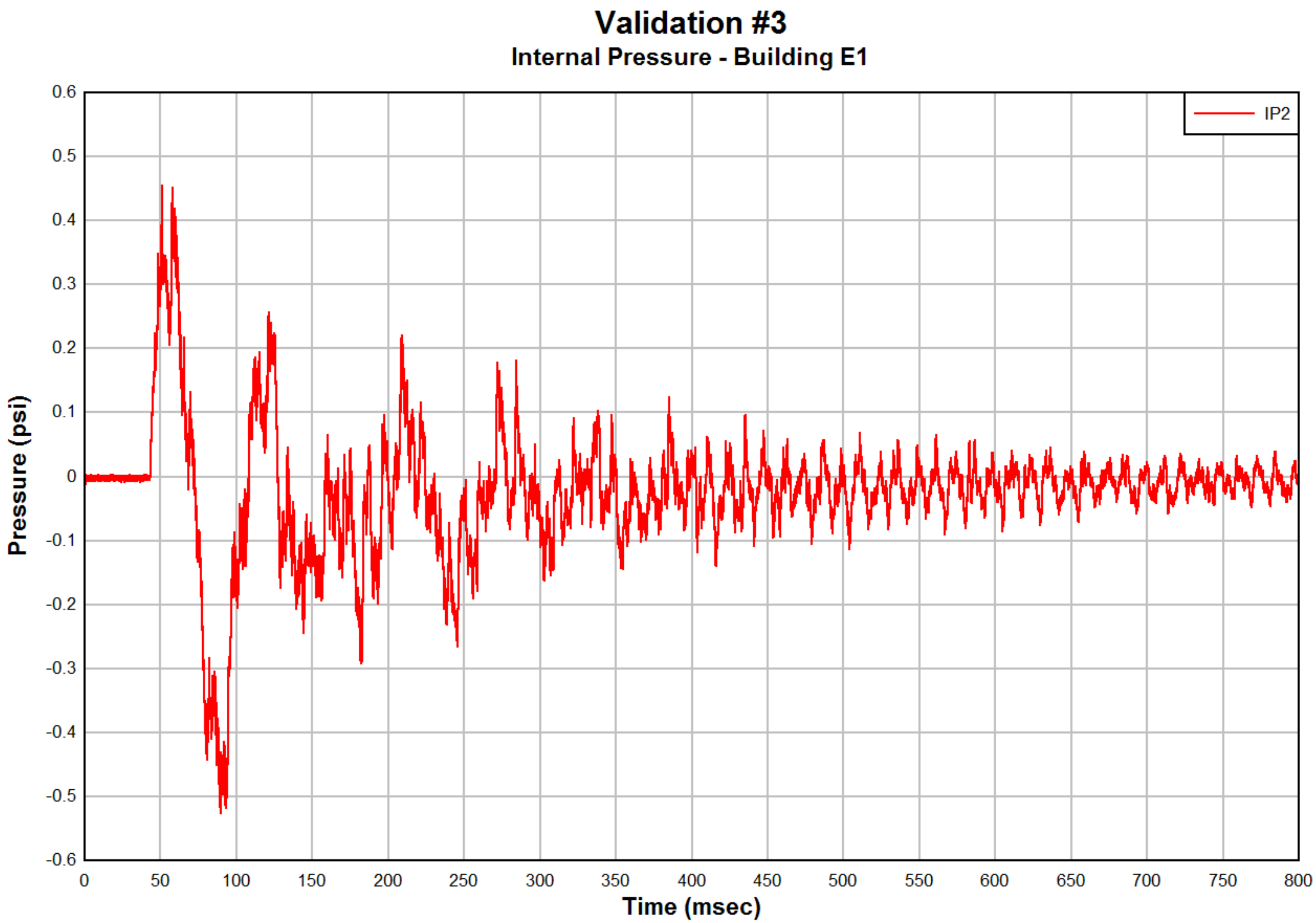
Validation #3 – Building E1: Right Side Face Deflection Gauges



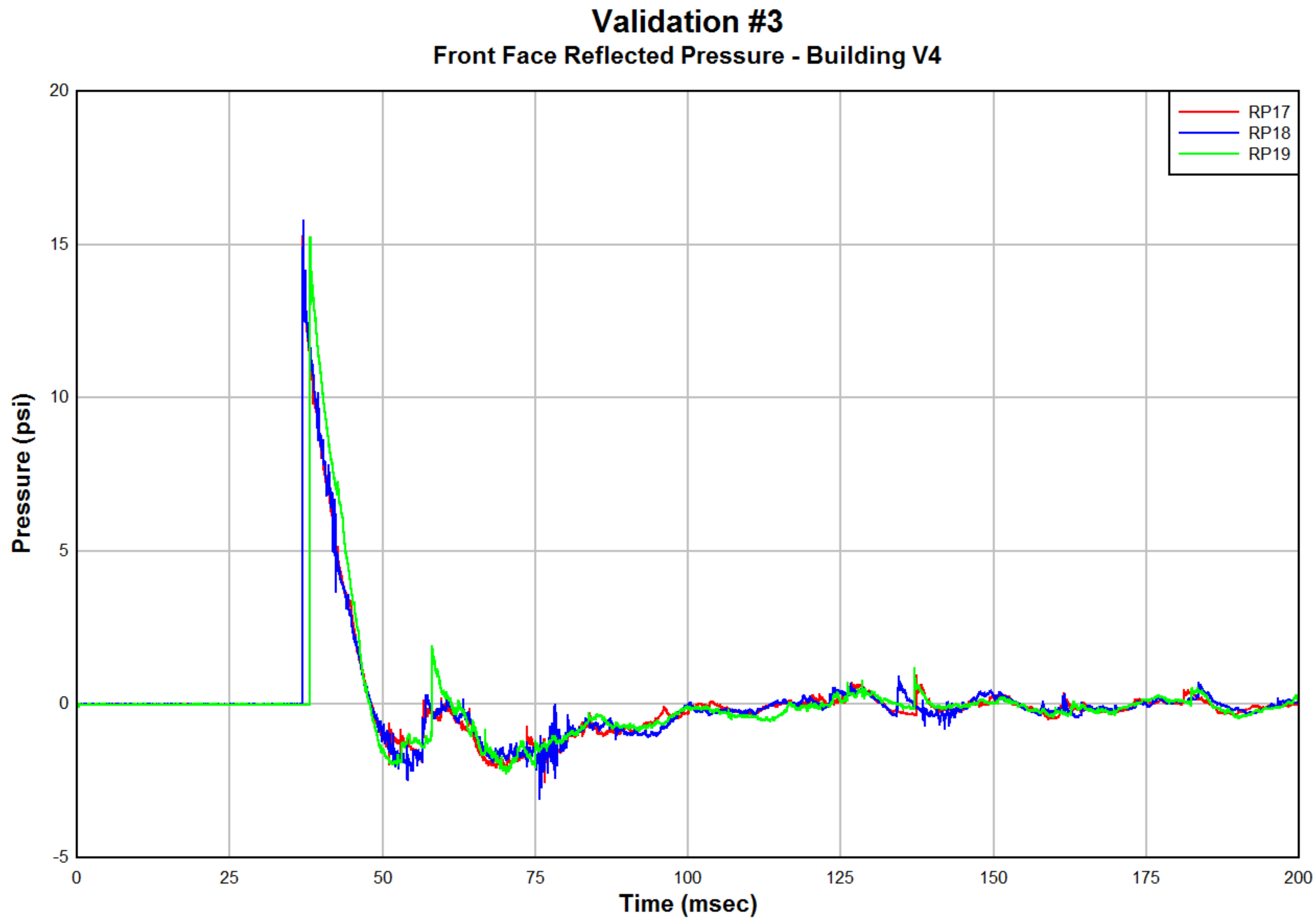
