DEVELOPMENT OF TIME-VARYING WIND UPLIFT TEST PROTOCOLS FOR RESIDENTIAL WOOD ROOF SHEATHING PANELS

By

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To Patricia, Jeff, Christina, Fitz, Mimi and Laddie

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Abstract of Thesis Presented to the Graduate School
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DEVELOPMENT OF TIME-VARYING WIND UPLIFT TEST PROTOCOLS FOR RESIDENTIAL WOOD ROOF SHEATHING PANELS

By

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Following the damage caused by Hurricane Andrew (1992) several research institutions and universities made an effort to improve the performance of residential wood roof sheathing panels in extreme wind events. The outcome of this research was predicted panel uplift resistance based on observed performance under varying static test methods. Changes in panel construction have resulted in improved wind uplift resistance, however failures are still observed below design level wind speeds. It is the hypothesis of this investigation that some residential wood roof sheathing panel failures are due to current test methods overestimating panel resistance. Previous wood roof sheathing panel uplift tests have used uniform static pressure on newly constructed panels. However aged wood roof sheathing panels are subjected to temporally varying pressure loading in the field. As part of this research standardized test protocols are developed for static and temporally varying (dynamic) pressure loading. Panels fabricated in the lab and harvested from existing construction are tested with static and dynamic pressure. Over 170 panels were tested statically, 38 of which were harvested from existing construction, and 34 panels were tested statically vs. dynamically. It is found that dynamic loading reduces uplift capacity of wood roof sheathing panels approximately 20%, and that age or weathering do not appear to effect the uplift resistance of panels.

CHAPTER 1
INTRODUCTION

Motivation

The damaging effects of hurricanes making landfall along the eastern seaboard of the United States have been well documented in the past 20 years, beginning with Hurricane Hugo (1989). Hurricanes annually cause billions of dollars in property loss in coastal regions of the south-east United States (van de Lindt et al. 2007). Examples of hurricane damage were observed in Hurricane Hugo in 1989, Hurricane Andrew in 1992 and most recently during the eight land falling hurricanes of 2004 and 2005 seasons. Approximately 160 billion dollars (normalized to 2005) in damages occurred in the 2004-05 hurricanes from Hurricanes Jeanne, Frances, Ivan, Charley, Rita, Wilma and Katrina (Pielke et al. 2008). This significant damage highlights a need to better understand the effects of Hurricane events on society. Figure 1-1 summarizes the total damages due to wind events by decade from 1900-2005 normalized to 2005 US dollar by Pielke et al. (2008), which suggests that this level of impact is not uncommon.

Wind damage to light-frame wood structural systems is responsible for a significant proportion of the observed damage from hurricane events. Sparks (1991b) observed that extreme winds in Hurricane Hugo were responsible for 60% of the insured losses. Further, a large proportion of the damage caused to light-framed wood structures is concentrated in damage to the roof systems. Baskaran and Dutt (1997) estimated that 95% of all monetary losses from Hurricanes Iniki and Andrew in 1992 resulted from the failure of roof system materials.

The construction of wood roofs typically consists of wood sheathing nailed to wood trusses or rafters that are attached to the walls of the building. Wood roof sheathing panels provide a structural diaphragm which resists lateral loads and encloses the structure when used in conjunction with a roof covering system. Post disaster investigations have found that roof

sheathing wind uplift failures lead to structural damage, due to loss of lateral stiffness (FEMA 1993), and extensive water damage (FEMA et al. 2005a). The damage caused by wood roof sheathing failure presents considerable monetary risk to the state of Florida considering that approximately $1.5 trillion in building stock are exposed to hurricane events (Pinelli et al. 2004).

Typically, the wind uplift tests used for commercial roofs and roofing systems (i.e. ASTM E15-92, UL 1897 and FM 4470) are based on physical tests using uniform static pressure applied to a sample until the specimen fails. However no such test exists for residential roof structures. The current prescriptive standards for wind uplift resistance of wood roof construction in the International Building Code (ICC 2006) were developed from experimental research conducted by Cunningham (1993), Schiff et al. (1994) and others. These tests which are basis for current design against wind uplift, which were developed five to seventeen years ago, lack the dynamic characteristics necessary to replicate actual wind forces generated during a hurricane.

It has been observed from post hurricane investigations that roof systems fail at wind speeds below design level. For example FEMA (2006) reported roof damage to homes in Alabama and Mississippi during Hurricane Katrina occurred in 120 mph winds although the design wind speeds for the areas were 140 mph (Figure 1-2). Reasons suggested for this include poor construction quality, environmental effects which accelerate loss of strength, natural variability in wood properties and under-estimation of the actual wind forces that occur during hurricanes. Another reason for this disconnect between in-field and predicted performance may be related to limitations of current wind uplift test protocols that predict uplift performance.

Baskaran and Dutt (1997) demonstrated that dynamic loads along individual dowel-type fasteners in wood reduced their withdrawal capacity by 10 to 30%. This effect is not currently accounted for in panel uplift testing and it may result in overly conservative prediction of wind

uplift capacity of wood roof sheathing. Similarly the in service conditions of residential wood roof sheathing may reduce the uplift resistance. Studies have shown that moisture and high humidity can cause 38% to 75% reduction in the strength of nails and staples installed in sheathing to wood framing connection (Chow et al. 1990; Feldborg 1989; Pye 1995).

Judge and Reinhold (2002) conducted one of the first studies to investigate the wind uplift capacity of roof sheathing installed in residential structures. While their limited study of twelve harvested wood roof sheathing panels from nine homes in Horry County, SC is not able suggest any conclusions it provides precedence for uplift testing of harvested panels conducted as part of this investigation.

It is the hypothesis of this investigation that some premature failures of roof sheathing panels occur because current wind uplift test methods overestimate the resistance of wood roof sheathing panels to real wind loading. A second hypothesis is that wood roof sheathing panel uplift resistance is reduced due to in service environmental effects which may degrade wood material properties over the typical life (0-50 years) of a residential roof.

Objectives

In order to test these hypotheses, wind uplift pressure testing will be conducted to

- Evaluate the uplift resistance of new vs. aged (harvested) wood roof sheathing panels in order to assess the effect of in-service conditions such as moisture cycles and,
- Compare the wind uplift resistance of wood roof sheathing panels tested statically vs. dynamically in order to assess the effects of dynamic loading.

Organization of Thesis

Chapter 2 presents a literature review of the current knowledge of wind uplift pressure testing for residential roofs, construction and vulnerability of wood residential buildings. The structural load path associated with wind events in typical residential wood roof sheathing system construction and failure mechanisms of wood roof sheathing systems are established.

Standardized commercial wood roof sheathing test protocols are reviewed to provide basis for development of the wind uplift test protocol to be used in testing.

A new standardized wind uplift test protocol is developed in Chapter 3 to evaluate the wind uplift resistance of wood roof sheathing panels. The protocol include a static pressure test method that is used to compare results with previously reported studies in the literature, and a dynamic pressure test method that provides more realistic simulation of actual wind forces.

Through wind uplift testing this investigation will a) evaluate the uplift performance of new panels and aged wood roof sheathing panels harvested from existing residential structures, b) compare the failure mechanisms and failure pressures of roof panels tested using new static vs. dynamic pressure test protocols, c) evaluate the effect of retrofit on existing roof wind uplift capacity and d) evaluate the effect of specific gravity on failure pressure of wood roof sheathing. Descriptions of sample construction/harvesting and test equipment are presented in Chapter 4. Chapter 5 presents the results and analysis of wind uplift tests performed. A discussion of findings relating to the objectives is then presented in Chapter 6. Chapter 7 makes conclusions based on results and provides recommendations for future use in a standardized test method.

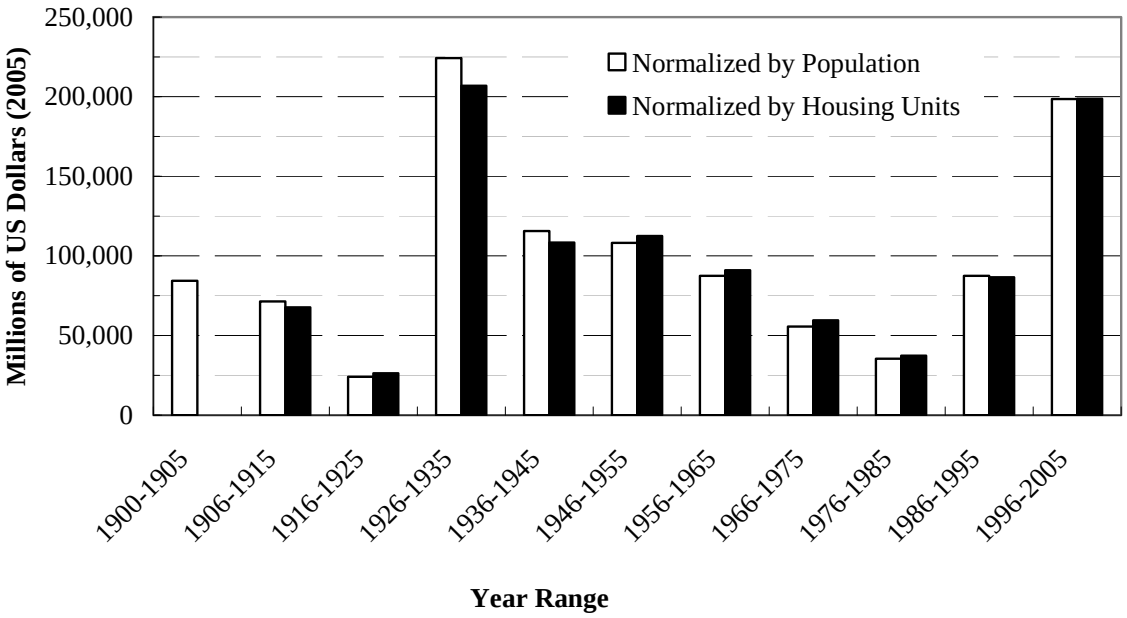


Figure 1-1. Total damages due to hurricane events by decade normalized to 2005 US currency by population vs. housing units presented in a paper by Pielke et al. (2008)

Figure 1-2. Structural damage due to loss of roof sheathing below design wind speeds, design wind speed 140 mph actual wind speed 120 mph Ono Island, AL (FEMA et al. 2005b)

CHAPTER 2
LITERATURE REVIEW

Wind Loads on Low-Rise Buildings

Natural winds exhibit considerable and rapid fluctuation in its velocity and direction, and this fluctuation (or turbulence) of the wind produces pressures on bluff bodies in its path that vary both spatially and in time. Wind tunnel studies have determined that the wind pressure vary over the building surface, with generally positive pressures being created on the windward walls and negative pressures on the leeward and side walls of the structure. In low-rise buildings which have sloped roof structures, fairly high negative pressures can be created in flow separation zones (at roof corners, ridges or edges) (Holmes 2001), see Figure 2-1. These negative pressures or suctions can be several times greater than the positive pressures on the windward walls. These suction forces are responsible for the wind damage to roof structures of low-rise structures observed after recent hurricanes. To estimate the pressure in the regions of flow separation Bernoulli's equation in the form (eq. 2-1),

$$C_p = 1 - \left(\frac{U}{U_0} \right)^2 \qquad (2\text{-}1)$$

is used where C_p is a non-dimensional pressure coefficient, U is the wind speed at the location being measured and U_0 is the reference wind speed. Strictly speaking Bernoulli's equation is not applicable in separated flow regions but it has been found that reasonable predictions can be found. The relationship of wind velocity to the resulting pressure force variations is described by the aerodynamic admittance function.

Wind speeds are highly variable and the resulting pressure forces depend greatly on the geometry of the structure making it impossible to solve for loads deterministically. Therefore stochastic methods are employed to envelope results to account for all possible combinations of

variables in determining design pressure loads. This enveloping process is conducted by analysis of both wind tunnel testing of structural models of different roof shapes and expected wind speeds for a particular location. Wind tunnel tests result in non-dimensional C_p pressure coefficients for a particular roof shape to be scaled by the design wind speed. Design wind speeds are developed by computer modeling to determine the probability of a wind speed being experienced at any location in the US. These wind speeds are defined at 10 meters (33 ft) above ground level in open exposure ,and must be adjusted to both the mean roof height and exposure condition to be applied to wind tunnel determined pressure coefficients. This stochastic analysis is summarized ASCE 7 (2006), which is the current reference manual used to determine wind uplift loading on low-rise residential structures.

The turbulent nature of wind loading results in “gusts” which can be significantly higher than mean wind speeds. These gusts are incorporated into design loads by categorizing the surface area a connection is designed to resist pressure loading over or “effective wind area”. The smaller the effective wind area subjected to a gust the larger the effect that gust has on the connection, because the gust load is not able to be distributed. This is accounted for in ASCE 7 by two categories main wind force resisting systems (MWFRS) and components and cladding (C&C), which are further broken down by actual effective wind areas. Design wind speeds are developed by computer modeling to determine the probability of a wind speed being experienced at any location in the US. These wind speeds are defined at 10 meters (33 ft) above ground level and must be adjusted to the mean roof height to be applied to wind tunnel determined pressure coefficients. Pressure loads are summarized in figures by cross-referencing effective wind areas by roof zones. The zones on a roof are determined from the analysis of wind tunnel studies and include field, edge, corner and overhang zones, where corner and overhang zones have the

highest load. Described above are the external pressure coefficients which are a result of building geometry, as pressure loading is a result of difference in pressure it is also necessary to know the internal pressure to determine a load. The internal pressure during wind loading is dependant on the openings in a structure through which wind can affect internal pressures. ASCE 7 accounts for this by using three categories open, partially enclosed and enclosed. ASCE 7 design pressures are determined from roof geometry, location, exposure, mean roof height, effective wind area and openings.

Response to Earlier Historical Hurricane Damage

Following Hurricane Hugo (in 1989) which caused $7 billion dollars in losses and Hurricane Andrew (in 1992) which caused $26 billion dollars in losses, several organizations initiated wind uplift testing on wood roof structural components. The results from those tests have informed many of the wind load design requirements for residential roof construction. The research was initiated after numerous observations of premature roof failures indicated the vulnerability of wood roofs on single-family residential structures.

Sparks (1991a) estimated that approximately 60% of the total damage from Hurricane Hugo occurred to residential buildings, the majority of which occurred to the roof structure. An analysis of insurance claims from Hugo found that for nearly all case-studies the vast majority (95%) of losses were a direct result of failure of roof materials (Amirkhanian et al. 1994). Keith and Rose (1992) observed that 24% of wood-framed residential homes in their study area (Miami, FL) lost at least one roof sheathing panel during Hurricane Andrew. The consequences of roof sheathing failure is significant for two reasons: 1) as sheathing fails the roof structure can lose its structural integrity resulting in structural failure and collapse and 2) sheathing loss creates openings in the roof that allow water intrusion that causes severe water damage to interior partitions and building contents (Cook Jr 1991)

The connection of wood sheathing to wood framing members was identified as a key factor in wood roof system failure. Experimental studies were conducted by Cunningham (1993), Schiff et al. (1994) and others to investigate and improve this connection. These studies used roof panels installed on a rigid pressure chamber where the chamber pressure is increased until failure occurs. Failure defined as the first failure of a nailed connection or fracture of the wood panel or wood framing member. Test results are reported as the mean failure pressure (psf) and the design (or allowable) wind uplift capacity is determined by dividing the mean failure pressure by a factor of safety (i.e. 2.0) or using the 5% exclusion value of the data. A full treatment of experimental studies into wind uplift loading of residential wood roof sheathing is provided in a later section.

Recent Hurricane Damage to the United States

Since 1900 the monetary loss associated with hurricanes in the United States is approximately $10 billion normalized to 2005 USD (Pielke et al. 2008), which suggests consistent potential for loss. Most recently the 2004 and 2005 storms have totaled over $150 billion dollars in damages (Pielke et al. 2008). Hurricane Katrina was the largest contributor causing over $125 billion of damage followed by Charley ($15 billion) and Ivan ($14 billion).

Wind damage during hurricane events has historically shown to be responsible for the majority of losses. For example Baskaran and Dutt (1997) estimated that 95% of losses from Hurricanes Iniki and Andrew in 1992 resulted from the failure of roof system materials under wind loading. However significant losses can occur from flood damage, which is caused by an increase in water levels which is referred to as a “storm surge”. Hurricane Katrina is an example where flood damage has far exceeded wind damage. The damaging effects of wind loading can reach further inland than storm surge limits and therefore present a risk to larger areas than flood damage.

Wood roof sheathing panels fail in hurricane wind loading by removal or separation of the sheathing from the wood framing members. Failure occurs when individual fastener connections fail or to a lesser extent when wood components fracture. The complete removal of a panel is not necessary to consider a panel to have failed, as water intrusion will increase even with partial panel removal as observed in Hurricane Hugo (Murden 1991) and elsewhere. Nail withdrawal from the framing member is the primary mode of failure observed in roof sheathing panels due to wind loading, with nail pull-through and fracture of the sheathing and/or wood framing observed to a lesser extent. Nail pull-through is a punching failure of the sheathing where the nail shank remains embedded in the wood framing while the sheathing is pulled away.

Hurricane Charley a category 4 hurricane on the saffir-simpson scale struck Punta Gorda, FL on August 13th , 2004 (FEMA et al. 2005a). The storm tracked north-west across Florida exiting in the Daytona Beach area approximately 10 hours later causing total flood and wind damages of approximately $16 billion. From a post disaster investigation it was observed that the majority of damages to residential structures occurred in homes constructed prior to code changes in 1994. This suggests the reduced fastener spacing and increased size have resulted in improved performance as compared to homes built after 1994.

Hurricane Ivan a category 3 hurricane on the saffir-simpson scale struck Alabama and Florida coastline on September 19th, 2004 (FEMA et al. 2005a). The storm tracked north west over the next week eventually exiting from the Delaware, Maryland, Virginia peninsula causing approximately $15 billion in losses. Again it was observed that the majority of damage occurred to construction built after 1994. It was observed that wood sheathing failures occurred in wind speeds below design level winds. For example roof sheathing failure was observed in 120 mph winds (estimated from FEMA HAZUS maps) at Ono Island, Alabama where current design wind

speed is 140 mph (Figure 2-2). Damage to roof sheathing was also observed in newer construction below the design wind speeds.

Hurricane Katrina struck southern Florida as a category 1 hurricane on August 25, 2005 then after strengthening in the Gulf of Mexico struck the Louisiana and Mississippi coasts as a category 3 hurricane (FEMA et al. 2006). As mentioned before the majority of the total estimated $125 billion in losses was caused by flooding. The majority of wind damage was observed in pre-1994 construction. Wind damage was also observed in new construction below design level wind speeds (van de Lindt et al. 2007).

The extensive monetary losses experienced from recent hurricanes have put the affected communities under considerable strain to cope with recovery. What's more the homes destroyed by either the resulting structural or water intrusion damage cause disruptions to thousands of residents. It is estimated that Hurricane Katrina destroyed over 300,000 single family homes in Louisiana and Mississippi displacing 450,000 thousand residents (FEMA et al. 2006).

Construction of Residential Houses

Low-rise residential wood roof structures are the focus of this investigation and the load path associated with resisting wind uplift pressure loads defined above is discussed in this section. The typical residential structure is composed of a roof system which transfers wind load to the wall system and then on to the foundation (Figure 2-3). The structural roof sheathing system is composed of wood sheathing attached to wood framing members. Roof sheathing is the part of the building envelope which provides weather resisting barrier to the structures and in many cases its lateral stiffness. Wind loading on roof sheathing panels are resisted by individual fasteners (nails or staples), which are installed into the wood framing. Roof structures can consist of individual rafters or more commonly metal-plate connected trusses.

These components are fastened to the walls using toe-nailed connection in older homes or using metal strap connections in contemporary construction. Typical spacing of wood framing roof members is 24 in. on center. Oriented strand board (OSB) sheathing have been used as roof sheathing the 1980's (Feng et al. 2008). OSB is a composite material consisting of small wood chips glued together and oriented in random directions. Plywood is also a composite material composed of layers of thin wood sheets adhered together in alternating orthogonal directions.

Current International Building Code specifies the use of ½ in. thick plywood sheathing (ICC 2006). The code also specifies the minimum fastener sizes and spacing to meet design wind uplift capacity for residential wood roofs. Fastening schedule written as x in. / y in., defines the spacing of fasteners installed along the exterior panel edge the nail spacing's on interior panel members. The minimum code requirements for installing wood roof sheathing systems for building construction before 1994 were 2 in. long nails with 0.113 in. diameter (6d Common) at 6 in. / 12 in, see Figure 2-4. Approximately 86% of the current building stock are constructed before building code modifications were introduced in 1994 to improve the wind resistance of structures (US Census Bureau 2003). Wood roof sheathing systems built after 1994 in high wind zones in Florida are required to use a 2-1/2 in. long 0.113 in. diameter annularly threaded nails (8d ring shank) at 6 in. / 6 in. or 4 in. / 4 in. at gable ends (ICC 2004).

Many residential roofs overhang the exterior walls along the eaves and at gable ends. These overhangs are typically 12 to 18 in. long. Light soffit materials are installed below the cantilever wood members and the soffits also have vents that allow air flow into the attics. This overhang allows pressure to act on both faces of the sheathing extending into the overhang which has been observed to be the location of increased failures (Figure 2-5).

Wind Uplift Behavior of Residential Wood Roof Sheathing Panels

Design of roof sheathing systems currently relies on full-scale system testing to measure roof wind uplift capacities. Residential wood roof systems have no industry-recognized test protocols for determining their wind uplift capacities. Currently accepted loading methods use static pressures applied uniformly to the full-scale test specimen. It appears that residential wood roof sheathing design is based on a limited number test studies conducted in the 1990s in response to extensive damage associated with roof materials observed in Hurricanes Hugo (1989), Andrew (1992), & Iniki (1992).

The majority of the published work from 1994 through 2008 on residential wood wind uplift testing was carried at research institutions; Clemson University (Jones 1998; Mizzell 1994; Murphy et al. 1996; Sutt 2000), FIU (IHRC 2004), while other testing was performed at the APA (Cunningham 1993), NAHB (NAHB 2003) and Stanley-Bostich (Reinhold et al. 2003).

The wind uplift capacity or structural resistance of wood roof panels can only be determined by wind uplift testing. Currently there are no analytically based methods for predicting the wind uplift capacity of wood roof sheathing panels. The current design method is based on static uplift pressure tests to determine failure capacities of specimens. Fastener load is calculated by the product of the failure pressure (psf) and the tributary area (ft^2), which results in units of pounds. A sample of tributary areas for a panel fastened with a 6 in. / 12 in. spacing can be seen in Figure 2-6.

Variability in Uplift Capacity Testing

The lack of a standard residential roof sheathing uplift test protocol is a possible cause for the variability in pressure test results. The natural variation of wood strength, moisture content and the wide variety of fasters which may also contribute to observed variability in results, which range from 26 psf for a 6d smooth shank at 6 in. / 12 in. to 397 psf for a 8d ring shank nail at 6

in. / 6in. Tests were performed using pneumatic air pressure or air bags and the failure pressure was defined as the pressure at which the first sign of structural distress or failure occurred. The results were reported as the ultimate failure pressure from which nail type and fastener schedule design values were determined by dividing the ultimate pressure by a factor of safety (typically 2.0) or 5% exclusion values (Kallem 1997).

Table 2-1 summarizes results of studies into wind uplift failure capacities conducted from 1993 to 2004. It can be seen that the mean failure pressure increases with nail size and reduced fastener spacing, and there appears to be similar performance of wood roof panels fabricated using OSB or plywood sheathing.

Comparison of these results are somewhat misleading because all of the differences in individual test methods used to obtain failure pressures such that reported tests lack a common basis to make comparisons. NAHB (2003) tests were conducted using air bags, while the remaining test used actual air pressure. The use of air-bags may result in different transfer of load to the sheathing that pressure loading does. Cunningham (1993) tested only one specimen per roof configuration. A further difference was observed even within institutions, such as Mizzell (1994) used a 40 psf/min rate increase versus the near-instantaneous loading that Kallem (1997) and Sutt (2000) relied upon.

Mean failure pressures vary significantly between studies. Panel attached with 6d smooth shank fasteners at a 6 in. / 12 in. schedule have mean failure pressures of 55 psf (15/32" plywood, Cunningham 1993), 65 psf (5/8" plywood, Cunningham 1993)) and 33 psf (plywood, Kallem 1997). Panels attach with 8d smooth shank fasteners attached at 6 in. / 12 in. had mean failure capacities of 79 psf (15/32" plywood, Sutt 2000), 67 psf (7/16 OSB, Sutt 2000) and 110 psf (1/2" plywood, IHRC 2004).

All told 22 wind uplift failure pressure studies found involved a total of 220 test specimens and the author was not able to find any other studies available in literature. Despite possible changes to materials that have been made over past 16 years the results from the 1990s and early 2000s are the basis of recent probabilistic studies and reliability models (i.e. Li and Ellingwood 2009 etc.). The current minimum fastener schedules in the Florida Building Code (ICC 2007) appear to still be based on these early test results. In successive versions of the Florida Building Code fastener schedules strengthened and nail dimensions increased in order to increase the wind uplift capacity of roofs. However, given the wide variation in methodologies used and spread of failure capacities observed it is difficult to have confidence in the test values used to develop these prescriptive guidelines.

Retrofit Measures

Retrofit of wood roof sheathing has been investigated at Clemson University for spray applied adhesives (Jones 1998). Samples were constructed with 4 ft by 8 ft by 15/32 in. plywood or 19/32 in. OSB attached to 2 in. by 4 in. framing members with 6d smooth shank nails at 6 in. / 12 in. Spray applied adhesive was applied in several combinations of full-bead and partial-bead between sheathing and framing members. Table 2-2 presents selected results from this study. It was found that significant improvement can be achieved through this approach.

In Field Testing

Judge and Reinhold (2002) reports on the destructive testing of several homes in Horray County, SC. Ten wood roof panels were tested and are presented in Table 2-3. Panel construction tested was connected with staples, 8d sinker nails, and 16d nails. Sheathing was 0.465 in. plywood, 7/16 in. OSB and 8 in. wide wood planking. Failure pressures ranged from 110 psf for staples spaced 3 in. to 5 in. apart to an extremely high 450 psf failure pressure for the planking.

Building Codes

Building codes specify the minimum requirements for construction of buildings within a jurisdiction. Their purpose is to protect life safety and provide a minimum standard for performance to protect public/private property. The design and construction of low-rise residential structures in Florida is governed by the Florida Building Code (ICC 2007) which has adopted ASCE 7 for reference as the minimum design loads of buildings. Currently the FBC specifies the construction of residential wood roof sheathing in high wind zones to be ½ in. plywood attached with 8d ring shank nails spaced at 6 in. Prior to 1994 equivalent provisions specified the connection of wood roof sheathing to be 6d smooth shank nails spaced at 6 in. / 12 in. Table 2-4 summarizes residential wood roof sheathing attachment in Florida from 1988 to present.

Wind Uplift Resistance of Dowel Type Fastener Connections in Wood

In previous sections the failure modes of roof sheathing panels are defined as the individual fastener connections failing in withdrawal or pull-through. This section reviews the current state of knowledge on withdrawal strength of roof fastener connections. The National Design Specifications for Wood Construction (NDS) (AF&PA 2005) requires that withdrawal capacities of all nail types & sizes which shall be used to resist uplift in roof sheathing be experimentally determined per ASTM D-1761 “Standard Test Methods for Mechanical Fasteners in Wood” (ASTM 2006a). In addition the nail pull-through capacity of wood based panel products is considered in the design of wood roof sheathing, and ASTM D-1037 “Evaluating Properties of Wood-Based Fiber and Particle Panel Materials” (ASTM 2006c) is specified.

ASTM D-1761 Protocol

ASTM D-1761 test protocol “Standard Test Methods for Mechanical Fasteners in Wood” determines the withdrawal strength of nails under a constant withdrawal rate of 0.1 in. / minute,

which is applied to failure. The test specimens are wood prisms with the nail driven at right angles to the face. Load is applied directly to the nail head wood prism is clamped to the testing machine. A minimum of 10 samples are tested with expected range of coefficient of variation (COV) between 15 to 30%. Larger number of samples is needed to obtain COVs in the 5 to 10% range due to the inherent variability in wood.

Effect of Rate of Loading on Withdrawal Resistance

Kallem (1997) conducted an investigation to determine the effect of nail withdrawal rate on the nail ultimate failure capacity. The test procedure used by Kallem was based on ASTM 1761 protocol and he compared strength versus six nail withdrawal rate; namely 0.1, 0.5, 1.0, 5.0, 10.0 in. per minute and near instantaneous~1/8 to a 1/10 of a second. Results were then analyzed for each 10 sample series. Samples were constructed with 2-1/2 in. 8d smooth shank nails (0.131 in. diameter) installed in spruce pine fir. Mean withdrawal capacities ranged from 153 to 165 lbs and COVs ranged from 21 to 34%. It found that for the given data there was no apparent relationship between rate of loading and withdrawal strength; however they note that the study is not conclusive and further testing should include conditioning of wood.

Further investigation was conducted at by Sherman (2000) which used four stages of loading (0.05, 0.1, 1.0, and 10.0 in./min.) to determine the effect of loading rates on fastener withdrawal strength in wood. Sample were constructed with two wood species, southern yellow pine and spruce pine fir, and three fastener, had driven 8d smooth shank nails, power driven 8d smooth shank nails and hand driven 8d ring shank nails, for 20 repeats each. Again there was no increase in strength as rate of loading increased, in fact withdrawal loads were slightly higher for the slowest loading rate. Results do not suggest a relationship between loading rate and fastener withdrawal strength.

Pull-Through Testing

The pull-through strength of OSB and plywood panels and effects of aging have been investigated in order to determine the appropriate failure mode for wood roof sheathing. Chui and Craft (2002) investigated pull-through strengths of roof sheathing system with plywood or OSB per ASTM D-1037 (ASTM 2000). It was found that unless sheathing, both OSB and plywood, is sufficiently thick (i.e. greater than 1/2 in.) the pull-through strength of 2-3/8 in. common and 3 in. common roof sheathing fasteners will be the controlling failure mechanism. Roof sheathing fastener strength is a function of both pull-through & withdrawal failure modes, where the lower capacity controls.

Age and Weathering Effects

Age and weathering effects on roof sheathing fastener withdrawal strength has been investigated in laboratory settings in several studies discussed below. Feldborg (1989) conducted long term withdrawal testing of annularly threaded and smooth shank nails to evaluate the effects of long term loading and humidity cycling. Nails were driven to three different depths and loaded with constant withdrawal load for two years while using five different humidity cycling treatments. After two years the withdrawal strengths of samples which did not fail were obtained by short term tests. It is found that long term loading has little effect on withdrawal capacity but that alternating humidity does reduce withdrawal capacity.

Chow et al. (1990) tested 6d smooth shank nails and 2 in. long 16 gauge staples for withdrawal and pull-through strengths after being exposed to (1) long term (5 years) weathering outside and (2) accelerated aging in the laboratory per ASTM D-1037 (1978) by soaking in water and drying in cycles. Results from plywood and OSB samples are summarized in Table 2-5. Mean withdrawal strength from 16 samples ranged from 31% to 125% of the corresponding original strength, and mean pull-through resistance ranged from 75% to 99% of the

corresponding original pull through strength. A possible reason for increased withdrawal strength is suggested to be due to corrosion.

Pye (1995) addressed the effects of nail driving/coating, shank type, framing species, and heat cycles. In testing he used 2 in. by 5 in. by 15/32 in. plywood sheathing attached to a 2 in. by 4 in. by 4-1/2 in. wood framing members of southern yellow pine and spruce pine fir. Pye found by testing different fasteners that coated or power driven nails had a higher withdrawal capacity than uncoated, hammer-driven nails. Ring shank nails had a 51% higher capacity than similar diameter smooth shank nails. Fasteners installed in southern yellow pine wood had the highest capacity except in the case of heat testing. Heat cycles ranging from 1 to 48 hours in duration were found to have the most significant effect, producing approximately 56% reduction, on fastener withdrawal capacity as compared with samples not exposed to heat cycling.

Nail Withdrawal Tests on Existing Residential Buildings

Laboratory investigations (Feldborg 1989, Chow et al. 1990 and Pye 1995) have found that humidity and heat reduces ultimate withdrawal strength of nails and staples installed in wood framing members under laboratory conditions. However the effect of humidity and heat cycles on the in-service performance of mechanical fasteners has not been well defined.

Sutt et al. (2000) developed a portable nail extraction device to measure nail withdrawal strength of nails installed in existing roof structures. The device uses a 2000 lb load cell connected to metal jaws that engage the nail heads (after sheathing has been removed), and lever arms used to pull this assembly vertically upwards. The load cell was connected to a digital readout that provides peak withdrawal load from the nail (Figure 2-7). Sutt et al. collected nail withdrawal loads from 200 samples taken from a single residential structure in Anderson, SC. The 2-3/8 in. long 0.113 in. diameter nails were installed in SYP and SPF. The mean nail withdrawal load and coefficient of variation for SYP results were 68 lbs/in. and 56%

respectively. The mean nail withdrawal load and coefficient of variation for SPF results were 42 lbs/in. and 59% respectively. The rather high variability in withdrawal loads and low mean capacities obtained by Sutt's work was attributed to aging effects associated with in-service conditions. Sutt also suggested that the withdrawal capacity of the nails were lower than expected from NDS (AF&PA 2005) design values also due to the effects of age and weathering of the wood.

Dynamic Loading

Investigations into the effects of uniform dynamic wind loading on flexible thermoplastic roofing systems attached with mechanical fasteners have been conducted by (Baskaran and Dutt 1997; Baskaran et al. 1999b). In this study component testing of individual nails installed in a ½ in. of plywood and OSB sheathing was conducted by applying sinusoidal load to the nails. When compared to static loading it was found that the mean withdrawal loads for the dynamic loading were reduced by 13 to 30 %.

The effects of dynamic loading on axially loaded dowel type fastener in wood is relatively unknown. The only such research has been conducted by Baskaran and Dutt (1997) described above. The varying load may cause damage to the frictional interface of the nail surface and the wood material. This type of damage may be similar to fatiguing effects in other materials where the number and amplitude of cycles determines the amount of damage.

The cumulative damage model used to describe the interaction of multiple durations at different amplitude was proposed by Miner (Benham and Warnock 1976) can be described by,

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \ldots = 1 \tag{2-2}$$

Where n_1 is the number of cycles at one stress level which would require N_1 cycles to cause failure, and n_2 is the number of cycles at a second stress level which would require N_2

cycles to cause failure. The idea is that the sum of the ratio of numbers of cycles at each stress level should sum to 1.0 or total damage.

In order to use this fatigue model for dowel type fasteners installed in wood it is necessary to determine the failure or stress level – number of cycles (S-N) curve, which would define how many cycles at a stress level it would take to fail a connection. To extend this fatigue model to wood roof sheathing uplift capacity it is first necessary to understand how pressure load is distributed to individual fasteners. As both the withdrawal strength of individual nails and the distribution of pressure load through a sheathing panel are extremely variable processes the problem would need to be solved through stochastic methods requiring large numbers of repeats.

Related Research at the University of Florida

This thesis seeks to advance the understanding of the structural performance of wood roof sheathing systems through component based testing of new and existing wood roof panel specimens. The study follows a research direction initiated through the Florida Coastal Monitoring Program (FCMP). FCMP was initiated at Clemson University in 1998 with three main goals; (a) the characterization of wind field in real loading conditions, (b) quantifying the resistance of existing residential construction structural components and (c) laboratory simulation of hurricane loading characteristics in component testing (Jesteadt 2006). Jesteadt developed and summarized methods to harvest existing residential wood roof sheathing panels. These procedures were used for removal and testing in this investigation to evaluate the effects of aging and weathering.

Summary

This chapter presented testing procedures, review of damage levels observed to wood structures in hurricanes, wind load on low-rise structures, structural behavior of wood roof

panels at ultimate wind uplift loads, and source of the previous wood roof sheathing panel uplift tests used as comparison in this study.

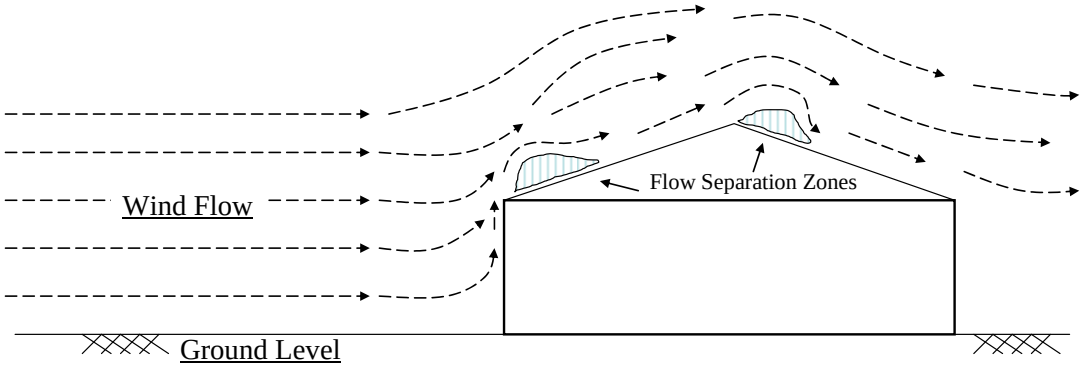


Figure 2-1. Resulting pressure loads from wind loading of residential structures

Figure 2-2. Structural damage due to loss of roof sheathing below design wind speeds, design wind speed 140 mph actual wind speed 120 mph Ono Island, AL (FEMA et al. 2005b)

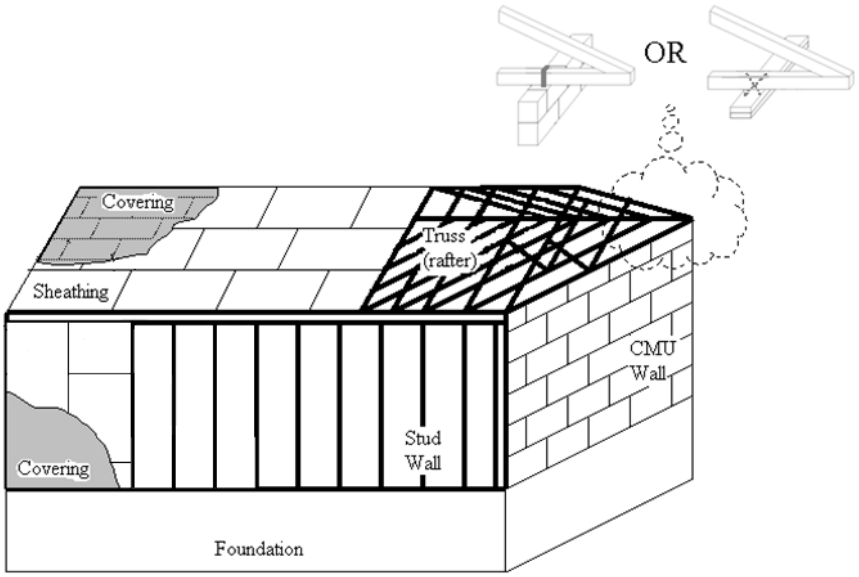


Figure 2-3. Typical residential structure construction with wood roof sheathing attached to metal plate trusses or rafter, which are attached to walls with either metal straps or toe-nails

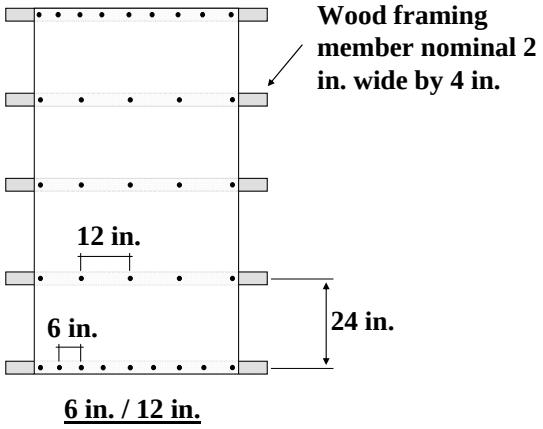


Figure 2-4. Fastener schedule and construction of roof panel designed by pre-1994 building code

Figure 2-5. Loss of roof sheathing at overhang locations, Hurricane Katrina (130 mph) long beach Mississippi (FEMA et al. 2006)

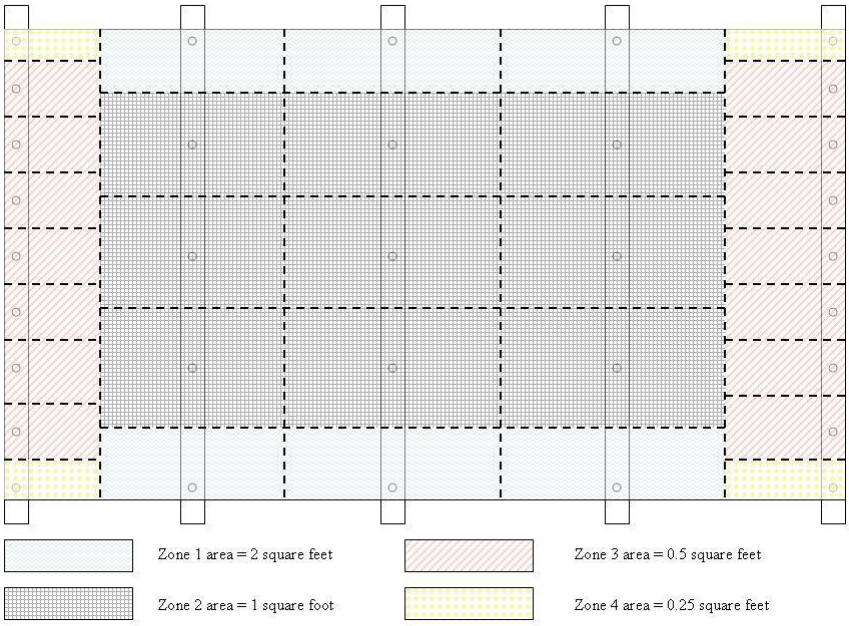


Figure 2-6. Tributary areas for individual fastener installed in a 6 in. / 12 in. spaced roof sheathing panel

Table 2-1. Wind Uplift Failure Pressure Research Conducted on Wood Roof Sheathing Panels (1993 through 2004)

Reference	*Sheathing Thickness, in.*	*Sheathing Type*	*Wood Member Size*	*Wood Member Species*	*Nail Size*	*Nail Spacing, in./in.*	*Number of Samples*	*Average Uplift, psf*	*COV*	*Distribution*	*Loading Regime*
(Cunningham 1993)	15/32	5-ply plywood	2x4	Douglas-fir or larch	6d Common	6/12	1	55	N/A	N/A	Monotonic 30-40 psf/min
	15/32	5-ply plywood	2x4	Douglas-fir or larch	8d Common	6/12	1	130	N/A	N/A	
	7/16	OSB	2x4	Douglas-fir or larch	6d Common	6/12	1	65	N/A	N/A	
	5/8	4-ply plywood	2x4	Douglas-fir or larch	8d Common	6/12	1	105	N/A	N/A	
	15/32	5-ply plywood	2x4	Douglas-fir or larch	6d Common	6/6	1	120	N/A	N/A	
	5/8	4-ply plywood	2x4	Douglas-fir or larch	8d Common	6/6	1	218	N/A	N/A	
	5/8	4-ply plywood	2x4	Douglas-fir or larch	8d Ring Shank	6/6	1	397	N/A	N/A	
(Mizzell 1994)	15/32	Plywood	2x4	SPF #2 or better	6d Common	6/12	4	26	0.09	Normal	Step: Increase 1 psf increments and hold for 1.5 sec
	15/32	Plywood	2x4	SPF#2 or better	8d Common	6/12	10	61	0.11	Normal	
	15/32	Plywood	2x4	SPF #2 or better	8d Common	6/6	10	107	0.16	Lognormal	
	19/32	Plywood	2x4	SPF#2 or better	8d Common	6/6	10	115	0.28	Lognormal	
	19/32	OSB	2x4	SPF #2 or better	8d Common	6/6	10	77	0.27	Normal	
(Murphy et al. 1996)	15/32	OSB	N/A	SYP #2 or better	8d Common	6/6	30	131	0.14	Normal	Step: Increase 1 psf for 1.5 sec
(Kallem 1997)	NA	4-ply Plywood	2x4	SYP	6d Common	6/12	14	33	0.22	Normal	Rapid Monotonic (failure in “less than 16 seconds”)
(Jones 1998)	19/32	OSB	2x4	SYP/SPF #2 or better	8d Common	6/12	10	87	0.28	Lognormal	Monotonic
	15/32	CDX Plywood	2x4	SYP/SPF #2 or better	8d Common	6/12	9	80	0.17	Normal	
(Sutt 2000)	15/32	Plywood	2x4	SYP	8d Common	6/12	7	79	0.09	N/A	Rapid Monotonic – Failure occurs within 10-45 sec
	7/16	OSB	2x4	SYP	8d Common	6/12	7	67	0.15	N/A	
(NAHB 2003)	7/16	OSB	2x6	SPF #1/#2	8d Common	6/12	3	228	0.07	N/A	Monotonic: 20 psf/min
(IHRC 2004)	1/2	CDX Plywood	2x4	SYP	8d Common	6/12	49	110	0.17	Lognormal	Monotonic
	1/2	CDX Plywood	2x4	SYP	8d Ring Shank	6/12	50	140	0.17	Normal	

Table 2-2. Summary of selected uplift pressure testing results from Jones (1998) investigation into adhesive retrofit methods for residential wood roof sheathing

	Control			Full-bead both sides			% Increase in Mean Failure Pressure
	# of Panels	Mean Failure Pressure (psf)	COV	# of Panels	Mean Failure Pressure (psf)	COV	
OSB	10	87	28%	4	185	16%	113%
Plywood	9	72		10	213		196%

Table 2-3. Summary of existing residential roof sheathing uplift testing conducted by Judge and Reinhold (2002)

House	Sheathing and Framing	Fastener	Spacing	# of Panels	Failure Pressure (psf)
1	0.465 Ply, 2" x 4" @ 24" o.c.	2.5" 0.113" dia.	6" / 8-12"	1	127
2	0.465 Ply, 2" x 6" @ 16" o.c.	2.5" 0.113" dia.	6" / 8-10"	2	232
3	7/16" OSB, rafter @ 24" o.c.	Staples	3-5"	1	105
4	7/16" OSB, rafter @ 24" o.c.	Staples	3-5"	2	110
5	rafter @ 24" o.c.	2.5" 0.113" dia.	7-9"/5.5-7.5"	1	196
6	rafter @ 24" o.c.	2.5" 0.113" dia.	7-9"/5.5-7.5"	2	119
7	8" Plank, rafter @ 16" o.c.	two 8d and one 16d nail per rafter		1	450

Table 2-4. Summary of building code requirements for roof sheathing design in Florida (1988 to current)

Year	Building Code	Nail	Min. Sheathing Thickness	Fastening Schedule
1988	South Florida Building Code (Dade County 1988)	6d common 8d common	½ in. or less greater than ½ in.	6 in. / 12 in.
1994	South Florida Building Code(Dade County 1994)	8d common	up to 19/32 in.	6 in. / 12 in., and 4" at gable ends
1997	Standard Building Code (SBCCI 1997)	6d common 8d common	½ in. or less 19/32 in. or greater	6 in. / 12 in.
2000	International Building Code (ICC 2000)	8d common	¾ in. or less	6 in. / 12 in.
2004	Florida Building Code (ICC 2004)	8d common	¾ in. or less	6 in. / 6 in., and 4" at roof corners
2004	Florida Building Code High-Velocity Hurricane Zone (ICC 2004)	8d ring shank	Minimum 19/32 in.	6 in. / 6 in., and 4" at gable ends
2006	International Building Code (ICC 2006)	8d common	¾ in. or less	4 in. / 8 in.
2007	Florida Building Code High-Velocity Hurricane Zone (ICC 2007)	8d ring shank	Minimum 19/32 in.	6 in. / 6 in., and 4" at gable ends

Table 2-5. Percentage of original strength summary of Chow et al. investigation of ageing effects on fastener withdrawal and pull-through resistance, (Chow et al. 1990)

Sheathing	Fastener	Withdrawal		Pull-Through	
		Outdoor	Lab	Outdoor	Lab
Plywood C-D grade	Nail	38%	38%	85%	77%
	Staple	61%	30%	87%	99%
OSB (pine)	Nail	31%	125%	75%	82%
	Staple	56%	18%	83%	97%

Notes: Nail-6d smooth shank; Staple-2 in. long 16 gauge staples; Lab aging per ASTM D-1037 (78)

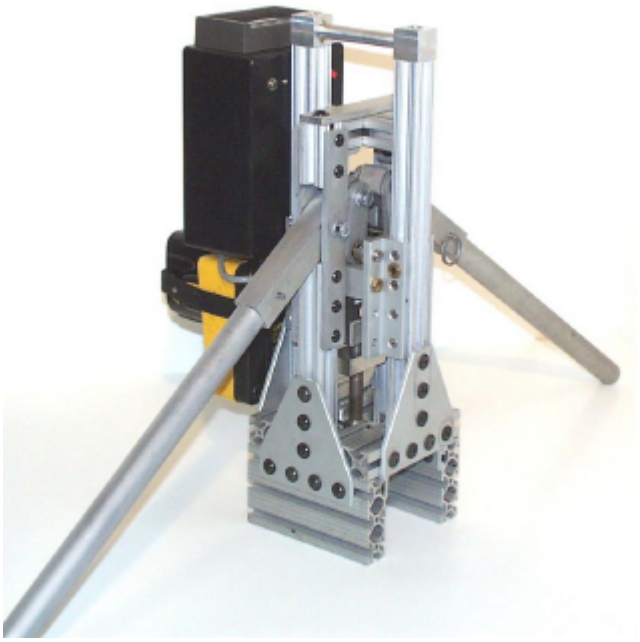


Figure 2-7. Nail Extraction Device Developed at Clemson University by Sutt (2000)

CHAPTER 3
DEVELOPMENT OF STANDARD WIND UPLIFT METHOD WITH STATIC AND DYNAMIC LOADING

The intent of any standard wind uplift test protocol should be to replicate the real effects of wind loading on building components, under controlled laboratory conditions, that the building component would experience in an extreme wind event. The test protocol should also be repeatable, consistently reproducing the same test conditions in every test, so that reasonable comparisons can be made for performance of any roof or wall system. For this reason simple uniform static test protocols have been used for the majority of commercial and residential roof sheathing tests. These tests assume that extreme wind loading can be modeled as a pseudo-static pressure load on a rigid building. Most wind uplift tests ignore the dynamic characteristics of the wind, assuming that structural component are rigid and therefore will not have a significant response to fluctuating loads (Holmes 2001). However it may not be an accurate assumption because it relies on the rigidity of the structure not the components. Residential wood structures are rigid systems but the stiffness of individual building cladding components (i.e. wood roof sheathing panels) may not be as rigid. Wind flow produces pressures and structural loads that vary temporally and spatially but these features are not represented in static test methods.