Validation #3 – Building E1: Roof Deflection Gauge



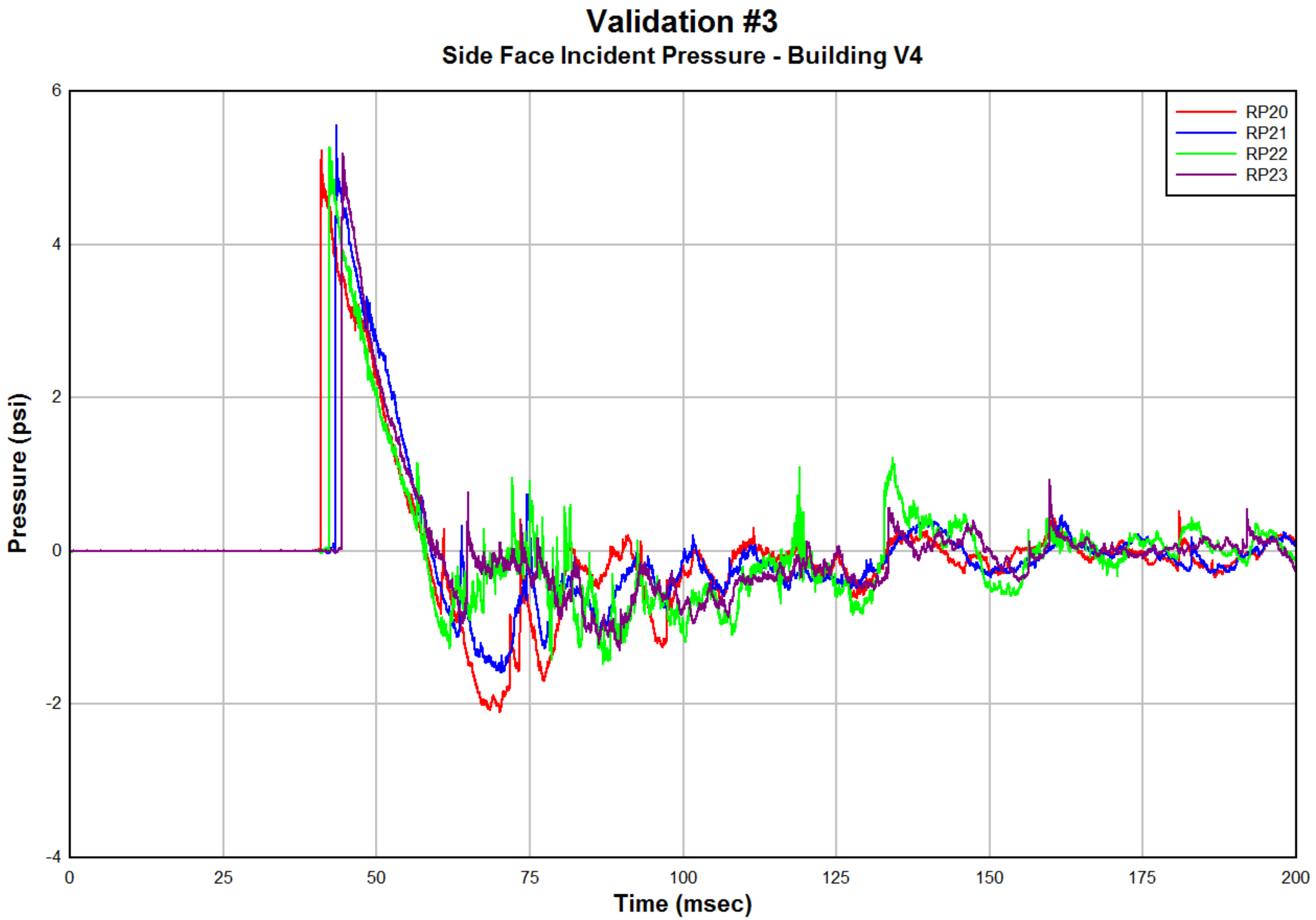
Validation #3– Building E1: Internal Pressure Gauge