In order to evaluate wind uplift performance of wood roof sheathing panels a test protocol must be selected that reasonably recreates wind load conditions. As discussed in the previous chapter, no such protocol currently exists and still no widely accepted test methods are available for wood roof sheathing. Thus a first stage of this research was to develop a suitable wind uplift test protocol. It was decided to develop both a static test protocol having clear-cut guidelines, as well as a dynamic test protocol, to test the hypothesis that fluctuating wind load reduces the ultimate resistance of wood roof panels.

This chapter presents the development of the test protocol and two loading functions that will be used in the experimental studies following this chapter. The test protocol is called the ***University of Florida - Wood Roof Sheathing Uplift Test*** (UF-WRSUT) Protocol with Method A – Static Pressure and Method B – Dynamic Pressure. The main features of the UF-WRSUT Protocol are as follows:

In order to provide a basis for comparison the parameters effecting panel uplift resistance addressed are,

- The pressure traces are fully described to enable accurate reproduction,
- The number of repeats in each test series is established (a minimum of 10 repeats is recommended)
- Failure modes are observed and recorded,
- The material properties (specific gravities and moisture contents) of the wood sheathing and components are recorded,
- Details of geometric dimensions and characteristics of the fasteners are recorded,

During this investigation the above requirements were added incrementally as knowledge of critical parameters was gained through (1) a literature review of previous findings and (2) experience gained during testing. Therefore not all testing presented in this investigation adheres to these requirements. Additionally some requirements are not able to be followed due to limitations such as a lack of sufficient harvested samples, this is noted in the discussion.

UF Wood Roof Sheathing Uplift Test (UF-WRSUT) Protocol

Several experimental studies have been previously used to determine the wind uplift resistance of wood panels (Cunningham 1993; IHRC 2004; Kallem 1997; Mizzell 1994). For the purposes of this research a “static” pressure test is defined when the chamber pressure does not cycle or fluctuate. Static pressure tests may include periods of constant pressures held for specific lengths of time or periods where pressure is monotonically increasing or decreasing.

The UF-WRUST test protocol was adapted from the existing standard test protocol, ASTM E330-02 for determining the structural performance of exterior curtain walls and windows and doors (ASTM 2004b). ASTM E330 is intended only for evaluating the structural performance associated with the specified test specimen and not the structural performance of adjacent construction. This test method was the selected starting point instead of using wind uplift test protocols for commercial roofing tests because of the typical duration of each test was comparable to durations of previous studies. ASTM E330 acknowledges the time-dependency of strength and deflection characteristics in some materials and so it recommends testing assemblies for the actual time duration to which it would be exposed.

The test is conducted by sealing a test specimen against one face of a closed test chamber to which air is supplied or exhausted at a sufficient rate to maintain the pressure difference across the specimen. The ultimate failure pressure of the panel is the maximum pressure that it sustains without failure. Failure of the panel can be by nail withdrawal or pull through or by fracture of one or more wood framing members or of the sheathing. During a hurricane event a roof structure would typically experience several hours of elevated wind speeds and increasing gustiness, however typically wind speeds approach or exceed the design wind speeds of the house only during a few time periods. It is assumed that the extreme wind speeds are the sole cause of roof damage, and therefore the effect of lower intensity pressure duration and fluctuations are neglected. The 10 second period wind pressure is then representing the period in which peak pressure acts on the structure to cause damage. The static test will reproduce the most severe peak pressure but not the sustained buffeting that the roof may experience.

The UF-WRSUT, Method A Static Pressure Test Protocol

The UF-WRSUT, Method A Static test protocol uses the step-and-hold procedure identified by ASTM E-330 procedure B with a 10-second pressure plateau. The pressure control

can be applied manually or under computer-control. For the computer-control approach it is found that two 20 second long initial stabilizing pressure steps are needed; at 5 psf and again at 15 psf. The chamber pressure is increased in 15 psf increments, held for 10 seconds and increased again until failure. A typical pressure trace is shown in Figure 3-1.

The following adjustments to the ASTM E330 test protocol were made:

- Include initial pressure stabilization time steps of 5 psf and 15 psf for 20 sec. each.
- Apply pressure in one direction only; i.e. either suction (reduced chamber pressure) or pressure (increased chamber pressure), but not both.
- Eliminate deflection readings to observe permanent panel deformations.
- Eliminate the 60 second recovery period for stabilization during testing.
- Determine and report the moisture content and specific gravity of wood members.
- Record and report the nail properties (length, shank diameter, head diameter, coating material and deformed shank pattern).

Panel failure is defined as any permanent separation between sheathing and framing member, sheathing fracture, split in framing member or failure of nails (in withdrawal or pull through). The chamber pressure is monitored and continuously recorded during the test and the peak instantaneous pressure at failure is recorded. The determination of the peak instantaneous failure pressure is discussed in a subsequent section. Each panel is inspected after the tests and the locations and failure mechanisms of the fasteners are noted.

UF WRSUT, Method B Dynamic Pressure Test Protocol

The dynamic wind pressure trace was developed to better simulate the wind pressure fluctuations observed in actual wind loading. There are a few existing dynamic pressure test protocols used for roof sheathing, for example SIDGERS (Baskaran et al. 1999a). These test traces apply uniformly distributed pressures in regular cycles, amplitude and constant frequency, to the specimen. The more recent SIDGERS was developed using rain-flow analysis of wind

tunnel pressure data. The UF-WRSUT, Method B was also developed from a wind tunnel study but the characteristics of the fluctuations were retained.

Development of the UF-WRSUT, Method B dynamic trace is based on time histories of pressure fluctuations collected from wind tunnel data conducted at Clemson Universities boundary layer wind tunnel facility. The study (Datin and Prevatt 2007) used a 1:50 scale model of a 30 ft by 60 ft gable roof residential structure. The building had a mean roof height of 13 ft 6 in. and a 4 in 12 (18.4º) slope, Figure 3-2, and 387 taps were installed.

Upwind terrain was modeled as suburban terrain exposure, (z_o = 0.22 m) with a turbulence intensity of 24% at mean roof height in the tunnel (with building model removed). Only selected highlights of this experiment are reported here, and further details can be obtained in Datin and Prevatt (2009). For this study a simulated pressure trace was developed from the measured pressure coefficients at pressure tap #002, located at (16.75 in., 7.75 in.) from the ridge corner at full-scale, see Figure 3-3.

The dynamic pressure trace was developed using the reduced frequency relationship to provide an equivalent full-scale time step. This is done because wind tunnel measurements are taken at a faster rate than the pressure control system can process, therefore requiring that the data be compressed while retaining the frequency content of the wind tunnel measurements. A 10 second period pressure fluctuation was chosen so that it contained at least three peak pressure excursions of near equal and highest magnitude during the period. The wind pressure coefficient trace was converted to full-scale pressure trace and the highest peak pressure was matched to a pressure plateau (i.e. 15 psf, 30 psf, 45 psf, etc.) as in Method A static pressure trace, see Figure 3-4.

Failure of Wood Roof Sheathing Panels

The predominant failure mode for wood roof panels is nail withdrawal, followed by nail pull through failures and to a lesser extent fracture of the wood framing member or sheathing. Typically connections fail in fastener withdrawal or fastener head pull-through failure modes. The failure of an entire panel is a result of multiple connections failing in which individual connections progressively fail after the initial fastener. However the removal of an entire roof sheathing panel from the framing members is not necessary to consider the panel as failed since even partial removal provides a path for water flow into the structure. Additionally the nailed (dowel-type) fastener connections to wood, loaded axially in withdrawal is a very brittle one in which nearly all strength is lost after minimal displacement (Forest Products Laboratory 1999). Currently it is not possible to relate the load distribution behavior of fasteners in wood sheathing panels to failure pressures because there have been no studies conducted that directly measured the nail withdrawal load. Further it is not always possible to relate field performance of roof panels to failure pressures given the large number of parameters (construction quality, actual wind speed, wood specific gravity and condition, nail type and length, and spacing's etc.) for which no data is available. Further post-disaster studies do not usually detail the actual failure mechanisms observed in the panels.

It is proposed that to establish the relationship of fastener type and spacing, sheathing, and framing condition on roof panel performance the location and modes of failure for all fastener failures must be observed. Qualitative considerations such as the location of the connection failures within the panel and the type of failure (rapid versus slow) may also be important. It is noted however, that such visual observations of failure modes after the panel has failed cannot determine the location of the initial fastener to fail.

Determination of Panel Failure Pressure

The panel failure pressure recorded is the instantaneous peak pressure recorded at failure where panel failure is determined by examination of pressure trace. Pressure for panels tested in the static phase of the investigation were predominantly recorded with a peak measurement pressure gauge, therefore failure pressure for these samples is simply the peak instantaneous pressure from the test. The peak instantaneous pressure is selected as the panel failure pressure because it is the largest pressure the panel could withstand before permanent damage was inflicted.

A pressure transducer was incorporated to recorded time-histories for the ccSPF retrofit test series from which the peak instantaneous pressure was found (Figure 3-5). The benefit of the pressure time-history is that if for some reason the pressure spiked after panel failure this false peak pressure could be identified and discarded.

Panels tested in the static vs. dynamic phase of the investigation used a computer controlled system which measured the pressure time-history. The program shuts off if it determined there was significant leakage in the system resulting in a deviation from the target pressure. Pressure time-histories were evaluated to determine the peak instantaneous pressure which preceded failure. Figure 3-6 presents a sample of the pressure time-history for a panel fastened at 6 in. / 12 in. All time-histories of dynamically tested panels are presented in Appendix E.

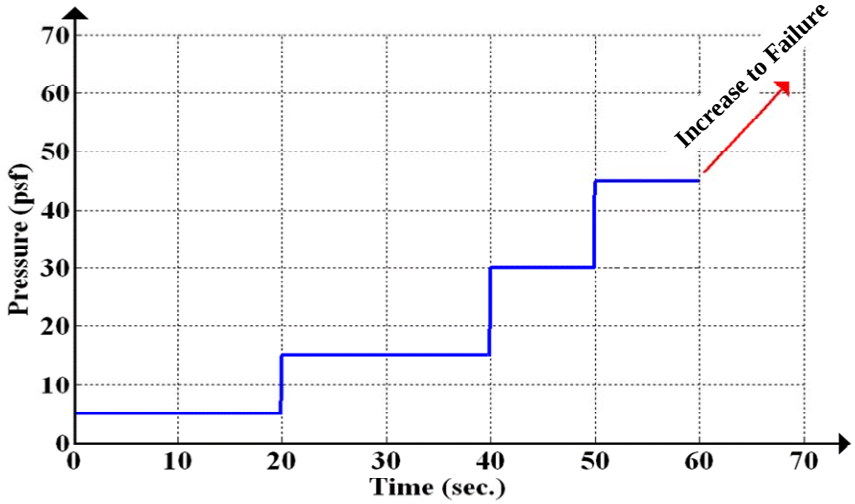


Figure 3-1. UF-WRSUT static pressure trace. (5 psf initial static pressure is included in the trace)

A B

Figure 3-2. Gable roof model A) installed in Wind Tunnel and B) close-up of model

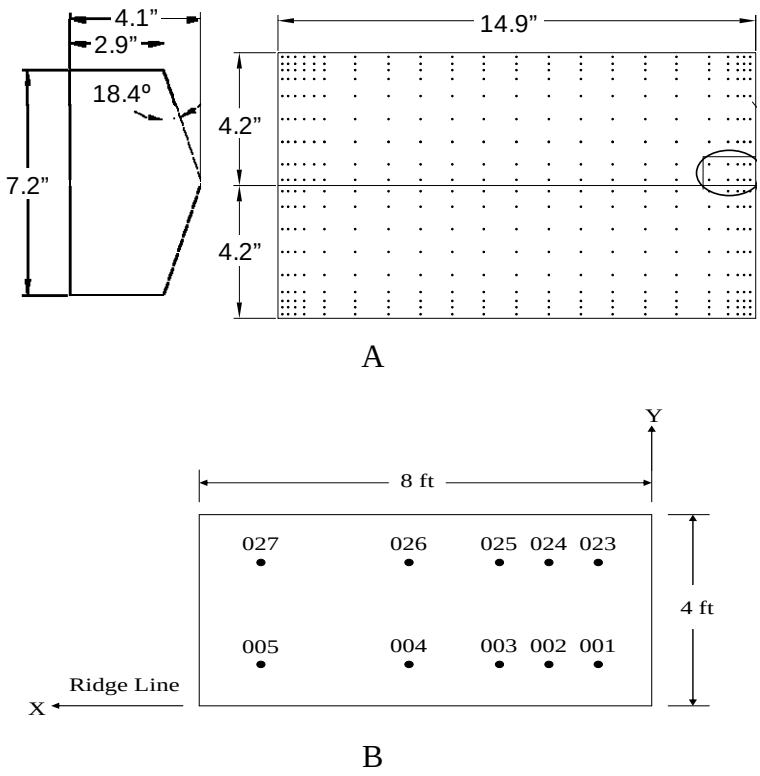


Figure 3-3. Pressure tap location A) 1:50 model scale (62 ft by 35 ft full scale) and B) full scale representation of panel

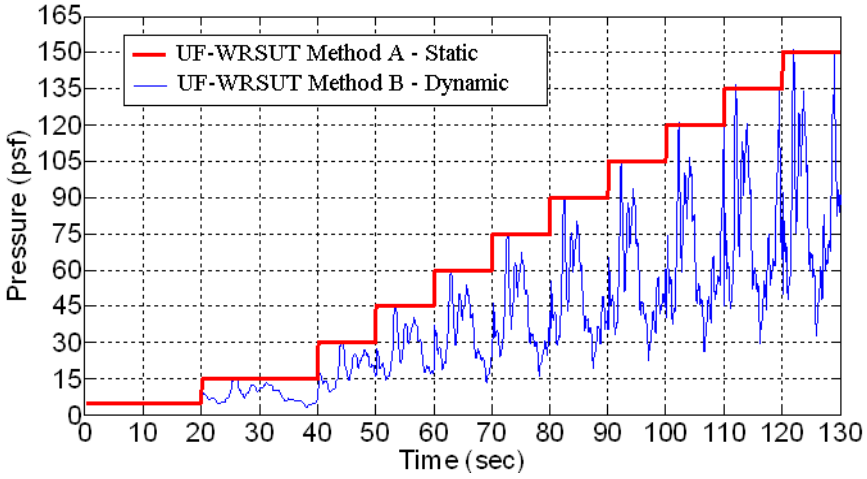


Figure 3-4. Comparison of static vs. dynamic pressure traces

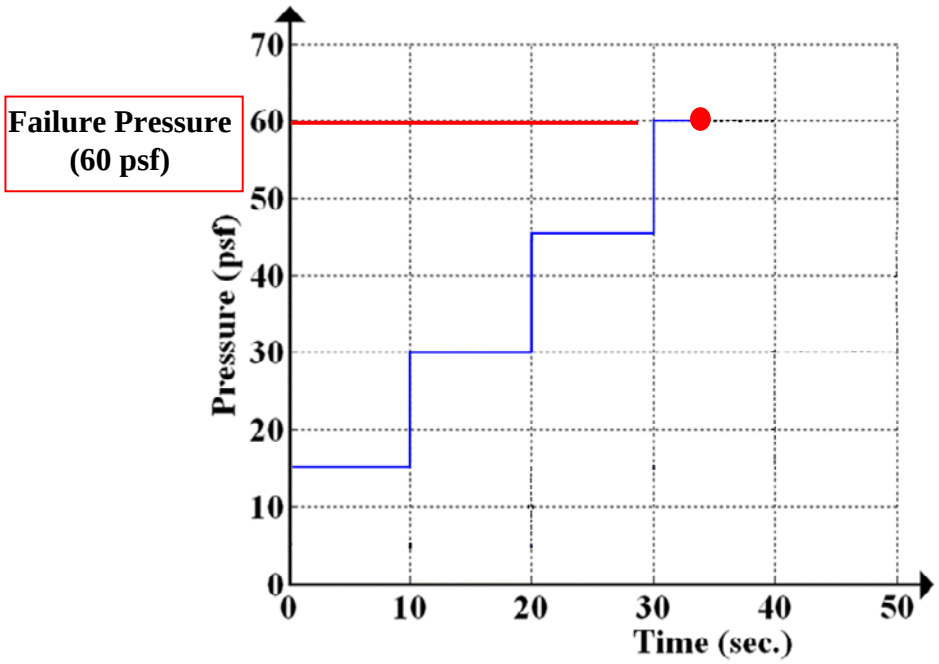
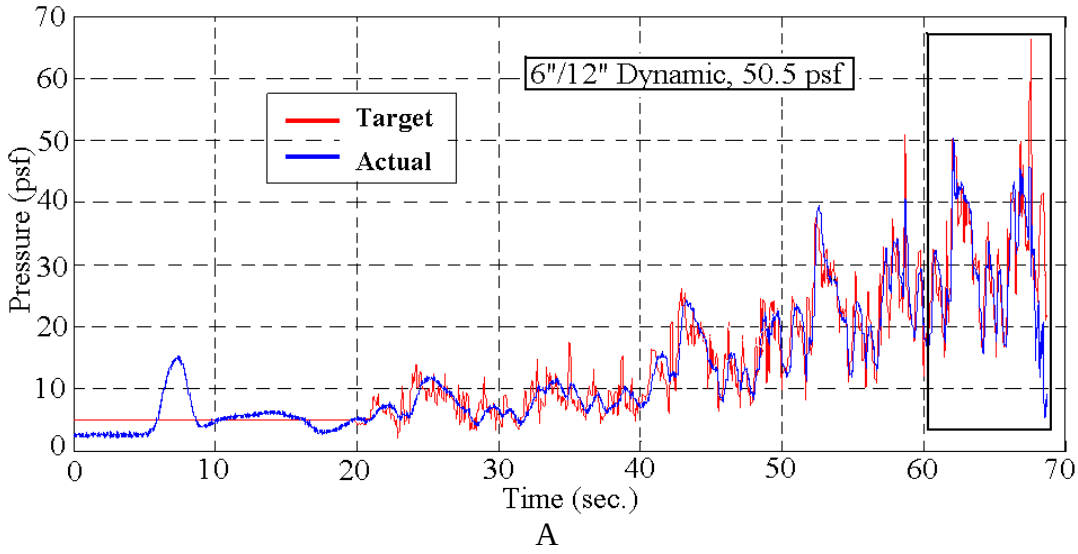
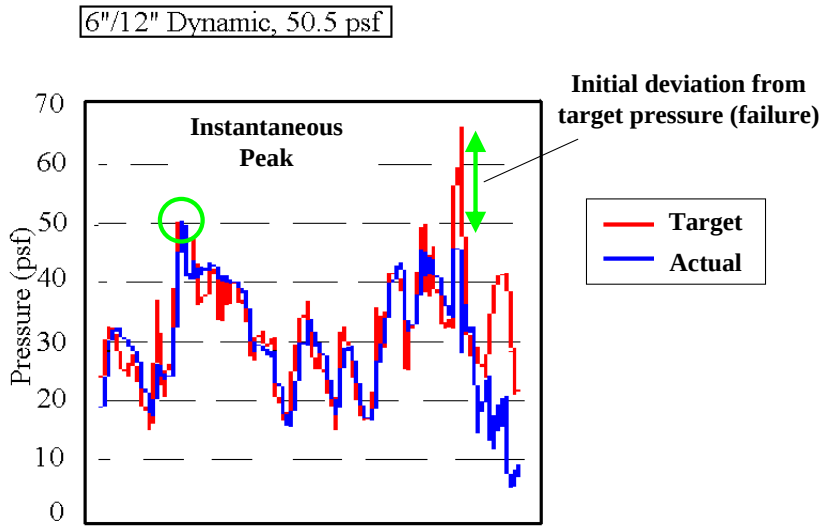


Figure 3-5. Diagram of instantaneous failure pressure for panel tested statically



A



B

Figure 3-6. Sample of target and actual pressure time-history for panels tested with the PLA system A) full time-history and B) close up of instantaneous peak and deviation from target pressure

CHAPTER 4
EXPERIMENTAL SETUP

Wind uplift pressure tests were conducted using 4 ft by 8 ft roof sheathing panels tested on a steel pressure chamber. Tests were conducted on both laboratory-fabricated panels and roof panels harvested from existing structures. The description of sample panel construction, harvesting, preparation, instrumentation, test equipment and procedures used in testing are presented in this chapter.

Panel Test Series and Construction

Laboratory fabricated panels were constructed in two groups with slightly different arrangements and different retrofit methods. The first group evaluated the effects of fastener type and spacing on uplift resistance of panels tested statically. The second compared the uplift resistance of panel tested statically vs. dynamically. Aged roof panels were harvested from 12 existing residential houses through the FCMP program over a year period from 2006 to 2009, and tested in two groups with different retrofit methods. Harvested panels have relatively small data sets due to the difficulties in procuring them, which is detailed below.

Panel Test Series

The first phase of testing is conducted on both laboratory fabricated panels and harvested panels, which is summarized in Figure 4-1. Testing was conducted to evaluate the developed test method (UF-WRSUT) and the effect of aging on panel uplift capacity. The second phase of testing is also conducted on both laboratory fabricated panels and harvested panels, which is summarized in Figure 4-2. Testing was conducted to evaluate the effect of dynamic loading on panel uplift capacity.

Laboratory Fabricated Panels (New)

Test panel specimens were fabricated using a 4 ft by 8 ft by ½ in. thick oriented strand board (OSB) sheathing fastened to 2 in. by 4 in. nominal dimensions southern yellow pine (SYP) framing members spaced 24 in. apart. The framing members were at least 4 ft-6 in. long in order to span the short dimension of the pressure chamber, see Figure 4-3. Nails were installed using a air compressor driven nail-gun, Stanley Bostitch Model FL21P. The air pressure was set at 70 psi to 80 psi and adjusted periodically to avoid over-driving the nails.

In the static phase of testing panels were constructed with three nail sizes; a) smooth shank 0.131 in. diameter 2-1/2 in. long nail (8d smooth shank nail) b) annularly threaded 0.113 in. diameter 2-1/2 in. long nail (8d ring shank nail) and c) smooth shank 0.113 in. diameter 2-3/8 in. long nail (2-3/8 in. 6d smooth shank nail). The 8d smooth shank and 8d ring shank nails were installed in three fastener schedules 6 in. / 12 in., 6 in. / 8 in., and 6 in. / 6 in. with approximately 15 replications in each set. The 2-3/8 in. 6d smooth shank nails were installed at 6 in. / 12 in. only with 15 replications, see Figure 4-4. To evaluate the structural benefit of retrofitting existing panels two arrangements of closed cell spray applied polyurethane foam (ccSPF) adhesive were tested. The ccSPF was installed on panels originally constructed with 8d ring shank nails spaced at 6 in. / 12 in. The panels were installed with (1) a fillet of ccSPF along each side of framing member to sheathing interface (see Figure 4-5 a) and (2) a full 3 in. thick layer of ccSPF filling each space between framing members (see Figure 4-5 b). A series of control panels were also tested with the same materials to provide a direct comparison, see Table 4-1 for a summary of panel series.

In the static vs. dynamic phase of testing laboratory fabricated panels were constructed with one nail type attached with two fastening schedules and one retrofit method. Hot dipped galvanized 2 in. 6d (0.113 in. diameter) smooth shank nails were used to match dimension of the

nails found in harvested panels. Two fastening schedules were used 6 in./12 in. and 6 in. /6 in. with 10 replications each, where half were tested statically and the other half were tested dynamically (Table 4-1). A 2 mils. thick plastic sheet was inserted between the wood stud and OSB sheet during fabrication to allow samples to be tested with positive chamber pressure (Figure 4-6), discussed below in installation of panel section. To evaluate the effect of dynamic loading on retrofitted panels four additional panels were constructed with the 2 in. 6d common nails at 6 in. /12 in., then installed with 8d ring shank nails between the 6d smooth shank nails.

Harvested Panels (Aged)

To evaluate the effect of in-service conditions on wind uplift resistance of residential structures panels were harvested from existing homes. Panels were removed from thirteen existing residential wood roof structures located in four communities throughout Central Florida, see Figure 4-7 for locations of structures. A total of 48 panels were harvested from 2006 thru 2009. Testing is broken into two groups the first conducted statically to evaluate the effect of age and two retrofit measures. The second test group is tested statically vs. dynamically in order to evaluate the effect of dynamic loading on roof sheathing wind uplift capacity. Panel attachment varies significant, in fastener type or spacing, resulting in limited sample sizes and relatively few direct comparisons to laboratory fabricated samples. In order to obtain panels researchers must wait for homes purchased by local or state governments to be made available before demolition. This is a slow process and results in researches getting what is available, which is rarely the ideal sample. Additionally the process of removing panels prohibits large numbers of panels to be removed from a single roof.

To access panels shingles are removed from the necessary section of roof. Panels are removed by cutting the sheathing around the sample panel with a circular-saw to expose framing members (Figure 4-8 a). This prevents any adjacent panel to be used as a sample panel therefore

keeping the number of panels removed from a single roof small, see Figure 4-8 c. The sample panel is then removed from the roof structure by cutting the exposed framing (Figure 4-8 b). Care is taken to brace the panel only on the sheathing during removal to prevent damage to fastener connections. The panel is then lowered to ground level to remove any additional truss members connected with metal plates, while only holding the sheathing. Metal truss plates are cut with a grinder while bracing the framing member to protect the fastener connections. Panels are braced only by the sheathing during transportation to prevent racking of the framing.

All harvested panels tested in the first group (38) to evaluate the effect of in-service conditions on wood roof sheathing wind uplift resistance consisted of 4 ft by 8 ft by ½ in. thick plywood sheathing. Sheathing is attached to 2 in. by 4 in. wood truss or rafter members spaced 24 in. apart with 1.5 in. staples, 2.5 in. staples, 2.5 in. long 0.131 in. diameter (8d) smooth shank nails and 2 in. long 0.113 in. diameter (6d) smooth shank nails at various spacing ranging from 4 in. / 4 in. to 6 in. / 12 in. detailed in Table 4-2. Panels were harvested from twelve homes located in three cities in Central Florida (Port Orange, Crystal City and Bartow) and ranged in age from 29 to 33 years. Twenty five harvested of the panels were tested in their existing configuration and thirteen panels were retrofitted. Retrofitting of panels was performed to simulate the economical strengthening of existing roof sheathing. Panels were retrofitted using one of two methods; a) twelve were retrofitted using 8d-ring shank nails installed at a 6 in. / 12 in. spacing which can be performed when a roof covering is replaced (retrofit A-1, see Figure 4-9), and b) one panel was retrofitted using a 3 in. thick layer of closed-cell spray-applied polyurethane foam (ccSPF) which can be installed through the attic at any time (Figure 4-10).

Ten panels were harvested from the same home in order to evaluate the effect of dynamic loading on wind uplift resistance of wood roof sheathing. Construction of panels are 4 ft by 8 ft

by 7/16 in. thick OSB attached to 2 in. by 4 in. framing members spaced 24 in. apart. Sheathing was fastened with 1-1/2 in. long 14 gauge staples spaced at 4 in. interior and exterior. Six panels were tested in existing conditions, two statically and four dynamically. The remaining four panels, two static and two dynamic, were retrofitted with 8d ring shank nails spaced between a 6 in. / 12 in. fastening schedule (retrofit A-2, see Figure 4-9) to correspond to static vs. dynamic testing of laboratory fabricated panels. Table 4-2 summarizes the test series.

Specific Gravity and Moisture Content Measurements

Specific gravity and moisture content of wood members are identified in Chapter 2 as parameters effecting the wind uplift resistance of wood roof sheathing. Therefore specific gravity measurements were taken for all static vs. dynamic testing to evaluate their effect on uplift resistance. Moisture content measurements were taken only for sample harvested from existing structures. Each reported framing member specific gravity or moisture content is the average of three samples, but methods for collecting samples were different between laboratory fabricated panels and harvested panels.

Oven dry specific gravity measurements were determined for framing members following procedures outlined in ASTM D 2395 (2006b), Method A. The equation for specific gravity presented in Method A (#2) is

$$SG = \frac{KW}{\left(1+\left(\frac{M}{100}\right)\right)Lwt} \tag{4.1}$$

which calculates the "green" specific gravity or in other words the specific gravity relative to the moisture content at the time of testing. It was that the definition used in the wood design manual NDS (AF&PA 2005) would be selected, which specifies the oven dry specific gravity. The term

$$\frac{W}{\left(1+\left(\frac{M}{100}\right)\right)} \tag{4.2}$$

is defined as the calculated oven-dry weight of specimen based on moisture content measurements. Due to the selection of oven dry specific gravity as the primary result of testing the oven-dry weight was directly measured. The resulting equation is

$$SG = \frac{KW}{Lwt} \tag{4.3}$$

where K = 27.68 unit conversion, W = dry weight (lbs), L = length (in.), w = width (in.), and t = thickness (in.). The oven used for drying is a 3488M Model Lab-line Instruments convection oven. Vernier calipers were used to measure the specimen volume and an Omega WSB – 8150 scale with a maximum capacity of 15 Kg was used to measure mass.

Moisture content measurements were conducted on harvested static vs. dynamic tested samples per ASTM D4442 Method B secondary oven dry method. Method B was selected due to its similarity to the specific gravity method used, therefore samples were simultaneously tested for specific gravity and moisture content. The precision of measurements is assumed to be the nearest whole percent because precision and bias calculations (section 5) were not made. Additionally endpoint measurements were not used because the drying time specified in the specific gravity was followed.

Framing members of panels fabricated in the laboratory are constructed with ten foot boards purchased together, prepared and allowed to dry for 2-3 weeks. Each 10 ft board was cut to make 2 framing members. Three samples were collected from each ten foot board, one on either end and one in the middle (Figure 4-11), and used to determine the specific gravity of both framing members cut from the ten foot board. ASTM D2395 specifies that specimens be cut at

least 18 in. from the ends of boards to “avoid effects of end drying”, however as oven dry specific gravity was being calculated this was not followed.

Framing members of panels harvested from existing structures are cut longer than needed for uplift testing in order to get specific gravity/moisture content samples. Three samples were removed from the ends of framing members, see Figure 4-12. End drying effects do not apply due to the fact that framing members were cut from the middle of the roof structure, therefore moisture content measurements were taken.

Uplift Testing Equipment

The objectives of testing as part of this thesis are to evaluate the effects of dynamic loading and in-service conditions on wood roof sheathing wind uplift resistance. In order to do this a test method was developed with static and dynamic load traces, which is detailed in Chapter 3. The equipment with which testing is conducted is developed through-out the testing conducted. All panel samples were tested in the same pressure chamber. The pressure chamber is a steel chamber measuring 4 ft. – 6 in. wide by 8 ft. – 6 in. long and 6 in. deep. Laboratory fabricated and harvested panels tested statically are tested with manual control of chamber pressure. Pressure is measured with a peak measurement gauge for all statically tested harvested panels and most statically tested laboratory fabricated panels. The pressure gauge was replaced with a pressure transducer for some statically tested laboratory fabricated panels. Panel tested with manual pressure control the pressure is supplied with two vacuum pumps in series, detailed below. Static vs. dynamic testing required higher accuracy of pressure control therefore a different system was used (Pressure Load Actuator), which is described below. As part of the feed-back control system pressure time-histories were recorded with a pressure transducer.

Panel Installation

Pressure is applied in one of two different methods, either negative or positive applied pressure depending on whether the specimen was lab built or harvested from an existing house. In both arrangements an uplift load is applied to the panel.

The negative pressure arrangement was used for all panels tested statically and harvested panels tested statically vs. dynamically. Panels are placed on top of the chamber then covered with 2 mil plastic leaving extra folds at framing to allow continuous contact of plastic and sheathing. Duct tape is used to seal plastic to chamber, see Figure 4-12. Then a shop-vac is used to remove the excess air in system, which can affect pressure control. After loading is completed the panel is removed from chamber and inspected for fastener failure type and location.

Laboratory fabricated panels tested statically vs. dynamically are tested in the positive pressure arrangement. Panels tested with positive pressure are constructed with plastic between the sheathing and framing. Panels are placed inside the chamber sheathing up to restrain the panel. Shims and wood blocking are then used to restrain the panel from movement during loading. Finally a steel frame is placed over the plastic and clamped tight, see Figure 4-13. After loading, the panel is removed from the chamber and inspected for fastener failure type and location.

Manual Control

Laboratory fabricated and harvested panels tested statically were controlled manually. Pressure was supplied with two 15 CFM US Vacuum pumps (model CP15) connected in series with PVC tubing to the pressure chamber supply pressure, see Figure 4-14. Pressure is controlled manually with a gate valve (Figure 4-14 a) which is closed to reduce the chamber pressure. Pressure measurements were initially recorded with an Omega DPG8000 general

purpose digital peak load measurement pressure gauge, later pressure measured with an Omega, Model # PX243A-2.5BG5V pressure transducer.

Pressure Load Actuator

Static vs. dynamic testing requires a more accurate system to control chamber pressure. The accurate development and repeatability of the dynamic pressure trace was only possible because of the unique capabilities of the test apparatus used. The Pressure Loading Actuator (PLA) system utilizes a 12 hp regenerative blower and computer-controlled feedback loop to actuate a 3 port valve with 5 arrangements, see Figure 4-15. The active valve control system and blower enable highly responsive control of chamber pressure resulting in a chamber pressure that closely follows the large amplitude pressure fluctuations and 10 Hz response of the simulated dynamic wind pressure. The PLA system was developed by Cambridge Consultants for the University of Western Ontario to investigate wind loading of full scale structures. Development of the PLA system can be found in Kemp (2008). The system was designed to provide pressure loading to boxes (chambers) ranging from 1 to 64 ft^2, which are placed on a structure allowing spatially varying dynamic pressure loading of an entire structure. This ongoing study is detailed in Bartlett et al. (2007). The PLA system enabled accurate repeatable dynamic pressure loading of wood roof sheathing panels, see Figure 4-16.

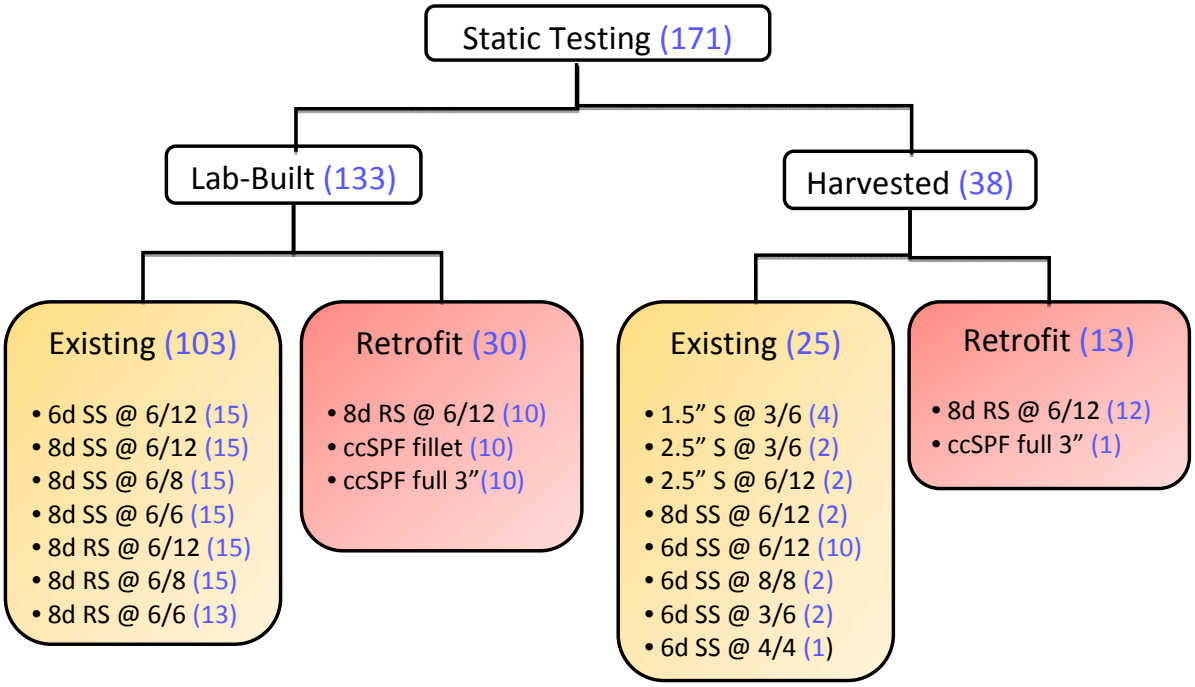


Figure 4-1. Summary of panel series tested in the static phase of this study

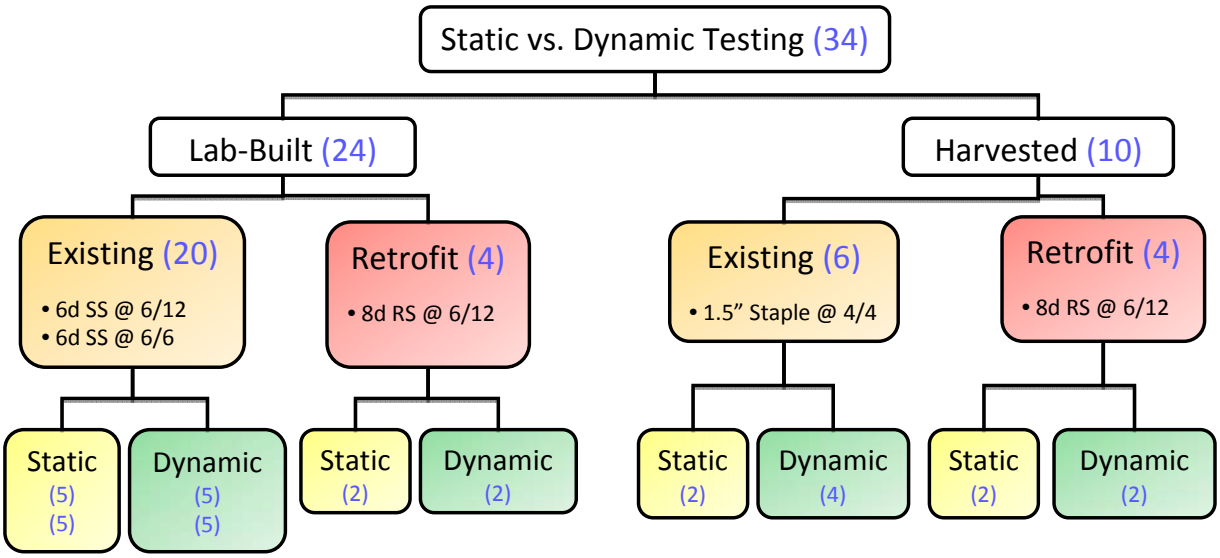


Figure 4-2. Summary of panel series tested in the static vs. dynamic phase of this study

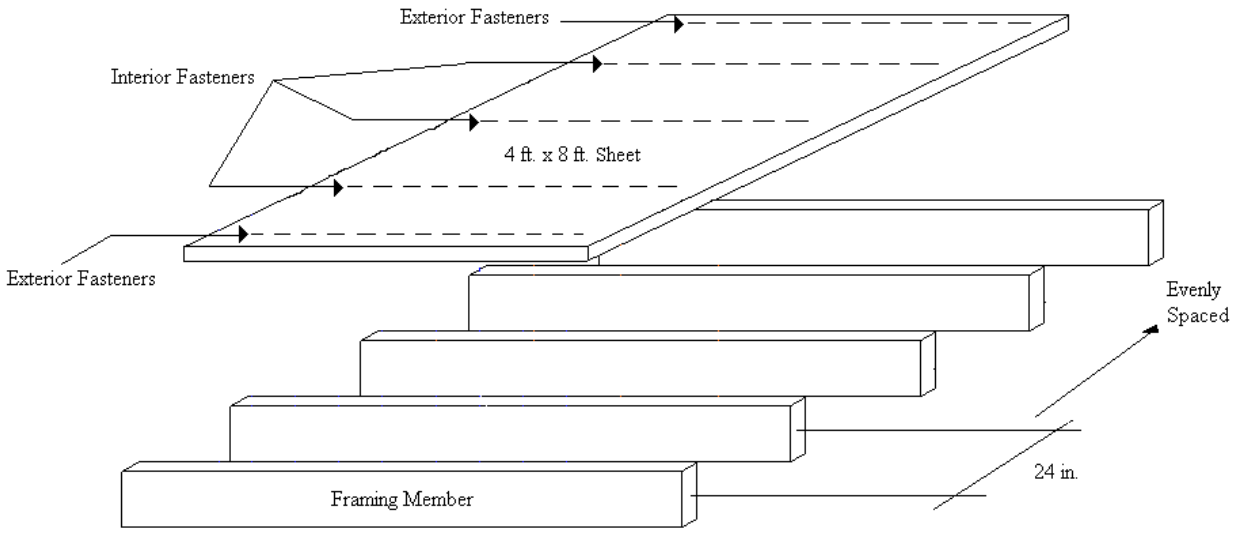


Figure 4-3. Laboratory fabricated panel construction, 6 in. / 12 in fastener schedule shown

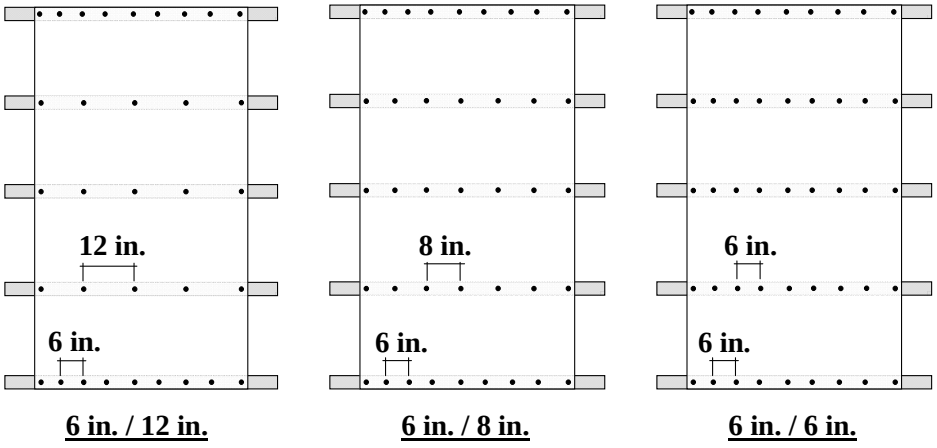
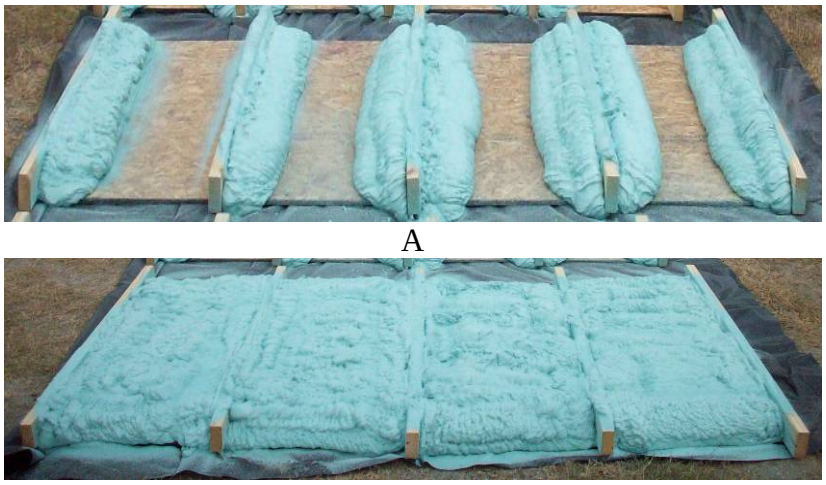


Figure 4-4. Laboratory panel fastening schedule (6 in. / 12 in., 6 in. / 8 in. and 6 in. / 6 in.)



A

B

Figure 4-5. Static ccSPF retrofit construction of A) fillet sample set and B) full 3 in. sample set

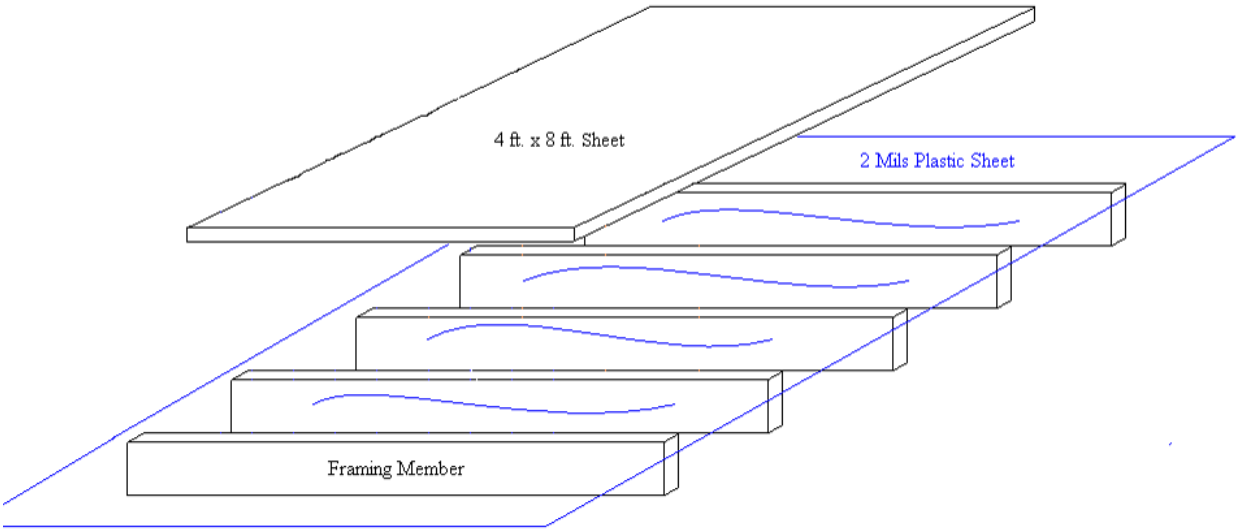


Figure 4-6. Panel construction for static vs. dynamic testing with 2 mils. thick plastic sheet placed between sheathing and wood framing member during fabrication

Table 4-1. Laboratory fabricated (New) panel series tested, constructed with ½ in. OSB and 2 in. by 4 in. southern yellow pine # 2 or better

Fastener	Spacing	# of Panels
Static Panel Series		
2-3/8 in. 6d smooth shank nail	6 in. / 12 in.	15
2-1/2 in. 8d smooth shank nail	6 in. / 12 in.	15
2-1/2 in. 8d smooth shank nail	6 in. / 8 in.	15
2-1/2 in. 8d smooth shank nail	6 in. / 8 in.	15
2-1/2 in. 8d ring shank nail	6 in. / 12 in.	15
2-1/2 in. 8d ring shank nail	6 in. / 8 in.	15
2-1/2 in. 8d ring shank nail	6 in. / 6 in.	13
2-1/2 in. 8d ring shank nail	6 in. / 12 in.	10
2-1/2 in. 8d ring shank nail, with Ret. B-1	6 in. / 12 in.	10
2-1/2 in. 8d ring shank nail, with Ret. B-2	6 in. / 12 in.	10
Static vs. Dynamic Panel Series		
2 in. 6d smooth shank nail	6 in. / 12 in.	10
2 in. 6d smooth shank nail	6 in. / 6 in.	10
2 in. 6d smooth shank nail, with Ret. A-2	6 in. / 12 in.	10

Table 4-2. Harvested panel series tested

Source	Age (years)	Existing	Retrofit	
		# of Panels	# of Panels	Retrofit Method
Static Tested Panel Series				
1.5 in. staple @ 3 in. / 6 in. spacing				
Port Orange #1	-	4	2	A-1
2.5 in. staple @ 3 in. / 6 in. spacing				
Bartow #1	-	2	-	-
Bartow #2	-	2	2	A-1
2.5 in. smooth shank nail @ 6 in. / 12 in. spacing				
Crystal River #1	36	2	2	A-1
2 in. smooth shank nail @ 6 in. / 12 in. spacing				
Port Orange #2	29	3	1	B-2
Bartow #3	33	2	2	A-1
Port Orange #3	-	2	1	A-1
Port Orange #4	-	2	-	-
2 in. smooth shank nail @ 8 in. / 8 in. spacing				
Port Orange #5	-	2	-	-
2 in. smooth shank nail @ 3 in. / 6 in. spacing				
Bartow #4	-	1	2	A-1
Port Orange #6	-	2	-	-
2 in. smooth shank nail @ 4 in. / 4 in. spacing				
Bartow #5	33	1	1	A-1
Static vs. Dynamic Tested Panel Series				
1.5 in. staple @ 4 in. / 4 in. spacing				
Debary #1	15	6	4	A-2

Notes:

Retrofit Method A-1. 8d ring shank nail @ 6 in. / 12 in. spacing

Retrofit Method A-2. 8d ring shank nail between 6 in. / 12 in. spacing

Retrofit Method B-2. 3 in. thick layer of spray applied polyurethane adhesive

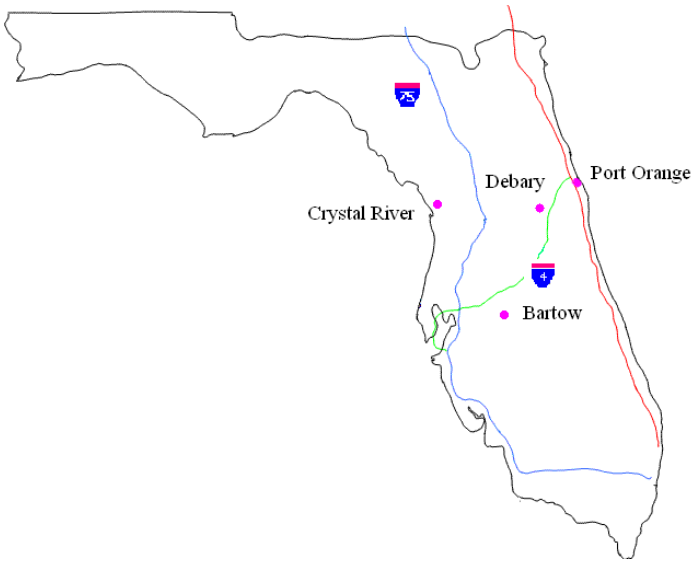


Figure 4-7. Locations of residential structures where panels were harvested

A B C

Figure 4-8. Pictures of harvested panel removal, A) Expose framing members, B) Cut framing members and C) Roof after panels removed

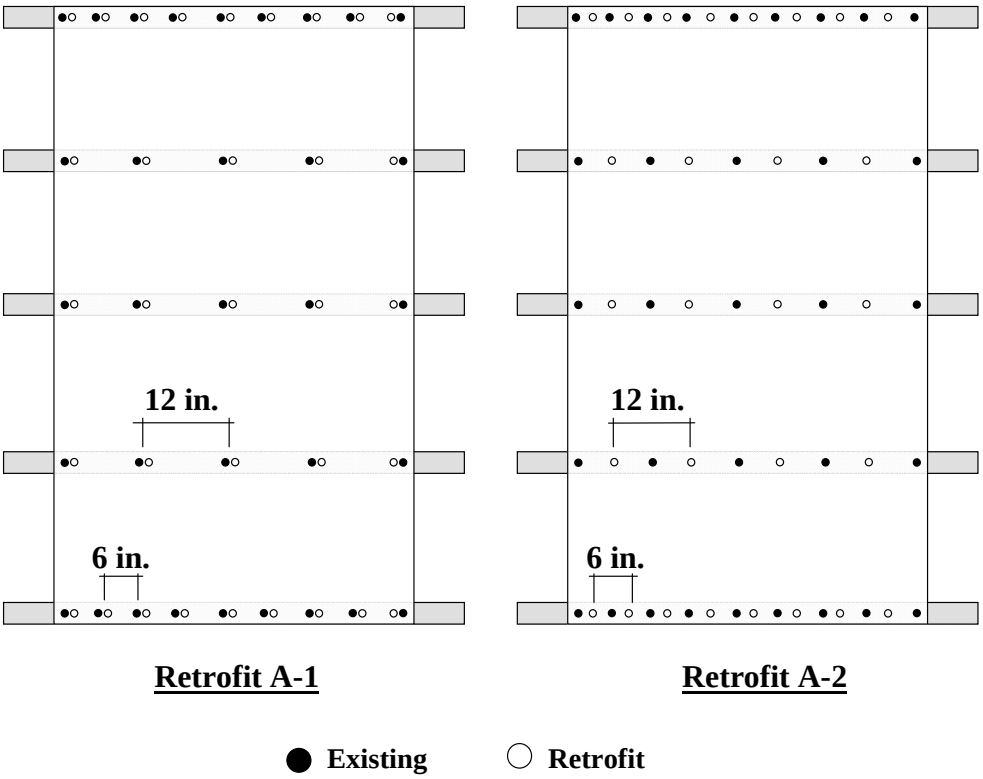


Figure 4-9. Comparison of retrofit measures A-1 and A-2

Figure 4-10. ccSPF retrofit of an existing residential structure in Port Orange, FL

Table 4-2. Summary of harvested panels tested statically and statically vs. dynamically

Sheathing Attachment Method	House Address	Existing	Retrofit	
		# of Panels	# of Panels	Retrofit Method
Statically vs. Dynamically Tested Panels, constructed with 7/16 in. OSB				
1.5 in. staple @ 4 in. / 4 in.	Debary #1	6	4	A-2
Statically Tested Panels, constructed with ½ in. Plywood				
1.5 in. staple @ 3 in. / 6 in.	Port Orange #1	4	2	A-1
2.5 in. staple @ 3 in. / 6 in.	Bartow #1	2	NA	NA
2.5 in. staple @ 6 in. / 12 in.	Bartow #2	2	2	A-1
2.5 in. smooth shank nail @ 6 in. / 12 in.	Crystal River #1	2	2	A-1
2 in. smooth shank nail @ 6 in. / 12 in.	Port Orange #2	3	1	B-2
2 in. smooth shank nail @ 6 in. / 12 in.	Bartow #3	2	2	A-1
2 in. smooth shank nail @ 6 in. / 12 in.	Port Orange #3	2	1	A-1
2 in. smooth shank nail @ 6 in. / 12 in.	Bartow #4	1	2	A-1
2 in. smooth shank nail @ 4 in. / 4 in.	Bartow #5	1	1	A-1
2 in. smooth shank nail @ 3 in. / 6 in.	Port Orange #6	2	NA	NA
2 in. smooth shank nail @ 8 in. / 8 in.	Port Orange #5	2	NA	NA
2 in. smooth shank nail @ 6 in. / 12 in.	Port Orange #4	2	NA	NA

Notes: Retrofit A-1 is 8d ring shank nails installed at 6 in. / 12 in. spacing, Retrofit A-2 is 8d ring shank nails installed between a 6 in. / 12 in. spacing, and Retrofit B-2 is ccSPF installed in a full 3 in. thick layer

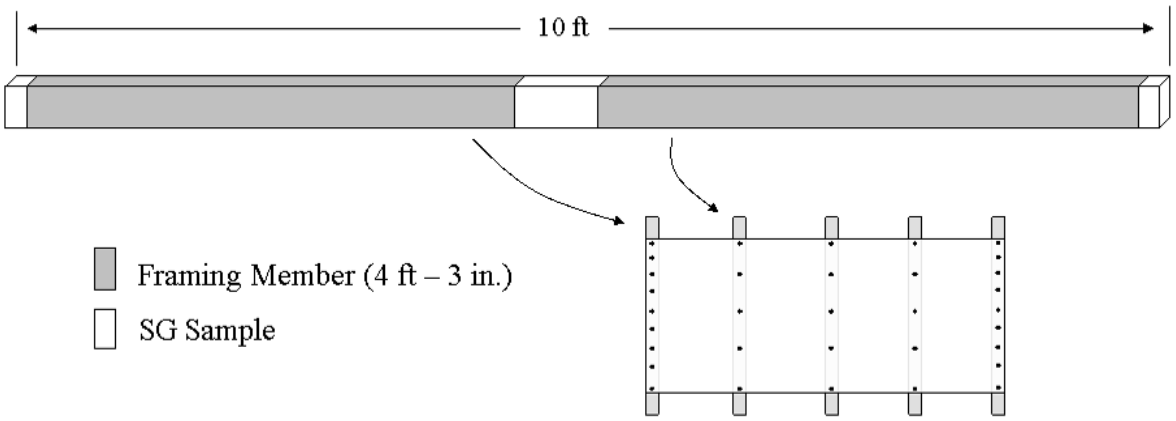


Figure 4-11. Laboratory fabricated panel specific gravity sample locations

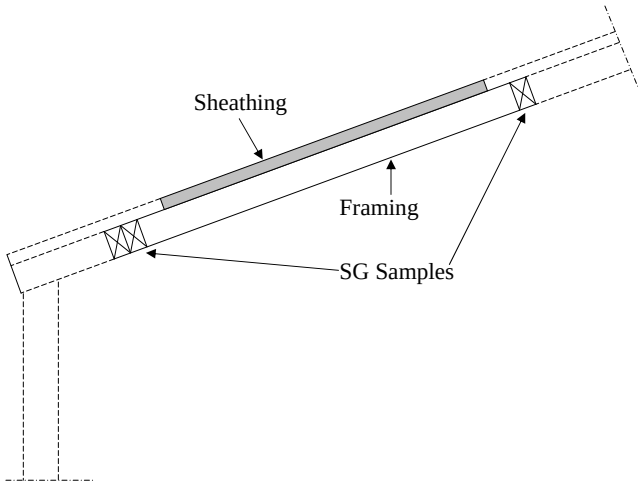


Figure 4-12. Harvested panel specific gravity and moisture content sample locations

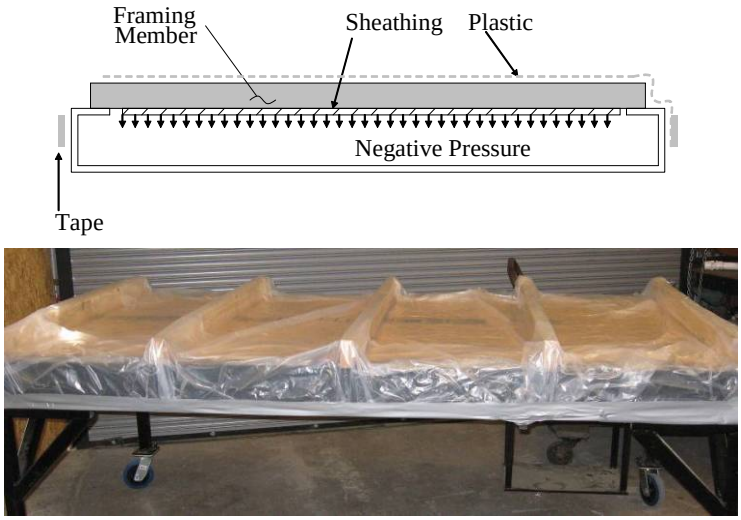


Figure 4-13. Panel installed in pressure chamber with negative pressure setup

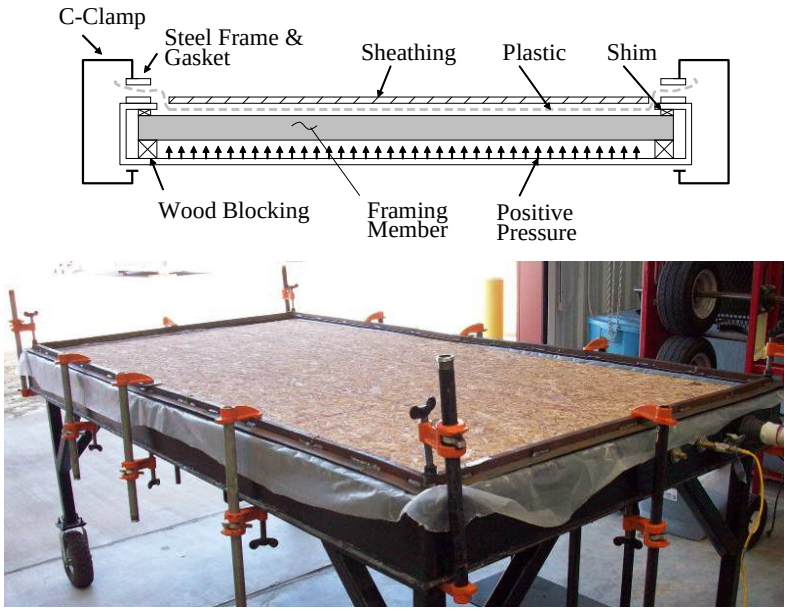


Figure 4-14. Panel installed in pressure chamber with positive pressure setup

A B

Figure 4-15. Manual control of pressure chamber (A gate valve control and (B vacuum pump in series

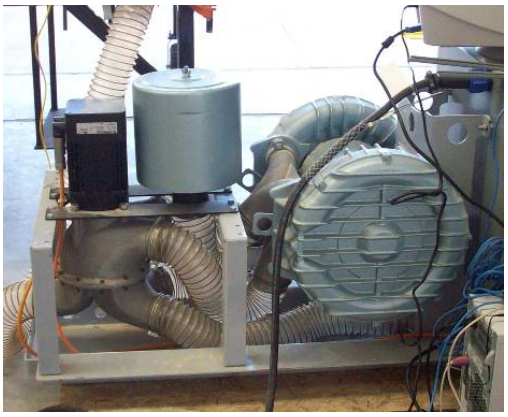


Figure 4-16. Pressure Load Actuator (PLA)

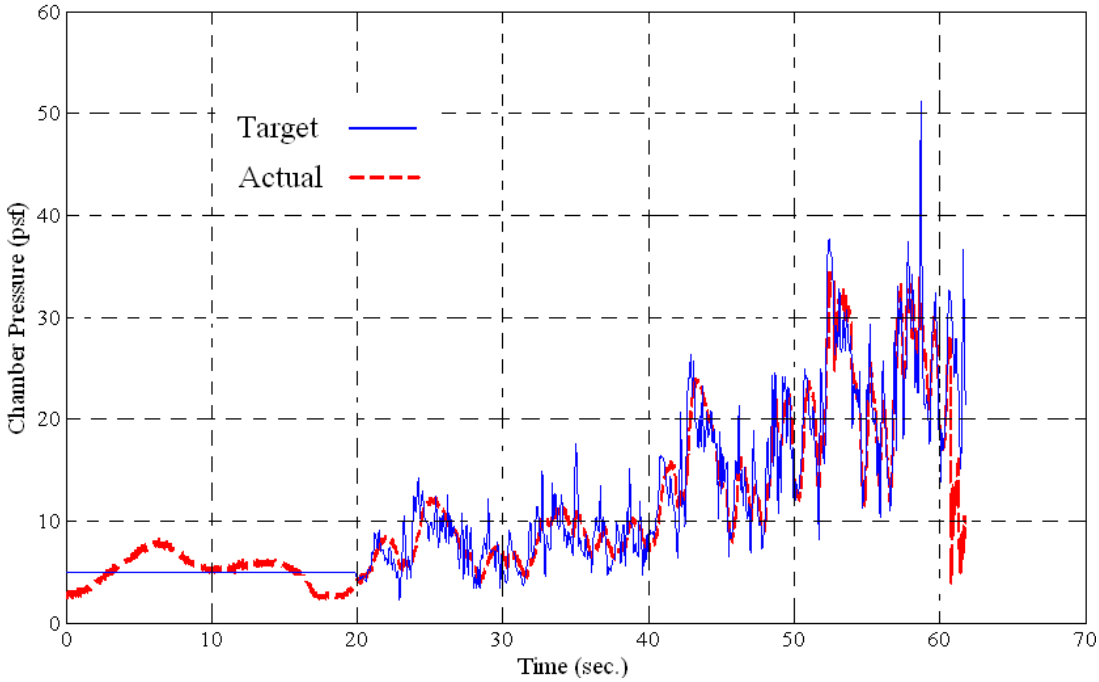


Figure 4-17. Comparison of dynamic target vs. actual chamber pressure used with the PLA

CHAPTER 5
RESULTS AND ANALYSIS

This chapter presents results from wood roof sheathing uplift pressure testing of panels fabricated in the laboratory and harvested from existing LFWS in Florida and analysis corresponding analysis. The effects of fastener type, fastener spacing, in-service conditions, retrofit measures and dynamic loading on panel uplift resistance are evaluated. Testing is broken into three groups (1) panels fabricated in the laboratory tested statically (2) panels harvested tested statically and (3) panels fabricated in the laboratory and harvested tested to directly compare static vs. dynamic loading.

Laboratory Static Panel Uplift Tests

Static testing of laboratory fabricated panels is conducted using the UF-WRSUT developed in Chapter 3. The maximum chamber pressure during testing is measured with the Omega peak measurement pressure gauge and is reported as failure capacity. The effects of fastener type, fastener spacing and ccSPF retrofit on newly constructed roof sheathing panel uplift resistance are investigated.

Evaluation of Fastener Size and Spacing on Uplift Capacity

Table 5-1 presents mean uplift capacities of panels constructed with 8d ring shank, 8d smooth shank and 2-3/8 in. long 6d smooth shank nails. Panels fastened with 8d ring shank and 8d smooth shank nails are constructed with 6 in. / 12 in., 6 in. / 8 in. and 6 in. / 6 in. fastener schedules. Panels constructed with 2-3/8 in. 6d smooth shank nails are constructed with only a 6 in. / 12 in. fastener schedule. A control series of 8d ring shank nails which is used for evaluation of ccSPF retrofit discussed below is also presented.

Figure 5-1 shows graphically the mean uplift failure capacities of laboratory fabricated panels tested with 8d ring shank, 8d smooth shank and 6d smooth shank nails at the three nailing

schedules used. It is seen that as interior fastener spacing decreases the mean uplift capacity increases. Panels fastened with 8d ring shank nails increase 45% to 57% in mean uplift strength (174/161 to 252 psf) from 6 in. / 12 in. to 6 in. / 6 in. fastening schedules. Panels fastened with 8d smooth shank nails increased 59% in mean uplift strength (129 psf to 205 psf) from 6 in. / 12 in. to 6 in. / 6 in. fastening schedules. Additionally it is found that this relationship between uplift capacity and fastener spacing appears to be linear for panels attached with 8d ring shank or smooth shank nails. The slopes for 8d ring shank and 8d smooth shank are 13.6 psf/in. and 12.5 psf/in. respectively suggesting that the relationship between fastener spacing and uplift capacity is independent of fastener type.

Ring shank nails are found to have higher uplift strength than smooth shank nails. Results show a 27% mean increase in strength from 8d smooth shank to 8d ring shank. Additionally it is found that 8d smooth shank nails have a higher uplift capacity than 2-3/8 in. 6d smooth shank nails. Only the 6 in. / 12 in. fastener schedule was tested for 2-3/8 in. long 6d common nails so no conclusive trends can be found; however it was found that uplift strength is increased 74% from 6d smooth shank nails to 8d smooth shank nails.

Failure pressure results of laboratory fabricated panel tested statically provide a control series which can be used for comparative purposes. In the following section results are compared to

- Previous studies for assessment of developed test method,
- Design fastener withdrawal strengths for evaluation of design assumptions,
- ccSPF attached panel for evaluation of strength benefits of ccSPF,
- Harvested panels for evaluation of in-service effects,
- Retrofitted harvested panels to evaluate strength benefits,
- Dynamically tested panels to determine the effects of dynamic loading.

Comparison of Results with Previous Studies

Results from static UF-WRSUT are compared with results identified in Chapter 2 in order to assess the relative performance of the developed test method with previous methods. Due to difficulties in making direct comparisons previously discussed in Chapter 2 results presented in Table 5-1 are compared to ranges of similar construction presented in literature review. Previous studies mean and 5% exclusion failure pressures with sufficient numbers of samples are compared to results.

Panel results from previous studies constructed with 6d smooth shank fasteners attached at 6 in. / 12 in. yielded mean failure pressures ranging from 26 psf to 55 psf, which is lower than static UF-WRSUT results having a mean failure pressure of 74 psf. Panel results from previous studies constructed with 8d smooth shank fasteners attached at 6 in. / 12 in. and 6 in. / 6 in. had ranges of mean failure pressures of 61 psf to 228 psf and 77 psf to 218 psf respectively. Static UF-WRSUT results of panels fastened with 8d smooth shank nails at 6 in. / 12 in. yielded a mean failure pressure of 129 psf and panels attached at 6 in. / 6 in. had a mean failure pressure of 205 psf; both of which fall within the corresponding ranges found from literature. Panels constructed with 8d ring shank fasteners had only one data set for each spacing of 6 in. / 12 in. and 6 in. / 6 in. Results from previous testing of panels constructed with 8d ring shank nails at 6 in. / 12 in. had a mean failure pressure of 140 psf, and the one panel fastened with 8d ring shank nails at 6 in. / 6 in. had a failure pressure of 397 psf. Panels constructed with 8d ring shank nails tested per UF-WRSUT for 6 in . / 12 in. and 6 in. / 6 in. fastener schedules resulted in mean failure pressures of 174 psf and 252 psf, which are lower than results from previous testing.

Table 5-2 presents a comparison of the mean and 5% exclusion failure pressures of laboratory fabricated static UF-WRSUT results and results of previous studies which had a sufficiently large sample size. Sample sizes from statically tested laboratory fabricated panels as

part of this study were 13 to 15 panels and for comparative purposes previous studies selected had similar or larger sample sizes; details of selected studies can be found in Chapter 2. The 5% exclusion failure pressure is presented to give a comparison of the variability in results. A statistical analysis of laboratory fabricated static UF-WRSUT found that all data sets fit a normal distribution at the 0.05 confidence level using an Anderson-Darin test. Using the normal cumulative distribution function the 5% value was calculated for results from static UF-WRSUT and all previous studies selected had normal distributions.

Static UF-WRSUT results from panels installed with 6d smooth shank fasteners at 6 in. / 12 in. are significantly higher than results from Kallem (1997), which had 14 samples, for both mean and 5% exclusion failure pressures. Panels attached with 8d smooth shank fasteners at 6 in. / 12 in. UF-WRSUT mean failure pressure result is within 18% of a 49 panel data set by IHRC (IHRC 2004) of similar construction. Statically tested UF-WRSUT panels attached with 8d smooth shank fastener spaced at 6 in. / 6 in. (205 psf mean and 170 psf 5% exclusion) are significantly higher than a 30 panel data set (131 psf mean and 101 psf 5% exclusion) of similar construction by Murphy et al. (1996). Statically tested UF-WRSUT panels attached with 8d ring shank fastener spaced at 6 in. / 12 in. (174 psf mean and 137 psf 5% exclusion) are significantly higher than a 50 panel data set (140 psf mean and 101 psf 5% exclusion) of similar construction by IHRC (IHRC 2004).

Results presented in Tables 5-1 and 5-2 show that laboratory fabricated panels tested with static UF-WRSUT yield failure pressures which in the same range as previous results considering that direct comparisons cannot be made. First, UF-WRSUT results are both above and below results from previous studies. Second, results from previous studies vary by the same magnitude between each other as they do to UF-WRSUT results. Third, the coefficients of

variation for static UF-WRSUT results (7% to 22%) are similar to previous testing (7% to 28%). Finally, while it is found that while difference in mean failure pressures can vary significantly the closest match with previous studies are from datasets as large or larger than those tested with UF-WRSUT. Given that result are comparable to previous studies, which wood roof sheathing panel design is based on, UF-WRSUT results can be compared with design values.

Evaluation of Calculated Fastener Resistance

Table 5-3 presents mean fastener failure loads calculated based on the assumption of tributary area for laboratory fabricated panels tested statically, where the maximum tributary area of fasteners installed in the panel is used. Panel failure pressure are dived by a factor of safety of 2.0 for design pressure then multiplied by the maximum tributary area to obtain calculated withdrawal strengths. Resulting failure loads are compared to design withdrawal strength based on NDS strengths (Table 11.2C) (AF&PA 2005). Withdrawal strengths from the NDS are multiplied by the built in factor of safety (5) and depth of penetration for each fastener, which is 2 in. for 8d smooth shank or ring shank nails and 1.875 in. for 2-3/8 in. 6d smooth shank nails. For example, 8d smooth shank fasteners have a design withdrawal value of 41 (lbs/in.) and a depth of penetration of 2 in. when installed through ½ in. OSB, is

$$41\left(\frac{lbs}{in.}\right)\times 2(in.)\times 5(SF) = 410lbs \qquad (5\text{-}1)$$

Figure 5-2 summarizes the effect of fastener spacing on mean panel failure pressure vs. mean fastener failure load. Calculated fastener failure load results show that as nail spacing is reduced mean withdrawal loads decrease, for both the 8d ring shank and smooth shank nails. It is observed that failure loads for the 8d ring shank nail decreased from 175 lbs to 126 lbs as the interior fastener spacing reduced from 12 in. to 6 in. o.c. Similarly nail failure loads decreased from 130 lbs to 103 lbs for 8d common nail as fastening spacing decreased from 12 in. to 6 in.

Calculated fastener failure results are significantly lower than design withdrawal strengths found from NDS strengths, ranging from 67% to 77% less for 8d ring shank and 2-3/8 in. 6d smooth shank respectively. Results suggest that 8d ring shank nails are roughly 27% greater than 8d smooth shank nails, however design withdrawal strength for 8d ring shank nails are 6% less than 8d smooth shank nails.

Results suggest that fastener withdrawal strength increases as spacing increases. However a study by Dao and van de Lindt (2008), in which the effect of fastener spacing is investigated by quantifying the applied moment of a nail due to the changes in eccentricity/spacing, suggests that fastener withdrawal loads would decrease with increased fastener spacing. These findings, while preliminary are important as they presents an alternative interpretation of test results from the commonly held notion that nail failure load is not changeable by the structural system itself. Results suggest that current approaches are not modeling the behavior of fastener strength within a panel. The limitation of the results is that no measurement of fastener load at the time failure were made to confirm the estimates based on failure pressure. It is likely that a large sample set is needed in order to investigate this behavior therefore a standardized test method is necessary to provide consistent results. Additionally it is likely that dynamic characteristics of wind loading effect this behavior and are therefore necessary to capture the true behavior.

Evaluation of ccSPF Retrofit

The need to address deficiencies in uplift resistance of the current building stock is established in Chapter 2. Two retrofit measures are investigated within this study first 8d ring shank nails installed at a fastener schedule of 6 in. / 12 in. and second ccSPF adhesive. Laboratory fabricated panels attached with ccSPF adhesive are tested statically to establish a baseline of their performance, Table 5-4.

The mean uplift capacity of panels connected with 8d ring shank nail at 6 in. / 12 in. was 161 psf with a coefficient of variation of 18%, which compares well with results presented in Table 5-1. This mean uplift capacity is increased to 202 psf when a fillet of ccSPF (ret. B-1) is applied resulting in a 25% increase capacity. When ccSPF was applied in a full 3 in. thick layer (ret. B-2) the increased effect was negligible with the mean uplift capacity increasing to 209 psf.

Results suggest that from the standpoint of improving structural strength there is little added benefit from the fillet to full coverage of ccSPF. Results provide a baseline for comparison with,

- Harvested panels retrofitted ccSPF to determine the effects of adhesion with aged panels,
- New and harvested panels retrofitted with 8d ring shank nails to compare the strength benefits.

Failure Mechanisms Observed

As detailed in Chapter 2 the uplift design of wood roof sheathing is based on the withdrawal strength of the individual fasteners. In order to assess the validity of this assumption that fastener connecting wood roof sheathing to framing fail in withdrawal panels were examined after testing for failure mode and location of all failures. Figure 5-3 presents a sample of the failure mode and location information of 8d smooth shank and ring shank nails at 6 in. / 6 in., which is collected for all laboratory fabricated panel tested statically. The framing members attached to the 4 ft. by 8 ft. sheathing is represented by the lines, on-top of which a shape corresponding to a particular failure mode is placed on each fastener that failed. Observations were made after panel was removed from the chamber. Full results are found in Appendix B.

The dominant failure mode of laboratory fabricated panels tested with the static UF-WRSUT is presented in Figure 5-4. It is found that failure of OSB panels fastened with smooth shank nail tend to be dominated by the withdrawal failure mode. Alternatively it is found that

panels attached with ring shank nails tend to be dominated by the pull-through failure mode. Additionally it is seen that as fastener spacing decreases pull-through failure mode becomes more dominant, however the effect of spacing is less significant than fastener type. The addition of ccSPF as a retrofit measure is seen to cause a more even distribution of panel failure modes.

Results will provide a basis for comparison of behavior of fasteners during failure. Comparisons of failure behavior will be made for new and harvested panels attached with different fasteners tested statically vs. dynamically.

Harvested Static Panel Uplift Test

Static testing of panels harvested from 12 homes throughout Central Florida was conducted using the UF-WRSUT developed in Chapter 3. The maximum chamber pressure during testing recorded with the Omega peak measurement pressure gauge is reported as failure capacity. The effects of age/weathering, fastener type, fastener spacing and retrofitting on existing panel constructions uplift resistance are investigated.

Evaluation of Existing Panel Uplift Resistance

Table 5-5 presents full results for static UF-WRSUT panels harvested from LFWS located in Central Florida in their existing condition, and the fastener failure loads calculated by the assumption of tributary area. Fastener failure loads are calculated by multiplying the failure pressure by the maximum tributary area in the panel; for example the 1.5 in. staples at 4 in. / 4 in. have a maximum tributary area of 2/3 ft^2 which is multiplied by the failure pressure of 93 psf to get a fastener failure load of 62 lbs. It is seen that panels attached with 2 in. smooth shank nails at 4 in. / 4 in. had the highest uplift capacity, which corresponds to the smallest tributary area seen. The fastener with the highest withdrawal capacity is the 2.5 in. (8d) smooth shank nail, which corresponds to the largest tributary area and fastener size. The most common panel attachment harvested was 6d smooth shank nails spaced at 6 in. / 12 in. having a range of mean

uplift capacities of 39 psf to 102 psf, which contains the laboratory fabricated result of 74 psf. Harvested panels attached with 8d common smooth shank nails at 6 in. / 12 in. had a mean uplift capacity of 107 psf which considering the small sample size (2) is similar to the laboratory fabricated result of 129 psf

Figure 5-5 presents a comparison of the scatter of static uplift capacity results from harvested vs. new panels. Harvested panels attached with 1.5 in. staples are attached at two fastener schedules having tributary areas of two thirds and one ft^2, which yield similar results. Harvested panels attached with 2.5 in. staples are attached at a 6 in. / 12 in. schedule and yield results lower than the 1.5 in. staples, which is reasonable considering that due to tributary area the 2.5 in. staples experience load levels as much as three times greater than the 1.5 in. staples. Harvested panels attached with 2 in. 6d smooth shank fasteners are attached with four fastener schedules (2, 1-1/3, 1 and 2/3 ft^2). As discussed in Chapter 4 panels constructed with 6d smooth shank should nominally be attached with a 6 in. / 12 in. schedule however spacing is somewhat variable in the field. Comparing harvested and new 6d smooth shank fastened panels it is seen that harvested panels have a slightly higher mean but contain laboratory fabricated panel results. This increased capacity of harvested panels can be attributed to the smaller loads on individual fasteners due to the closer spacing of harvested panels. Harvested panels attached with 8d smooth shank fasteners are attached at a 6 in. / 12 in. schedule and have 17% lower mean uplift capacity than new panels, however only two samples exist for harvested results and both fall within range of new panel results.

Figure 5-6 presents a comparison of mean uplift capacity vs. calculated fastener withdrawal load for panels attached with staples and nails at different fastening schedules. The change in fastening is best thought of as change in tributary area because fastener withdrawal

loads are calculated by multiplying the tributary area by panel uplift capacity. Panels attached with 1.5 in. staples experienced a decrease in mean uplift capacity and calculated fastener load as tributary area decreases. Panels attached with 2 in. smooth shank nails had similar mean uplift capacities for fastening schedules 6 in. / 12 in. to 3 in. / 6 in., but calculated fastener withdrawal strength steadily decreased. From fastening schedules 3 in. / 6 in. to 4 in. / 4 in. panel uplift capacity increased and fastener withdrawal strength increased slightly, however only a single sample was tested at 4 in. / 4 in. spacing so this result may not be representative. This trend of decreasing fastener load as tributary area decreases is confirmed in laboratory fabricated static test results.

Results from panels harvested from existing structures and laboratory fabricated panels presented above provide a data for the evaluation of age or weathering effects on wood roof sheathing panel uplift capacity. These results show that statically tested uplift capacities of existing wood roof sheathing panels have similar uplift capacities as newly fabricated panels. This suggests that laboratory testing of newly fabricated roof sheathing panels can sufficiently approximate the in-service strength of wood roof sheathing panels.

Evaluation of Retrofit of Existing Panels

Table 5-6 presents mean uplift capacity results from a comparative study of panels harvested from existing LFWS located in Central Florida in existing condition vs. retrofitted. All panels are tested statically per UF-WRSUT. Panels are retrofitted with 8d ring shank nails spaced at 6 in. / 12 in. installed as if a panel had no existing fasteners (Retrofit A-1). When existing fastener locations coincided with the 8d ring shank retrofit fastener locations nails were installed directly next to the existing fastener. One panel was retrofitted with a full 3 in. thick layer of ccSPF adhesive (Retrofit B-2).

Figure 5-7 presents a comparison of statically tested harvested panels in existing and retrofit conditions. Results are averaged by individual source houses to account for any variation between source houses of similar construction in the comparison. Harvested panel which were retrofitted were found to have significantly increased uplift capacities. Panels retrofitted with 8d ring shank nails spaced at 6 in. / 12 in. had an average increase in capacity of over 2.5 times. Variations in retrofitted uplift capacity are seen to be independent of the existing fastener. Panels retrofitted with ccSPF retrofitted increased 2.4 time however only 1 sample was tested.

Results presented in this section provide data for the evaluation of the initial strength benefits of retrofitting existing construction. It is seen that retrofitting with one of the two methods used can increase the uplift capacity of existing panels immediately after retrofits are installed significantly.

Comparison of Laboratory vs. Harvested Specific Gravity Samples

Specific gravity measurements (167 laboratory samples and 178 harvested samples totaling 345 samples) were taken for all panels tested in static vs. dynamic testing. The mean specific gravity of laboratory samples is 0.52 and 0.57 for harvested panels, which is similar to the NDS specified specific gravity for southern yellow pine of 0.55. However, the mean values of two sets of specific gravity are not statistically equal (p-value on the order of 10^{-8}). The specific gravities of the framing members used in the lab-built panels were best fit by a lognormal distribution (Figure 5-8a), and for the harvested panels (15 years old), a normal distribution was a better fit (Figure 5-8b). Rammer et al. (2001) found that normal and lognormal distributions fit the specific gravity of southern yellow pine.

Table 5-7 presents specific gravities calculated in an attempt to determine if framing member specific gravity affects the wind uplift capacity of wood roof sheathing. Three specific gravities are calculated; the first is a mean of all framing members, the second is a mean of all

framing members which failed and the third is the mean of the framing members which were observed or determined to fail first. In the third case if it was not possible to observe which framing member failed first, then the framing members with relatively larger numbers of fastener failures were used. The normalized panel failure pressure was calculated by dividing each panel failure pressure by the maximum failure pressure for the given data set.

It is found that the specific gravity of framing members collected from in-service wood roof systems are higher than wood framing members which were structural #2 grade or higher. This suggests that there is no degradation of wood properties once installed in the field. An analysis of the effects of specific gravity to uplift capacity suggests no correlation, however limitations in testing make it difficult to pinpoint the initial failure and corresponding specific gravity. It is found that if specific gravity effects the uplift capacity of wood roof sheathing it is not a dominant factor.

Static vs. Dynamic Panel Uplift Test

The effects of dynamic loading characteristics on existing and retrofitted panels both fabricated in the laboratory and harvested from existing construction are investigated. Laboratory fabricated panels fastened with 6d smooth shank fasteners at 6 in. / 12 in. and 6 in. / 6 in. schedules from the same sample population are tested both statically and dynamically. Then laboratory fabricated “existing” panels attached with the 6d smooth shank fasteners at 6 in. / 12 in. “retrofitted” with 8d ring shank nails are tested both statically and dynamically. Panels harvested from existing construction in Central Florida fastened with 1.5 in. staples at 4 in. / 4 in. are tested both statically and dynamically. Harvested panels are retrofitted with 8d ring shank nails and tested statically and dynamically. All static vs. dynamic samples are tested with the PLA system detailed in Chapter 4. Results recorded are pressure time-histories framing member

specific gravity, failure mode and location, moisture content for only harvested panels and defects in harvested panel constructions (i.e. missing fasteners).

Comparison of Static vs. Dynamic Laboratory Fabricated Panel Uplift Test

Table 5-8 presents results from static vs. dynamic UF-WRSUT of laboratory fabricated panels. Failure pressures of statically tested laboratory fabricated panels attached with 2 in. long 6d smooth shank nails spaced at 6 in. / 12 in. (62 psf) are similar to results obtained in static UF-WRSUT results (74 psf). Similarly statically tested panels retrofitted with 8d ring shank nails yield a mean failure pressure (183 psf) close to that of static UF-WRSUT results of similar construction (174 psf and 161 psf).

Figure 5-9 presents a comparison of statically vs. dynamically tested panels fabricated in the lab. As expected from static testing of panels fabricated lab, panels having 6 in. / 12 in. fastening schedules failed at lower pressures than the panels having 6 in. / 6 in. fastening schedules for both static tests (62 psf vs. 108 psf) and dynamic tests (52 psf vs. 90 psf). Similarly as expected from static testing of panels retrofitted with 8d ring shank nails, panels retrofitted with 8d ring shank nails had a higher failure pressure than in existing condition for both static (62 psf vs. 183 psf) and dynamic (52 psf 167 psf) loading. A reduction of 16% was found in uplift capacity of panels in existing condition tested dynamically. The capacity of retrofitted panels was reduced 9% from static to dynamic loading.

Results from this section provide a comparison of statically vs. dynamically tested panels, and the effect of dynamic loading on new 8d ring shank retrofitted panels. It is seen that dynamic loading reduces the uplift capacity of new panels constructed with smooth shank fasteners 16% and new panels constructed with ring shank fasteners 9%. This suggests that dynamic loading reduces wood roof sheathing uplift strength and that the effects are lessened for annularly threaded nails.

Comparison of Static vs. Dynamic Harvested Panel Uplift Test

Table 5-9 presents results from static vs. dynamic UF-WRSUT of panels harvested from a single existing LFWS in Florida. Failure pressures of statically tested harvested panels attached with 1.5 in. long staples (93 psf) are less than results obtained in static UF-WRSUT results (120 psf). Similarly statically tested panels retrofitted with 8d ring shank nails yield a mean failure pressure (168 psf) less than that of static UF-WRSUT results of similar construction (270 psf). However retrofitted panel uplift capacity fall within the range of statically tested laboratory fabricated panels attached with 8d ring shank nails at 6 in. / 12 in. (174 psf and 161 psf).

Figure 5-10 presents a comparison of statically vs. dynamically tested panels harvested existing construction. As expected from static testing of panels retrofitted with 8d ring shank nails, panels retrofitted with 8d ring shank nails had a higher failure pressure than in existing condition for both static (93 psf vs. 168 psf) and dynamic (72 psf vs. 126 psf) loading. A reduction of 23% was found in uplift capacity of panels in existing condition tested dynamically. The capacity of retrofitted panels was reduced 26% from static to dynamic loading.

Results from this section provide data for the evaluation of the effect of dynamic loading on harvested panel in existing and retrofitted conditions. It is seen that dynamic loading reduces harvested panels in existing conditions by 23% and panels in retrofitted conditions by 26%. This increase in effect for panels retrofitted with 8d ring shank nails is contradictory to laboratory fabricated panel results, however as detailed in Chapter 4 the panels used for retrofitting were the panel with most missing fasteners. Thus harvested retrofitted panels effectively had larger tributary areas than those of laboratory fabricated retrofitted panels making them more susceptible to dynamic loading. However limited sample sizes prevent conclusive findings. It is clear that dynamic loading effects the uplift capacity of wood roof sheathing panels.

Peak Pressure vs. Failure Pressure

Table 5-10 presents a summary of the peak pressures and the failure pressure of all panels tested statically vs. dynamically. This is investigated to determine if panel failure occurs at a peak pressure or later at some reduced pressure. Dynamically tested results are the focus of this comparison but statically tested results are provided for comparison. If a panel fails at a peak pressure or pressure spike then the failure is assumed to be due to the large loading rate. However if a panel fails after a pressure spike at a lower pressure then failure is assumed to be due to some accumulation of damage in the nail to wood connection. It is found that half (9) of the dynamically tested panels failed at a pressure spike and are therefore due to the loading rate. It is found that the half of the dynamically tested panels failed after a pressure spike at a reduced pressure, which were on average is 87% of the peak pressure recorded.

For dynamically tested panels the failure pressure compared to peak pressure it can be seen that the failure pressure is lower than the peak pressure by 3-9%. Statically tested panels show little difference from peak to failure pressure. Therefore the reduction from dynamically to statically tested panel resistance is increased when using the failure pressure instead of peak pressure, which is what has typically been used in previous static tests. Table 5-11 presents the reductions from static to dynamic resistance for both the peak and failure pressures. As expected the magnitude of the reductions from static panel resistance are increased when calculated with the failure pressures

Analysis of Variance of Results

Results are analyzed to determine if the control variables fastener type, fastener spacing, age, date tested, retrofit and type of loading have a statistically significant effect on failure pressure in this section. This is done by analyzing the variance within data series to test the null hypothesis that all the samples come from the same sample population. Analysis of Variance

(ANOVA) test is a measure of the error between some number of data series by judging statistically if the differences in the sample means are different. With the limited number of samples there is a risk that the theoretical behavior of a combination of variables is not being captured in results therefore the analysis is conducted for datasets of four samples or greater. Similarly the methods employed assume that the distribution of the results is normal which has only been verified for laboratory fabricated samples in the static phase of testing constructed with 8d ring shank, 8d smooth shank and 6d smooth shank. All series used in analysis of variation are presented in Table 5-12.

Procedure

To analyze the variance in the different datasets the comparison is broken down into two parts (1) the error due to comparison or treatment and (2) the error due to the differences in individual samples within a given dataset. The general procedure to analyze the variance (ANOVA) of several datasets is to first calculated the sum of the squares error for the treatment (SST) by,

$$SST = n\sum_{i}\left(\bar{y}_{i.} - \bar{y}_{..}\right)^2 \tag{5-2}$$

Where $\bar{y}_{i.}$ is the mean of an individual series, $\bar{y}_{..}$ is the mean of all the series combined, *n* is the number of samples in the series and *i* is the number of the current treatment. Next the sum of the squares error for those variations which are not due to the treatment (SSE) by,

$$SSE = \sum_{ij}\left(y_{ij} - \bar{y}_{i.}\right)^2 \tag{5-3}$$

Where y_{ij} is the *j*th term of the *i*th series or treatment and $\bar{y}_{i.}$ is the mean of the *i*th series or treatment. Then unbiased estimates of the variance of the error between series are calculated for the error due to the treatment (MST) and the error due to differences in sample means (MSE),

$$MST = \frac{SST}{t-1} \tag{5-4}$$

$$MSE = \frac{SSE}{N-t} \tag{5-5}$$

Where *t* is the number of treatments and *N* is the total number of samples in all series. Therefore the ratio of the two unbiased mean square errors MST to MSE (F-value) should approach 1.0 if the series were from similar sample populations expressed as,

$$F-value = \frac{MST}{MSE} \tag{5-6}$$

Simply stated this means that if the variations of the mean values of each dataset about the overall mean are the same as the variations of the individual samples to their corresponding mean, then the samples can be said to come from the same population. To assess the probability that the F-value is close to 1.0 a p-value analysis is used where the probability that the F-value will be close to the expected value (p-value) is determined and compared to a level of confidence. To assess the probability that the F-value will be close to 1.0 the p-value must be less than the confidence limit α. For example if a p-value is greater than 0.05 then one can say "the data is from the same population with 95% confidence" ($95\% = (1-\alpha)\cdot 100\%$). A critical $F_{critical}$-value is calculated corresponds to the confidence limit meaning that if the F-value is greater than the $F_{critical}$-value then the datasets come from different sample populations.