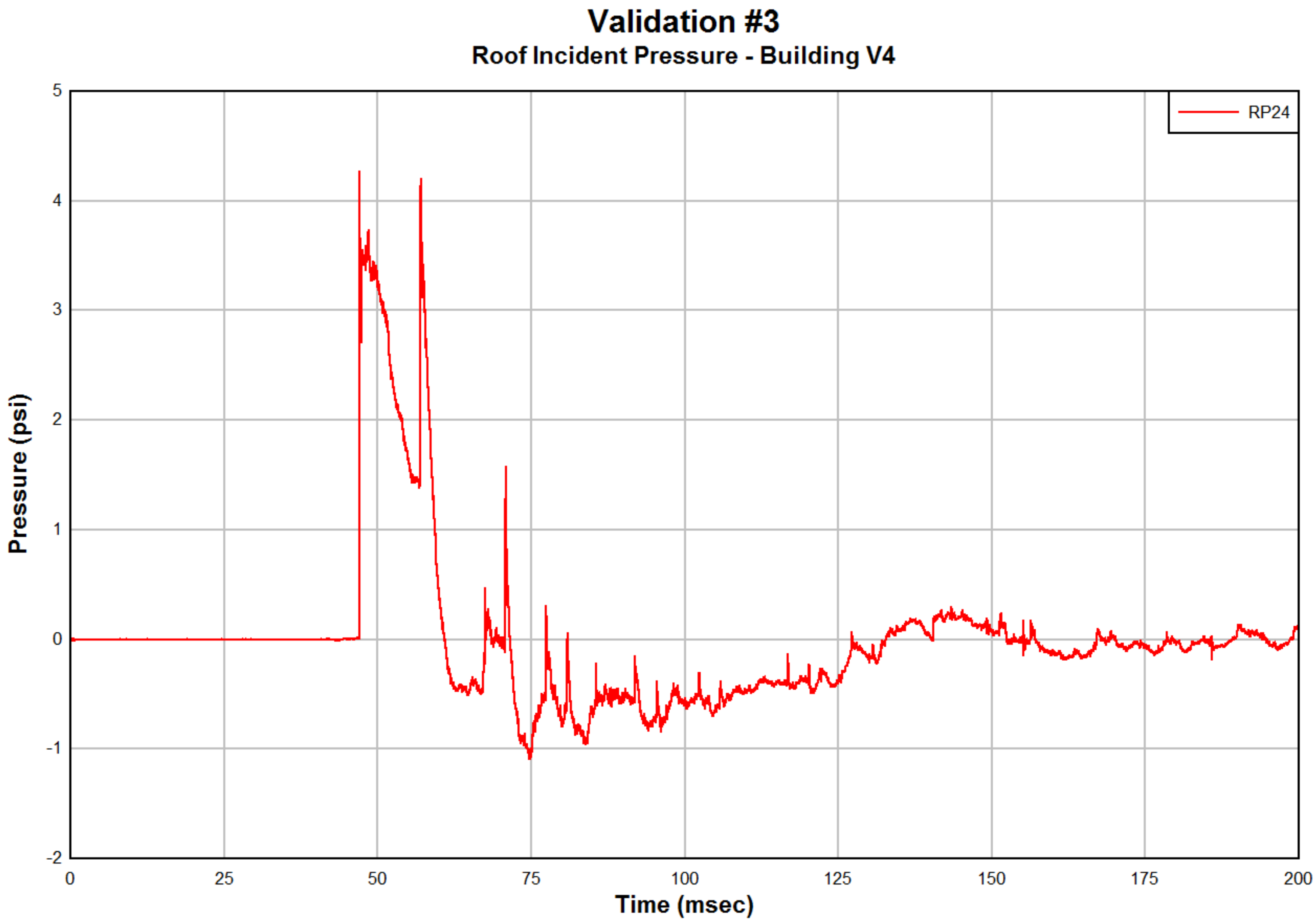
Validation #3 – Building V4: Front Face Reflected Pressure Gauges



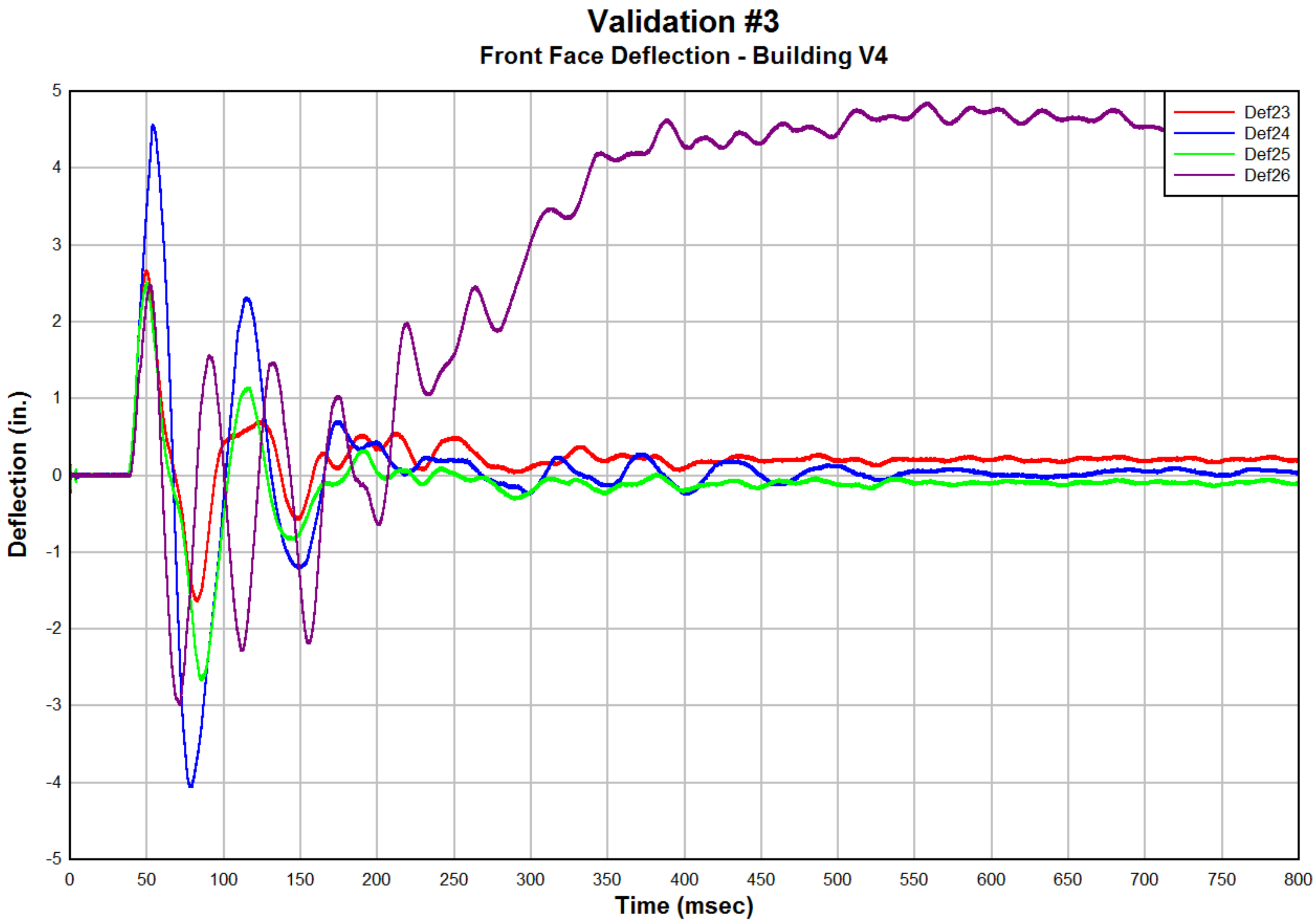