To further analyze the result the Bonferroni Method (Ott and Longnecker 2004) is applied to determine if individual datasets are similar to each other. The Bonferroni Method compares the variance between datasets in the same way described above except that it does this for each individual dataset. This is done by adjusting the confidence interval for the individual comparisons by dividing the α by *C*, where *C* is,

$$C = \frac{I(I-1)}{2} \tag{5-7}$$

and I is the number of treatments or comparisons. By dividing the α by C the problem becomes more conservative, therefore with large numbers of comparisons there is a risk of rejecting the null hypothesis when it is true. The problem is evaluated by adjusting the comparison of the series means by adding and subtracting the critical t-value (eq. 5-8).

$$\overline{X}_{i.} - \overline{X}_{j.} \pm t_{N-1,\alpha/(2C)} \sqrt{MSE\left(\frac{1}{J_i} + \frac{1}{J_j}\right)} \tag{5-8}$$

Where $t_{N-1,\alpha/(2C)}$ is calculated from the "Students t-table", $\overline{X}_{i.}$ is the mean of the series i, J_i is the number of observation in series i, $\overline{X}_{j.}$ is the mean of the series j and J_j is the number of observation in series j. If zero lies between the limits of equation 6 then the series means are found from the same population because the difference is zero within the confidence interval.

Laboratory Fabricated Panels Tested Statically

A comparison of the variances between all laboratory fabricated panel datasets in the static phase of testing is conducted using the procedure described above with to determine if the different datasets came from the same sample population. Table 5-13 presents the ANOVA table which finds that the datasets come from different sample populations. Table 5-14 presents a full comparison of the individual datasets to each other using the Bonferroni Method, which is summarized in Table 5-15. It was found that the two 8d ring shank series spaced at 6 in. / 12 in. are from the same sample population. It was found that the two ccSPF retrofit arrangements (fillet and full 3 in.) were from the same sample population. Additionally it was found that 8d ring shank nails at 6 in. / 12 in. are from the same population as 8d smooth shank nails at 6 in. / 8

in. Similarly 8d ring shank nails at 6 in. / 8 in. are found to be from the same population as 8d smooth shank nails at 6 in. / 6 in. All remaining populations were found to be different.

Harvested Panels Tested Statically

A comparison of variances for all panels constructed with 6d smooth shank panel spaced at 6 in. / 12 in. was conducted to determine if harvested panel results came from the same sample population as newly constructed results. This comparison included harvested and laboratory fabricated panels from the static phase of testing, as well as laboratory fabricated panels from the static vs. dynamic phase of testing. Table 5-16 presents the ANOVA table for the four datasets. It is found that all datasets constructed with 6d smooth shank nails at 6 in. / 12 in. come from the same sample population. Table 5-17 presents the full results for comparison using the Bonferroni Method, which suggest again that all datasets come from the same sample population.

A comparison of all statically loaded panels retrofitted with 8d ring shank nails spaced at 6 in. / 12 in. versus existing panel with 8d ring shank nails spaced at 6 in. / 12 in. was conducted using the unequal variance t-test. The t-test is identical to the analysis of variance procedure described above except that only two samples are compared. The full T-table is presented in Table 5-18 and it is found that the two results are not from the same sample population for a confidence level of 95%. This suggests that the original fasteners installed at the time of construction make a difference

Laboratory Fabricated Panels Tested Statically vs. Dynamically

A comparison of variances for panels fastened with 6d smooth shank nails spaced at both 6 in. / 12 in. and 6 in. / 6 in. from the static vs. dynamic phase of testing are compared. The peak pressure recorded during a test and the pressure which the panel failed at were both used. Table 5-19 and Table 5-21 present ANOVA tables for the peak and failure pressures respectively,

which both suggests that all statically and dynamically tested panels regardless of fastener spacing come from the same sample population. Table 5-20 and Table 5-22 present the Bonferroni analysis for the peak and failure pressures respectively, and again both suggest that statically and dynamically tested panels are from the same population for both 6 in. / 12 in. and 6 in. / 6 in. spacings.

Table 5-1. Results of static UF-WRSUT of laboratory fabricated panels fastened with 2-3/8 in. long 6d smooth shank, 8d smooth shank and 8d ring shank nails

Fastener Schedule	8d Ring Shank Nails			8d Smooth Shank Nails			6d Smooth Shank Nails		
	# of Panels	Mean Failure (psf)	COV	# of Panels	Mean Failure (psf)	COV	# of Panels	Mean Failure (psf)	COV
6 in. / 12 in.	10 15	161.0 174.4	18% 13%	15	129.4	11%	15	74.3	22%
6 in. / 8 in.	15	216.1	17%	15	175.1	11%	NA	NA	NA
6 in. / 6 in.	13	252.4	7%	15	205.3	10%	NA	NA	NA
Notes:									

Notes: Ring Shank Nails - 2.5 in. long 0.113 in. dia., 8d Smooth Shank Nails - 2.5 in. long 0.131 in. dia., and 6d Smooth shank - 2.375 in. long 0.113 in. dia.

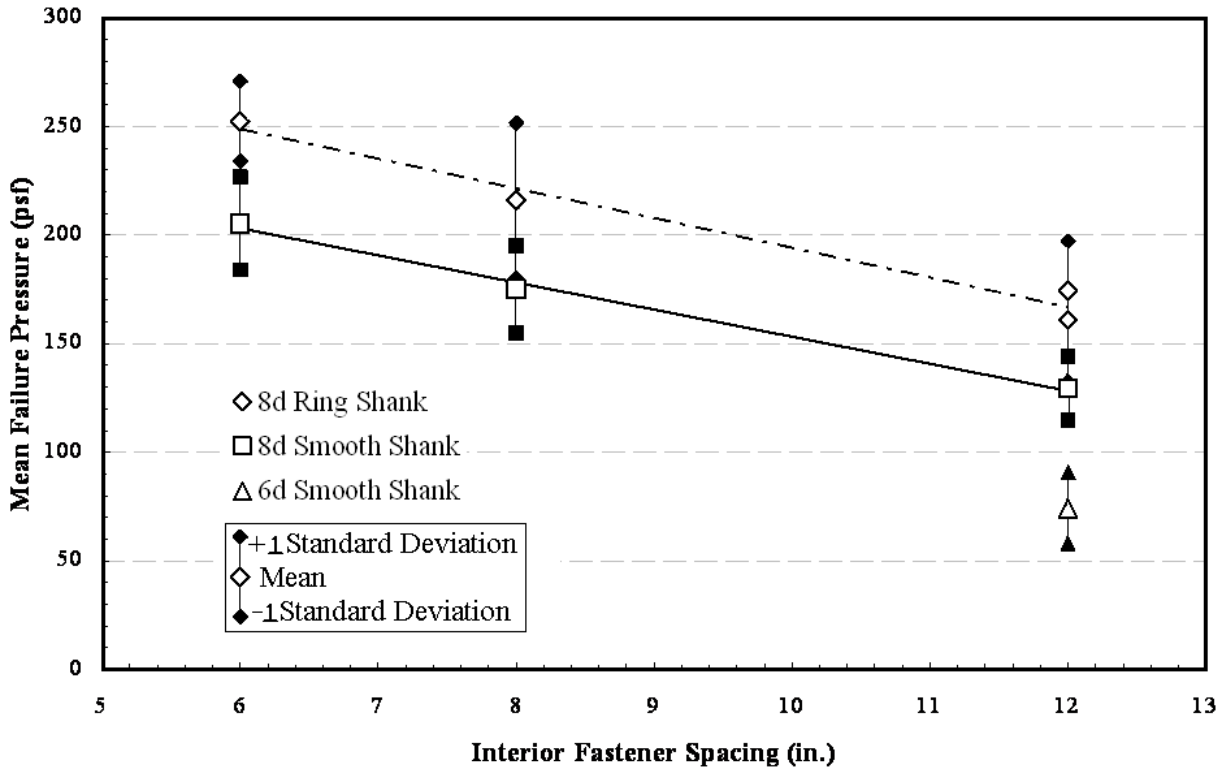


Figure 5-1. Mean failure pressures of laboratory fabricated panels tested statically

Table 5-2. Comparison of mean and 5% exclusion value failure pressures for panels fabricated in the lab tested statically UF-WRSUT vs. previous studies

Panel Attachment	Current Study		Previous Results	
	Mean Capacity	5% Exclusion Value	Mean Capacity	5% Exclusion Value
6d Smooth Shank @ 6"/12"	74	47	(A) 33	21
8d Smooth Shank @ 6"/12"	129	105	(B) 110	N/A
8d Smooth Shank @ 6"/6"	205	170	(C) 131	101
8d Ring Shank @ 6"/12"	174	137	(B) 140	101

NOTES: A - Study by Kallem (1997) w/ plywood, B - Study by IHRC (1004) w/ plywood and C - Study by Murphy et al. (1996) w/ OSB

Table 5-3. Mean of calculated maximum fastener loads based on tributary area

Fastening Schedule	6 in. / 12 in. (lbs)	6 in. / 8 in. (lbs)	6 in. / 6 in. (lbs)	Design Withdrawal Load (lbs)
8d Ring Shank	175	144	126	387
8d Smooth Shank	130	117	103	410
6d Smooth Shank	75	NA	NA	328

Notes:

(1) Specific gravity of 0.55 assumed for design withdrawal load (2) Ring shank (0.113 in. diameter) design withdrawal strength determined by linear extrapolation (38.67 lbs/in.) (AF&PA 2005)

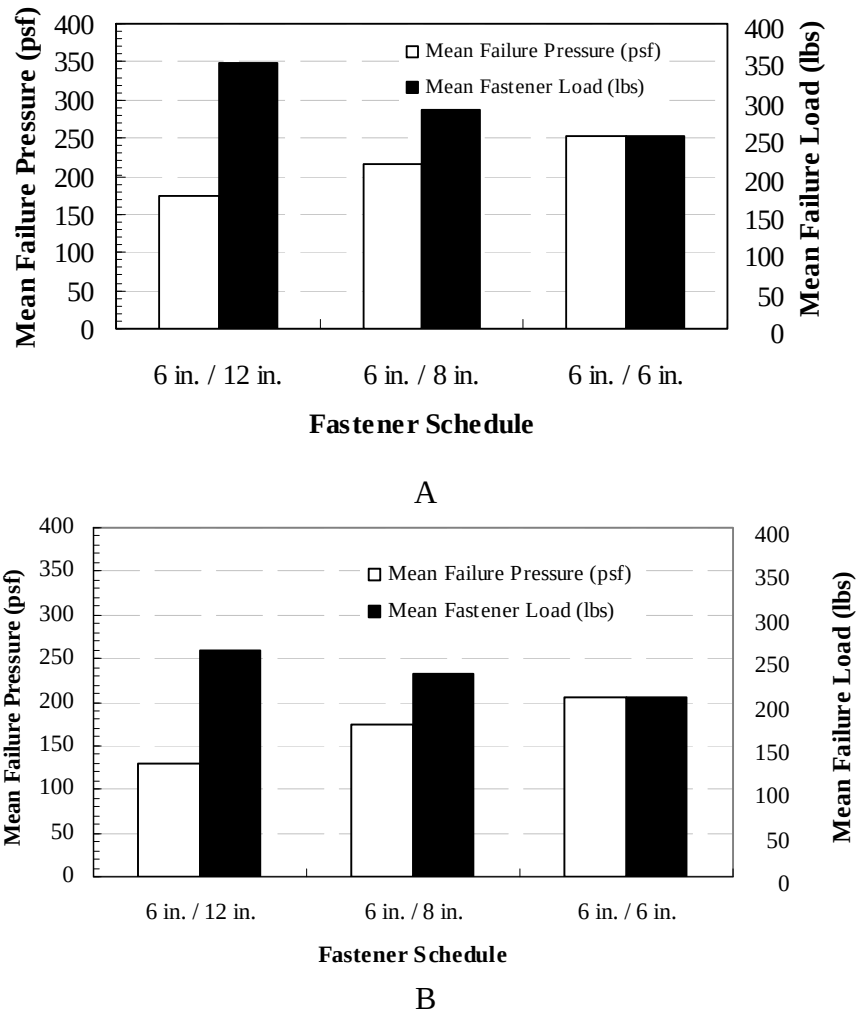
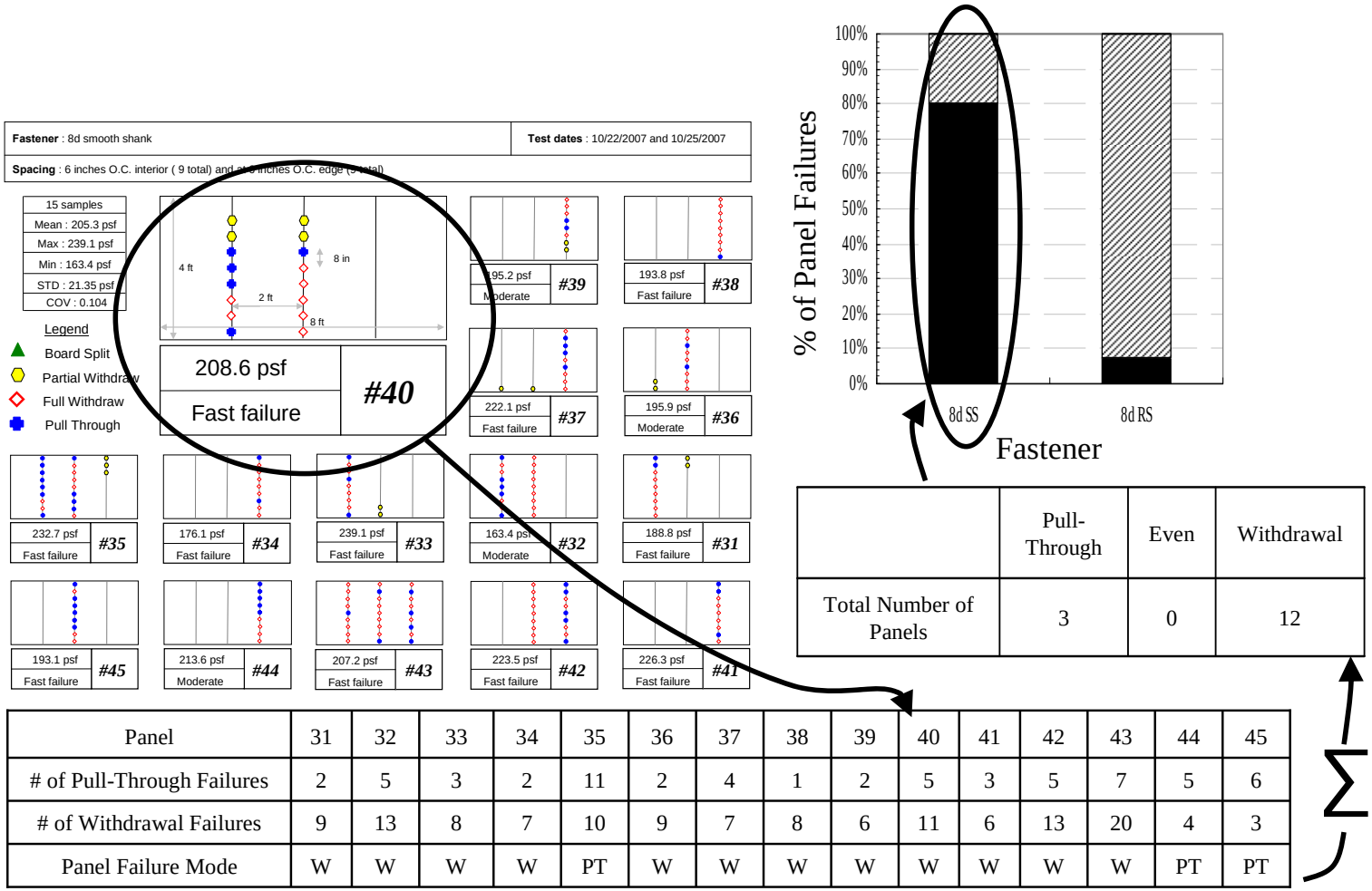


Figure 5-2. Summary of mean panel failure (psf) vs. calculated fastener failure (lbs) for A) 8d ring shank nails and B) 8d smooth shank nails attached at 6 in. / 12 in., 6 in. / 8 in. and 6 in. / 6 in. schedules

Table 5-4. Results of static UF-WRSUT of laboratory fabricated panels fastened with 8d ring shank nails at 6 in. / 12 in. retrofitted with ccSPF adhesive

	# of Panels	Mean Failure Pressure (psf)	COV
Control Series	10	161	18%
Fillet Retrofit (B-1)	10	202	14%
Full 3 in. Retrofit (B-2)	10	209	19%



	Pull-Through	Even	Withdrawal
Total Number of Panels	3	0	12

Panel	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45
# of Pull-Through Failures	2	5	3	2	11	2	4	1	2	5	3	5	7	5	6
# of Withdrawal Failures	9	13	8	7	10	9	7	8	6	11	6	13	20	4	3
Panel Failure Mode	W	W	W	W	PT	W	W	W	W	W	W	W	W	PT	PT

Figure 5-3. Sample of failure mode and location calculations from Statically tested 8d smooth shank nails at 6 in. /12 in.

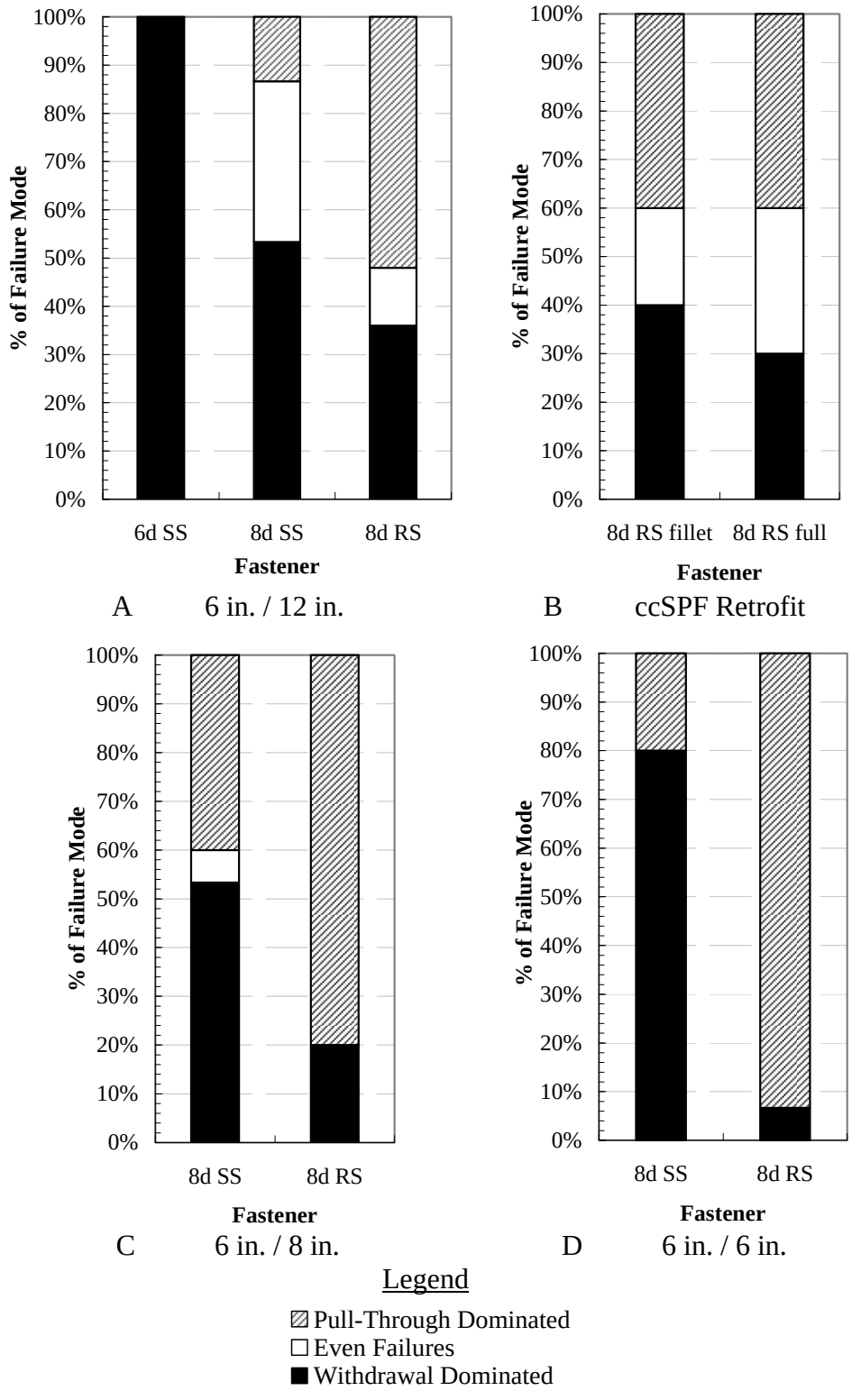


Figure 5-4. Distribution of Laboratory Fabricated Statically Tested Panels Dominated by Withdrawal or Pull-through Failure modes A) 6 in. / 12 in. spacing, B) ccSPF retrofit of 6 in. / 12 spacing, C) 6 in. / 8 in. spacing and D) 6 in. / 6 in. spacing

Table 5-5. Measured failure pressure and calculated fastener failure load of statically tested panels harvested from existing structures in Central Florida

House	Failure Pressure (psf)	Tributary Area (ft^2)	Fastener Withdrawal Load (lbs)
Staples: 1.5 in. staple @ 4 in. / 4 in. spacing, ½ in. OSB			
1.5 in. Staple-7	93	0.67	62
1.5 in. Staple-8	93	0.67	62
Mean	93		62
Staples: 1.5 in. staple @ 3 in. / 6 in. spacing, ½ in. Plywood			
1.5 in. Staple-1	137	1	137
1.5 in. Staple-2	132	1	132
1.5 in. Staple-3	117	1	117
1.5 in. Staple-4	93	1	93
Mean	120		120
Staples: 2.5 in. staple @ 3 in. / 6 in. spacing, ½ in. Plywood			
2.5 in. Staple-1	66	1	66
2.5 in. Staple-2	97	1	97
2.5 in. Staple-3	30	1	30
2.5 in. Staple-4	39	1	39
Mean	58		58
Nails: 2.5 in. smooth shank nail @ 6 in. / 12 in. spacing, ½ in. Plywood			
8d Smooth Shank-46	112	2	224
8d Smooth Shank-47	101	2	202
Mean	107		213
Nails: 2 in. smooth shank nail @ 6 in. / 12 in. spacing, ½ in. Plywood			
6d Smooth Shank-16	134	2	268
6d Smooth Shank-17	98	2	196
6d Smooth Shank-18	74	2	148
6d Smooth Shank-20	53	2	106
6d Smooth Shank-21	132	2	264
6d Smooth Shank-29	46	2	92
6d Smooth Shank-30	31	2	62
6d Smooth Shank-36	59	2	118
6d Smooth Shank-37	90	2	180
Mean	80		159
Nails: 2 in. smooth shank nail @ 8 in. / 8 in. spacing, ½ in. Plywood			
6d Smooth Shank-34	83	1.33	111
6d Smooth Shank-35	62	1.33	83
Mean	73		97

Table 5-5. Continued

House	Failure Pressure (psf)	Tributary Area (ft^2)	Fastener Withdrawal Load (lbs)
Nails: 2 in. smooth shank nail @ 3 in. / 6 in. spacing, ½ in. Plywood			
6d Smooth Shank-24	117	1	117
6d Smooth Shank-32	62	1	62
6d Smooth Shank-33	64	1	64
Mean	81		81
Nails: 2 in. smooth shank nail @ 4 in. / 4 in. spacing, ½ in. Plywood			
6d Smooth Shank-27	149	0.67	99

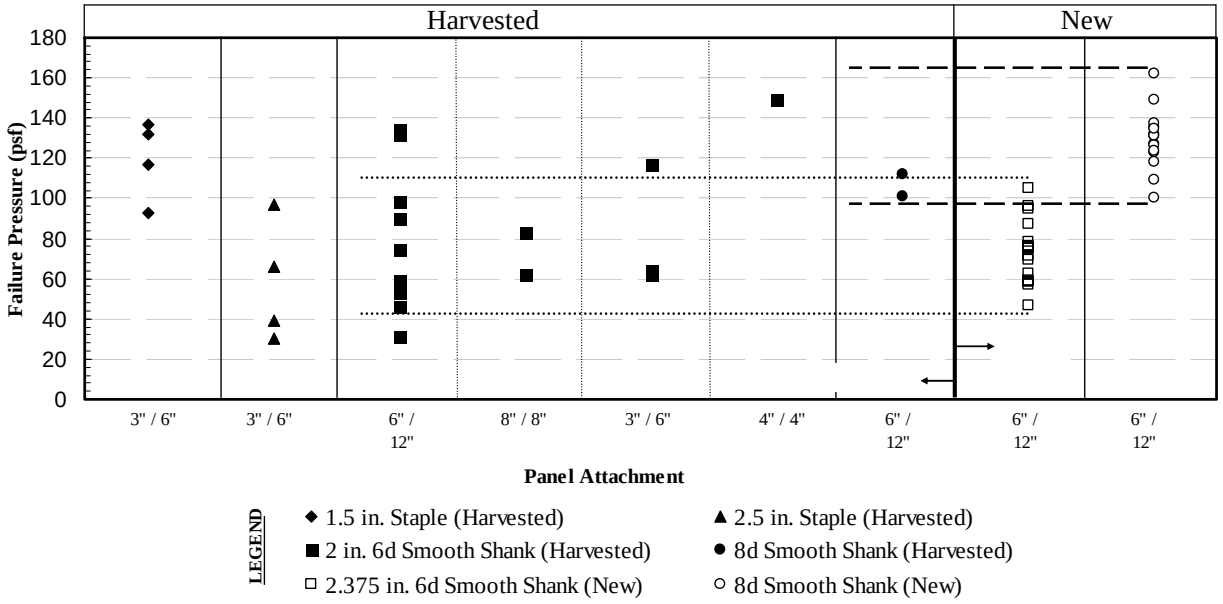
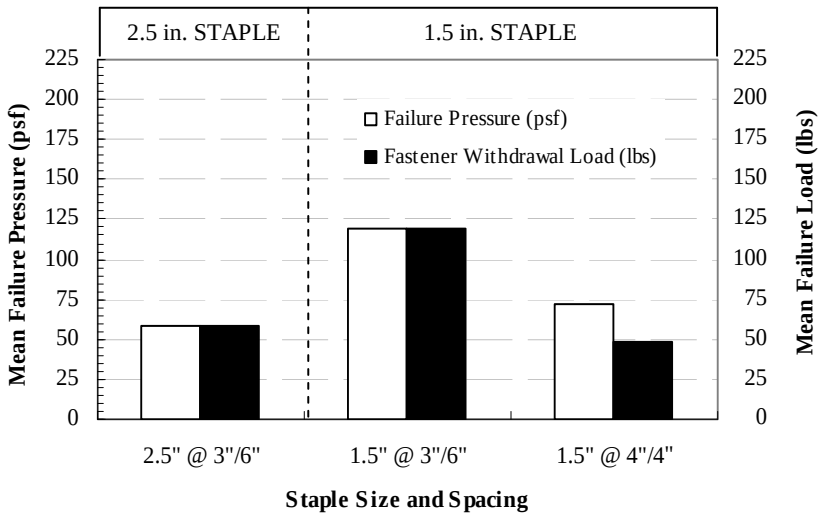


Figure 5-5. Comparison of statically tested harvested and new panels full results of failure pressure



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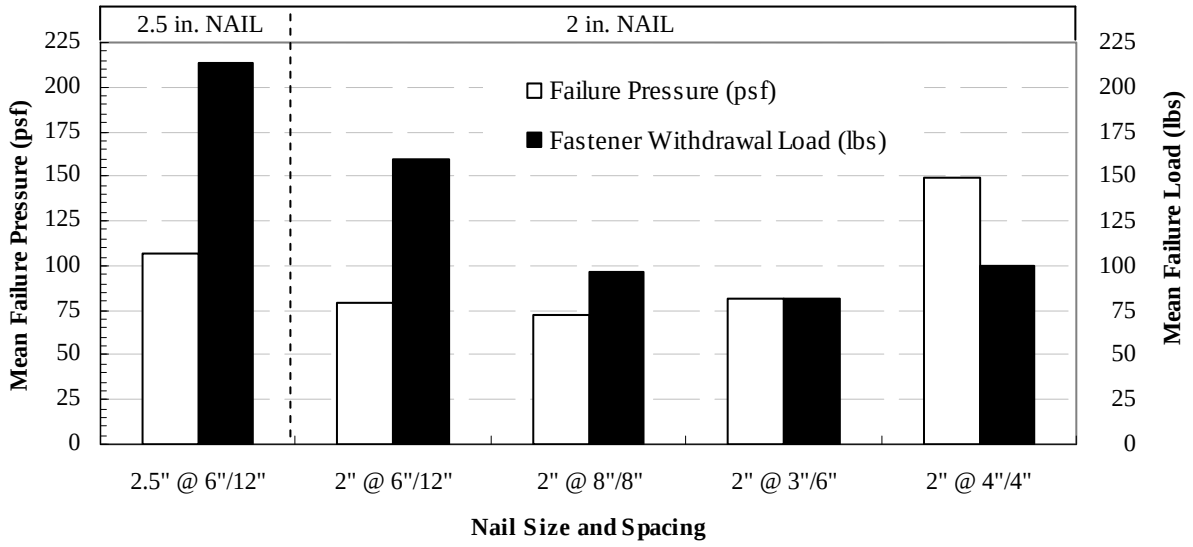


B

Figure 5-6. Comparison of mean uplift capacities with mean calculated fastener failure loads for harvested panels attached with a) staples and b) nails

Table 5-6. Mean failure pressures of static UF-WRSUT of panels harvested from existing LFWS located in Central Florida and retrofitted existing panels

Source	Existing		Retrofit			% increase in Mean Failure Pressure
	# of Panels	Mean Failure Pressure (psf)	# of Panels	Retrofit Method	Mean Failure Pressure (psf)	
Staples: 1.5 in. staple @ 3 in. / 6 in. spacing, ½ in. plywood						
Port Orange #1	4	120	2	A-1	270	125%
Staples: 2.5 in. staple @ 3 in. / 6 in. spacing, ½ in. plywood						
Bartow #2	2	35	2	A-1	167	377%
Nails: 2.5 in. smooth shank nail @ 6 in. / 12 in. spacing, ½ in. plywood						
Crystal River #1	2	107	2	A-1	200	87%
Nails: 2 in. smooth shank nail @ 6 in. / 12 in. spacing, ½ in. plywood						
Port Orange #2	3	102	1	B-2	250	145%
Bartow #3	2	93	2	A-1	218	134%
Port Orange #3	2	39	1	A-1	140	259%
Nails: 2 in. smooth shank nail @ 3 in. / 6 in. spacing, ½ in. plywood						
Bartow #4	1	117	2	A-1	169	44%
Nails: 2 in. smooth shank nail @ 4 in. / 4 in. spacing, ½ in. plywood						
Bartow #5	1	149	1	A-1	197	32%

Notes:

Retrofit Method A-1. 8d ring shank nail @ 6 in. / 12 in. spacing

Retrofit Method B-2. 3 in. thick layer of spray applied polyurethane adhesive

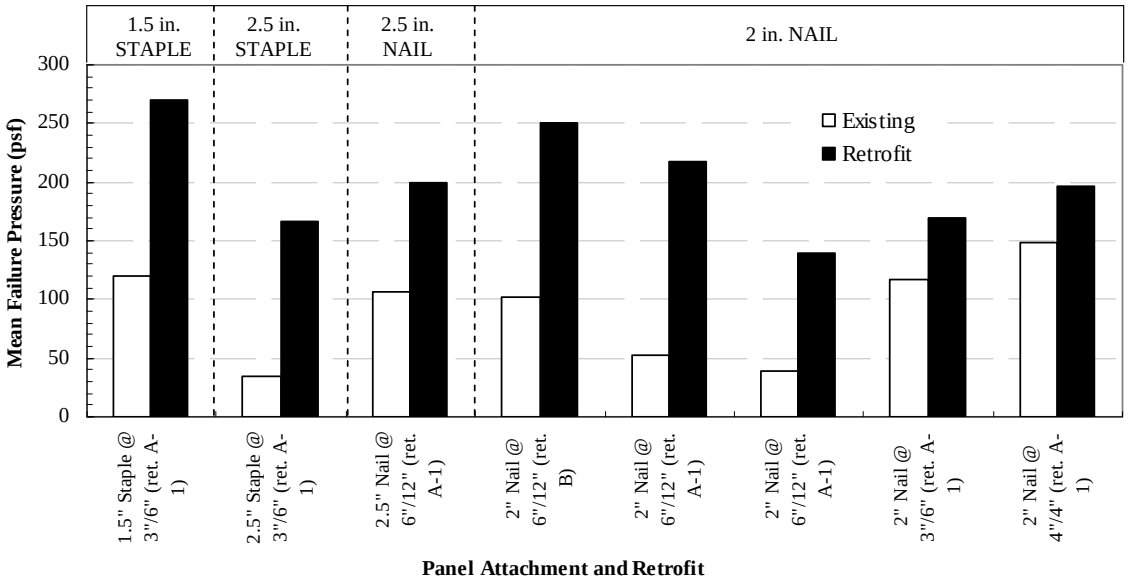


Figure 5-7. Comparison of statically tested harvested panels in existing vs. retrofit conditions separated by individual source houses

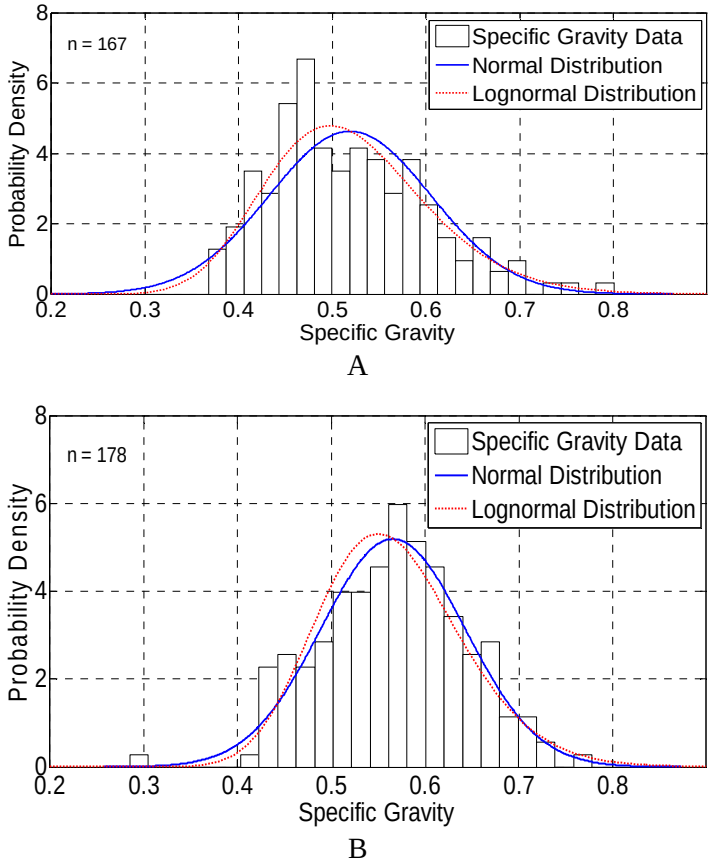


Figure 5-8. Probability distribution of Specific Gravity of Panels Tested in Phase 2 for (A New Panels and (B Harvested Panels 15 years old

Table 5-7. Summary of specific gravities calculated to investigate effect of specific gravity on panel wind uplift resistance

ID	Statically Tested				Dynamically Tested			
	Mean Failure Pressure (psf)	SG 1	SG 2	SG 3	Mean Failure Pressure (psf)	SG 1	SG 2	SG 3
6d Com. at 6"/12"	62	0.47	0.55	0.57	52	0.53	0.52	0.52
6d Com. At 6"/6"	108	0.52	0.55	0.54	90	0.54	0.55	0.54
6d Com. At 6"/12" Ret. A	183	0.47	0.46	0.48	167	0.55	0.58	0.63
1.5" 14 gauge Staple at 4"/4"	93	0.60	0.62	0.60	72	0.56	0.55	0.50

Notes: SG 1 calculated as mean of all framing members, SG 2 calc. as mean of all framing members which failed, and SG 3 calc. as mean of initial framing member(s) that failed

Table 5-8. Mean failure pressure of laboratory fabricated panels attached with 2 in. long 6d common nails tested statically and dynamically per UF-WRSUT

Fastener Schedule	Mean Specific Gravity of Framing	Static		Dynamic		Ratio of Dynamic to Static Mean Failure Pressures
		# of Panels	Mean Failure Pressure (psf)	# of Panels	Mean Failure Pressure (psf)	
6 in. / 12 in.	0.50	5	62	5	52	0.84
6 in. / 6 in.	0.53	5	108	5	90	0.83
6 in. / 12 in. ret. A-2	0.51	2	183	2	167	0.91

Notes:

Retrofit Method A-2. 8d ring shank nails installed between an existing fastener spacing of 6 in. / 12 in.

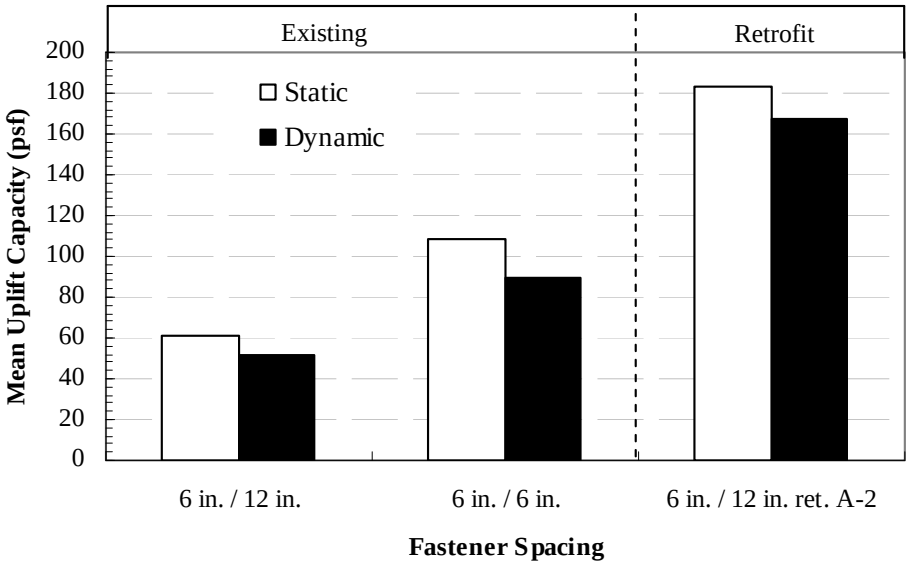


Figure 5-9. Comparison of statically vs. dynamically loaded laboratory fabricated panels attached with 2 in. long 6d smooth shank nails at 6 in. / 12 in. (existing and retrofit) and 6 in. / 6 in.

Table 5-9. Mean failure pressure of panels harvested from existing construction attached with 1.5 in. long staples at 4 in. / 4 in. tested statically and dynamically per UF-WRUT

Fastener Schedule	Mean Specific Gravity of Framing	Mean Moisture Content of Framing	Static		Dynamic		Ratio of Dynamic to Static Mean Failure Pressures
			# of Panels	Mean Failure Pressure (psf)	# of Panels	Mean Failure Pressure (psf)	
4 in. / 4 in.	0.56	9%	2	93	4	72	0.77
4 in. / 4 in. ret. A-2	0.58	9%	2	168	2	126	0.75

Notes:

Retrofit Method A-2. 8d ring shank nails installed between an existing fastener spacing of 6 in. / 12 in.

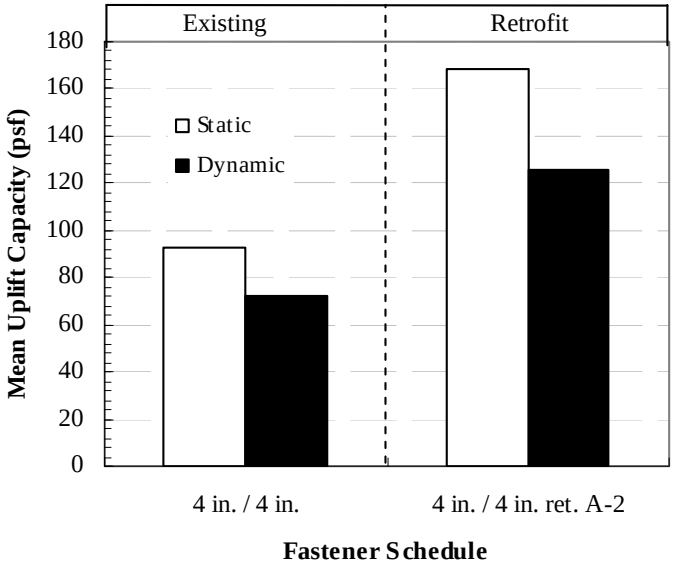


Figure 5-10. Comparison of statically vs. dynamically loaded panels harvested from existing construction attached with 1.5 in. long staples at 4 in. / 4 in. (existing and retrofit)

Table 5-10. Summary of comparison of peak vs. failure pressure for all static vs. dynamic panel results (34 panels)

Dynamically Tested Panels						Statically Tested Panels					
Panel	Peak (psf)	(sec.)	Failure (psf)	(sec.)	Ratio of Failure to Peak	Panel	Peak (psf)	(sec.)	Failure (psf)	(sec.)	Ratio of Failure to Peak
6d SS at 6 in. / 12 in. (Lab)											
6d SS-43	34.5	52	33	60	0.96	6d SS-38	47	52	47	52	
6d SS-44	51	68	48	72	0.94	6d SS-40	58	61	58	61	
6d SS-45	50	62	46	67.5	0.92	6d SS-41	64.8	61	63.3	70	98%
6d SS-46	58	72	58	72		6d SS-42	65	62	65	62	
6d SS-47	66	82	66	82		6d SS-39	73	70	73	70	
6d SS at 6 in. / 6 in. (Lab)											
6d SS-57	44	62	44	62		6d SS-53	106	90	106	90	
6d SS-58	103	111	103	111		6d SS-52	64	61	64	61	
6d SS-59	105	111.5	105	111.5		6d SS-56	134	111	134	111	
6d SS-60	72	82	53	85	0.74	6d SS-55	91	81	91	81	
6d SS-61	124	121	104	124	0.84	6d SS-54	135	110	135	110	
6d SS at 6 in. / 12 in. Retrofitted with A-2 (Lab)											
6d SS-50	164	171	164	171		6d SS-48	199	150	195	154	98%
6d SS-51	171	173.5	162	173.8	0.95	6d SS-49	167	130	167	130	
1.5 in. Staple at 4 in. / 4 in. (Harvested)											
1.5 in. Staple-9	93	102	93	102		1.5 in. Staple-7	93	80	90	82	97%
1.5 in. Staple-10	61	72	61	72		1.5 in. Staple-8	93	80	90	84	97%
1.5 in. Staple-11	52	62	37	66	0.71						
1.5 in. Staple-12	82	82	71	89	0.87						
1.5 in. Staple at 4 in. / 4 in. with Retrofitted with A-2 (Harvested)											
1.5 in. Staple-15	150	161	150	161		1.5 in. Staple-13	168	130	165	132	98%
1.5 in. Staple-16	102	102	90	105	0.88	1.5 in. Staple-14	169	130	165	136	98%

Table 5-11. Summary of reduction from statically tested to dynamically tested panel resistance for both peak and failure pressures

	Peak Pressure	Failure Pressure
6d SS at 6 in. / 12 in. (Lab)	16%	20%
6d SS at 6 in. / 6 in. (Lab)	15%	23%
6d SS at 6 in. / 12 in. Retrofitted with A-2 (Lab)	8%	10%
1.5 in. Staple at 4 in. / 4 in. (Harvested)	23%	27%
1.5 in. Staple at 4 in. / 4 in. with Retrofitted with A-2 (Harvested)	25%	27%

Table 5-12. Summary of series used in analysis of variance

Series	# of Panels	Mean (psf)	St. Dev.
Laboratory Fabricated Static			
8d RS 6/12 w/ ccSPF fillet	10	202	28
8d RS 6/12 w/ full ccSPF	10	209	39
8d RS 6/12	15	174	23
8d RS 6/12 ccSPF Control	10	161	28
8d RS 6/8	15	216	36
8d RS 6/6	13	252	18
8d SS 6/12	15	129	15
8d SS 6/8	15	175	20
8d SS 6/6	15	205	21
6d SS 6/12	15	74	16
Harvested Static			
6d SS 6/12 harvest	9	80	37
6d SS 6/12 lab	15	74	16
Laboratory Fabricated Static vs. Dynamic			
6d SS 6/12 S	5	62	10
6d SS 6/12 D	5	52	12
6d SS 6/6 S	5	108	29
6d SS 6/6 D	5	90	31

Table 5-13. ANOVA table for all laboratory fabricated static phase results (alpha = 0.05, therefore 95% confidence level)

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	301684	7	43097.72	81.87163	1.27E-39	2.098005
Within Groups	55272.63	105	526.406			
Total	356956.7	112				

Table 5-14. Bonferroni test full results for laboratory fabricated statically tested panels (alpha = 0.05)

Series A	Series B	Lower Bound	Upper Bound	Null Hypothesis
8d RS 6/12 w/ ccSPF fillet	8d RS 6/12 w/ full ccSPF	-43.98155544	30.18155544	Accept
	8d RS 6/12	-6.284007303	61.41734064	Accept
	8d RS 6/12 ccSPF control	3.91844456	78.08155544	Reject
	8d RS 6/8	-47.91067397	19.79067397	Accept
	8d RS 6/6	-85.26899174	-15.51562365	Reject
	8d SS 6/12	38.7699927	106.4713406	Reject
	8d SS 6/8	-6.944007303	60.75734064	Accept
	8d SS 6/6	-37.1440073	30.55734064	Accept
	6d SS 6/12	93.88265936	161.5840073	Reject
8d RS 6/12 w/ full ccSPF	8d RS 6/12	0.615992697	68.31734064	Reject
	8d RS 6/12 ccSPF control	10.81844456	84.98155544	Reject
	8d RS 6/8	-41.01067397	26.69067397	Accept
	8d RS 6/6	-78.36899174	-8.615623647	Reject
	8d SS 6/12	45.6699927	113.3713406	Reject
	8d SS 6/8	-0.044007303	67.65734064	Accept
	8d SS 6/6	-30.2440073	37.45734064	Accept
	6d SS 6/12	100.7826594	168.4840073	Reject
8d RS 6/12	8d RS 6/12 ccSPF control	-20.41734064	47.2840073	Accept
	8d RS 6/8	-71.9036299	-11.34970343	Reject
	8d RS 6/6	-109.378864	-46.53908473	Reject
	8d SS 6/12	14.77703677	75.33096323	Reject
	8d SS 6/8	-30.93696323	29.61696323	Accept
	8d SS 6/6	-61.13696323	-0.583036768	Reject
	6d SS 6/12	69.88970343	130.4436299	Reject
8d RS 6/12 ccSPF control	8d RS 6/8	-88.91067397	-21.20932603	Reject
	8d RS 6/6	-126.2689917	-56.51562365	Reject
	8d SS 6/12	-2.230007303	65.47134064	Accept
	8d SS 6/8	-47.9440073	19.75734064	Accept
	8d SS 6/6	-78.1440073	-10.44265936	Reject
	6d SS 6/12	52.88265936	120.5840073	Reject
8d RS 6/8	8d RS 6/6	-67.75219732	-4.912418063	Reject
	8d SS 6/12	56.40370343	116.9576299	Reject
	8d SS 6/8	10.68970343	71.2436299	Reject
	8d SS 6/6	-19.51029657	41.0436299	Accept
	6d SS 6/12	111.5163701	172.0702966	Reject

Table 5-14. Continued

Series A	Series B	Lower Bound	Upper Bound	Null Hypothesis
8d RS 6/6	8d SS 6/12	91.59308473	154.432864	Reject
	8d SS 6/8	45.87908473	108.718864	Reject
	8d SS 6/6	15.67908473	78.51886399	Reject
	6d SS 6/12	146.7057514	209.5455307	Reject
8d SS 6/12	8d SS 6/8	-75.99096323	-15.43703677	Reject
	8d SS 6/6	-106.1909632	-45.63703677	Reject
	6d SS 6/12	24.83570343	85.3896299	Reject
8d SS 6/8	8d SS 6/6	-60.47696323	0.076963232	Accept
	6d SS 6/12	70.54970343	131.1036299	Reject
8d SS 6/6	6d SS 6/12	100.7497034	161.3036299	Reject

Table 5-15. Bonferroni test summary for laboratory fabricated static tested results (alpha = 0.05)

	8d RS at 6/12, ccSPF fillet	8d RS at 6/12, ccSPF full	8d RS at 6/12	8d RS at 6/12 ccSPF control	8d RS at 6/8	8d RS at 6/6	8d SS at 6/12	8d SS at 6/8	8d SS at 6/6	6d SS at 6/12
8d RS at 6/12, ccSPF fillet	1	-	-	-	-	-	-	-	-	-
8d RS at 6/12, ccSPF full	1	1	-	-	-	-	-	-	-	-
8d RS at 6/12	1	0	1	-	-	-	-	-	-	-
8d RS at 6/12 ccSPF control	0	0	1	1	-	-	-	-	-	-
8d RS at 6/8	1	1	0	0	1	-	-	-	-	-
8d RS at 6/6	0	0	0	0	0	1	-	-	-	-
8d SS at 6/12	0	0	0	1	0	0	1	-	-	-
8d SS at 6/8	1	1	1	1	0	0	0	1	-	-
8d SS at 6/6	1	1	0	0	1	0	0	1	1	-
6d SS at 6/12	0	0	0	0	0	0	0	0	0	1

Notes: 1 = same population (accept null hypothesis), and 0 = different population (reject null hypothesis)

Table 5-16. ANOVA table for all panels fastened with 6d smooth shank nails spaced at 6 in. / 12 in. (alpha = 0.05, therefore 95% confidence level)

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	3076.918	3	1025.639	1.986536	0.137212	2.922277
Within Groups	15488.86	30	516.2954			
Total	18565.78	33				

Table 5-17. Bonferroni test full results for all panels fastened with 6d smooth shank nails spaced at 6 in / 12 in. (alpha = 0.5)

Series A	Series B	Lower Bound	Upper Bound	Null Hypothesis
6d SS 6/12 harvest	6d SS 6/12 lab	-21.66197671	32.46197671	Accept
	6d SS 6/12 S	-17.63496351	53.96429685	Accept
	6d SS 6/12 D	-8.176963514	63.42229685	Accept
6d SS 6/12 lab	6d SS 6/12 S	-20.37935052	45.90868385	Accept
	6d SS 6/12 D	-10.92135052	55.36668385	Accept
6d SS 6/12 S	6d SS 6/12 D	-31.13496506	50.05096506	Accept

Table 5-18. T-test result for panels retrofitted with 8d ring shank nails at 6 in. / 12 in. vs. panels with only 8d ring shank nails at 6 in. / 12 in.

	8d RS 6/12 ret.	8d RS 6/12 exist
Mean	192.789375	169.06
Variance	1886.39914	648.3875
Observations	16	25
Hypothesized Mean Difference	0	
df	22	
t Stat	1.978578735	
P(T<=t) one-tail	0.030258876	
t Critical one-tail	1.717144335	
P(T<=t) two-tail	0.060517752	
t Critical two-tail	2.073873058	

Table 5-19. ANOVA table for all panels tested in static vs. dynamic phase using peak pressure (alpha = 0.05, therefore 95% confidence level)

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	9952.879	3	3317.626	6.436434	0.004583	3.238872
Within Groups	8247.117	16	515.4448			
Total	18200	19				

Table 5-20. Bonferroni test full results for panels tested in static vs. dynamic phase using peak pressure (alpha = 0.05)

Series A	Series B	Lower Bound	Upper Bound	Null Hypothesis
6d SS 6/12 S	6d SS 6/12 D	-33.73835588	52.65435588	Accept
	6d SS 6/6 S	-89.85435588	-3.461644119	Reject
	6d SS 6/6 D	-71.32035588	15.07235588	Accept
6d SS 6/12 D	6d SS 6/6 S	-99.31235588	-12.91964412	Reject
	6d SS 6/6 D	-80.77835588	5.614355881	Accept
6d SS 6/6 S	6d SS 6/6 D	-24.66235588	61.73035588	Accept

Table 5-21. ANOVA table for all panels tested in static vs. dynamic phase using failure pressure (alpha = 0.05, therefore 95% confidence level)

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	9054.6535	3	3018.2178	5.782337	0.007095	3.238872
Within Groups	8351.552	16	521.972			
Total	17406.2055	19				

Table 5-22. Bonferroni test full results for panels tested in static vs. dynamic phase using failure pressure (alpha = 0.05)

Series A	Series B	Lower Bound	Upper Bound	Null Hypothesis
6d SS 6/12 S	6d SS 6/12 D	-32.408998	54.52899801	Accept
	6d SS 6/6 S	-88.208998	-1.27100199	Reject
	6d SS 6/6 D	-64.008998	22.92899801	Accept
6d SS 6/12 D	6d SS 6/6 S	-99.268998	-12.331002	Reject
	6d SS 6/6 D	-75.068998	11.86899801	Accept
6d SS 6/6 S	6d SS 6/6 D	-19.268998	67.66899801	Accept

CHAPTER 6
DISCUSSION

Wood roof sheathing panel system wind uplift design is based on prescriptive codes which specify acceptable panel construction materials and attachment. The determination of panel construction is based on pseudo-static pressure testing of full size newly constructed panels to failure. Wind uplift loading of wood roof sheathing systems in existing conditions is both dynamic and subjected to aging or weathering. It is the hypothesis of this investigation that pseudo-static pressure testing of newly constructed panels overestimates the wind uplift resistance, which accounts for a portion of the observed failures below design wind speeds.

In order to do this a standard test protocol was developed (UF-WRSUT) as none currently exist. The protocol has two procedures, Method A (static) and Method B (dynamic), in order to (1) provide a direct comparison of static and dynamic loading and (2) create a test protocol using simple equipment which can be utilized in most laboratories. The standard protocol is compared with previous studies which currently form the basis for wood roof sheathing uplift design. The static method is then used to evaluate the uplift resistance of panels harvested from existing structures in Florida. Finally laboratory constructed and harvested panels are tested with both methods in order to compare the uplift resistance of wood roof sheathing under dynamic loading. This chapter discusses findings from the experimental study to evaluate the effect of dynamic uplift loading and age or weathering on wood roof sheathing panel uplift capacity.

Analysis of Design Wind Speeds

As detailed in Chapter 2 the current wind uplift design of wood roof sheathing is based on previous wind uplift pressure testing. To compare results from this experimental study to design wind speeds ASCE 7-05 (2006) Method 2 – Analytical Procedure is worked in reverse. The author was unable to find a standard method for determining design uplift pressure, therefore a

factor of safety is applied to the mean failure pressure to obtain the design pressure. The 2.0 factor of safety is selected based on a paper by Sutt et al. (2003) which summarizes applicable research to panel uplift resistance. The paper proposes a design equation for residential wood roof sheathing based on individual fastener capacity, which applies a factor of safety of 2.0 for panel design pressures in addition to the factor of safety of 5.0 used in nail withdrawal design loads. Additionally 5% exclusion values are used for design pressure where available for comparison.

The structure which the panels are assumed to be installed in is a partially enclosed gable roof building with a mean roof height of 15 ft in exposure B conditions. Equation 6-22 (eq. 6-2) is rearranged to solve for the velocity pressure (q_z) (eq. 6-3), where the design pressure (p) is divided by the sum of the internal (GC_{pi} = +0.55) and external (GC_p = -2.6) product of gust effect factor and pressure coefficients for an effective wind area of 2 ft^2.

$$p = q_z\left[\left(GC_p\right) - \left(GC_{pi}\right)\right] \tag{6-2}$$

$$q_z = \frac{p}{\left[\left(GC_p\right) - \left(GC_{pi}\right)\right]} \tag{6-3}$$

Then equation 6-15 (eq. 6-4) is rearranged to solve for the design velocity (V_{design}) (eq. 6-5) where q_z is divided by the product of a unit conversion (0.00256), elevation coefficient (K_z = 0.7), topographic factor (K_{zt} = 1.0), directionality factory (K_d = 0.85) and importance factor (I = 1.0). Then the roof of that is the design velocity.

$$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V_{design}{}^2 \cdot I \tag{6-4}$$

$$V_{design} = \sqrt{\frac{q_z}{0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot I}} \tag{6-5}$$

Table 6-1 summarizes the calculated design wind speeds for laboratory fabricated UF-WRSUT Method A results with both 2.0 factor of safety and 5% exclusion value design pressures. From the analysis it is seen that the 2.0 factor of safety design pressure is more conservative than the 5% exclusion design pressure, where wind speeds are 11-25% less for the factor of safety design pressure. The current connection of residential wood roof sheathing in coastal areas is in Florida is 8d ring shank at 6 in. / 6 in. (ICC 2004) and 8d smooth shank at 4 in. / 8 in. (ICC 2006). ASCE 7-05 (ASCE 2006) design wind speeds for coastal areas in Florida range from 130-150 mph. Results corresponding to current construction practices, 8d ring shank at 6 in. / 6 in. and 8d smooth shank at 6 in. / in. panels, are compared to design wind speeds in coastal areas. Wind speeds calculated with the 5% exclusion design pressure are sufficient for both connections. It is seen that current connections of 8d ring shank nails spaced at 6 in. / 6 in. are sufficient to resist design wind speed in coastal areas. It is found that 6d smooth shank nails provide an insufficient resistance design wind speeds above 100 mph. This suggests that the current building stock constructed before 1994 is in need of retrofit measures to prevent significant losses due to hurricane loading.

Design pressure calculated with the 2.0 factor of safety are considered herein due to lack of 5% exclusion values for all results. Table 6-2 summarizes calculated design wind speeds for panels tested per UF-WRSUT Method B. It is seen that no panel configuration used is sufficient to be used in coastal areas. Panels retrofitted with 8d ring shank nails at 6 in. / 12 in. have increased calculated design wind speeds but do not meet ASCE 7-05 design wind speeds.

Dynamic testing was not conducted on current construction so conclusions cannot be made regarding the effect of dynamic loading on current construction. However it is shown that

dynamic loading reduces similar wood roof sheathing uplift resistance, therefore it is reasonable to assume that current construction uplift resistance would be reduced.

Effect of Aging or Weathering on Wind Uplift Resistance

Laboratory studies of wood roof sheathing connections have suggested that withdrawal strength can be effected by conditions which may be present in the field, detailed in Chapter 2. As part of this study the effects of aging or weathering on wind uplift resistance is investigated by testing uplift capacities of panel harvested from existing structures. Results do not suggest any significant reduction in wind uplift resistance of residential wood roof sheathing due to aging effects. Results show that harvested panels are within the range of similar laboratory fabricated results. Comparisons are difficult to make due to the limited sample sizes and variation in construction of harvested panel. As discussed in Chapter 4 it is difficult to obtain harvested panels and the researcher must take what is available. The effect of age or weathering is shown not to have an effect on the specific gravity of the wood framing of harvested samples. The specific gravity samples collected from existing structures is actually shown to have a higher specific gravity (0.57) than those of laboratory constructed samples (0.52).

Panels retrofitted with all of the four methods used experience increased wind uplift resistance. Retrofitting panels with 8d ring shank nails can be done when the roof covering is being replace with little effort. Considering the finding that age or weathering do not effect panel uplift capacity when fastened with nails, results suggests that the significant increase in capacity found in testing will be sustained over the life of the roof. Panels retrofitted with ccSPF have similar structural benefits and provide an option to retrofit the roof at any time, not just when being re-roofed. However the effects of age or weather on ccSPF structural capacity are not known.

Effect of Dynamic Nature of Wind Uplift Loading

The comparison of wind uplift performance of statically tested panels to similar panels tested dynamically is conducted in order to evaluate the effect of the dynamic nature of wind uplift loading on residential wood roof sheathing. Results showed approximately a 20% reduction in uplift capacity when dynamically tested panels are compared with statically tested ones. The amplitude and frequency content of the dynamic pressure trace derived from wind tunnel tests was representative of actual wind pressure fluctuations to represent as accurately as possible real wind flow. Current pseudo-static wind uplift testing is based on the assumption that the residential wood structure is a rigid which results in a high natural frequency, therefore the amplifying effects of dynamic loading can be neglected. While the natural frequency of residential wood structures is known the natural frequency of wood roof sheathing is currently unknown and may not be a rigid system. However displacement measurements were not taken from sheathing during testing making conclusions about the rigidity of sheathing impossible, and small sample sizes limit the statistical merit of the conclusions.

Results show that 50% of panels tested dynamically fail below the highest instantaneous pressure during testing suggesting a cumulative damage effect. It is thought that if a panel is able to take load after it receives a pressure spike then the panel failure is not due to the large loading rate associated with the pressure spike. When failure then occurs at a pressure lower than the preceding spike it is suggested that this is evidence of damage to the nail/wood interface accumulating. These results are preliminary and further testing is necessary to confirm the findings.

Wind Uplift Behavior of Residential Wood Roof Sheathing

Results showed an interesting fact in that current tributary area model used to estimate the nail withdrawal load of sheathing behavior. For all data sets the calculated fastener failure load

increased with decreasing fastener spacings. It was expected that the withdrawal load would be the same regardless of nail spacing since the same wood is used. A new model developed by Dao and van de Lindt (2008) suggests that withdrawal capacity of fasteners installed in sheathing is non-linearly dependant on fastener spacing, where as spacing increases the moment applied to the fastener increases. It is found that under uniform loading panel displacement is increased significantly and therefore field fastener strengths are reduced. This model is particularly interesting when considering the spatial variations present in wind uplift loading, where pressure may vary significantly over a 4 ft by 8 ft surface. Considering that moment reduces the withdrawal capacity of fasteners installed in wood then field nails would also experience moment.

Wood roof sheathing panels behavior is based on the assumption that fastener withdrawal strength with be the limiting factor in panel failure capacity. As discussed in Chapter 2 pull-through strengths of common wood roof sheathing are similar to fastener withdrawal strengths. It is therefore expected that as fastener withdrawal capacity increases, based on fastener type or spacing, the panel failure mode will change from withdrawal to pull-through. This trend is observed in test results for fastener type, where panels fastened with ring shank nails have more pull-through failures. It is observed that as fastener spacing increases the number of pull-through failures decrease, which suggests an inverse relationship of fastener spacing and withdrawal capacity. This finding supports Dao and van de Lindt findings that spacing effects fastener withdrawal capacity. The confirmation of fastener withdrawal strength reducing as spacing increases and the discrepancy of this finding with the tributary area calculated fastener load suggest that tributary is not appropriate for wood roof sheathing.

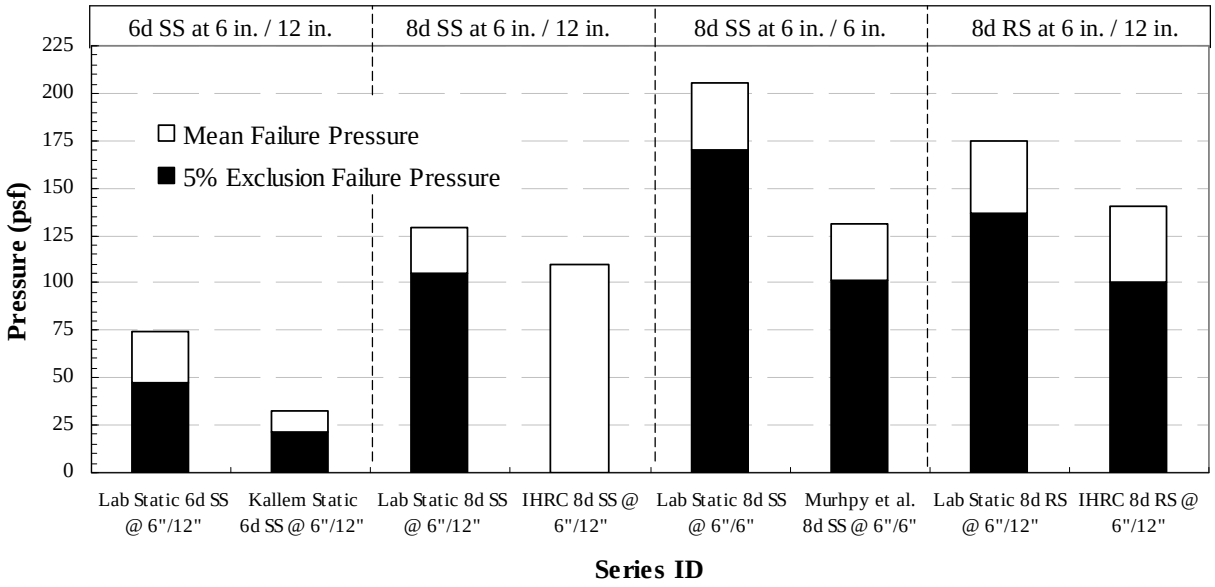


Figure 6-1. Comparison of lab static mean and 5% exclusion failure pressures to studies by Kallem (1997) w/ plywood, IHRC (2004) w/ plywood and Murphy et al. (1996) w/ OSB (IHRC 8d SS @ 6”/12” was not a normal distribution so 5% exclusion value is not provided)

Table 6-1. Comparison of design wind speeds calculated per ASCE 7-05 Method 2 for UF-WRSUT Method A results based on A) a factor of safety of 2.0 applied to the mean and B) 5% exclusion of the data (enclosed gable roof building in exposure B assumed, with a mean roof height of 15 ft)

Series	Mean Failure Pressure (psf)	Field Panels (mph)			Overhang Panels (mph)	
		Zone 1	Zone 2	Zone 3	Zone 2	Zone 3
6d SS @ 6"/12"	74.3	130	104	88	105	81
8d SS 6"/12"	129.4	171	137	116	139	107
8d SS 6"/8"	175.1	199	160	135	162	125
8d SS 6"/6"	205.3	>200	173	146	175	135
8d RS @ 6"/12"	174.4	191	153	130	155	120
8d RS @ 6"/12"	161.0	199	160	135	161	124
8d RS @ 6"/8"	216.1	>200	178	150	180	138
8d RS @ 6"/6"	252.4	>200	192	162	194	150
Fillet of ccSPF (B-1)	202.0	>200	172	145	174	134
Full 3" of ccSPF (B-2)	209.0	>200	175	148	177	136

A Factor of safety = 2.0

Series	5% Exclusion Failure Pressure (psf)	Field Panels (mph)			Overhang Panels (mph)	
		Zone 1	Zone 2	Zone 3	Zone 2	Zone 3
6d SS @ 6"/12"	74.3	146	117	99	119	92
8d SS @ 6"/12"	129.4	>200	175	148	177	136
8d SS @ 6"/8"	175.1	>200	>200	172	>200	159
8d SS @ 6"/6"	205.3	>200	>200	188	>200	174
8d RS @ 6"/12"	174.4	>200	>200	169	>200	156
8d RS @ 6"/8"	216.1	>200	>200	181	>200	167
8d RS @ 6"/6"	252.4	>200	>200	>200	>200	198

B 5% exclusion

Table 6-2. Comparison of design wind speeds calculated per ASCE 7-05 Method 2 for UF-WRSUT Method B results A) statically tested panels and B) dynamically tested panels (enclosed gable roof building in exposure B assumed, with a mean roof height of 15 ft)

	Mean Failure Pressure (psf)	Field Panel Design Wind Speed (MPH)			Overhang Panel Design Wind Speed (MPH)	
		Zone 1	Zone 2	Zone 3	Zone 2	Zone 3
Laboratory Fabricated Panels						
2" 6d SS @ 6"/12"	62	118	95	80	96	74
2" SS @ 6"/6"	108	156	126	106	127	98
2" SS @ 6"/12" Ret. A-2	183	204	163	138	165	127
Harvested Panels						
1.5" S @ 4"/4"	93	145	116	98	118	91
1.5" S @ 4"/4" Ret. A-2	168	195	157	132	158	122

A Static

	Mean Failure Pressure (psf)	Field Panel Design Wind Speed (MPH)			Overhang Panel Design Wind Speed (MPH)	
		Zone 1	Zone 2	Zone 3	Zone 2	Zone 3
Laboratory Fabricated Panels						
2" 6d SS @ 6"/12"	52	108	87	74	88	68
2" SS @ 6"/6"	90	143	115	97	116	89
2" SS @ 6"/12" Ret. A-2	167	194	156	132	158	122
Harvested Panels						
1.5" S @ 4"/4"	72	128	102	87	104	80
1.5" S @ 4"/4" Ret. A-2	126	169	136	115	137	106

B Dynamic

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The purpose of this research was to establish a relationship between laboratory test performance of wood roof sheathing and the failure mechanisms observed in the field after hurricanes. The reviewed literature identified no recognized test method for evaluating the wind uplift performance of these systems, and so a test method was proposed that standardizes current approaches used for wind uplift testing of residential wood roof sheathing. A dynamic pressure test protocol was also developed using pressure time histories collected in wind tunnel experiments. These two test methods (a Static and a Dynamic one) were used to compare the ultimate wind uplift resistance of new and existing wood roof sheathing panels. The design wind speeds associated with the failure pressures were determined in accordance with ASCE 7 minimum load provisions, and the following conclusions were made:

Comparison of Results with Previous Studies

Panels tested with standard wind uplift test protocol developed (UF-WRSUT) have failure pressures in the same order of magnitude as previously reported test results. For example 8d smooth shank nails at 6 in. / 12 in. yielded a mean failure pressure of 129 psf which is similar to a study by IHRC (IHRC 2004) which had a mean failure pressure of 110 psf. Additionally coefficients of variation from panel uplift results from UF-WRSUT (7% to 22%) are in similar range as observed in previous studies (7% to 28%).

Effect of In-Service and Environmental Effects on Roof Panel Strength

Test results have not shown significant loss in wind uplift strength of roof panels due to environmental conditions. Comparing panels fastened with 6d smooth shank nails at 6 in. / 12 in. spacing, harvested panels have a higher mean uplift capacity (80 psf) than laboratory

fabricated panels (74 psf). In the case of panels fastened with 8d smooth shank nails at 6 in. / 12 in. spacing harvested panels have a lower mean uplift capacity (107 psf) than laboratory fabricated panels (129 psf), however only two harvested panels were tested. A larger data set of existing panels would need to be tested to establish a statistical basis for this, however experience with wood materials suggests that once its moisture content is maintained below about 19% (as was the case in all the houses tested) the strength of the wood should not deteriorate. Further low moisture content in the wood also prevents the growth of fungi that permanently degrade wood strength.

Dynamic Load Effects on Wood Panel Strength

Dynamic loading is shown to reduce failure pressure results from 9% to 23%, which suggests a potential reduction due to dynamic loading. The reduction from static to dynamic testing is observed in both new panel construction and panels harvested from existing structures. An analysis of variance finds that all static vs. dynamic results come from the same sample population and a limited sample size is tested so no conclusive findings can be made. Further testing with larger numbers of samples are needed to extrapolate results to current wind uplift design of wood roof sheathing panels.

Recommendations

Previous residential wood roof sheathing uplift testing have used various methods which have contributed to a varying body results. The use of a standard test method is suggested as a for uplift testing of residential wood roof sheathing in order to reduce the error in comparisons due the test method.

It is recommended that further testing of both dynamic and age or weathering effects be continued as sample sizes were relatively small due to logistical and time constraints. It is recommended that an investigation be conducted to determine if cumulative damage occurs in

the connection between individual dowel type fasteners and the wood it is embedded in. As wood is found in to be an elastic material at low levels of energy cumulative damage effects may only occur above some threshold. Quantifying cumulative damage would require lab studies of nail samples subjected to cyclical loading of varying load levels and amplitude.

It is recommended that the distribution of pressure loads to individual fasteners in wood roof sheathing panels be investigated to relate the previous recommendation to panel system behavior.

APPENDIX A
FULL PANEL UPLIFT RESULTS

Table A-1. Summary of panel uplift test series

Panel Series	Fastener Schedule	Dates of Testing	Retrofit Type	Description
			1.5 in. Staple	
Port Orange #1	3 in. / 6 in.	-	A-1	Six harvested panels tested statically in EC (4) and RC (2)
Debary #1	4 in. / 4 in.	15 yrs	A-1	Ten harvested panels tested statically in EC (2) and RC (2) vs. dynamically in EC (4) and RC (2)
			2.5 in. Staple	
Bartow #1	3 in. / 6 in.	-	-	Two harvested panels tested statically in EC
Bartow #2	6 in. / 12 in.	-	A-1	Four harvested panels tested statically in EC (2) and RC (2)
			6d (0.113 in. diameter) Smooth Shank Nails, 2 in. and 2.375 in. long	
Laboratory #1-A	6 in. / 12 in.	New	-	Five laboratory fabricated panel tested statically
Laboratory #1-B	6 in. / 12 in.	New	-	Ten lab-built panels tested statically to have similar sample size as other lab-built panels
Port Orange #2	6 in. / 12 in.	28 yrs	B-2	Four harvested panels tested statically in EC (3) and RC (1)
Bartow #3	6 in. / 12 in.	32 yrs	A-1	Four harvested panels tested statically in EC (2) and RC (2)
Bartow #4	3 in. / 6 in.	32 yrs	A-1	Three harvested panels tested statically in EC (1) and RC (2)
Bartow #5	4 in. / 4 in.	32 yrs	A-1	Two harvested panels tested statically in EC (1) and RC (1)
Port Orange #3	6 in. / 12 in.	-	A-1	Three harvested panels tested statically in EC (2) and RC (1)
Port Orange #4	3 in. / 6 in.	-	-	Two harvested panels tested statically in EC (2)
Port Orange #5	8 in. / 8 in.	-	-	Two harvested panels tested statically in EC (2)
Port Orange #6	6 in. / 12 in.	-	-	Two harvested panels tested statically in EC (2)
Laboratory #2-A	6 in. / 12 in.	New	-	Ten laboratory fabricated panels tested statically (5) and dynamically (5) in EC
Laboratory #2-B	6 in. / 12 in.	New	A-2	Four laboratory fabricated panels tested statically (2) and dynamically (2) in RC
Laboratory #2-C	6 in. / 6 in.	New	-	Ten laboratory fabricated panels tested statically (5) and dynamically (5) in EC

Notes: EC = existing condition, RC = retrofit condition; Retrofit Methods A-1. 8d ring shank nail @ 6 in. / 12 in. spacing, A-2. 8d ring shank nails installed between an existing fastener spacing of 6 in. / 12 in., B-1. fillet of spray applied polyurethane adhesive, B-2. 3 in. thick layer of spray applied polyurethane adhesive

Table A-1. Continued

Panel Series	Fastener Schedule	Dates of Testing	Retrofit Type	Description
8d (0.131 in. diameter) Smooth Shank Nails, 2.5 in. long				
Laboratory #3-A	6 in. / 12 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC
Laboratory #3-B	6 in. / 8 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC
Laboratory #3-C	6 in. / 6 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC
Crystal River #1	6 in. / 12 in.	35 yrs	A-1	Four harvested panel tested statically in EC (2) and RC (2)
8d (0.113 in. diameter) Ring Shank Nails, 2.5 in. long				
Laboratory #4-A	6 in. / 12 in.	New	-	Ten laboratory fabricated panels tested statically in EC, for comparison with B and C
Laboratory #4-B	6 in. / 12 in.	New	B-1	Ten laboratory fabricated panels tested statically in RC
Laboratory #4-C	6 in. / 12 in.	New	B-2	Ten laboratory fabricated panels tested statically in RC
Laboratory #5-A	6 in. / 12 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC
Laboratory #5-B	6 in. / 8 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC
Laboratory #5-C	6 in. / 6 in.	New	-	Fifteen laboratory fabricated panels tested statically in EC

Notes: EC = existing condition, RC = retrofit condition; Retrofit Methods A-1. 8d ring shank nail @ 6 in. / 12 in. spacing, A-2. 8d ring shank nails installed between an existing fastener spacing of 6 in. / 12 in., B-1. fillet of spray applied polyurethane adhesive, B-2. 3 in. thick layer of spray applied polyurethane adhesive

Table A-2. Full uplift test results of panels fastened with 1.5 in. staples

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Port Orange #1	1.5 in. Staple-1	-	-	-	1.5 in.	3 in. / 6 in.	-	-	1/2 in. Ply	X	-	-	137	-
	1.5 in. Staple-2	-	-	-			-	-		X	-	-	132	-
	1.5 in. Staple-3	-	-	-			-	-		X	-	-	117	-
	1.5 in. Staple-4	-	-	-			-	-		X	-	-	93	-
	1.5 in. Staple-5	-	-	-			8d RS	6 in. / 12 in.		X	-	-	221	-
	1.5 in. Staple-6	-	-	-						X	-	-	319	-
Debary #1	1.5 in. Staple-7	15 yrs	0.56	8%	1.5 in. (14 gauge)	4 in. / 4 in.	-	-	7/16 in. OSB	X	-	W	93	76
	1.5 in. Staple-8		0.64	9%			-	-		X	-	W	93	76
	1.5 in. Staple-9	15 yrs	0.56	10%		4 in. / 4 in.	-	-		-	X	W	93	57
	1.5 in. Staple-10		0.55	9%			-	-		-	X	PT	61	33
	1.5 in. Staple-11		0.54	9%			-	-		-	X	W	52	25
	1.5 in. Staple-12		0.58	9%			-	-		-	X	W	82	42
	1.5 in. Staple-13	15 yrs	0.61	10%		4 in. / 4 in.	8d RS	6 in. / 12 in.		X	-	PT	168	152
	1.5 in. Staple-14		0.56	9%						X	-	W	169	152
	1.5 in. Staple-15	15 yrs	0.62	9%		4 in. / 4 in.	8d RS	6 in. / 12 in.		-	X	PT	150	90
	1.5 in. Staple-16		0.52	9%						-	X	PT	102	57

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even and RS = ring shank nail

Table A-3. Full uplift test results of panels fastened with 2.5 in. staples

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Bartow # 1	2.5 in. Staple-1	-	-	-	2.5 in.	3 in. / 6 in.	-	-	1/2 in. Ply	X	-	-	66	-
	2.5 in. Staple-2	-	-	-			-	-		X	-	-	97	-
Bartow # 2	2.5 in. Staple-3	-	-	-	2.5 in.	6 in. / 12 in.	-	-	1/2 in. Ply	X	-	-	30	-
	2.5 in. Staple-4	-	-	-			-	-		X	-	-	39	-
	2.5 in. Staple-5	-	-	-			8d RS	6 in. / 12 in.		X	-	-	165	-
	2.5 in. Staple-6	-	-	-						X	-	-	169	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even and RS = ring shank nail

Table A-4. Full uplift test results of panels fastened with 6d smooth shank nails

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #1-A	6d SS-1	New	-	-	2.375 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	X	-	-	75	-
	6d SS-2		-	-			-	-		X	-	-	105	-
	6d SS-3		-	-			-	-		X	-	-	71	-
	6d SS-4		-	-			-	-		X	-	-	76	-
	6d SS-5		-	-			-	-		X	-	-	47	-
Laboratory #1-B	6d SS-6	New	-	-			-	-		X	-	W	62	-
	6d SS-7		-	-			-	-		X	-	W	59	-
	6d SS-8		-	-			-	-		X	-	W	96	-
	6d SS-9		-	-			-	-		X	-	W	69	-
	6d SS-10		-	-			-	-		X	-	W	79	-
	6d SS-11		-	-			-	-		X	-	W	87	-
	6d SS-12		-	-			-	-		X	-	W	95	-
	6d SS-13		-	-			-	-		X	-	W	57	-
	6d SS -14		-	-			-	-		X	-	W	59	-
	6d SS-15		-	-			-	-		X	-	W	76	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even ; RS = ring shank nail and SS = smooth shank nail

Table A-4. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Port Orange #2	6d SS-16	28 yrs	-	-	2 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. Ply	X	-	-	134	-
	6d SS-17		-	-			-	-		X	-	-	98	-
	6d SS-18		-	-			-	-		X	-	-	74	-
	6d SS-19		-	-			ccSPF	Full (3 in.)		X	-	-	250	-
Bartow #3	6d SS-20	32 yrs	-	-	2 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. Ply	X	-	-	53	-
	6d SS-21		-	-			-	-		X	-	-	132	-
	6d SS-22		-	-			8d RS	6 in. / 12 in.		X	-	-	229	-
	6d SS-23		-	-						X	-	-	206	-
Bartow #4	6d SS-24	32 yrs	-	-	2 in. (0.113 in.)	3 in. / 6 in.	-	-	1/2 in. Ply	X	-	-	117	-
	6d SS-25		-	-			8d RS	6 in. / 12 in.		X	-	-	136	-
	6d SS-26		-	-						X	-	-	202	-
Bartow #5	6d SS-27	32 yrs	-	-	2 in. (0.113 in.)	4 in. / 4 in.	-	-	1/2 in. Ply	X	-	-	149	-
	6d SS-28		-	-			8d RS	6 in. / 12 in.		X	-	-	197	-
Port Orange #3	6d SS-29	-	-	-	2 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. Ply	X	-	-	46	-
	6d SS-30	-	-	-			-	-		X	-	-	31	-
	6d SS-31	-	-	-			8d RS	6 in. / 12 in.		X	-	-	140	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-4. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener Size	Existing Fastener Spacing	Retrofit Fastener Type	Retrofit Fastener Spacing	Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
Port Orange #4	6d SS-32	-	-	-	2 in. (0.113 in.)	3 in. / 6 in.	-	-	1/2 in. Ply	X	-	-	62	-
	6d SS-33	-	-	-			-	-		X	-	-	64	-
Port Orange #5	6d SS-34	-	-	-		8 in. / 8 in.	-	-		X	-	-	83	-
	6d SS-35	-	-	-			-	-		X	-	-	62	-
Port Orange #6	6d SS-36	-	-	-		6 in. / 12 in.	-	-		X	-	-	59	-
	6d SS-37	-	-	-			-	-		X	-	-	90	-
Laboratory #2-A	6d SS-38	New	0.48	-	2 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	X	-	W	47	32
	6d SS-39		0.49	-			-	-		X	-	W	73	61
	6d SS-40		0.44	-			-	-		X	-	W	58	46
	6d SS-41		0.44	-			-	-		X	-	W	65	60
	6d SS-42		0.51	-			-	-		X	-	W	65	46
	6d SS -43	New	0.50	-	2 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	-	X	W	35	23
	6d SS-44		0.56	-			-	-		-	X	W	51	32
	6d SS-45		0.52	-			-	-		-	X	W	51	29
	6d SS-46		0.51	-			-	-		-	X	W	58	32
	6d SS-47		0.54	-			-	-		-	X	W	66	40

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-4. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener Size	Existing Fastener Spacing	Retrofit Fastener Type	Retrofit Fastener Spacing	Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
Laboratory #2-B	6d SS-48	New	0.49	-	2 in. (0.113 in.)	6 in. / 12 in.	8d RS	6 in. / 12 in.	1/2 in. OSB	X	-	W	199	186
	6d SS-49		0.44	-						X	-	W	167	150
	6d SS-50		0.55	-			8d RS	6 in. / 12 in.		-	X	W	164	99
	6d SS-51		0.54	-						-	X	W	171	100
Laboratory #2-C	6d SS-52	New	0.50	-	2 in. (0.113 in.)	6 in. / 6 in.	-	-	1/2 in. OSB	X	-	PT	64	47
	6d SS-53		0.58	-			-	-		X	-	W	106	90
	6d SS-54		0.47	-			-	-		X	-	W	135	120
	6d SS-55		0.57	-			-	-		X	-	W	91	77
	6d SS-56		0.47	-			-	-		X	-	W	134	120
	6d SS-57		0.56	-			-	-		-	X	W	44	27
	6d SS-58		0.58	-			-	-		-	X	W	103	62
	6d SS-59		0.56	-			-	-		-	X	W	105	62
	6d SS-60		0.54	-			-	-		-	X	W	72	43
	6d SS-61		0.48	-			-	-		-	X	W	124	70

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-5. Full uplift test results of panels fastened with 8d smooth shank nails

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #3-A	8d SS-1	New	-	-	2.5 in. (0.131 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	X	-	W	127	-
	8d SS-2		-	-			-	-		X	-	W	162	-
	8d SS-3		-	-			-	-		X	-	E	131	-
	8d SS-4		-	-			-	-		X	-	PT	126	-
	8d SS-5		-	-			-	-		X	-	PT	123	-
	8d SS-6		-	-			-	-		X	-	E	135	-
	8d SS-7		-	-			-	-		X	-	W	124	-
	8d SS-8		-	-			-	-		X	-	W	149	-
	8d SS-9		-	-			-	-		X	-	E	132	-
	8d SS-10		-	-			-	-		X	-	E	109	-
	8d SS-11		-	-			-	-		X	-	W	137	-
	8d SS-12		-	-			-	-		X	-	W	100	-
	8d SS-13		-	-			-	-		X	-	W	131	-
	8d SS-14		-	-			-	-		X	-	E	135	-
	8d SS-15		-	-			-	-		X	-	W	118	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nails and SS = smooth shank nail

Table A-5. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #3-B	8d SS-16	New	-	-	2.5 in. (0.131 in.)	6 in. / 8 in.	-	-	1/2 in. OSB	X	-	PT	151	-
	8d SS-17		-	-			-	-		X	-	PT	180	-
	8d SS-18		-	-			-	-		X	-	E	178	-
	8d SS-19		-	-			-	-		X	-	PT	183	-
	8d SS-20		-	-			-	-		X	-	W	167	-
	8d SS-21		-	-			-	-		X	-	W	160	-
	8d SS-22		-	-			-	-		X	-	W	180	-
	8d SS-23		-	-			-	-		X	-	PT	192	-
	8d SS-24		-	-			-	-		X	-	W	145	-
	8d SS-25		-	-			-	-		X	-	W	221	-
	8d SS-26		-	-			-	-		X	-	W	172	-
	8d SS-27		-	-			-	-		X	-	PT	151	-
	8d SS-28		-	-			-	-		X	-	W	168	-
	8d SS-29		-	-			-	-		X	-	W	178	-
	8d SS-30		-	-			-	-		X	-	PT	202	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-5. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Spacing		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Type	Type	Spacing						
Laboratory #3-C	8d SS-31	New	-	-	2.5 in. (0.131 in.)	6 in. / 6 in.	-	-	1/2 in. OSB	X	-	W	189	-
	8d SS-32		-	-			-	-		X	-	W	163	-
	8d SS-33		-	-			-	-		X	-	W	239	-
	8d SS-34		-	-			-	-		X	-	W	176	-
	8d SS-35		-	-			-	-		X	-	PT	233	-
	8d SS-36		-	-			-	-		X	-	W	196	-
	8d SS-37		-	-			-	-		X	-	W	222	-
	8d SS-38		-	-			-	-		X	-	W	194	-
	8d SS-39		-	-			-	-		X	-	W	195	-
	8d SS-40		-	-			-	-		X	-	W	209	-
	8d SS-41		-	-			-	-		X	-	W	226	-
	8d SS-42		-	-			-	-		X	-	W	224	-
	8d SS-43		-	-			-	-		X	-	W	207	-
	8d SS-44		-	-			-	-		X	-	PT	214	-
	8d SS-45		-	-			-	-		X	-	PT	193	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-5. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Crystal River #1	8d SS-46	35 yrs	-	-	2.5 in. (0.131 in.)	6 in. / 12 in.	-	-	1/2 in. Ply	X	-	-	112	-
	8d SS-47		-	-			-	-		X	-	-	101	-
	8d SS-48		-	-			8d RS	6 in. / 12 in.		X	-	-	182	-
	8d SS-49		-	-						X	-	-	217	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through and E = even ; RS = ring shank

Table A-6. Full uplift test results of panels fastened with 8d ring shank nails

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #4-A	8d RS-1	New	-	-	2.5 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	X	-	W	115	-
	8d RS-2		-	-			-	-		X	-	W	177	-
	8d RS-3		-	-			-	-		X	-	E	144	-
	8d RS-4		-	-			-	-		X	-	PT	164	-
	8d RS -5		-	-			-	-		X	-	W	123	-
	8d RS-6		-	-			-	-		X	-	W	168	-
	8d RS -7		-	-			-	-		X	-	PT	153	-
	8d RS-8		-	-			-	-		X	-	PT	210	-
	8d RS-9		-	-			-	-		X	-	PT	174	-
	8d RS-10		-	-			-	-		X	-	PT	182	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank

Table A-6. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #4-B	8d RS-11	New	-	-	2.5 in. (0.113 in.)	6 in. / 12 in.	ccSPF	Fillet	1/2 in. OSB	X	-	W	204	-
	8d RS-12		-	-						X	-	PT	174	-
	8d RS-13		-	-						X	-	W	214	-
	8d RS-14		-	-						X	-	PT	192	-
	8d RS15		-	-						X	-	W	258	-
	8d RS-16		-	-						X	-	E	225	-
	8d RS-17		-	-						X	-	PT	211	-
	8d RS-18		-	-						X	-	W	166	-
	8d RS-19		-	-						X	-	PT	172	-
	8d RS-20		-	-						X	-	E	204	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-6. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #4-C	8d RS-21	New	-	-	2.5 in. (0.113 in.)	6 in. / 12 in.	ccSPF	Full (3 in.)	1/2 in. OSB	X	-	E	210	-
	8d RS-22		-	-						X	-	PT	241	-
	8d RS-23		-	-						X	-	PT	273	-
	8d RS-24		-	-						X	-	W	202	-
	8d RS-25		-	-						X	-	W	151	-
	8d RS-26		-	-						X	-	E	175	-
	8d RS-27		-	-						X	-	PT	196	-
	8d RS-28		-	-						X	-	E	165	-
	8d RS-29		-	-						X	-	PT	245	-
	8d RS-30		-	-						X	-	W	231	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-6. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener Size	Existing Fastener Spacing	Retrofit Fastener Type	Retrofit Fastener Spacing	Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
Laboratory #5-A	8d RS-59	New	-	-	2.5 in. (0.113 in.)	6 in. / 12 in.	-	-	1/2 in. OSB	X	-	W	186	-
	8d RS-60		-	-			-	-		X	-	W	151	-
	8d RS-61		-	-			-	-		X	-	W	115	-
	8d RS-62		-	-			-	-		X	-	PT	151	-
	8d RS-63		-	-			-	-		X	-	PT	166	-
	8d RS-64		-	-			-	-		X	-	PT	181	-
	8d RS-65		-	-			-	-		X	-	PT	188	-
	8d RS-66		-	-			-	-		X	-	PT	195	-
	8d RS-67		-	-			-	-		X	-	W	199	-
	8d RS-68		-	-			-	-		X	-	W	175	-
	8d RS-69		-	-			-	-		X	-	W	192	-
	8d RS-70		-	-			-	-		X	-	PT	172	-
	8d RS-71		-	-			-	-		X	-	PT	198	-
	8d RS-72		-	-			-	-		X	-	PT	186	-
	8d RS-73		-	-			-	-		X	-	E	161	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS =smooth shank nail

Table A-6. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #5-B	8d RS-44	New	-	-	2.5 in. (0.113 in.)	6 in. / 8 in.	-	-	1/2 in. OSB	X	-	PT	185	-
	8d RS-45		-	-			-	-		X	-	PT	193	-
	8d RS-46		-	-			-	-		X	-	PT	247	-
	8d RS-47		-	-			-	-		X	-	W	134	-
	8d RS-48		-	-			-	-		X	-	PT	239	-
	8d RS -49		-	-			-	-		X	-	PT	269	-
	8d RS-50		-	-			-	-		X	-	PT	207	-
	8d RS-51		-	-			-	-		X	-	PT	211	-
	8d RS-52		-	-			-	-		X	-	PT	245	-
	8d RS-53		-	-			-	-		X	-	W	224	-
	8d RS-54		-	-			-	-		X	-	PT	163	-
	8d RS-55		-	-			-	-		X	-	PT	237	-
	8d RS-56		-	-			-	-		X	-	W	222	-
	8d RS-57		-	-			-	-		X	-	PT	219	-
	8d RS-58		-	-			-	-		X	-	PT	247	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through, E = even, RS = ring shank nail and SS = smooth shank nail

Table A-6. Continued

	Panel Description	Age	Specific Gravity	Moisture Content	Existing Fastener		Retrofit Fastener		Sheathing	Static Loading	Dynamic Loading	Failure mode	Peak Uplift (psf)	10-sec. Mean Uplift (psf)
					Size	Spacing	Type	Spacing						
Laboratory #5-C	8d RS-31	New	-	-	2.5 in. (0.113 in.)	6 in. / 6 in.	-	-	1/2 in. OSB	X	-	PT	241	-
	8d RS-32		-	-			-	-		X	-	PT	268	-
	8d RS-33		-	-			-	-		X	-	PT	239	-
	8d RS-34		-	-			-	-		X	-	PT	253	-
	8d RS-35		-	-			-	-		X	-	PT	284	-
	8d RS-36		-	-			-	-		X	-	PT	253	-
	8d RS-37		-	-			-	-		X	-	PT	254	-
	8d RS-38		-	-			-	-		X	-	PT	241	-
	8d RS-39		-	-			-	-		X	-	PT	255	-
	8d RS-40		-	-			-	-		X	-	PT	248	-
	8d RS-41		-	-			-	-		X	-	PT	221	-
	8d RS-42		-	-			-	-		X	-	PT	239	-
	8d RS-43		-	-			-	-		X	-	PT	286	-

Notes: Dominant failure mode described by W = withdrawal, PT = pull-through and E = even ; RS = ring shank

APPENDIX B
FULL FAILURE MODE AND LOCATION INFORMATION

Figure B-1 thru B-12 present location and mode of each panel fabricated in the laboratory.

Results are grouped by series.