Validation #3 – Building V4: Side Face Incident Pressure Gauges



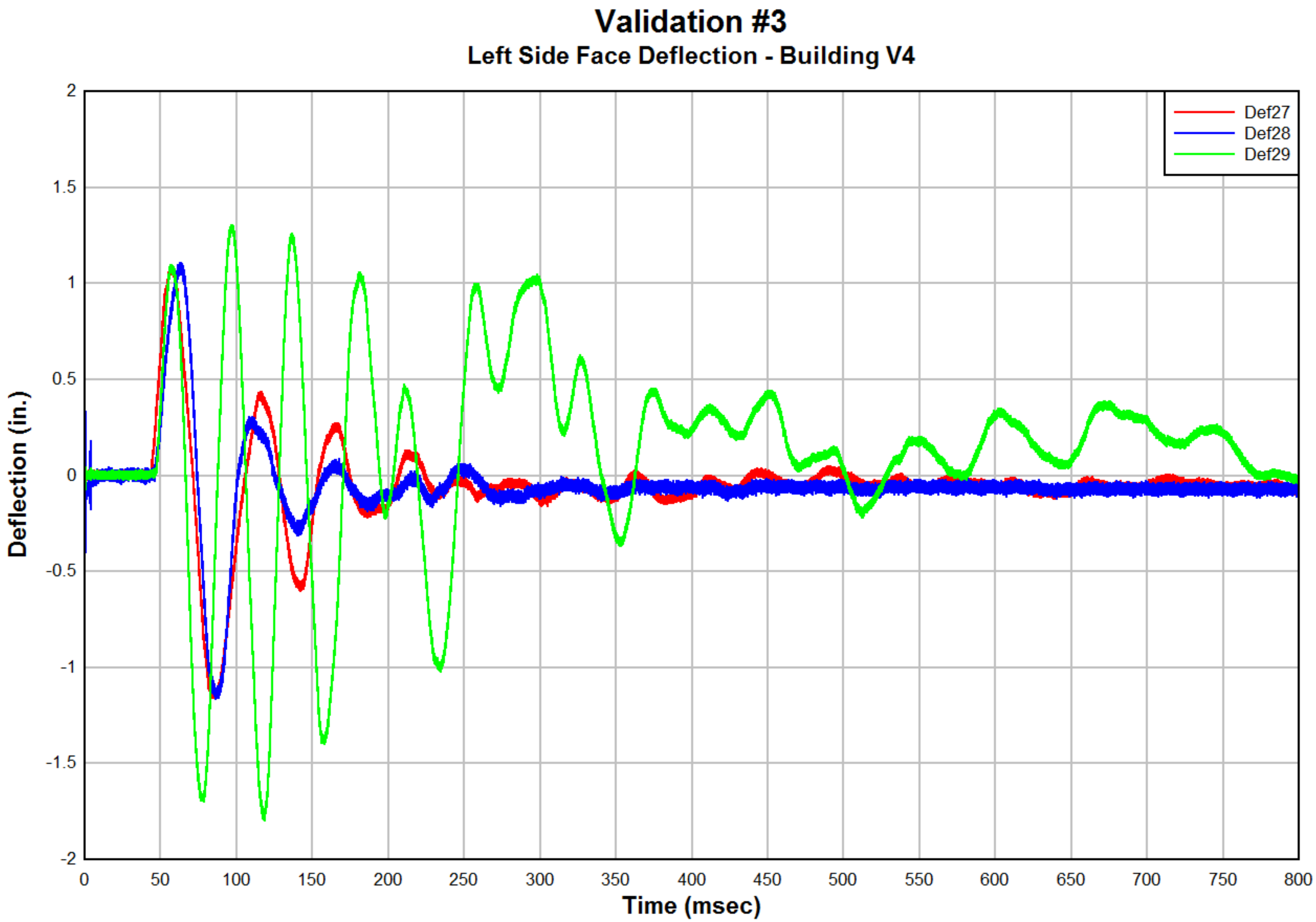
Validation #3 – Building V4: Roof Pressure Gauge