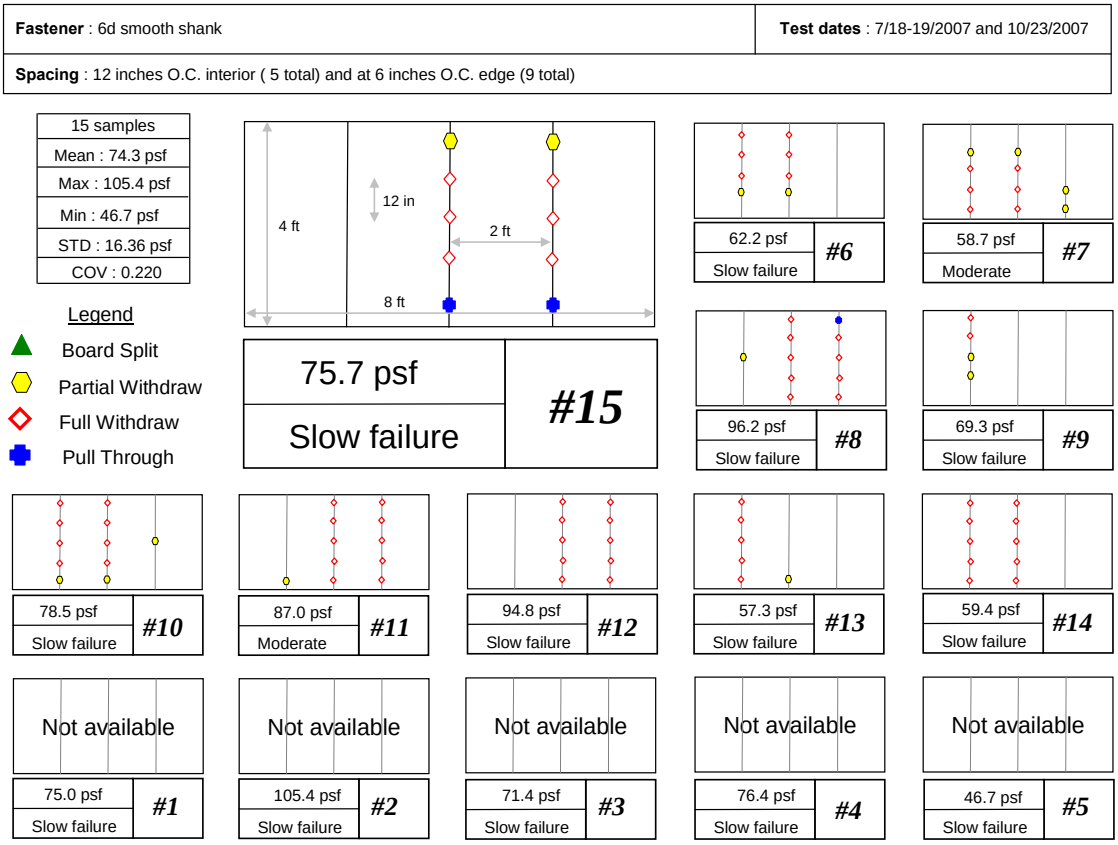


Figure B-1. Failure mode / location for panels fastened with 6d smooth shank nail spaced at 6 in. / 12 in. tested statically

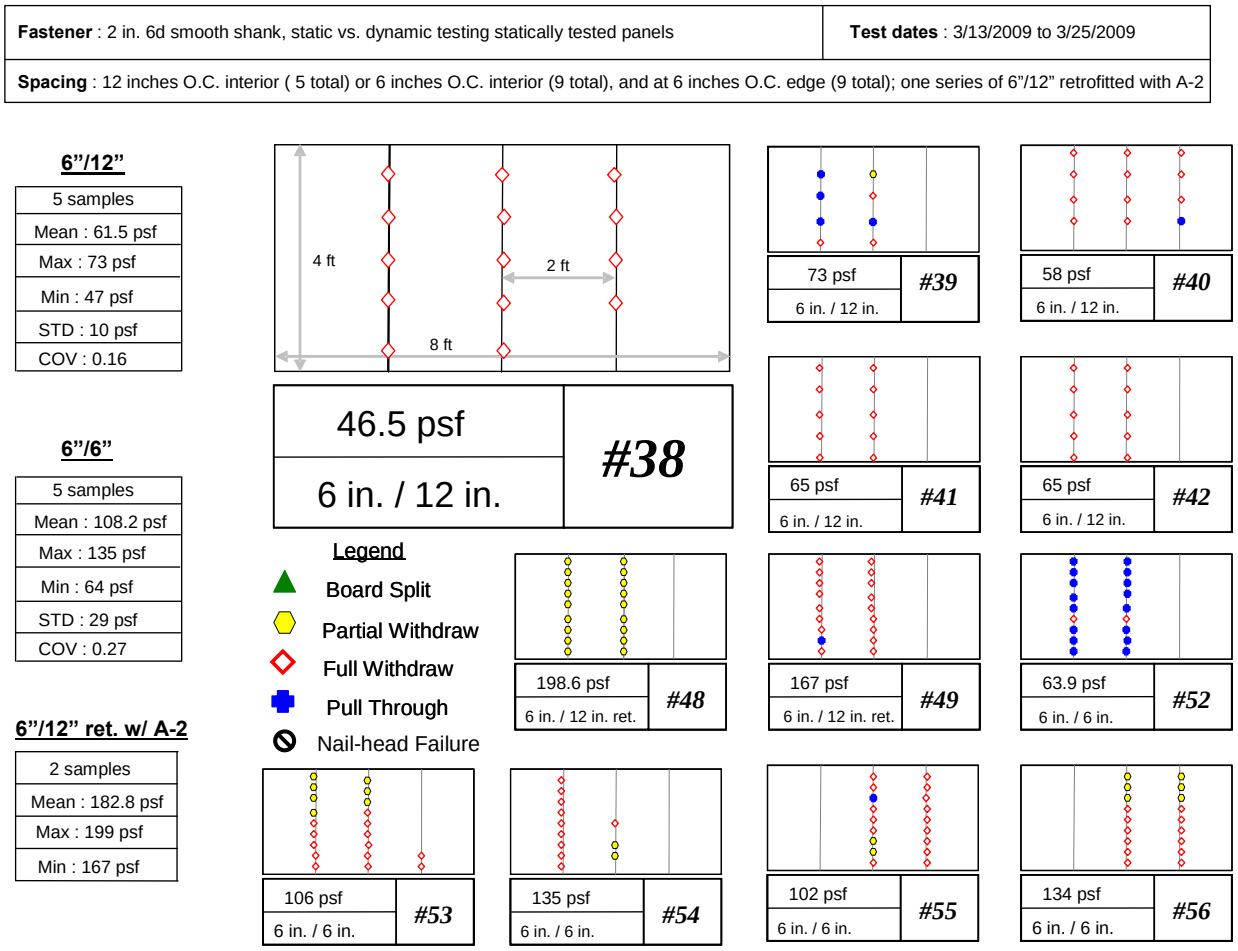


Figure B-2. Failure mode / location for panels fastened with 6d smooth shank nail spaced at 6 in. / 12 in., 6 in. / 12 in. retrofitted with ret. A-2, and 6 in. / 6 in. tested statically

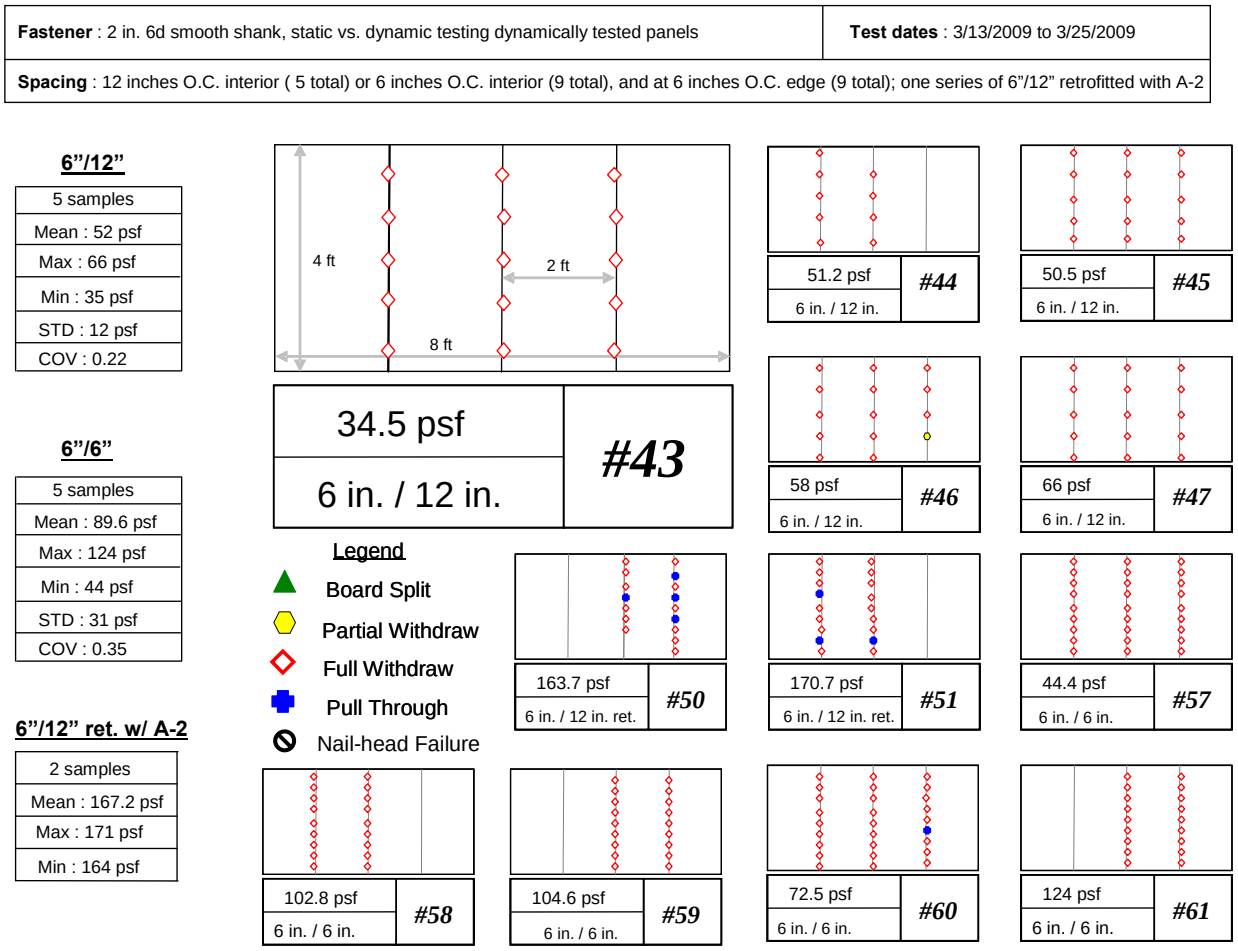


Figure B-3. Failure mode / location for panels fastened with 6d smooth shank nail spaced at 6 in. / 12 in., 6 in. / 12 in. retrofitted with ret. A-2, and 6 in. / 6 in. tested dynamically

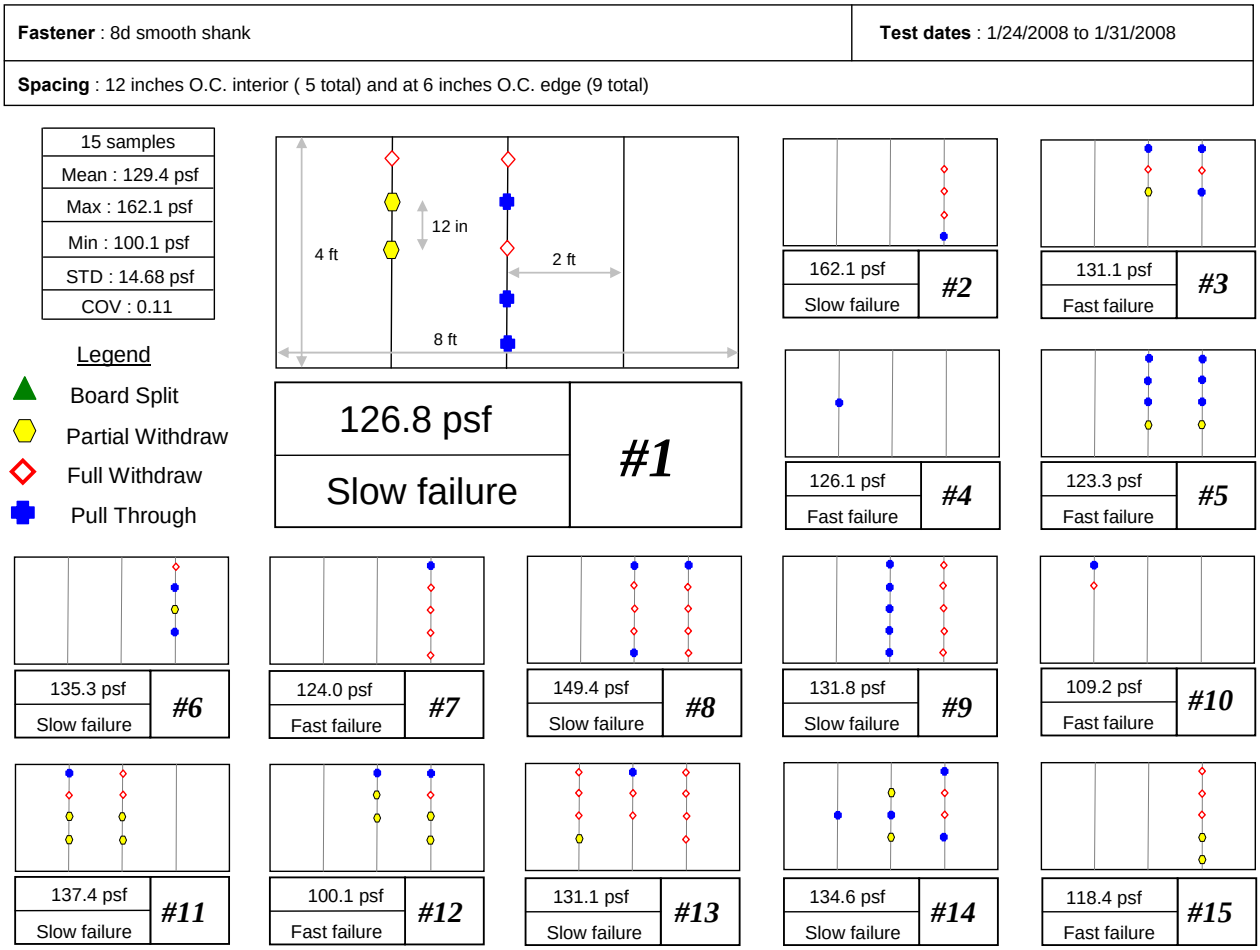


Figure B-4. Failure mode / location for panels fastened with 8d smooth shank nail spaced at 6 in. / 12 in. tested statically

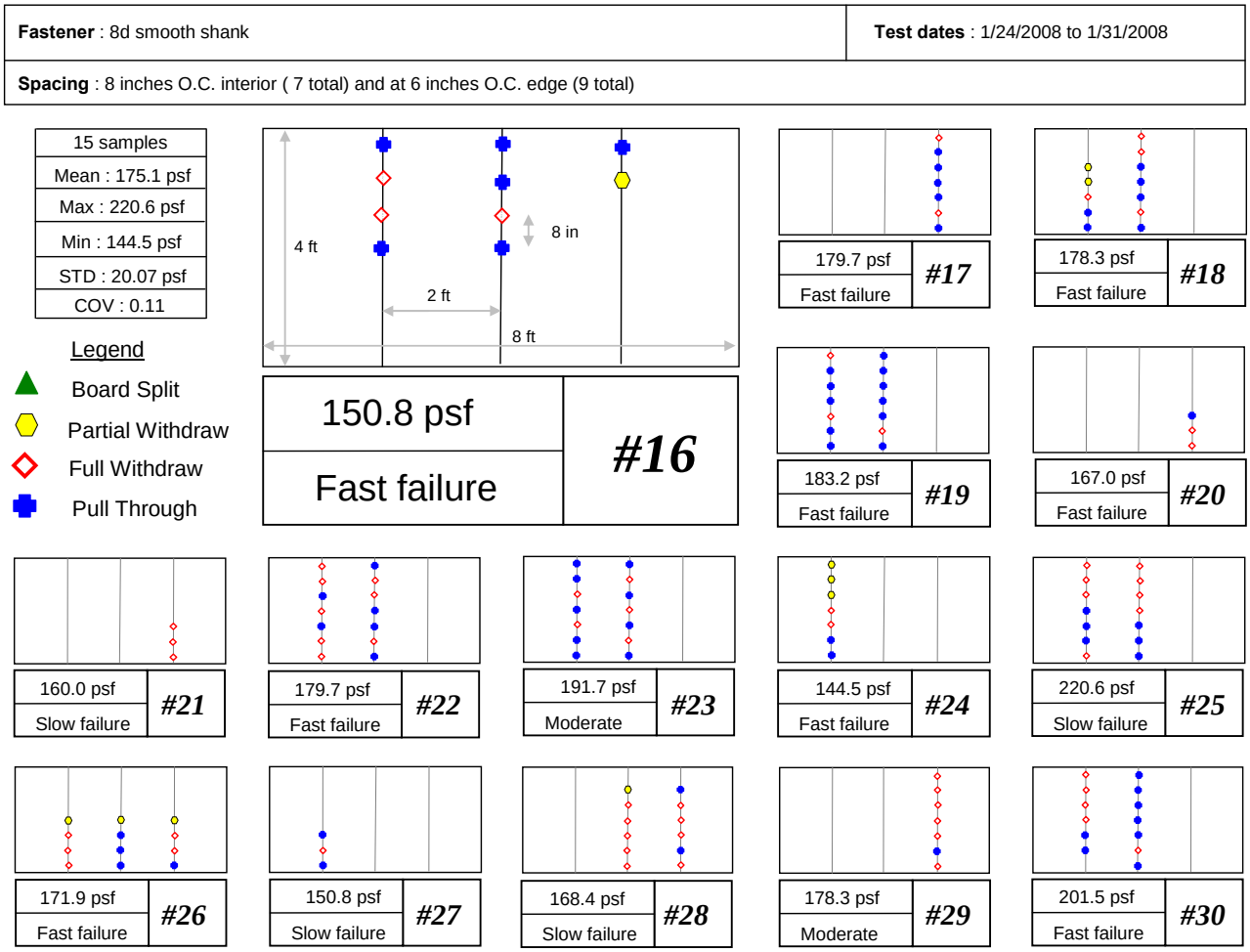


Figure B-5. Failure mode / location for panels fastened with 8d smooth shank nails spaced at 6 in. / 8 in. tested statically

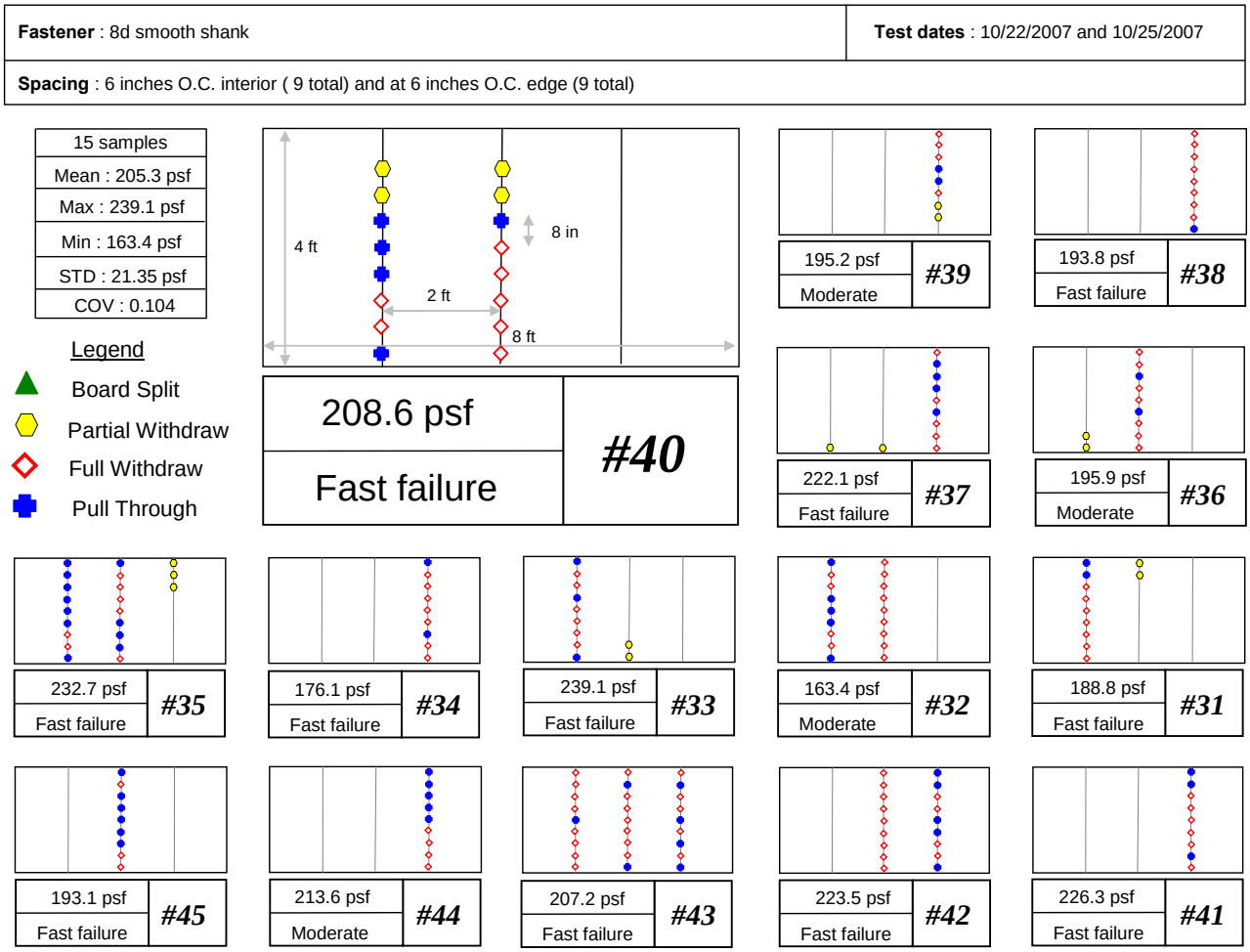


Figure B-6. Failure mode / location for panels fastened with 8d smooth shank spaced at 6 in. / 6 in. tested statically

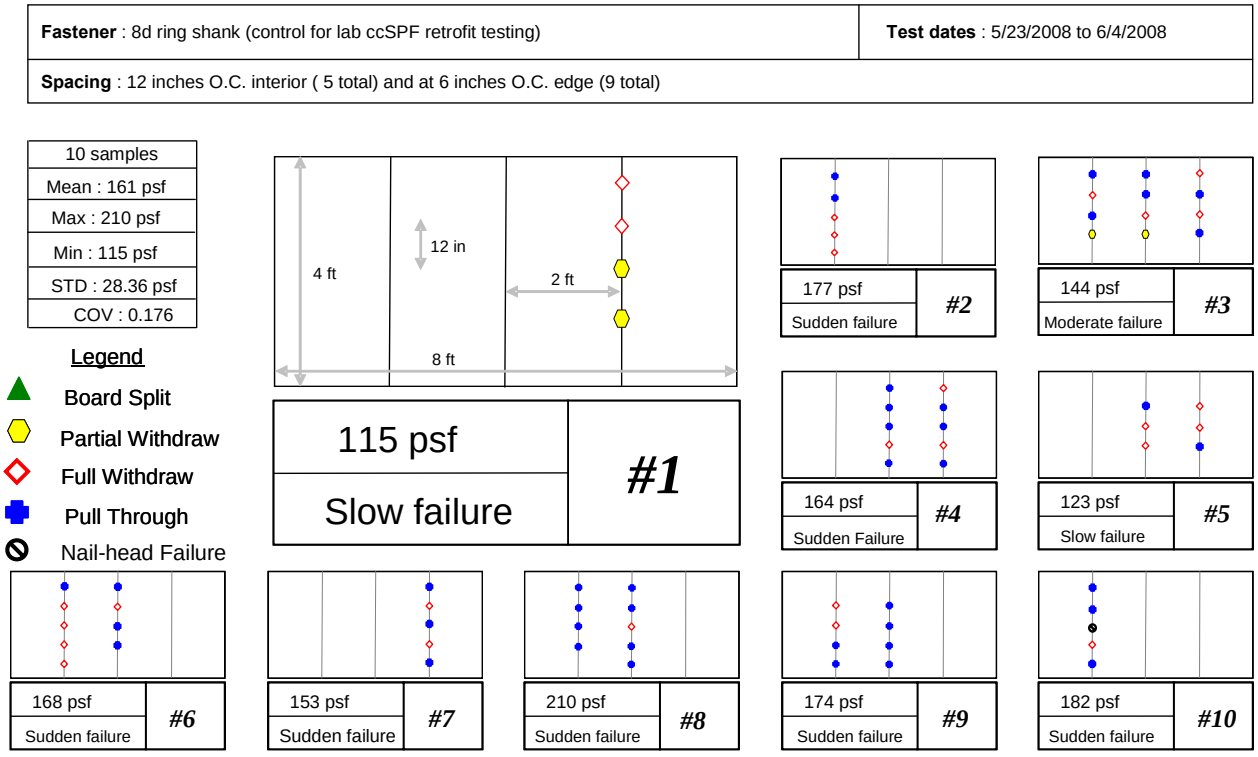


Figure B-7. Failure mode / location for panels fastened with 8d ring shank nails spaced at 6 in. / 12 in. tested statically

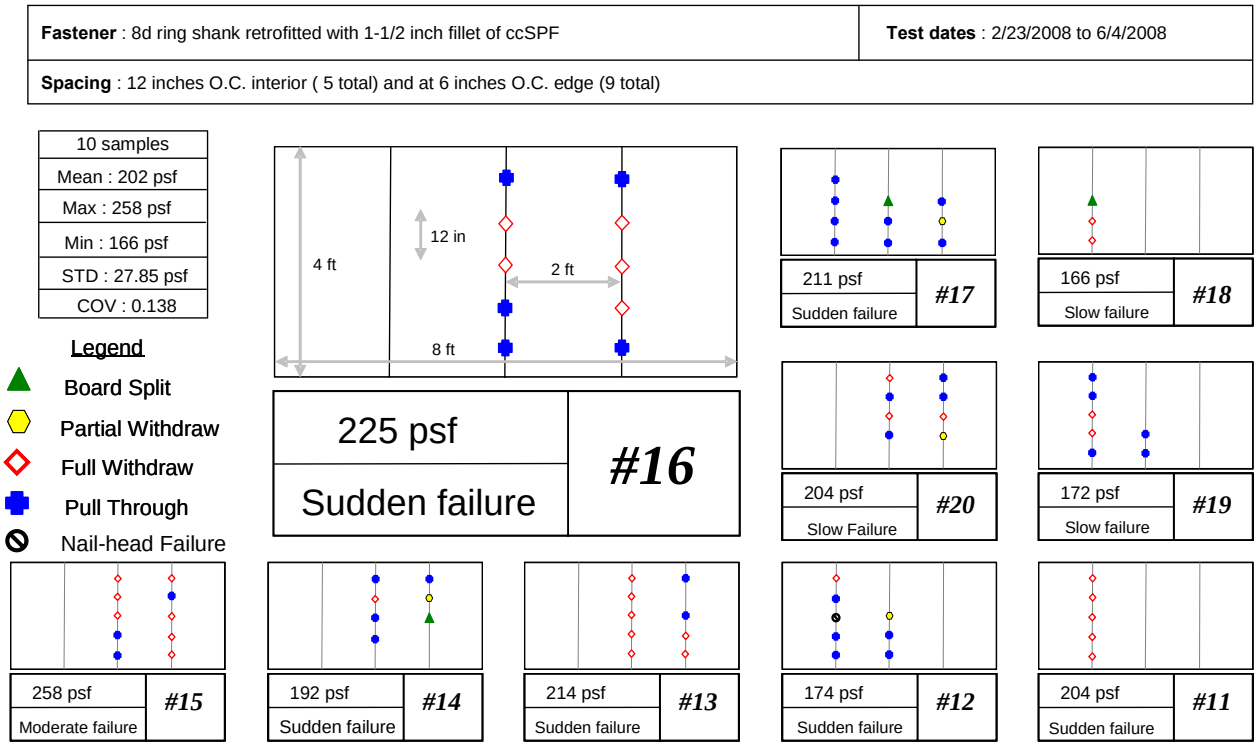


Figure B-8. Failure mode / location for panels fastened with 8d ring shank nails spaced at 6 in. / 12 in. retrofitted with 1-1/2 in. fillet of ccSPF tested statically

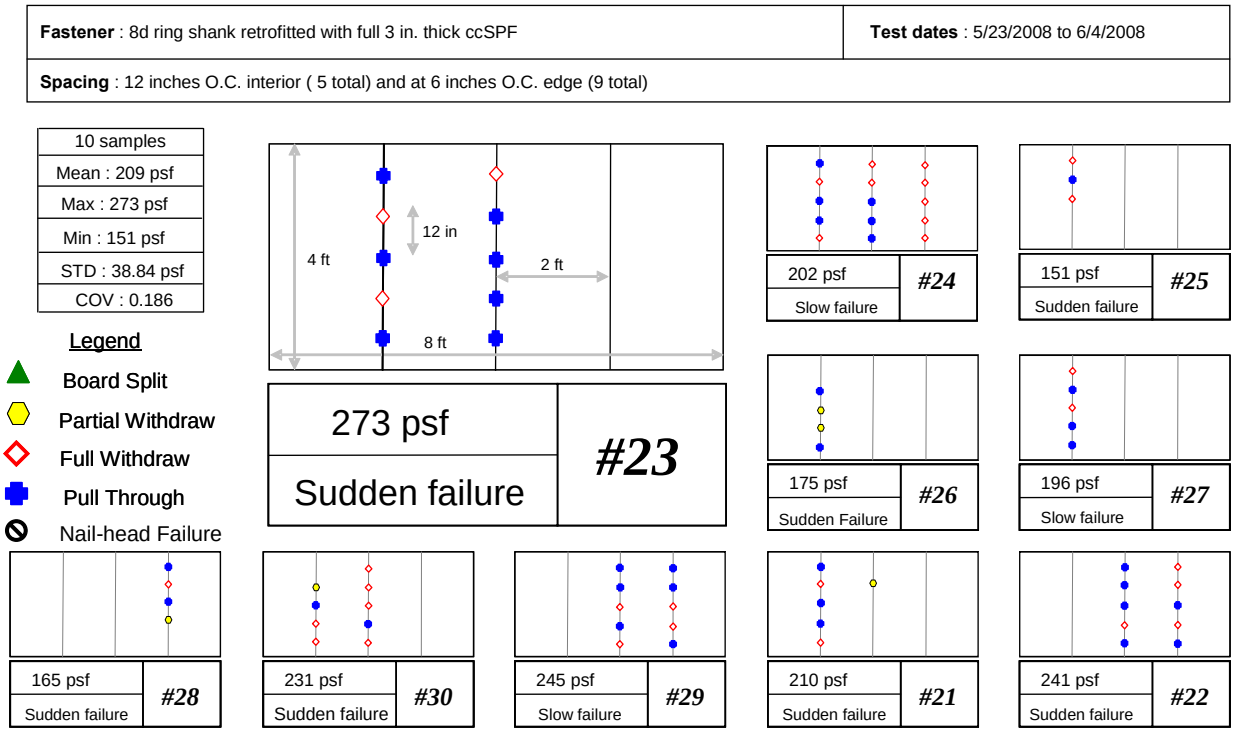


Figure B-9. Failure mode / location for panels fastened with 8d ring shank nails spaced at 6 in. / 12 in. retrofitted with full 3 in. thick layer of ccSPF tested statically

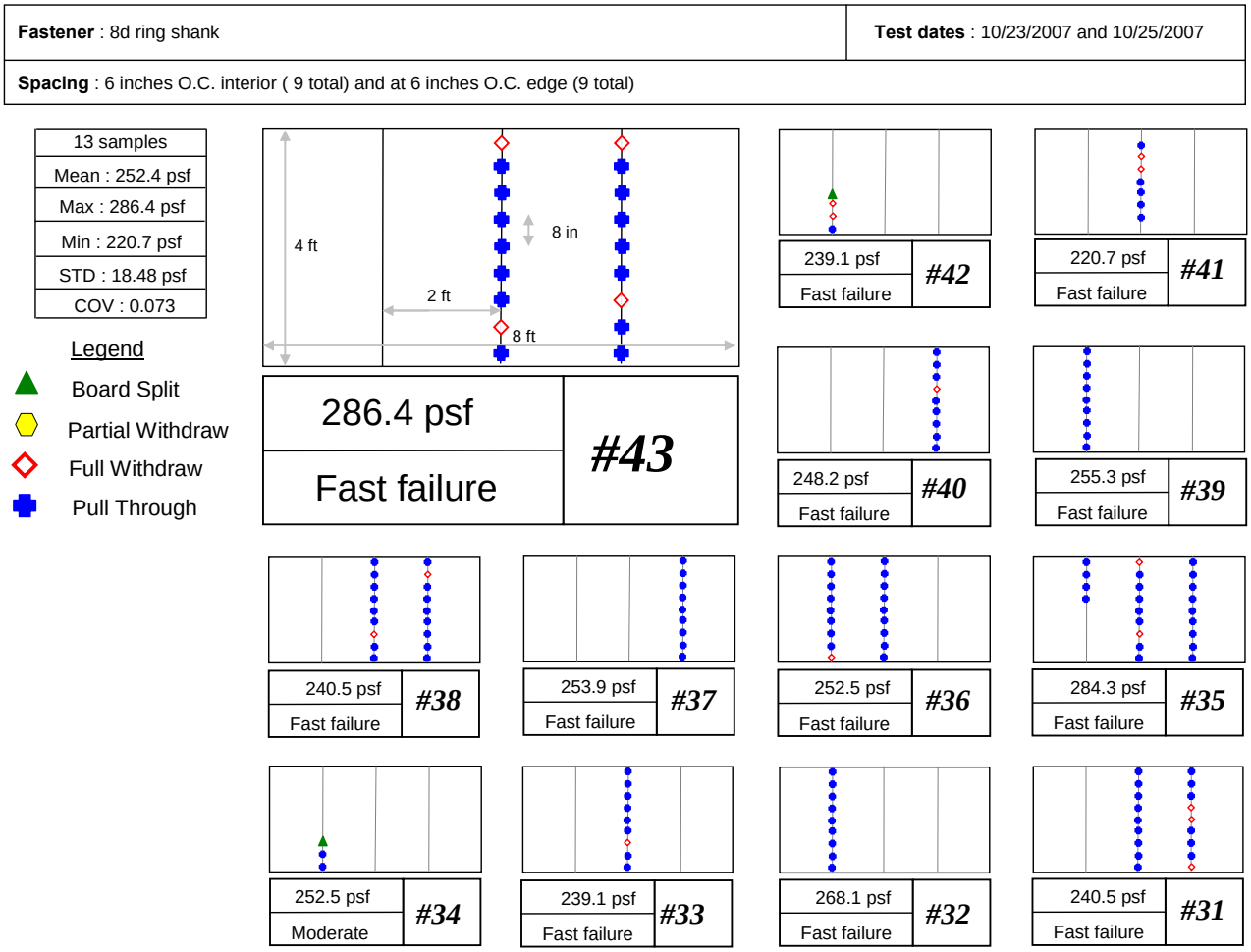


Figure B-10. Failure mode / location of panels fastened with 8d ring shank nails spaced at 6 in. / 6 in. tested statically

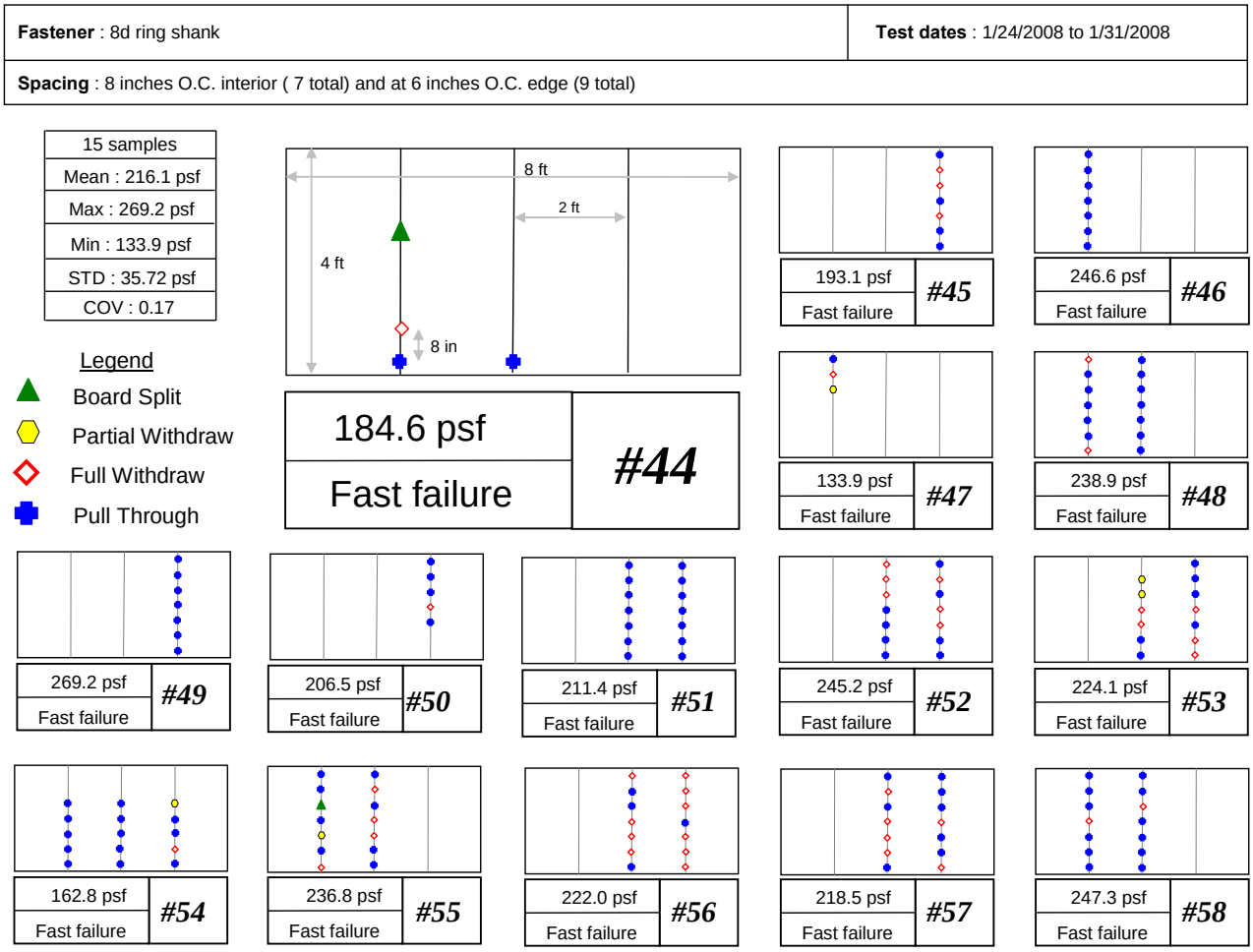


Figure B-11. Failure mode / location for panels fastened with 8d ring shank nails spaced at 6 in. / 8 in. tested statically

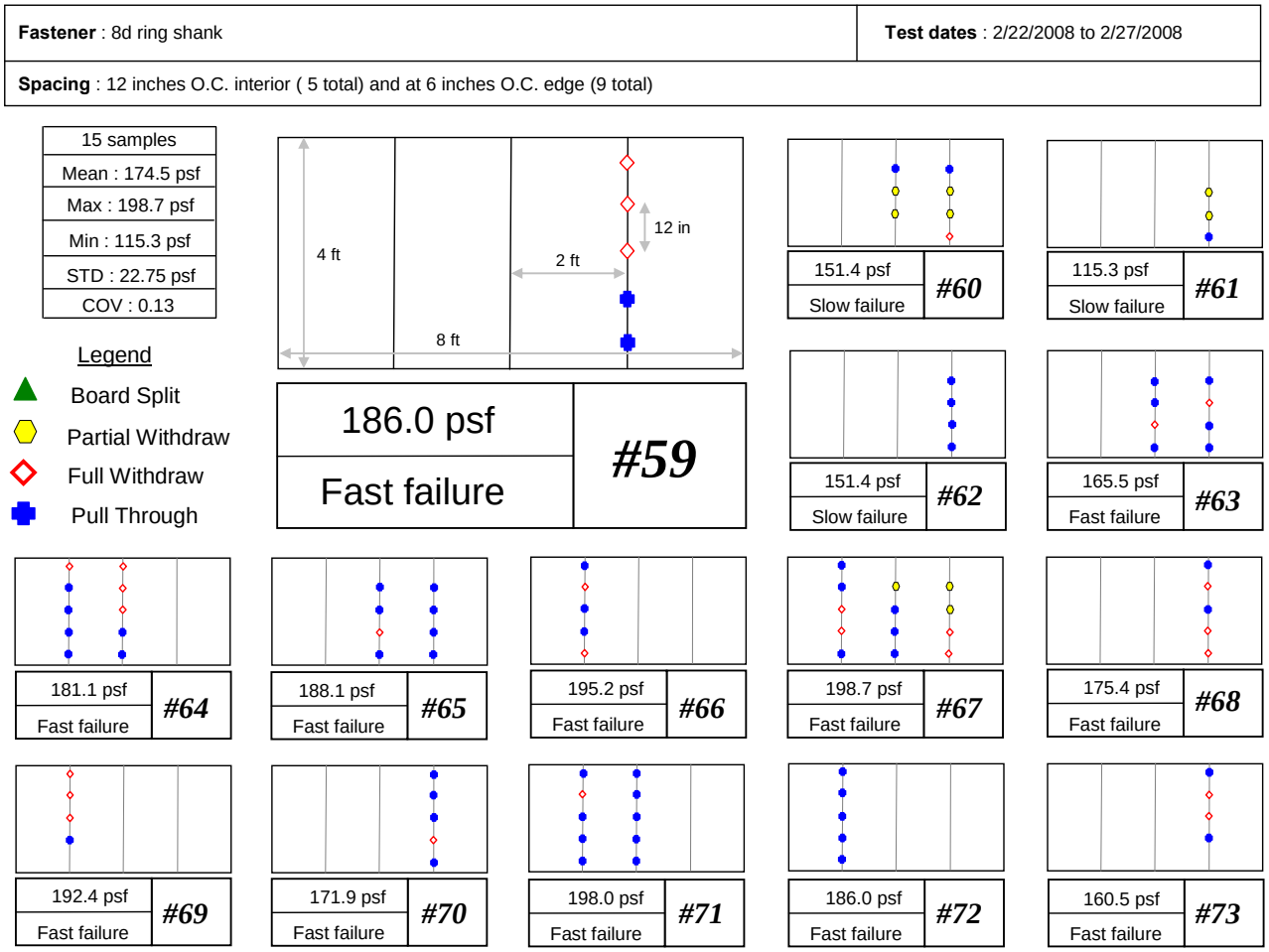


Figure B-12. Failure mode / location for panels fastened with 8d ring shank nails spaced at 6 in. / 12 in. tested statically

APPENDIX C
PANEL CONSTRUCTION OF STATIC VS. DYNAMIC TESTING OF HARVESTED PANELS

Fastener : 1.5 in. staple tested statically & dynamically, and retrofitted with A-2	**Test dates** : 5/21/2009 to 5/22/2009
Spacing : 4 inches O.C. interior (13 total) and at 4 inches O.C. edge (13 total); series of retrofitted with A-2	

4"/4" Dynamic

4 samples
Mean : 71.9 psf
Max : 93 psf
Min : 52 psf
STD : 19 psf
COV : 0.26

4"/4" Static

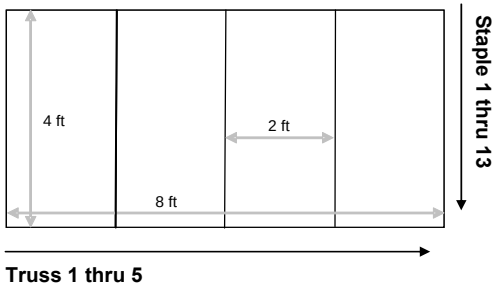
2 samples
Mean : 93 psf
Max : 93 psf
Min : 93 psf

4"/4" Dynamic ret. w/ A-2

2 samples
Mean : 126 psf
Max : 150 psf
Min : 102 psf

4"/4" Static ret. w/ A-2

2 samples
Mean : 168 psf
Max : 169 psf
Min : 167 psf



	Truss 1	Truss 2	Truss 3	Truss 4	Truss 5	Truss 1	Truss 2	Truss 3	Truss 4	Truss 5
	# 7 (92.8 psf), 4 in. / 4 in. Static					# 12 (81.6 psf), 4 in. / 4 in. Dynamic				
Missing Fastener		1, 5-8, 10		9				5,7		
Partially Embedded	10		3,4,9,10	2,3	2			9		
	# 8 (93 psf), 4 in. / 4 in. Static					# 13 (167.4 psf), 4 in. / 4 in. w/ ret. A-2 Static				
Missing Fastener	1	1			4,12		2-7	5		4,12
Partially Embedded		7,8	3-6,10					1,3,10		
	# 9 (93.3 psf), 4 in. / 4 in. Dynamic					# 14 (168.8 psf), 4 in. / 4 in. w/ ret. A-2 Dynamic				
Missing Fastener					3	4		2,6	7,8	11
Partially Embedded		3			6	11	4,6,7,10	3,7,12	11,12	3,5,10
	# 10 (60.9 psf), 4 in. / 4 in. Dynamic					# 15 (149.7 psf), 4 in. / 4 in. w/ ret. A-2 Dynamic				
Missing Fastener		11	1,2				2-12	9	6,10	
Partially Embedded	1								11	7
	# 11 (51.6 psf), 4 in. / 4 in. Dynamic					# 16 (101.9 psf), 4 in. / 4 in. w/ ret. A-2 Dynamic				
Missing Fastener		5,7,11					4,5,6,10		2-12	
Partially Embedded	9	1,10						9		

Figure C-1. Summary of missing fasteners from static vs. dynamic testing, Debary #1 series

Figure C-1 presents results from observations of occurrences and locations of missing fasteners taken at time of harvesting.

APPENDIX D
FULL SPECIFIC GRAVITY MEASUREMENTS

Table D-1. All specific gravity measurements taken from laboratory fabricated static vs. dynamic uplift testing

#	SG	#	SG	#	SG	#	SG		
1	0.372	43	0.655	85	0.455	127	0.409	number	168
2	0.429	44	0.601	86	0.585	128	0.468	mean	0.52
3	0.425	45	0.598	87	0.472	129	0.465	St-Dev	0.09
4	0.632	46	0.572	88	0.479	130	0.676	COV	18%
5	0.586	47	0.603	89	0.493	131	0.429		
6	0.694	48	0.506	90	0.632	132	0.421		
7	0.622	49	0.498	91	0.574	133	0.432		
8	0.656	50	0.548	92	0.672	134	0.539		
9	0.624	51	0.602	93	0.657	135	0.651		
10	0.500	52	0.532	94	0.615	136	0.587		
11	0.436	53	0.529	95	0.539	137	0.757		
12	0.428	54	0.509	96	0.515	138	0.787		
13	0.540	55	0.495	97	0.466	139	0.667		
14	0.573	56	0.480	98	0.491	140	0.539		
15	0.547	57	0.582	99	0.493	141	0.575		
16	0.457	58	0.593	100	0.482	142	0.582		
17	0.456	59	0.479	101	0.478	143	0.473		
18	0.468	60	0.644	102	0.411	144	0.508		
19	0.601	61	0.495	103	0.464	145	0.446		
20	0.591	62	0.609	104	0.482	146	0.457		
21	0.589	63	0.689	105	0.609	147	0.477		
22	0.532	64	0.523	106	0.579	148	0.377		
23	0.565	65	0.517	107	0.400	149	0.368		
24	0.577	66	0.486	108	0.406	150	0.378		
25	0.692	67	0.573	109	0.420	151	0.458		
26	0.408	68	0.574	110	0.462	152	0.440		
27	0.537	69	0.536	111	0.430	153	0.393		
28	0.573	70	0.390	112	0.454	154	0.589		
29	0.482	71	0.419	113	0.516	155	0.449		
30	0.458	72	0.412	114	0.563	156	0.523		
31	0.461	73	0.483	115	0.442	157	0.552		
32	0.471	74	0.613	116	0.450	158	0.537		

Table D-1. Continued

#	SG	#	SG	#	SG	#	SG
33	0.461	75	0.415	117	0.520	159	0.483
34	0.452	76	0.475	118	0.474	160	0.502
35	0.536	77	0.731	119	0.525	161	0.533
36	0.476	78	0.477	120	0.606	162	0.544
37	0.510	79	0.613	121	0.474	163	0.466
38	0.838	80	0.562	122	0.404	164	0.521
39	0.420	81	0.541	123	0.453	165	0.390
40	0.449	82	0.493	124	0.475	166	0.406
41	0.548	83	0.541	125	0.448	167	0.403
42	0.550	84	0.502	126	0.506		

Table D-2. All specific gravity measurements taken from harvested static vs. dynamic uplift testing

#	SG	#	SG	#	SG	#	SG		
1	0.513	46	0.519	91	0.563	135	0.592	number	178
2	0.562	47	0.590	92	0.709	136	0.570	mean	0.57
3	0.476	48	0.661	93	0.586	137	0.575	St-Dev	0.08
4	0.451	49	0.443	94	0.517	138	0.709	COV	14%
5	0.613	50	0.536	95	0.563	139	0.665		
6	0.590	51	0.513	96	0.572	140	0.585		
7	0.575	52	0.545	97	0.674	141	0.545		
8	0.601	53	0.501	98	0.558	142	0.566		
9	0.412	54	0.630	99	0.621	143	0.441		
10	0.619	55	0.595	100	0.549	144	0.489		
11	0.611	56	0.494	101	0.505	145	0.484		
12	0.435	57	0.646	102	0.495	146	0.503		
13	0.478	58	0.586	103	0.565	147	0.594		
14	0.424	59	0.485	104	0.549	148	0.511		
15	0.477	60	0.467	105	0.636	149	0.624		
16	0.484	61	0.535	106	0.481	150	0.506		
17	0.560	62	0.586	107	0.609	151	0.567		
18	0.508	63	0.593	108	0.619	152	0.620		
19	0.497	64	0.633	109	0.512	153	0.737		
20	0.462	65	0.523	110	0.482	154	0.532		
21	0.643	66	0.518	111	0.492	155	0.536		
22	0.617	67	0.778	112	0.575	156	0.542		

Table D-2. Continued

#	SG	#	SG	#	SG	#	SG
23	0.619	68	0.601	113	0.523	157	0.532
24	0.470	69	0.631	114	0.652	158	0.536
25	0.646	70	0.613	115	0.599	159	0.655
26	0.693	71	0.504	116	0.441	160	0.615
27	0.634	72	0.561	117	0.675	161	0.519
28	0.529	73	0.455	118	0.462	162	0.670
29	0.705	74	0.684	119	0.599	163	0.572
30	0.452	75	0.487	120	0.644	164	0.666
31	0.425	76	0.541	121	0.713	165	0.622
32	0.526	77	0.590	122	0.611	166	0.609
33	0.285	78	0.622	123	0.590	167	0.556
34	0.562	79	0.555	124	0.437	168	0.590
35	0.596	80	0.576	125	0.450	169	0.481
36	0.567	81	0.544	126	0.573	170	0.548
37	0.564	82	0.560	127	0.698	171	0.505
38	0.632	83	0.625	128	0.668	172	0.723
39	0.667	84	0.601	129	0.571	173	0.630
40	0.439	85	0.596	130	0.571	174	0.436
41	0.529	86	0.699	131	0.670	175	0.548
42	0.672	87	0.585	132	0.546	176	0.539
43	0.657	88	0.575	133	0.655	177	0.644
44	0.605	89	0.450	134	0.549	178	0.573
45	0.457	90	0.526				

APPENDIX E
STATIC VS. DYNAMIC PANEL TESTING, TARGET AND ACTUAL PRESSURE TIME-HISTORIES

- Laboratory Fabricated 6d Smooth Shank at 6 in. / 12 in. Static (5 panels)

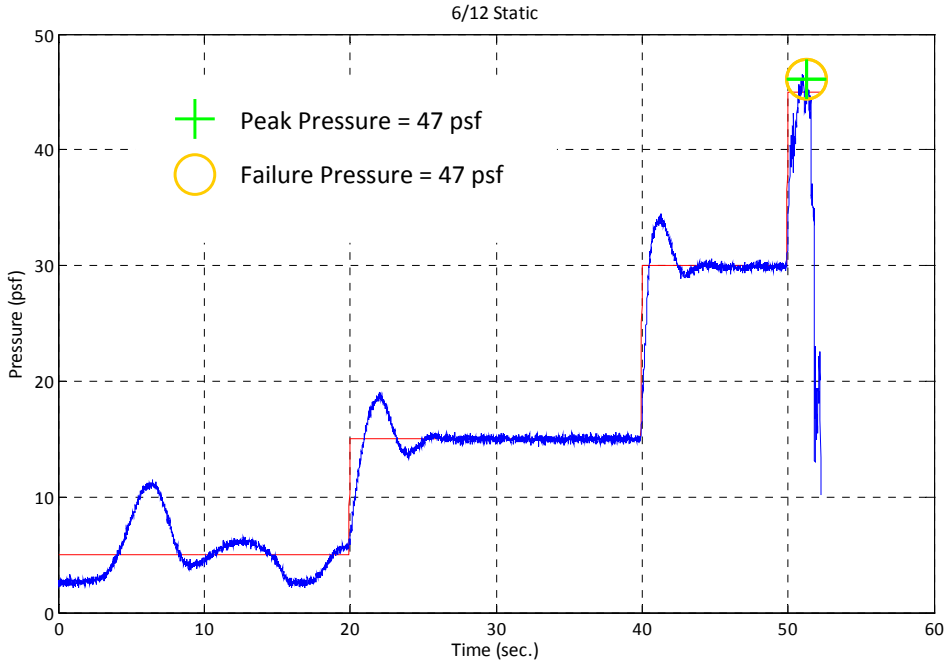


Figure E-1. Summary of pressure time-history for static 6d SS at 6/12 panel (6d SS-38)

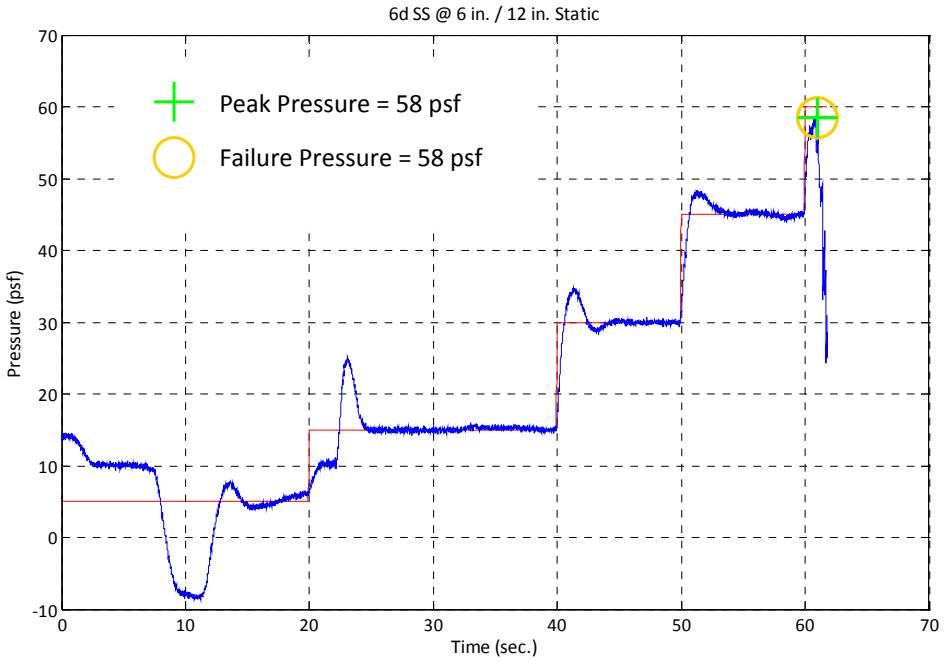


Figure E-2. Summary of pressure time-history for static 6d SS at 6/12 panel (6d SS-40)

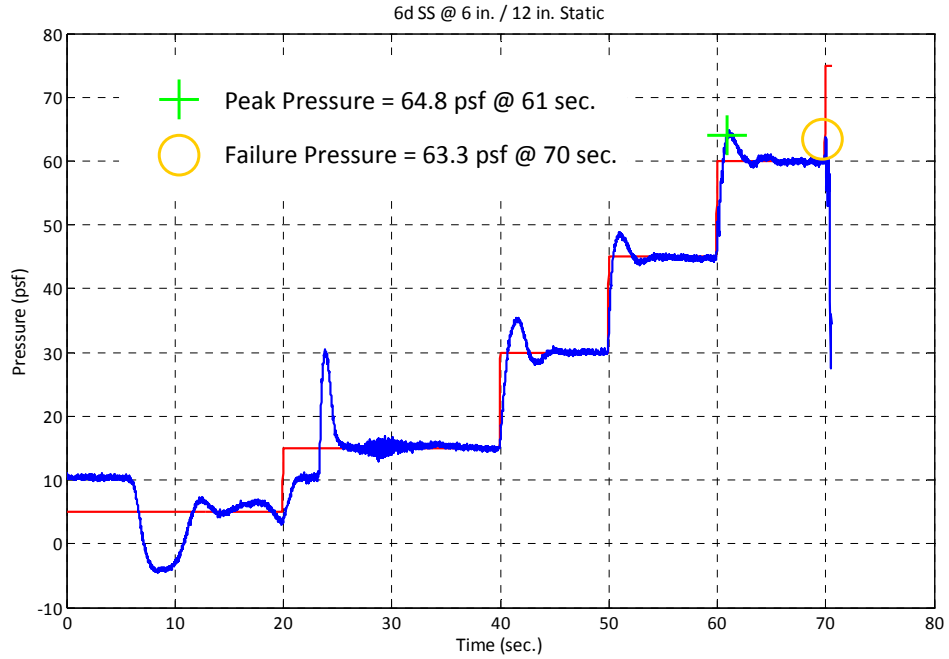


Figure E-3. Summary of pressure time-history for static 6d SS at 6/12 panel (6d SS-41)

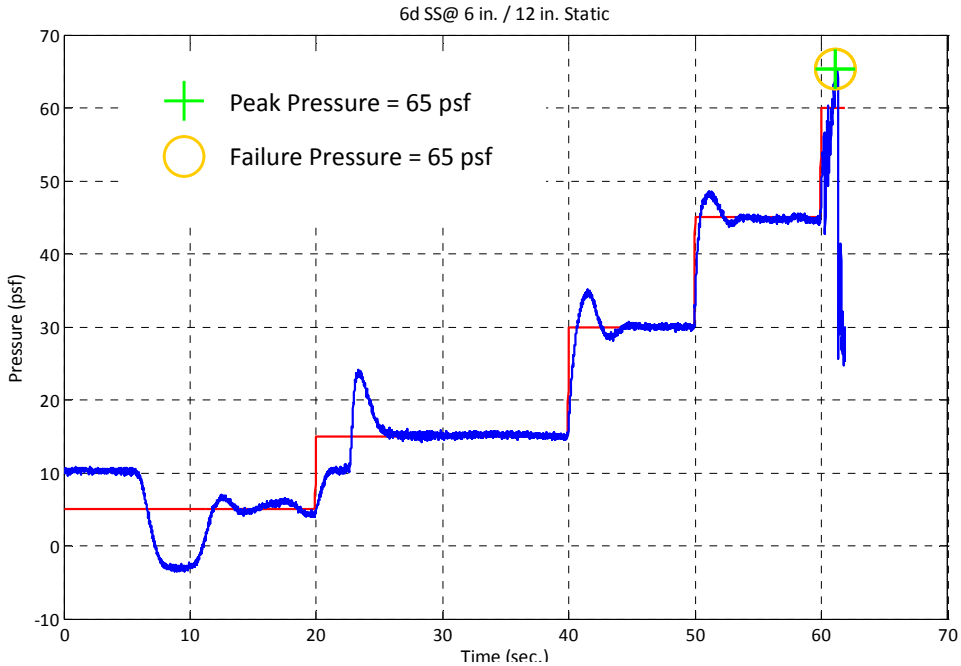


Figure E-4. Summary of pressure time-history for static 6d SS at 6/12 panel (6d SS-42)

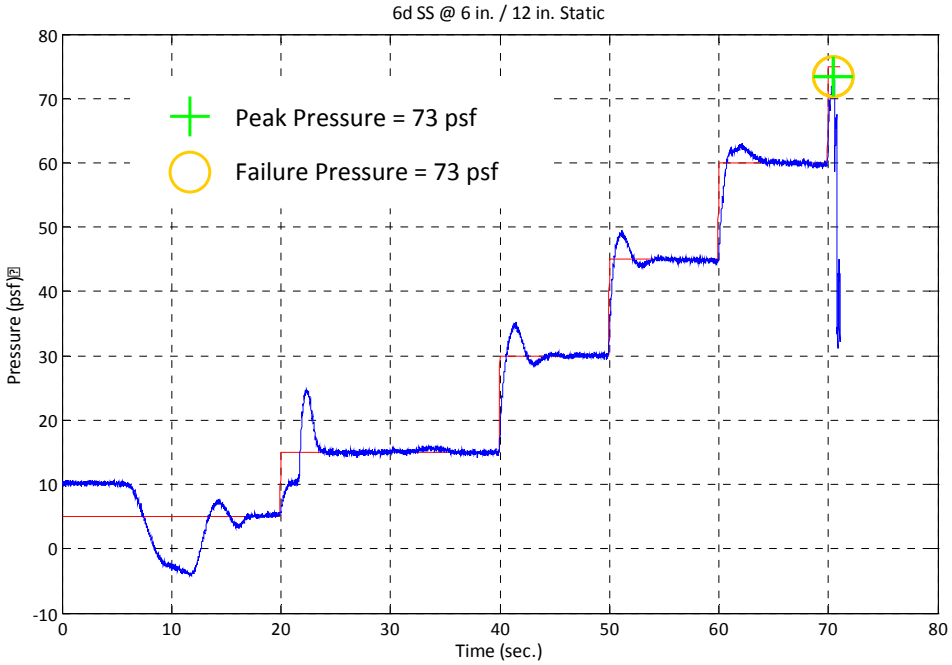
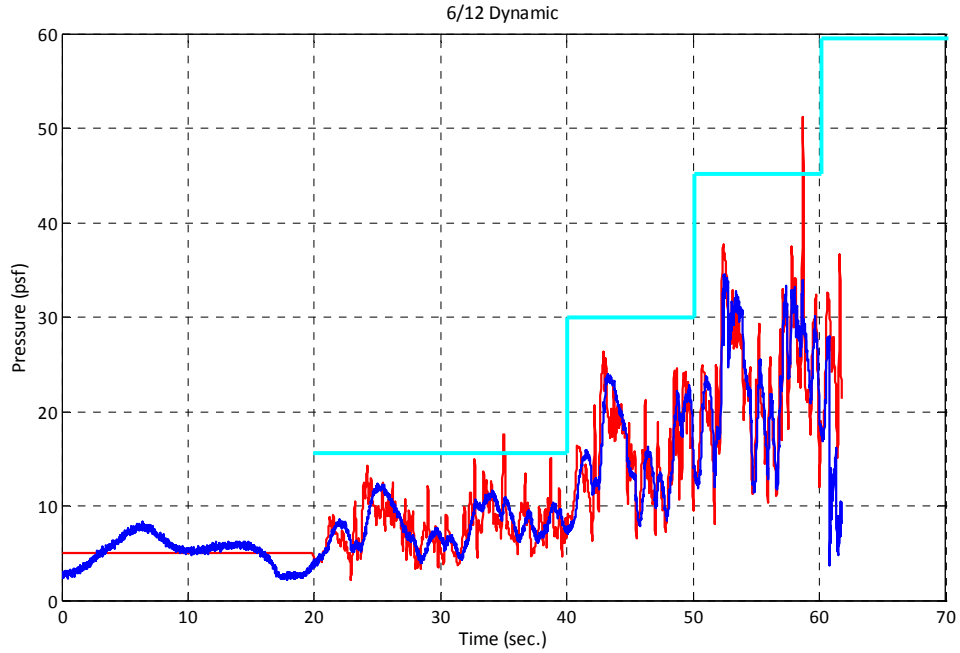


Figure E-5. Summary of pressure time-history for static 6d SS at 6/12 panel (6d SS-39)

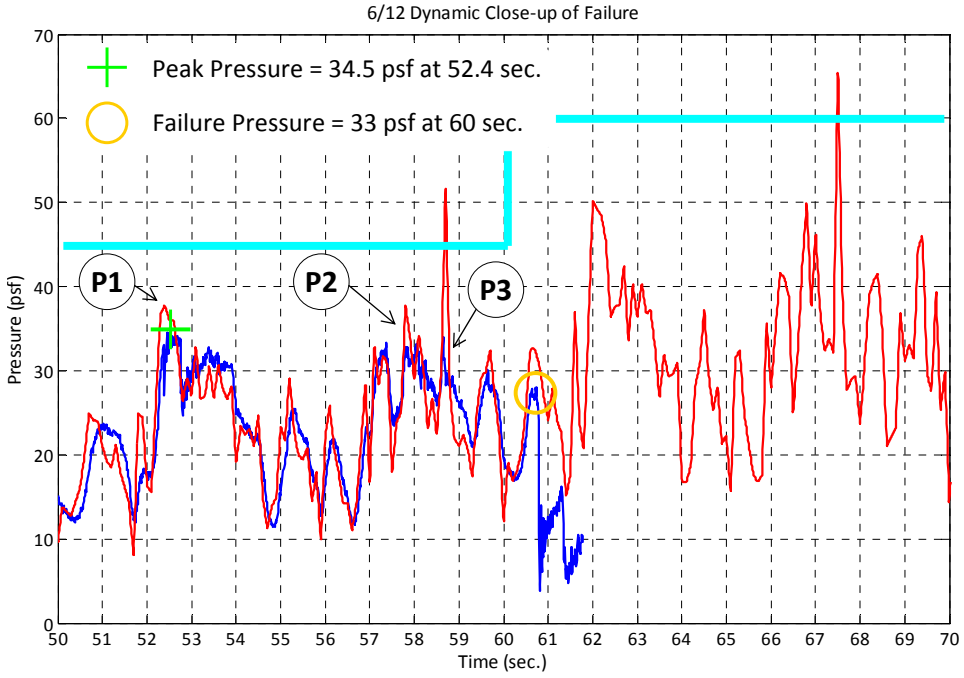
- Laboratory Fabricated 6d Smooth Shank at 6 in. / 12 in. Dynamic (5 panels)

Table E-1. Summary of dynamic pressure trace for 6d SS at 6 in. / 12 in. panels (blue – actual and red target)

	Peak 1	Peak 2	Peak 3	Peak	Failure	Ratio Failure to Peak
6d SS at 6 in. / 12 in. Dynamic						
6d SS-43	34.5 psf @ 52 sec. 37.7 psf @ 52 sec.	33 psf @ 58 sec. 38 psf @ 58 sec.	34 psf @ 59 sec. 51 psf @ 59 sec.	34.5 psf @ 52 sec. 37.7 psf @ 52 sec.	28 psf @ 60 sec. 33 psf @ 60 sec.	0.96
6d SS-44	48 psf @ 62 sec. 50 psf @ 62 sec.	47 psf @ 67 sec. 50 psf @ 67 sec.	51 psf @ 68 sec. 65 psf @ 68 sec.	51 psf @ 68 sec. 65 psf @ 68 sec.	48 psf @ 72 sec. 63 psf @ 72 sec.	0.94
6d SS-45	50 psf @ 62 sec. 50 psf @ 62 sec.	43 psf @ 66 sec. 41 psf @ 66 sec.	46 psf @ 67 sec. 49 psf @ 67 sec.	50 psf @ 62 sec. 50 psf @ 62 sec.	46 psf @ 67.5 sec. 66 psf @ 67.5 sec.	0.92
6d SS-46	51 psf @ 62 sec. 50 psf @ 62 sec.	51 psf @ 67.5 sec. 66 psf @ 67.5 sec.	46 psf @ 69 sec. 46 psf @ 69 sec.	58 psf @ 72 sec. 63 psf @ 72 sec.	58 psf @ 72 sec. 63 psf @ 72 sec.	1.00
6d SS-47	60 psf @ 72 sec. 63 psf @ 72 sec.	62 psf @ 76.5 sec. 79 psf @ 76.5 sec.	55 psf @ 79.5 sec. 62 psf @ 79.5 sec.	66 psf @ 82 sec. 75 psf @ 82 sec.	66 psf @ 82 sec. 75 psf @ 82 sec.	1.00

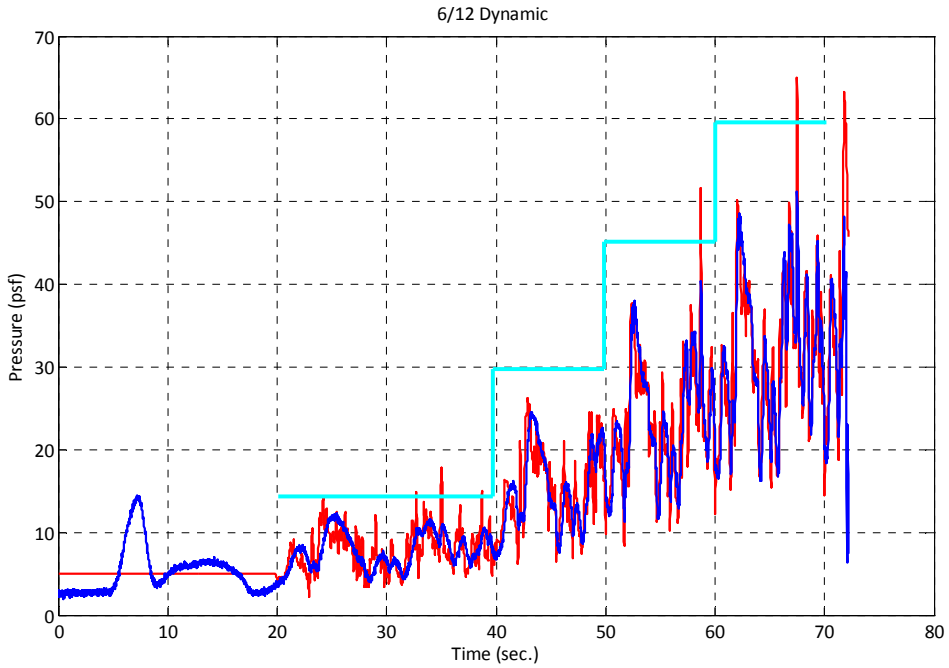


A

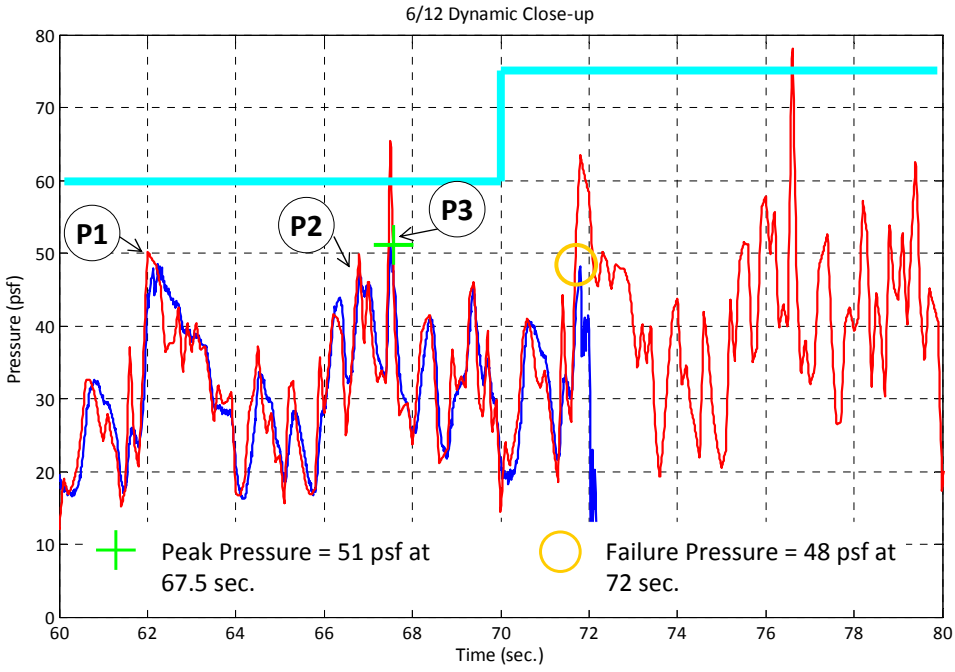


B

Figure E-6. Summary of pressure time-histories for dynamic 6d SS at 6/12 panel (6d SS-43) A) full time-history and B) close up of failure

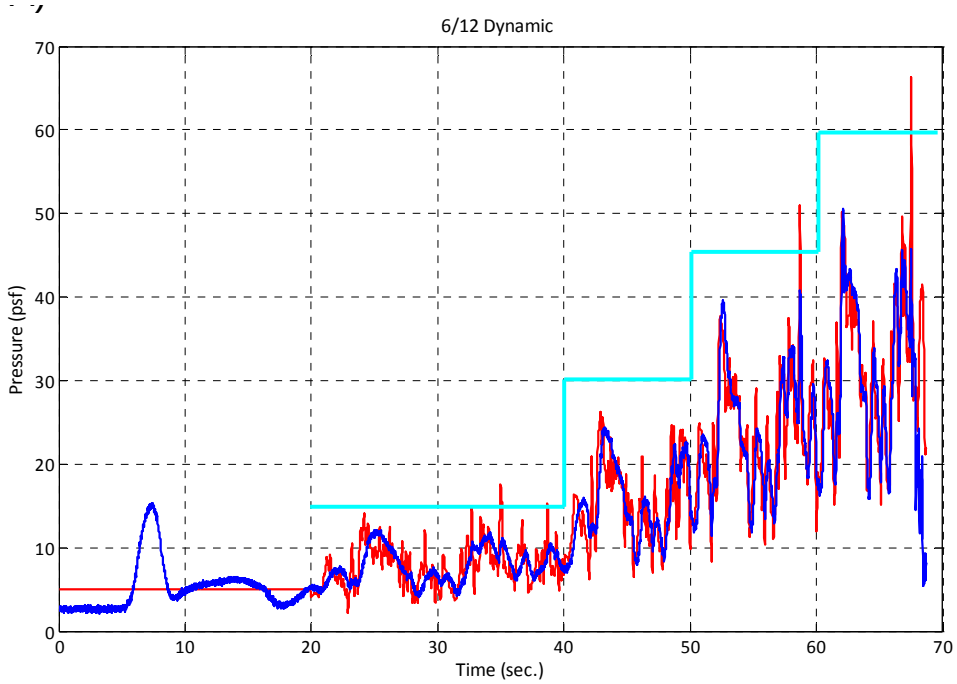


A

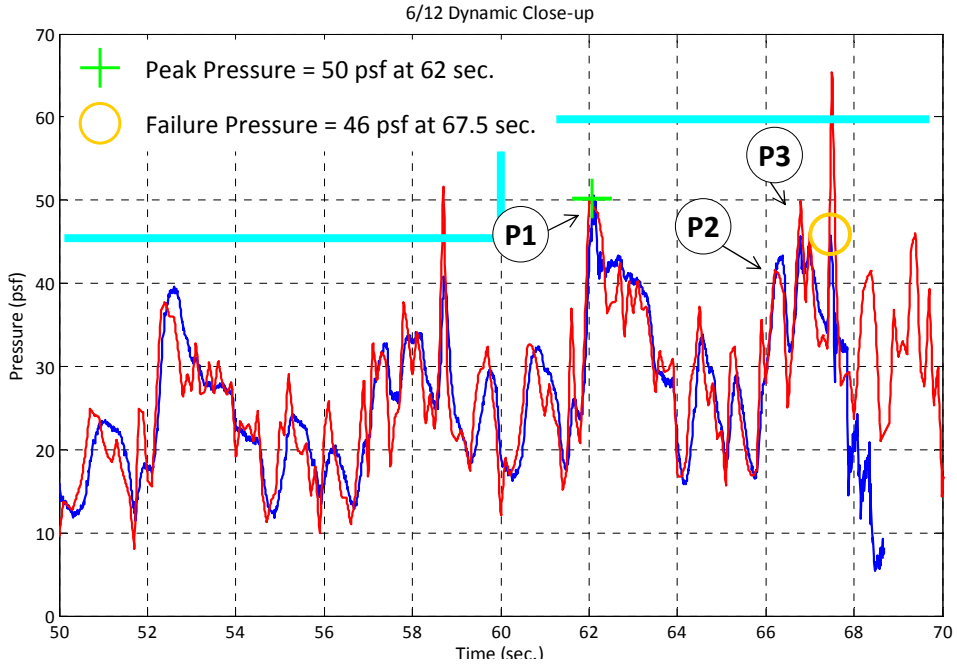


B

Figure E-7. Summary of pressure time-histories for dynamic 6d SS at 6/12 panel (6d SS-44) A) full time-history and B) close up of failure

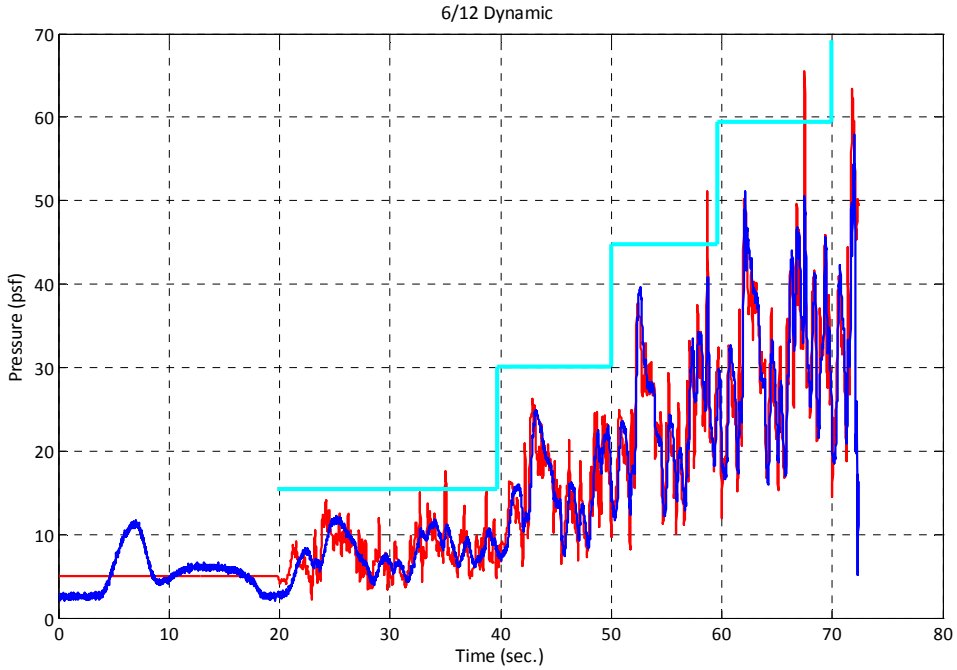


A

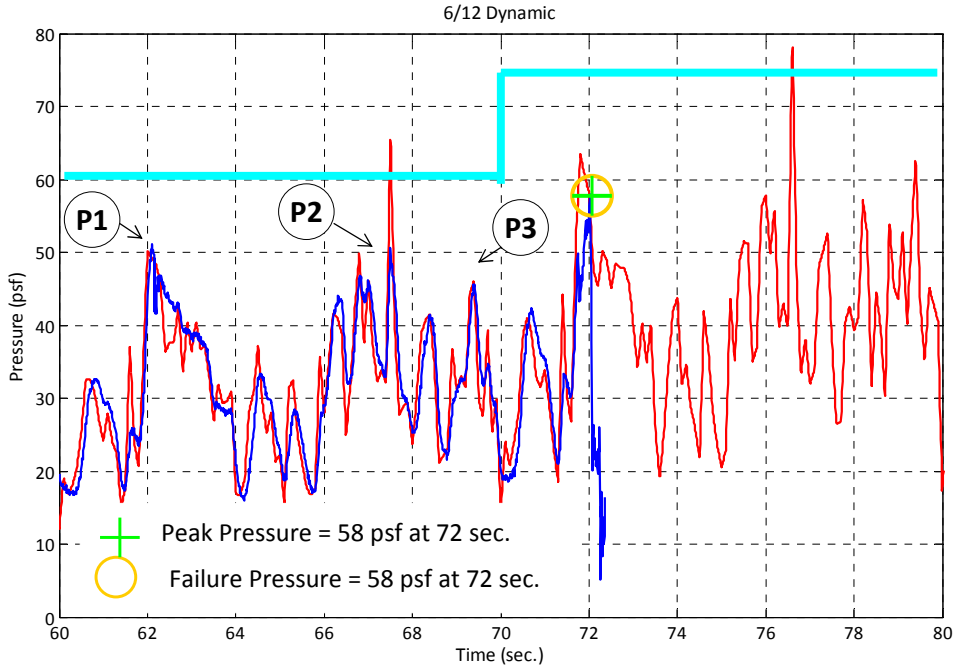


B

Figure E-8. Summary of pressure time-histories for dynamic 6d SS at 6/12 panel (6D SS-45) A) full time-history and B) close up of failure



A



B

Figure E-9. Summary of pressure time-histories for dynamic 6d SS at 6/12 panel (6d SS-46) full time-history and B) close up of failure

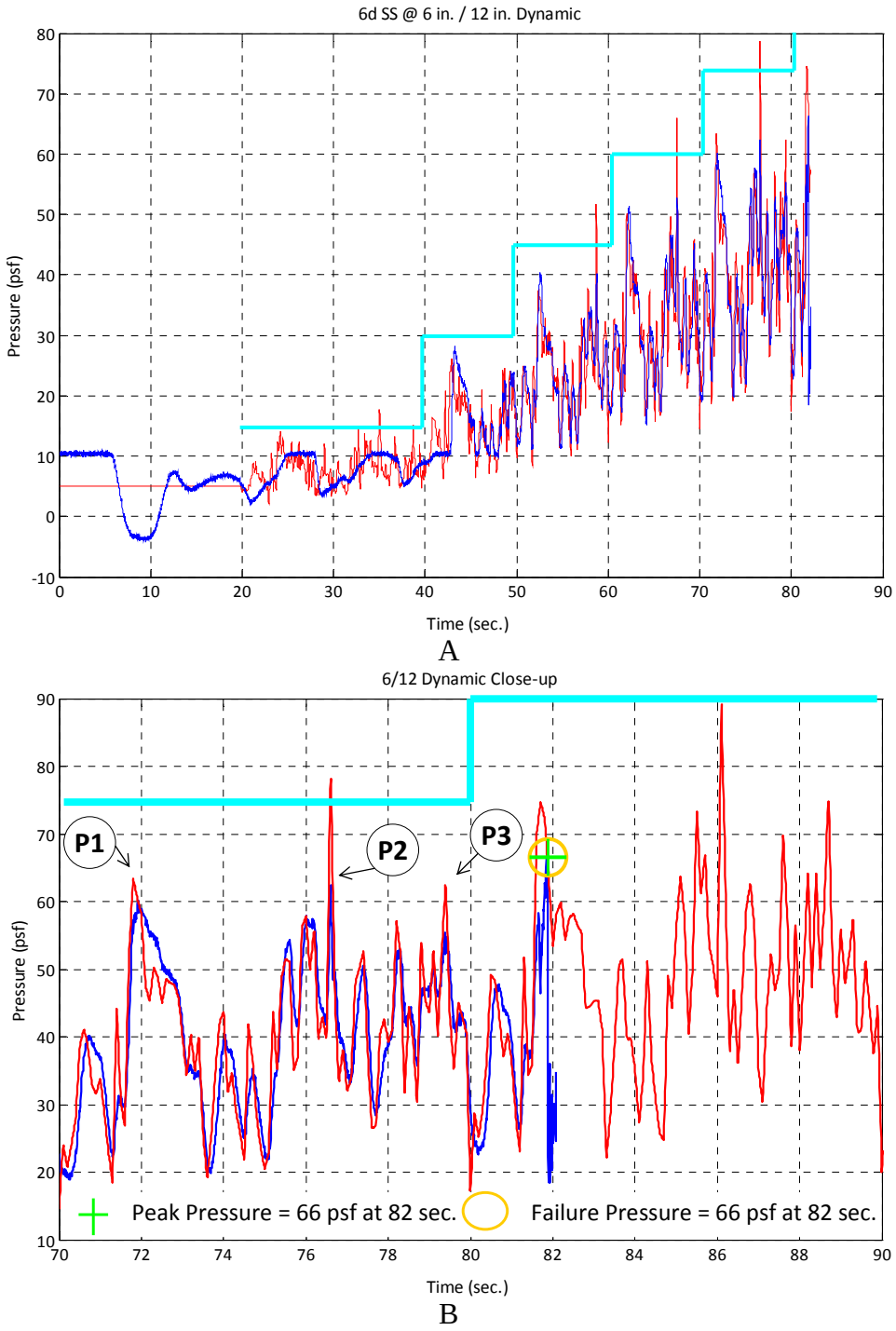


Figure E-10. Summary of pressure time-histories for dynamic 6d SS at 6/12 panel (6d SS-47) A) full time-history and B) close up of failure

- Laboratory Fabricated 6d Smooth Shank at 6 in. / 6 in. Static (5 panels)

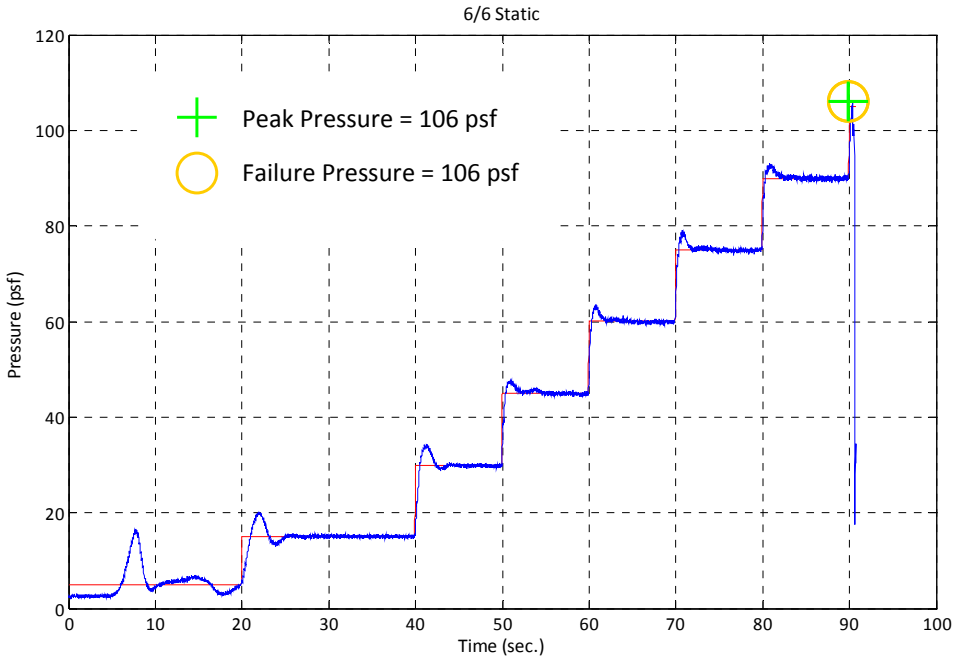


Figure E-11. Summary of pressure time-history for static 6d SS at 6/6 panel (6d SS-53)

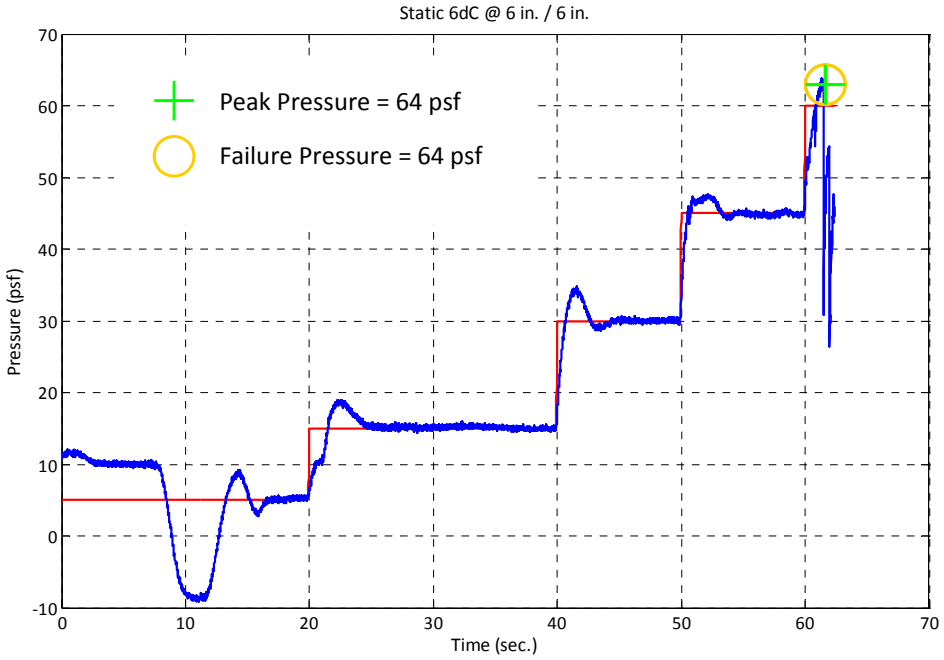


Figure E-12. Summary of pressure time-history for static 6d SS at 6/6 panel (6d SS-52)

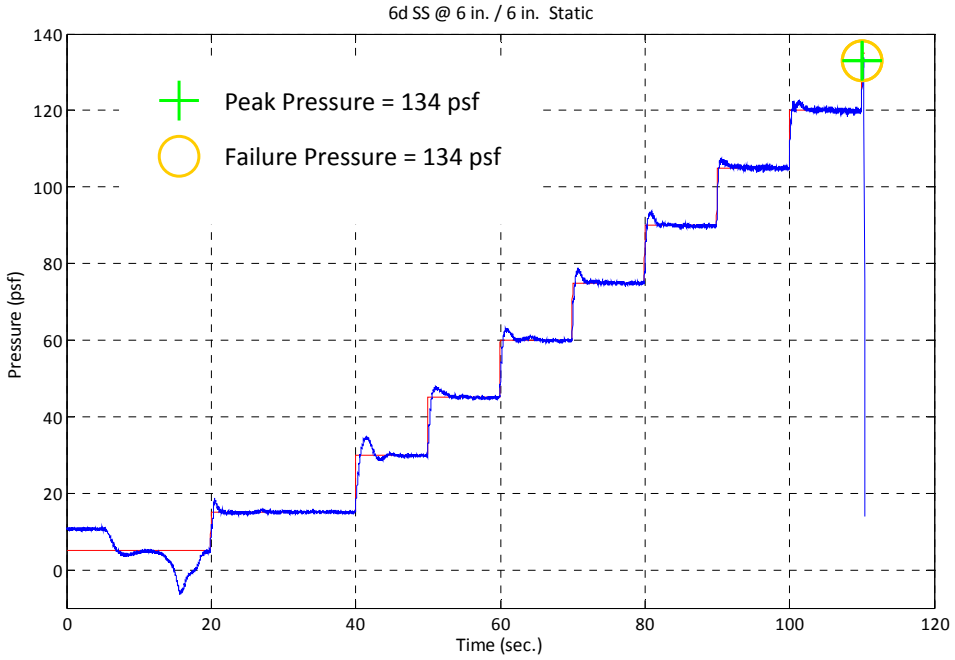


Figure E-13. Summary of pressure time-history for static 6d SS at 6/6 panel (6d SS-56)

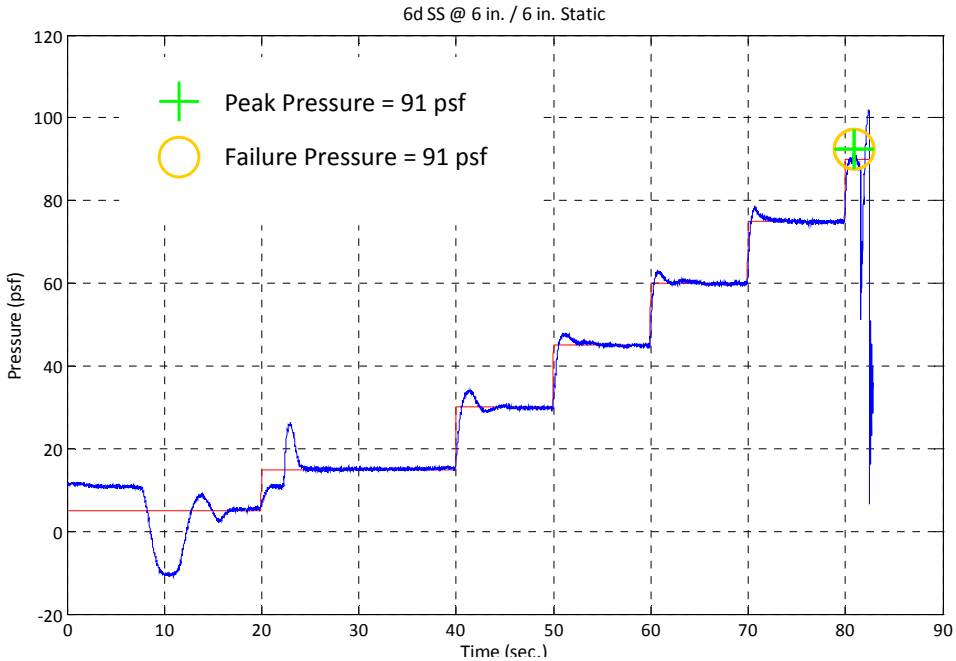


Figure E-14. Summary of pressure time-history for static 6d SS at 6/6 panel (6d SS-55)

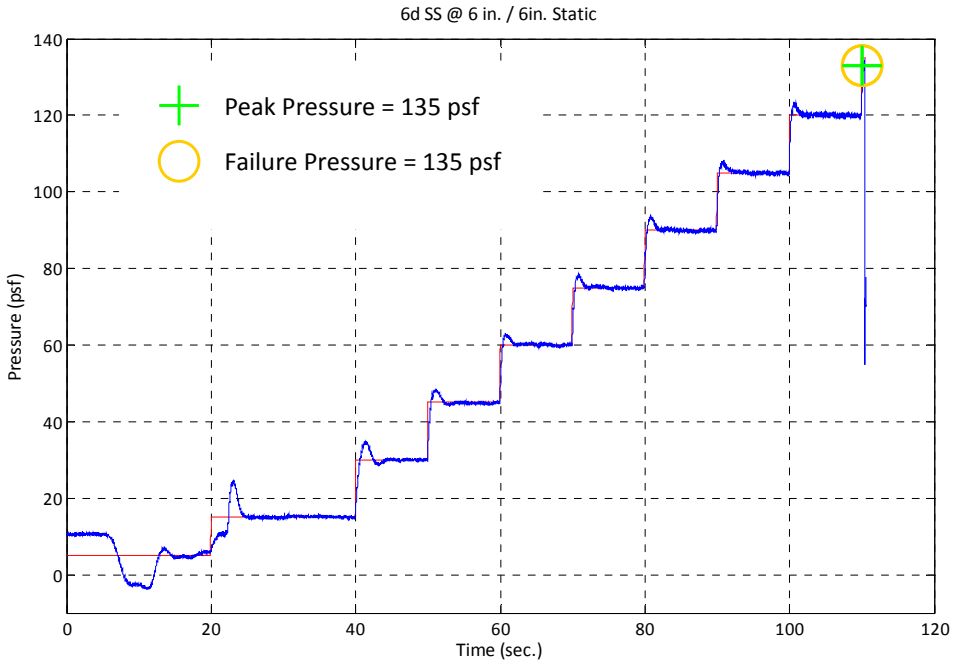
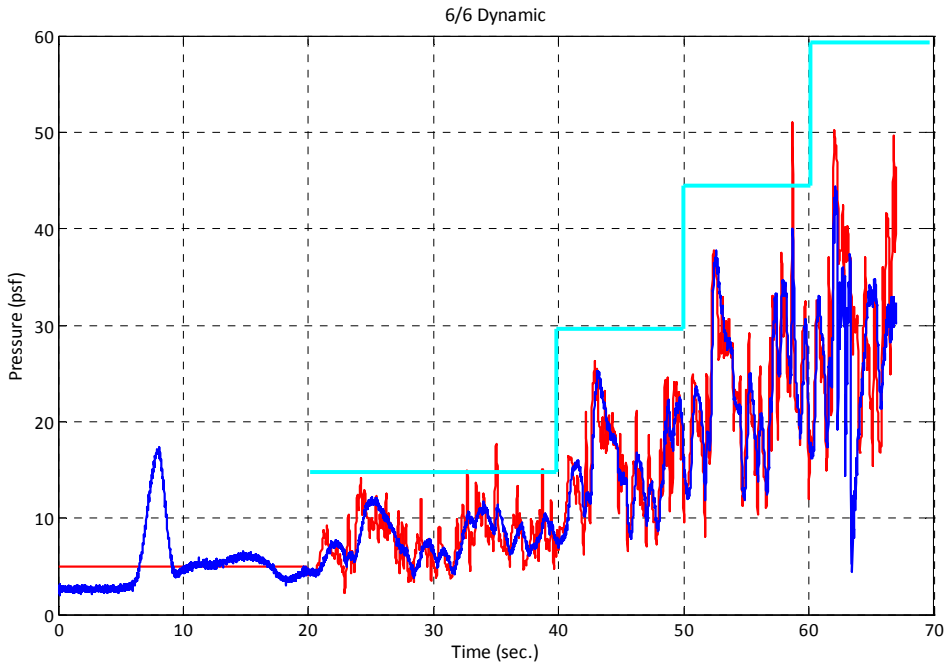


Figure E-15. Summary of pressure time-history for static 6d SS at 6/6 panel (6d SS-54)

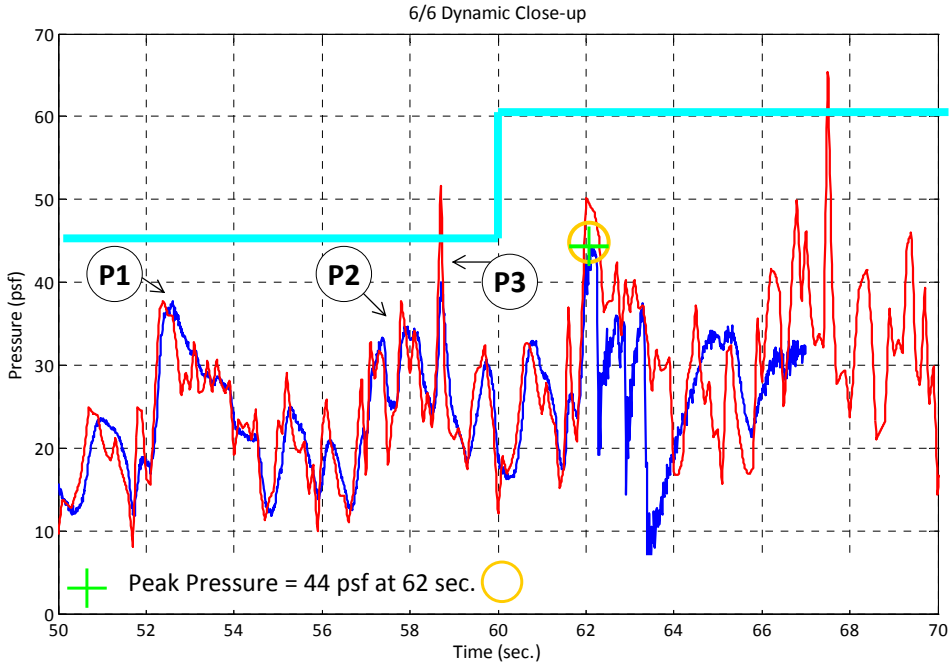
- Laboratory Fabricated 6d Smooth Shank at 6 in. / 6 in. Dynamic (5 panels)

Table E-2. Summary of dynamic pressure trace for 6d SS at 6 in. / 6 in. panels (blue – actual and red target)

	Peak 1	Peak 2	Peak 3	Peak	Failure
			6d SS at 6 in. / 6 in. Dynamic		
6d SS-57	38 psf @ 52 sec.	35 psf @ 58 sec.	40 psf @ 59 sec.	44 psf @ 62 sec.	44 psf @ 62 sec.
	38 psf @ 52 sec.	38 psf @ 58 sec.	52 psf @ 59 sec.	50 psf @ 62 sec.	50 psf @ 62 sec.
6d SS-58	101 psf @ 101.5 sec.	96 psf @ 105 sec.	92 psf @ 107 sec.	103 psf @ 111 sec.	103 psf @ 111 sec.
	98 psf @ 101.5 sec.	110 psf @ 105 sec.	97 psf @ 107 sec.	112 psf @ 111 sec.	112 psf @ 111 sec.
6d SS-59	102 psf @ 101.5 sec.	97 psf @ 105 sec.	93 psf @ 107.5 sec.	105 psf @ 111.5 sec.	105 psf @ 111.5 sec.
	98 psf @ 101.5 sec.	110 psf @ 105 sec.	97 psf @ 107.5 sec.	112 psf @ 111.5 sec.	112 psf @ 111 sec.
6d SS-60	64 psf @ 72 sec.	64 psf @ 77 sec.	60 psf @ 79 sec.	72 psf @ 82 sec.	59 psf @ 82.5 sec.
	63 psf @ 72 sec.	79 psf @ 77 sec.	62 psf @ 79 sec.	75 psf @ 82 sec.	60 psf @ 82.5 sec.
6d SS-61	114 psf @ 111 sec.	107 psf @ 115 sec.	102 psf @ 117 sec.	124 psf @ 121 sec.	104 psf @ 124 sec.
	112 psf @ 111 sec.	128 psf @ 115 sec.	114 psf @ 117 sec.	128 psf @ 121 sec.	102 psf @ 124 sec.