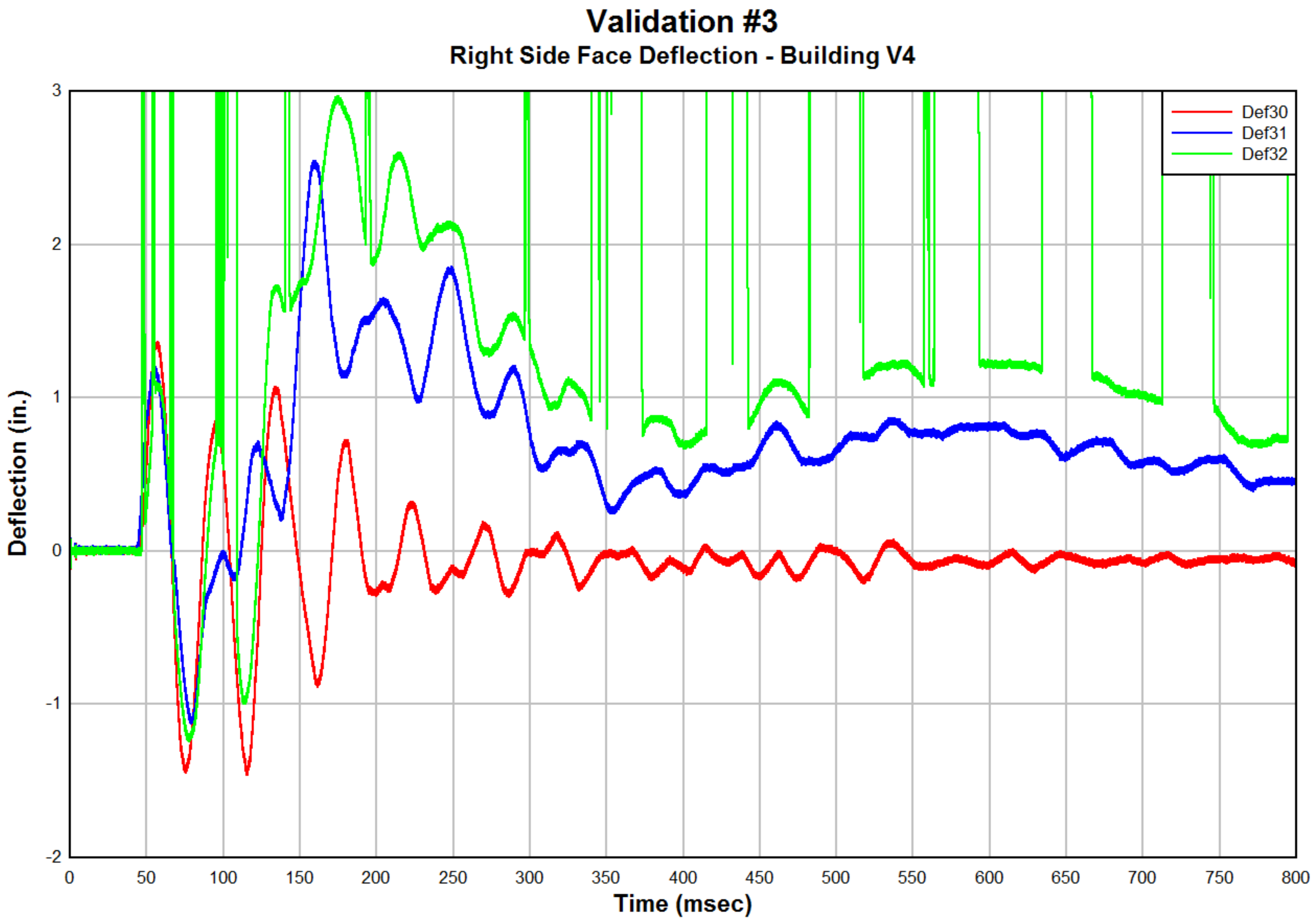
Validation #3 – Building V4: Front Face Deflection Gauges



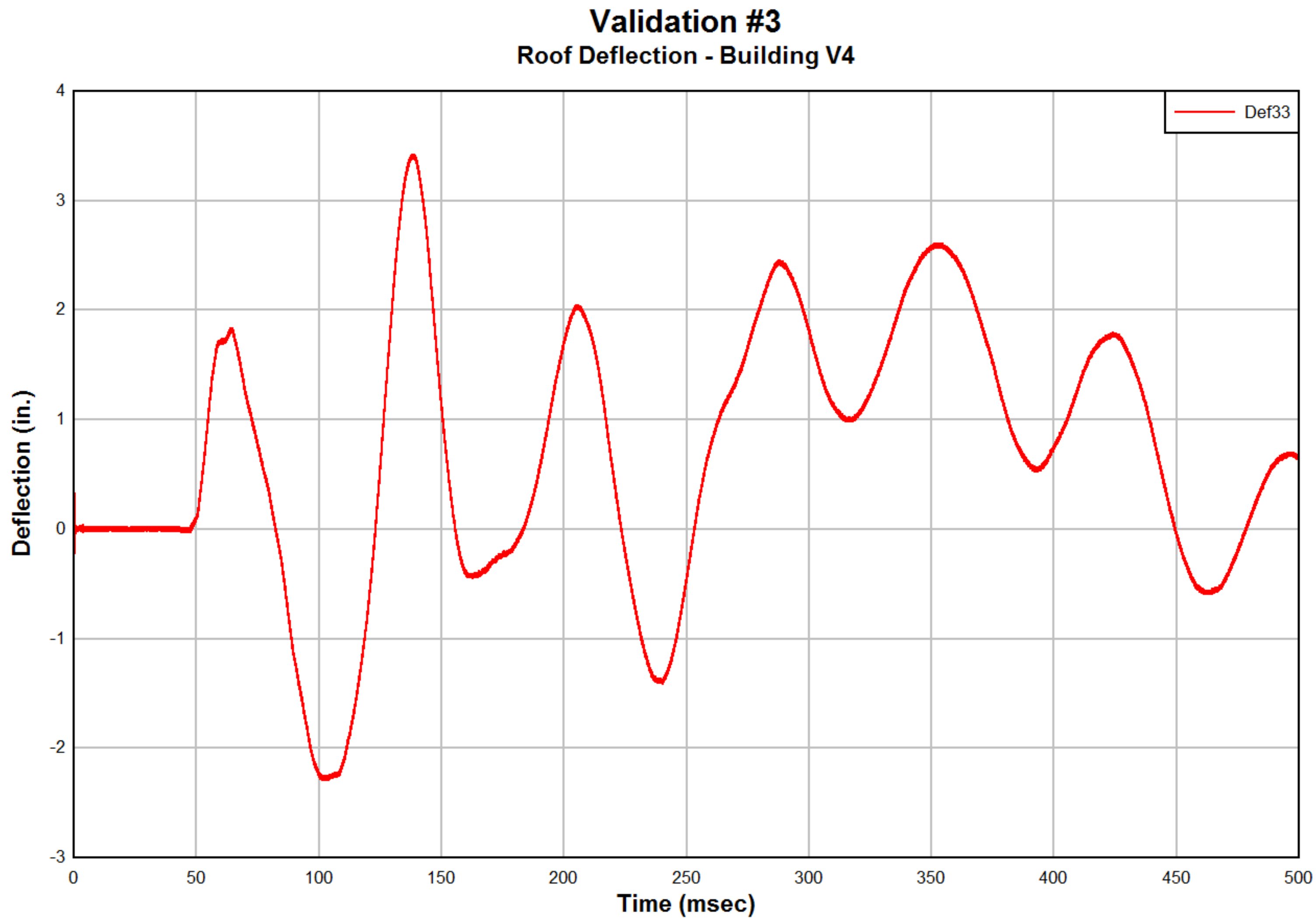
Validation #3 – Building V4: Left Side Face Deflection Gauges



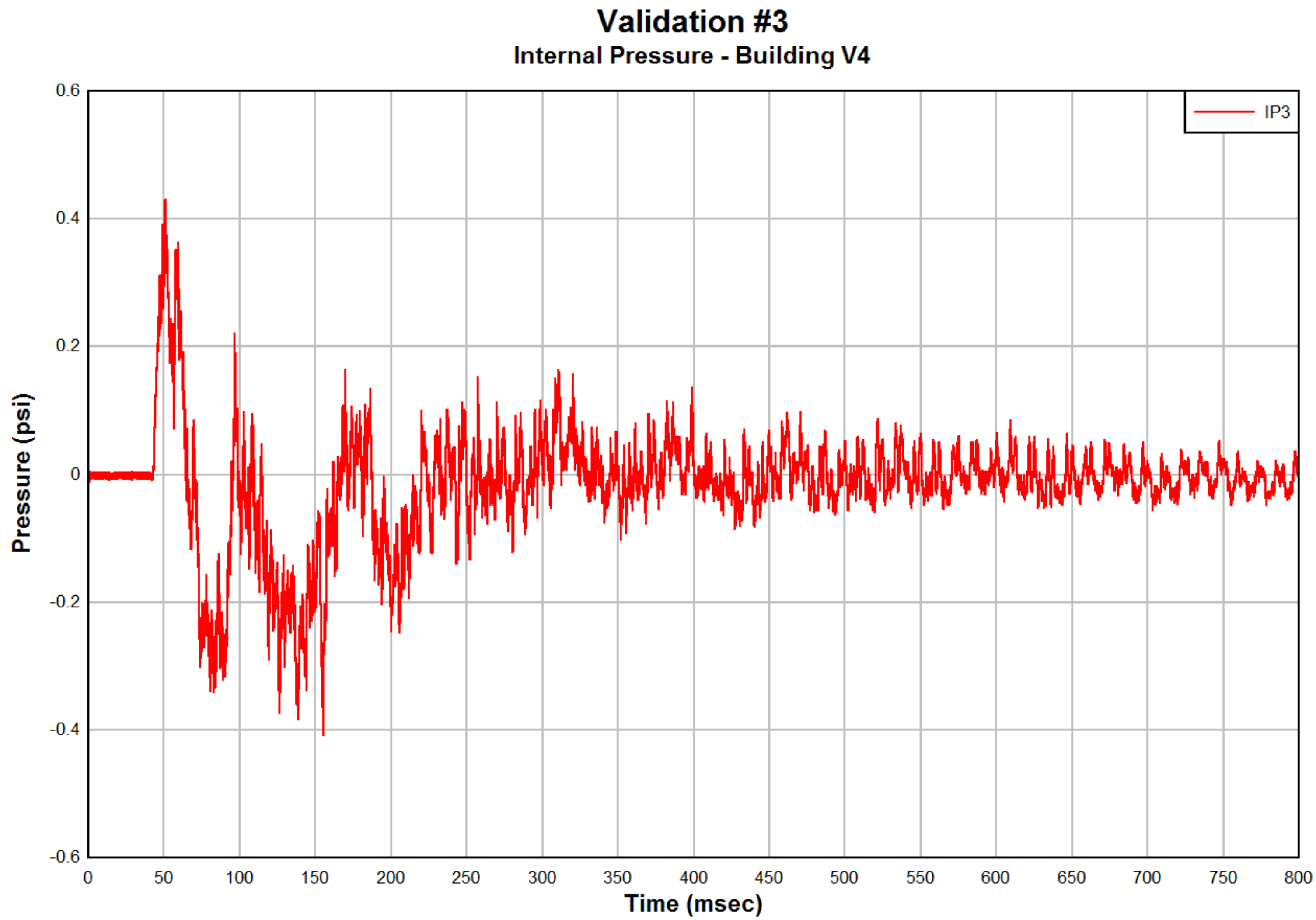
Validation #3 – Building V4: Right Side Face Deflection Gauges



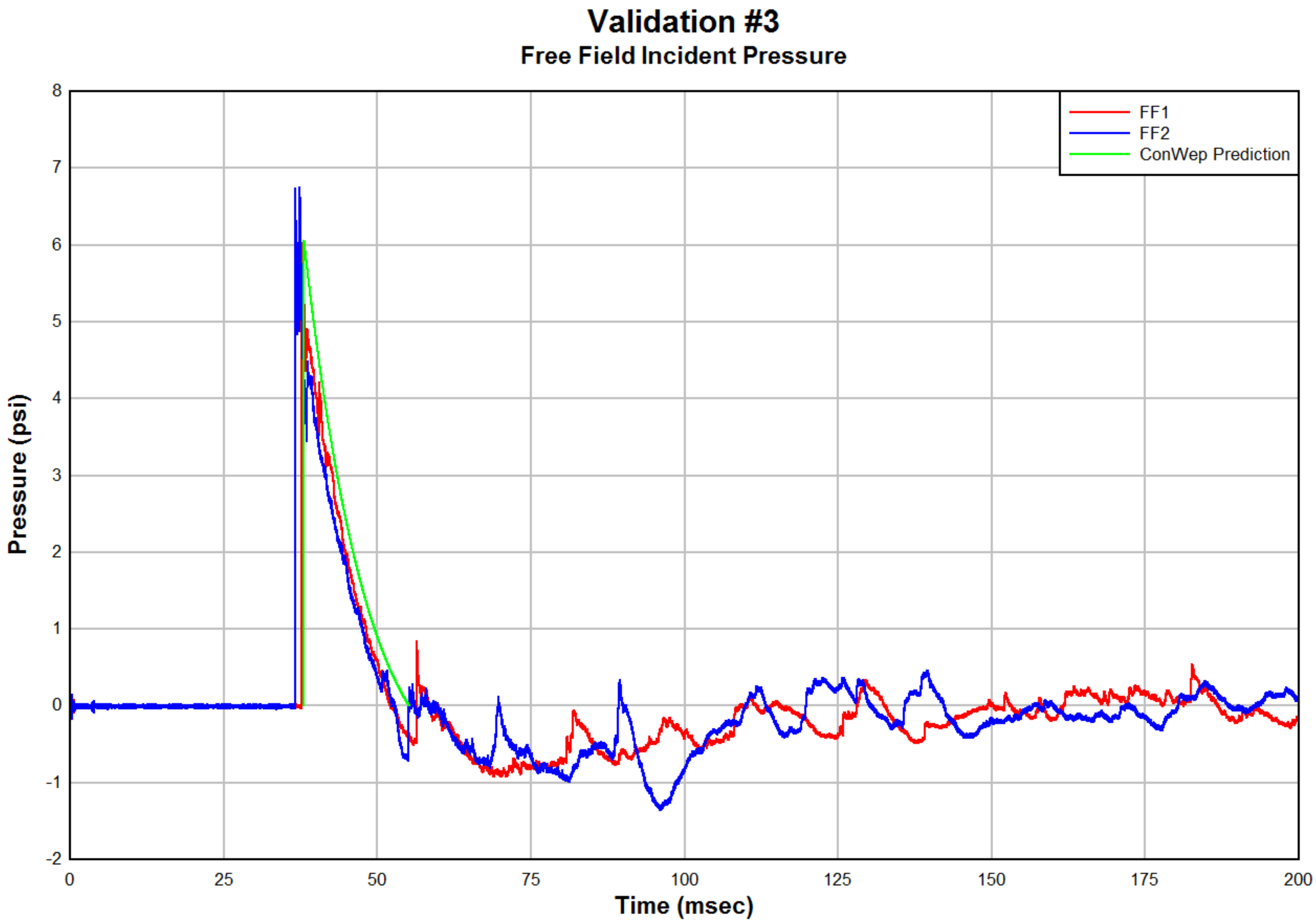
Validation #3 – Building V4: Roof Deflection Gauge



Validation #3 – Building V4: Internal Pressure Gauge



Validation #3: Free Field Incident Pressure Gauges



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.