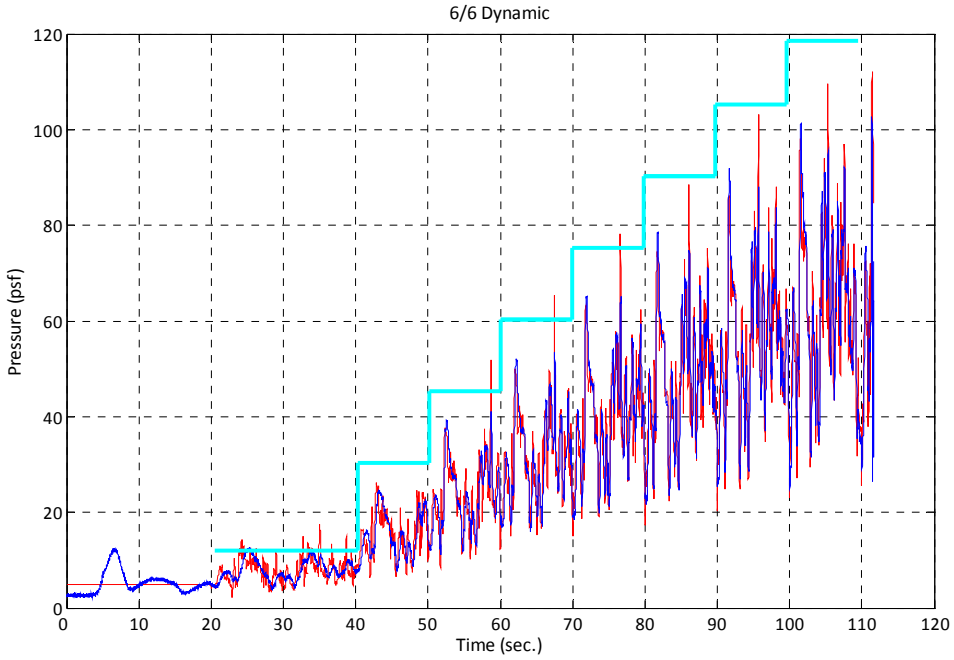


A

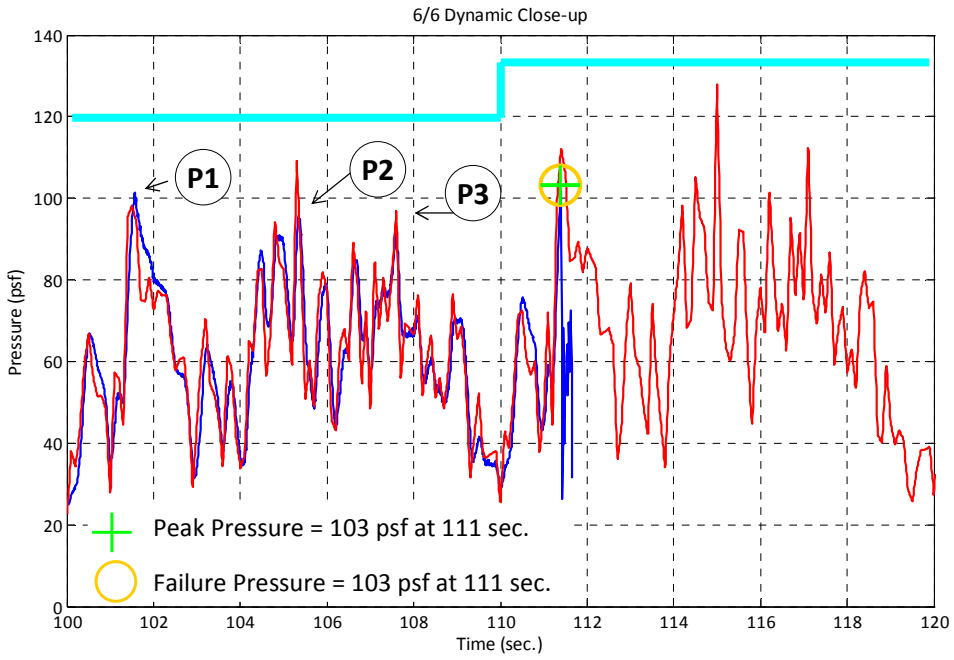


B

Figure E-16. Summary of pressure time-histories for dynamic 6d SS at 6/6 panel (6d SS-57) A) full time-history and B) close up of failure

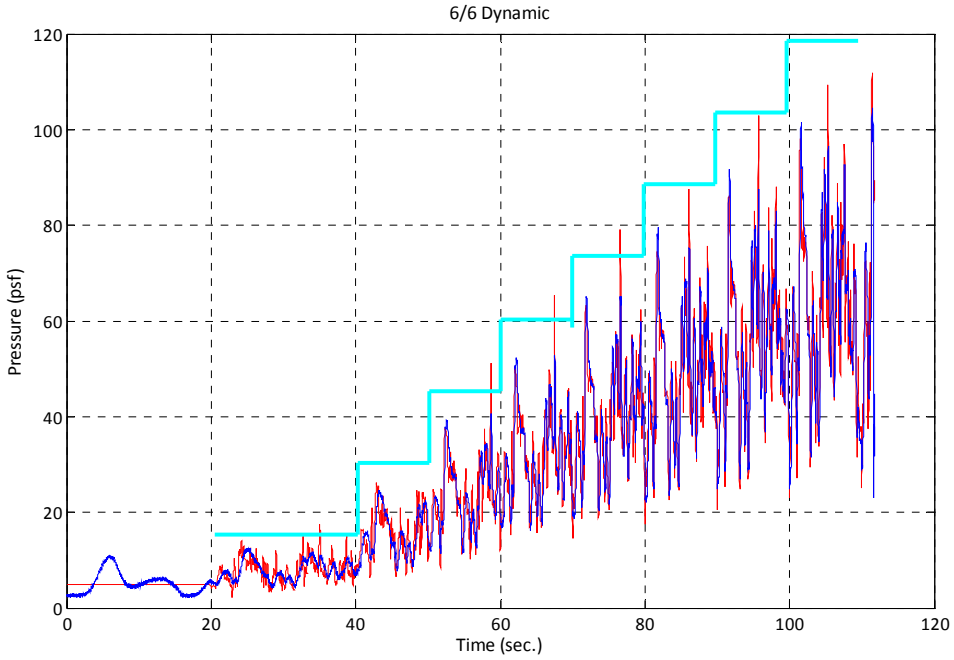


A

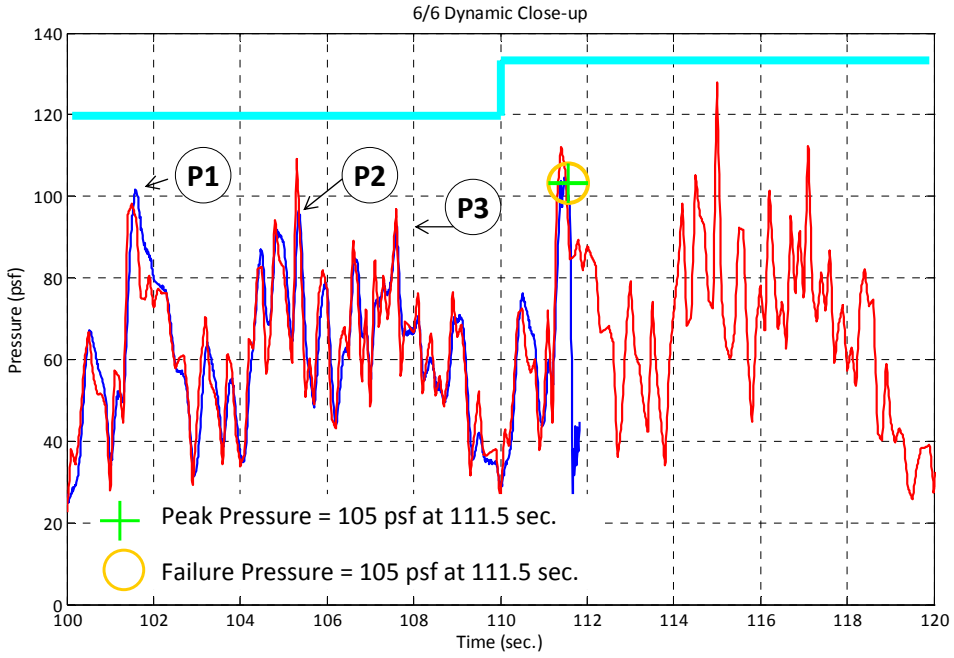


B

Figure E-17. Summary of pressure time-histories for dynamic 6d SS at 6/6 panel (6d SS-58) A) full time-history and B) close up of failure

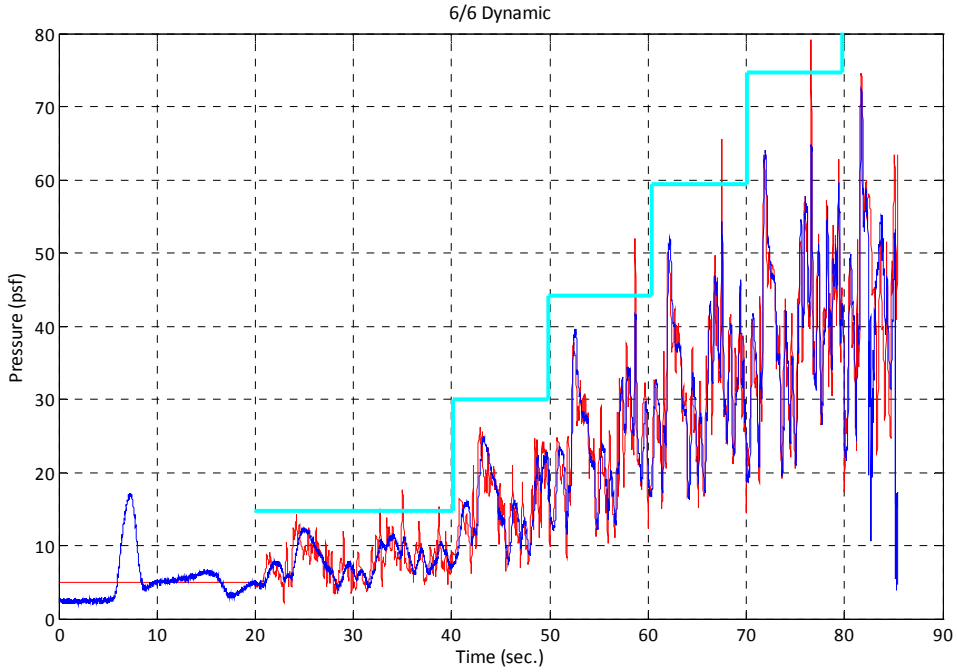


A



B

Figure E-18. Summary of pressure time-histories for dynamic 6d SS at 6/6 panel (6d SS-59) A) full time-history and B) close up of failure



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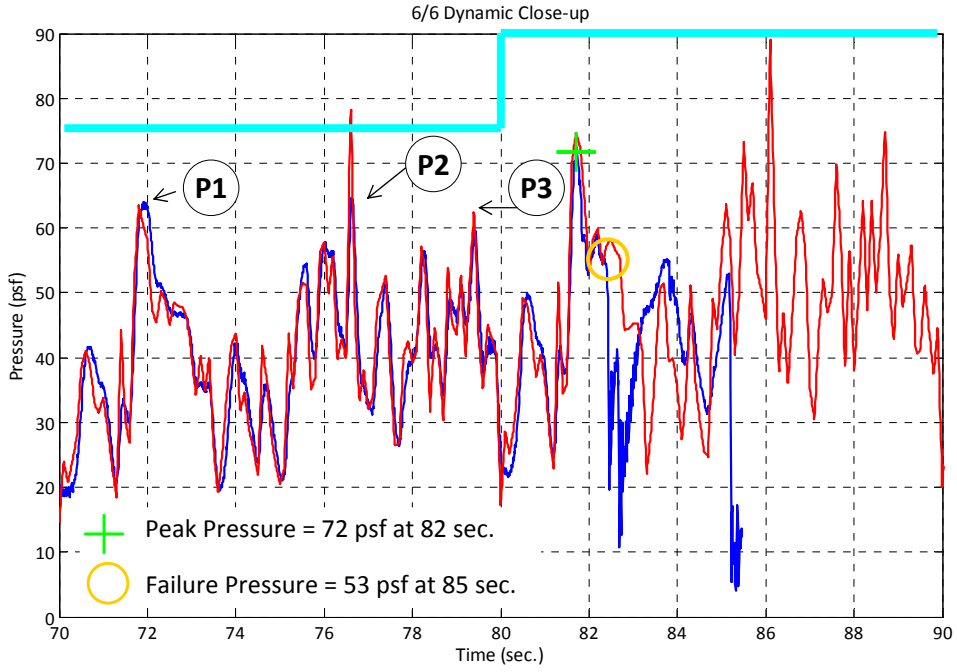
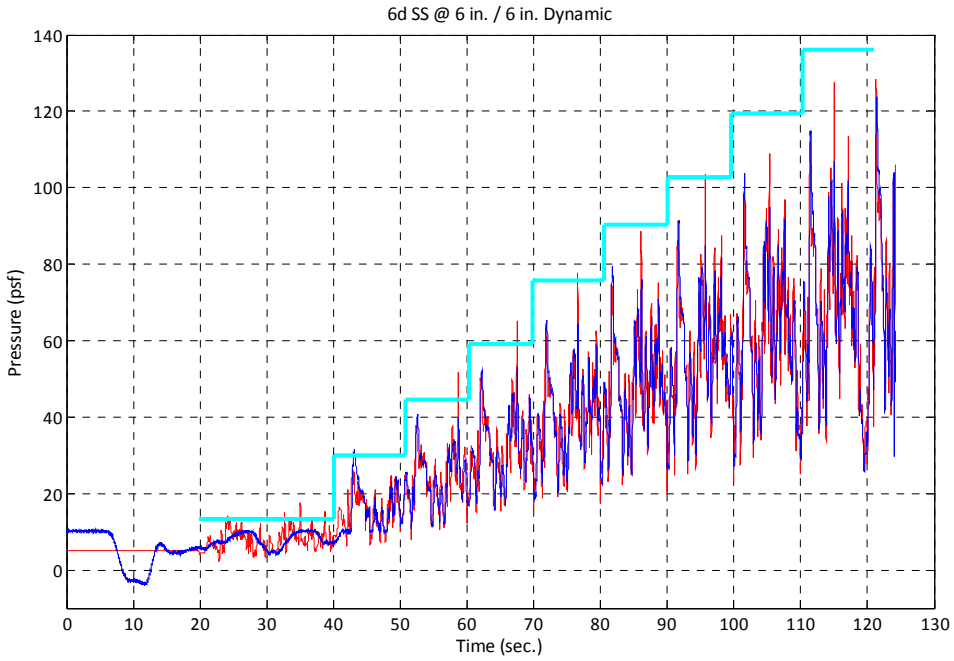


B

Figure E-19. Summary of pressure time-histories for dynamic 6d SS at 6/6 panel (6d SS-60) A) full time-history and B) close up of failure



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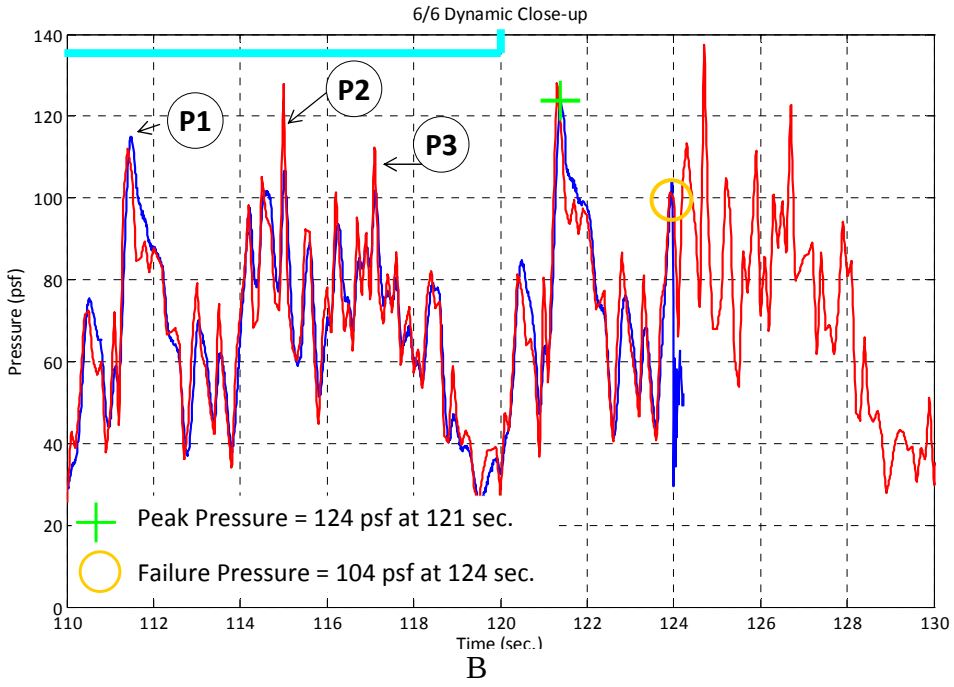


B

Figure E-20. Summary of pressure time-histories for dynamic 6d SS at 6/6 panel (6d SS-61) A) full time-history and B) close up of failure

- Laboratory Fabricated 6d Smooth Shank at 6 in. / 12 in. Ret. A-2 Static (2 panels)

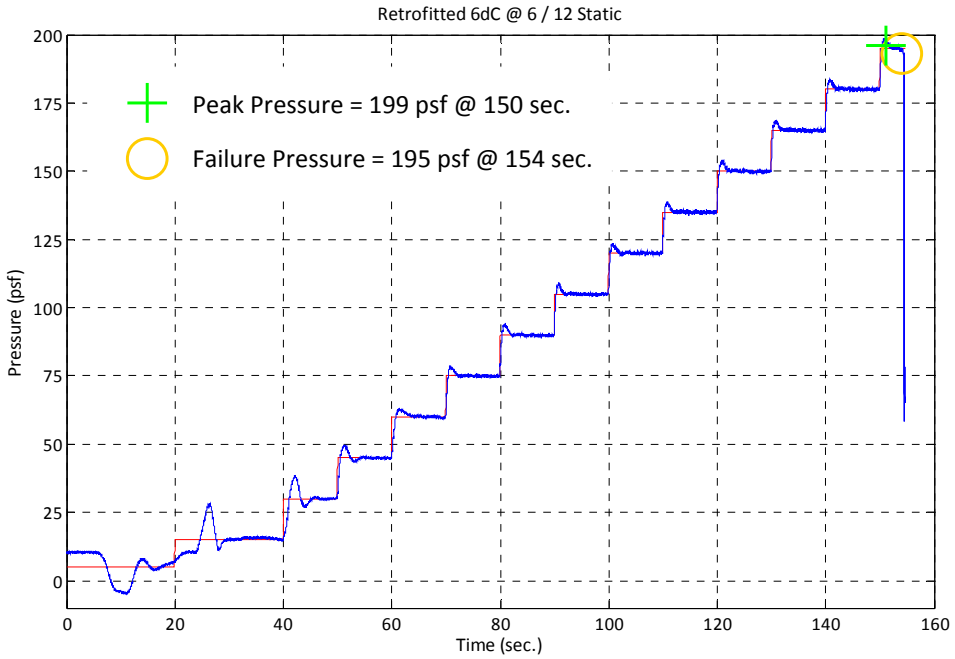


Figure E-21. Summary of pressure time-history for static 6d SS at 6/12 ret. A-2 panel (6d SS-48)

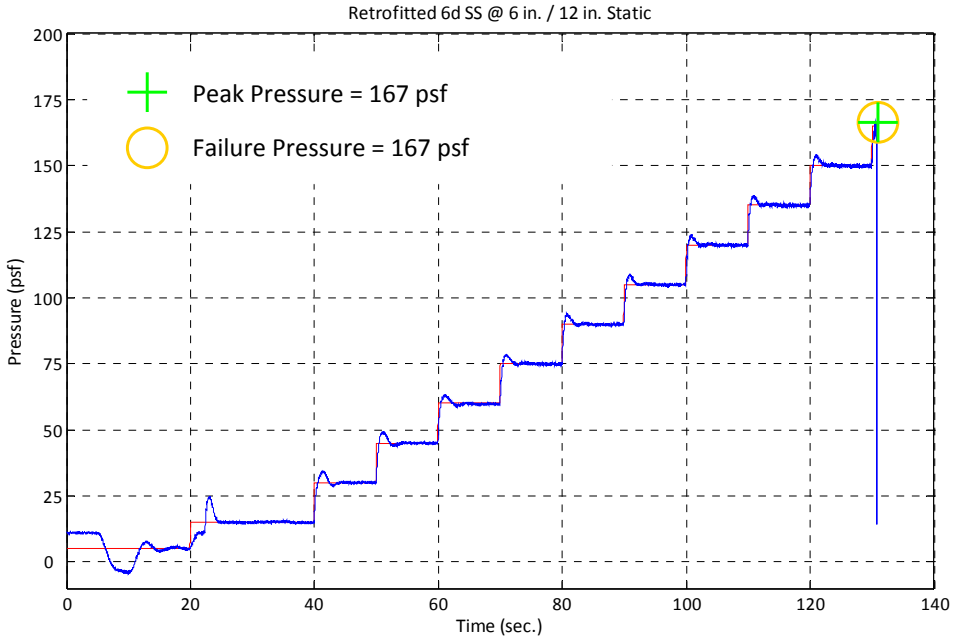
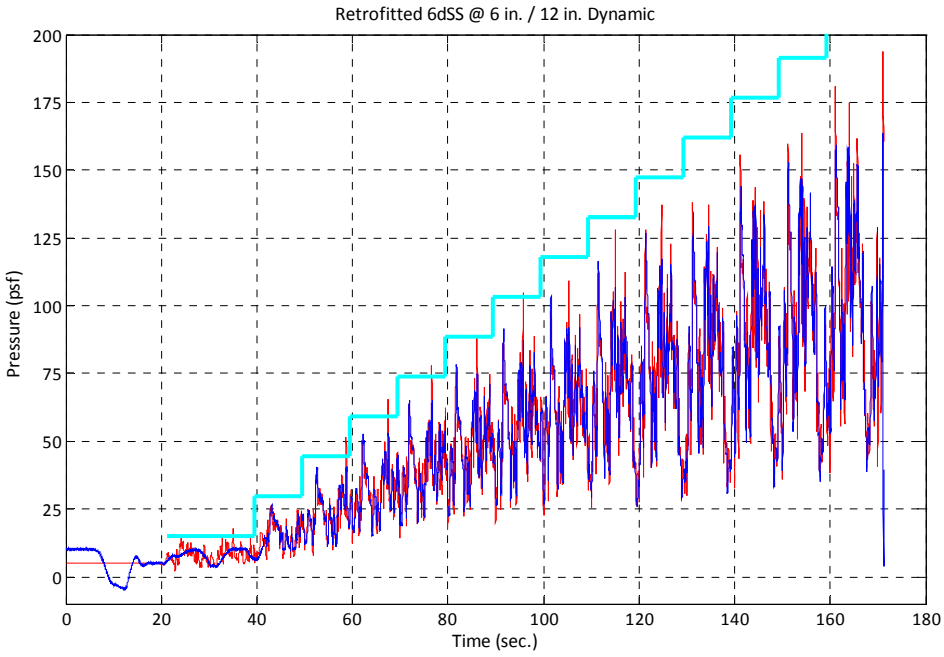


Figure E-22. Summary of pressure time-history for static 6d SS at 6/12 Ret. A-2 panel (6d SS-49)

- Laboratory Fabricated 6d Smooth Shank at 6 in. / 12 in. Ret. A-2 Dynamic (2 panels)

Table E-3. Summary of dynamic pressure trace for 6d SS at 6 in. / 12 in. retrofitted panels (blue – actual and red target)

	Peak 1	Peak 2	Peak 3	Peak	Failure
6d Retrofit Dynamic					
6d SS-50	159 psf @ 161 sec.	155 psf @ 164 sec.	162 psf @ 166 sec.	164 psf @ 171 sec.	164 psf @ 171 sec.
	181 psf @ 161 sec.	175 psf @ 164 sec.	152 psf @ 166 sec.	194 psf @ 171 sec.	194 psf @ 171 sec.
6d SS-51	160 psf @ 161 sec.	157 psf @ 164 sec.	152 psf @ 166 sec.	171 psf @ 173.5 sec.	162 psf @ 173.8 sec.
	181 psf @ 161 sec.	175 psf @ 164 sec.	162 psf @ 166 sec.	169 psf @ 173.5 sec.	174 psf @ 173.8 sec.



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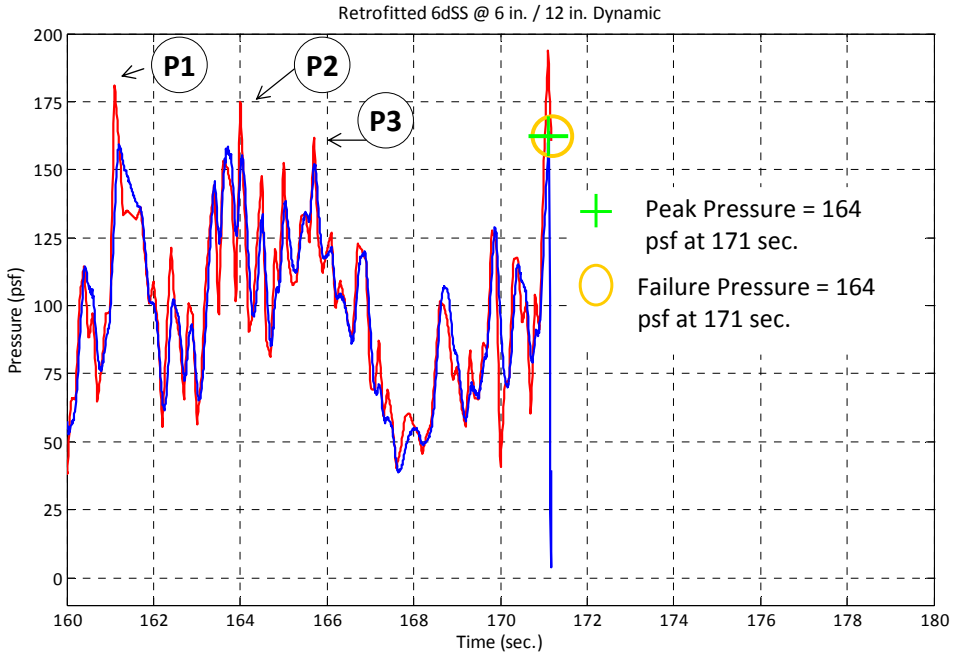
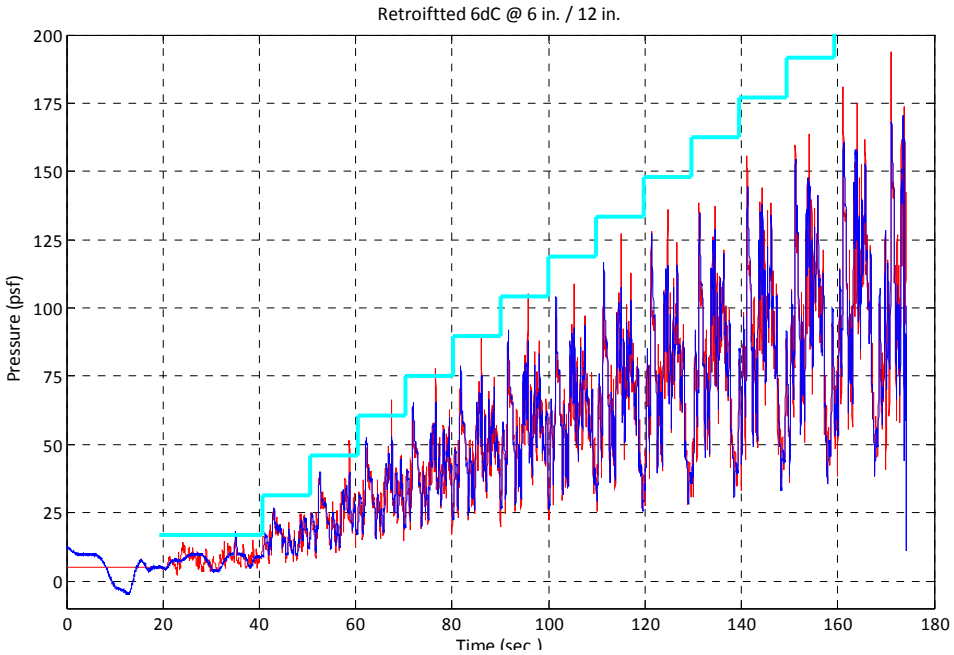


B

Figure E-23. Summary of pressure time-history for dynamic 6d SS at 6/12 ret. A-2 panel (6d SS-50) A) full time-history and B) close up of failure



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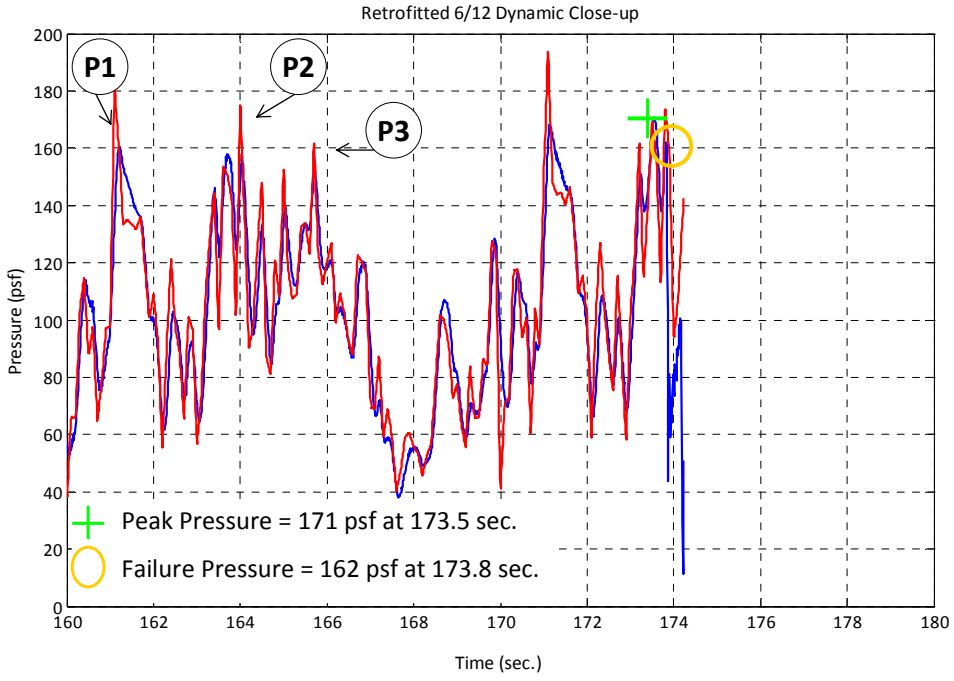


B

Figure E-24. Summary of pressure time-history for dynamic 6d SS at 6/12 ret. A-2 panel (6d SS-51) A) full time-history and B) close up of failure

- Harvested 1.5 in. Staple at 4 in. / 4 in. Static (2 panels)

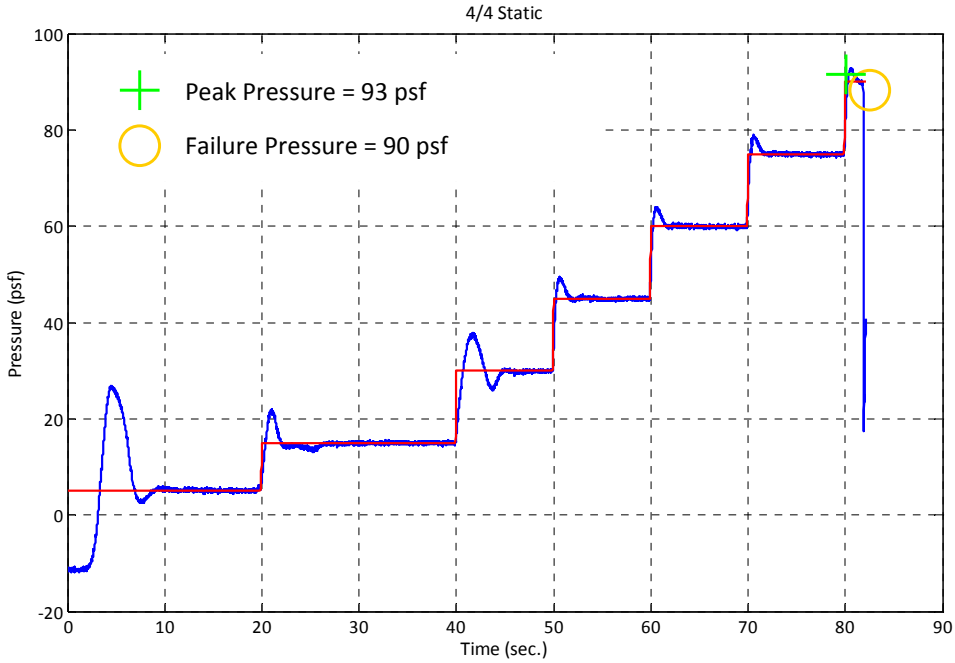


Figure E-25. Summary of pressure time-history for static 1.5 in. Staple at 4/4 (1.5 in. Staple-7)

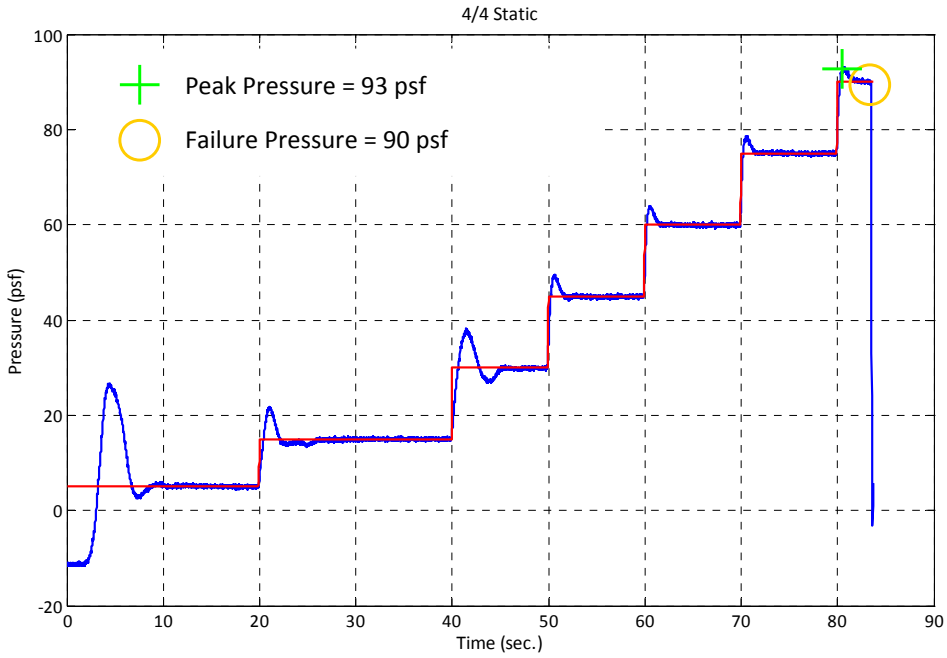
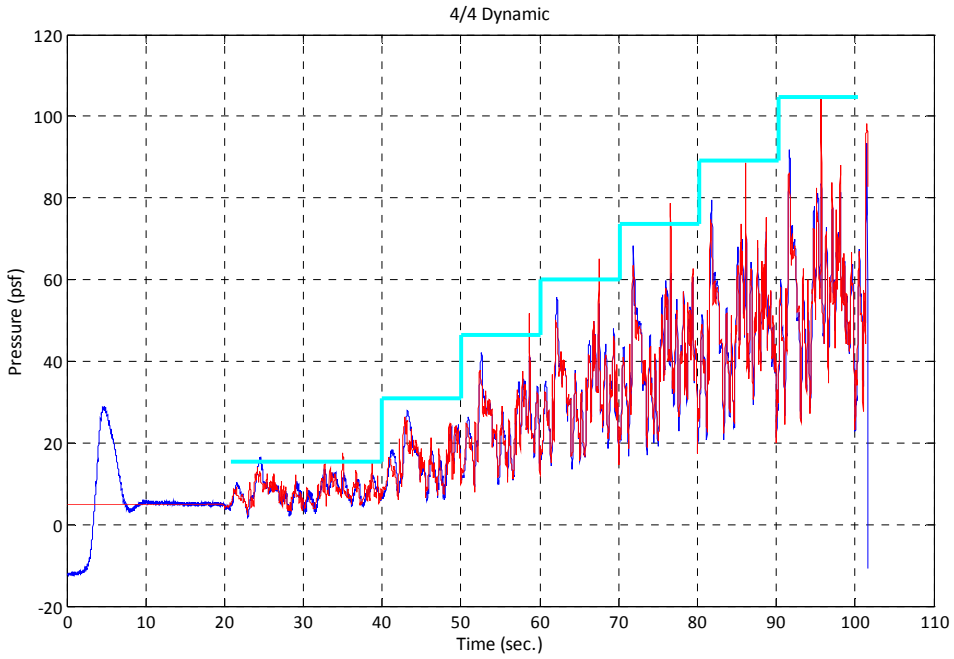


Figure E-26. Summary of pressure time-history for static 1.5 in. Staple at 4/4 (1.5 in. Staple-8)

- Harvested 1.5 in. Staple at 4 in. / 4 in. Dynamic (4 panels)

Table E-4. Summary of dynamic pressure trace for 1.5 in. Staples at 4”/4” panels (blue – actual and red target)

	Peak 1	Peak 2	Peak 3	Peak	Failure
1.5 in. Staple-9	92 psf @ 92 sec.	84 psf @ 96 sec.	82 psf @ 98 sec.	93 psf @ 102 sec.	93 psf @ 102 sec.
	86 psf @ 92 sec.	105 psf @ 96 sec.	89 psf @ 98 sec.	98 psf @ 102 sec.	98 psf @ 102 sec.
1.5 in. Staple-10	55 psf @ 62 sec.	49 psf @ 67 sec.	54 psf @ 68 sec.	61 psf @ 72 sec.	61 psf @ 72 sec.
	50 psf @ 62 sec.	49 psf @ 67 sec.	65 psf @ 68 sec.	63 psf @ 72 sec.	63 psf @ 72 sec.
1.5 in. Staple-11	42 psf @ 52 sec.	36 psf @ 58 sec.	42 psf @ 59 sec.	52 psf @ 62 sec.	37 psf @ 66 sec.
	38 psf @ 52 sec.	38 psf @ 58 sec.	52 psf @ 59 sec.	50 psf @ 62 sec.	41 psf @ 66 sec.
1.5 in. Staple-12	69 psf @ 72 sec.	63 psf @ 77 sec.	61 psf @ 79 sec.	82 psf @ 82 sec.	71 psf @ 89 sec.
	63 psf @ 72 sec.	78 psf @ 77 sec.	62 psf @ 79 sec.	75 psf @ 82 sec.	75 psf @ 89 sec.



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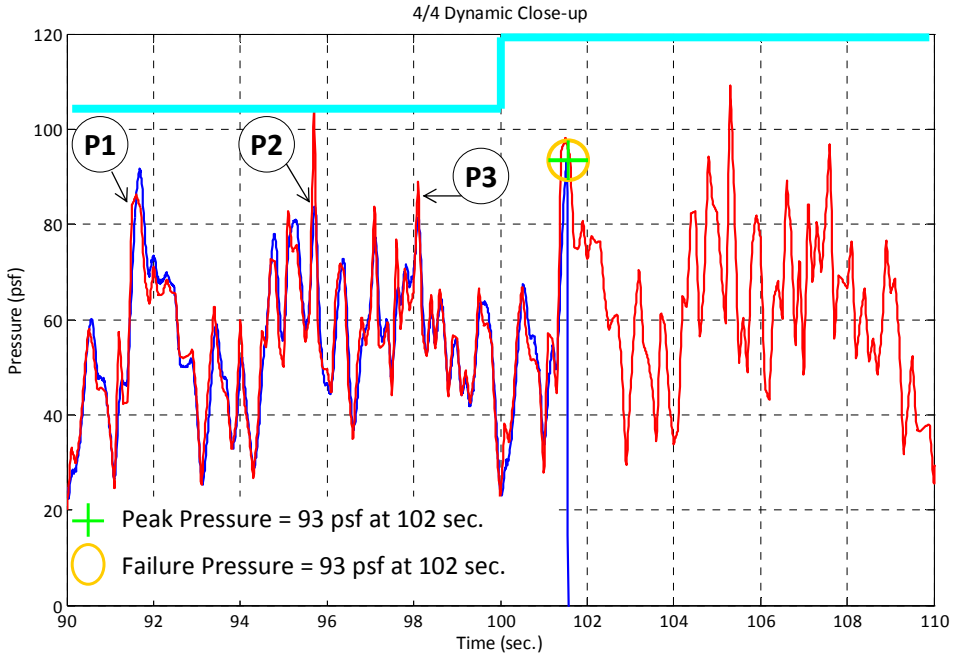
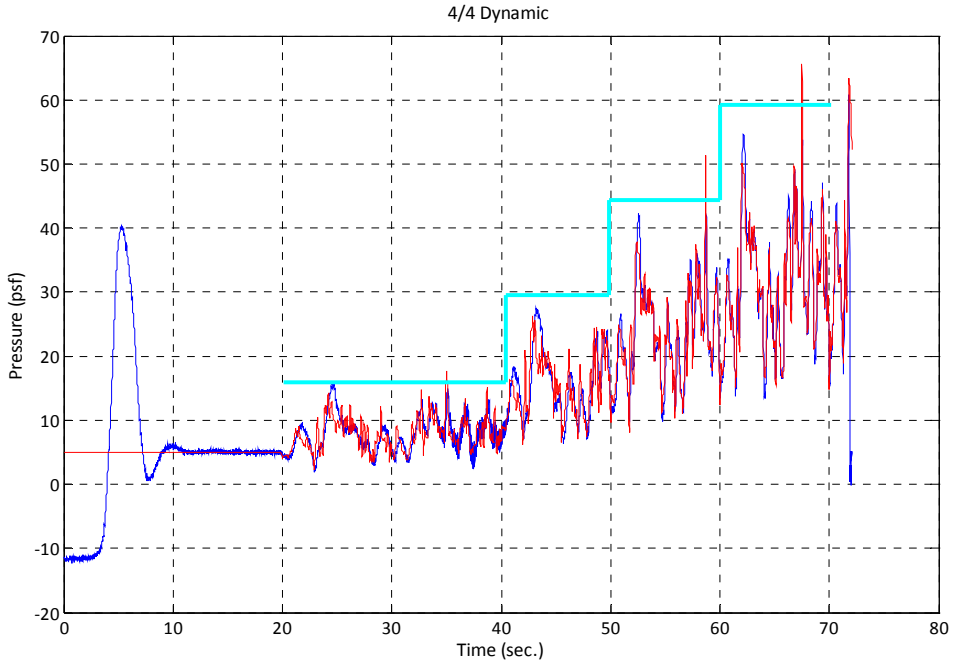


B

Figure E-27. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 (1.5 in Staple-9) A) full time-history and B) close up of failure



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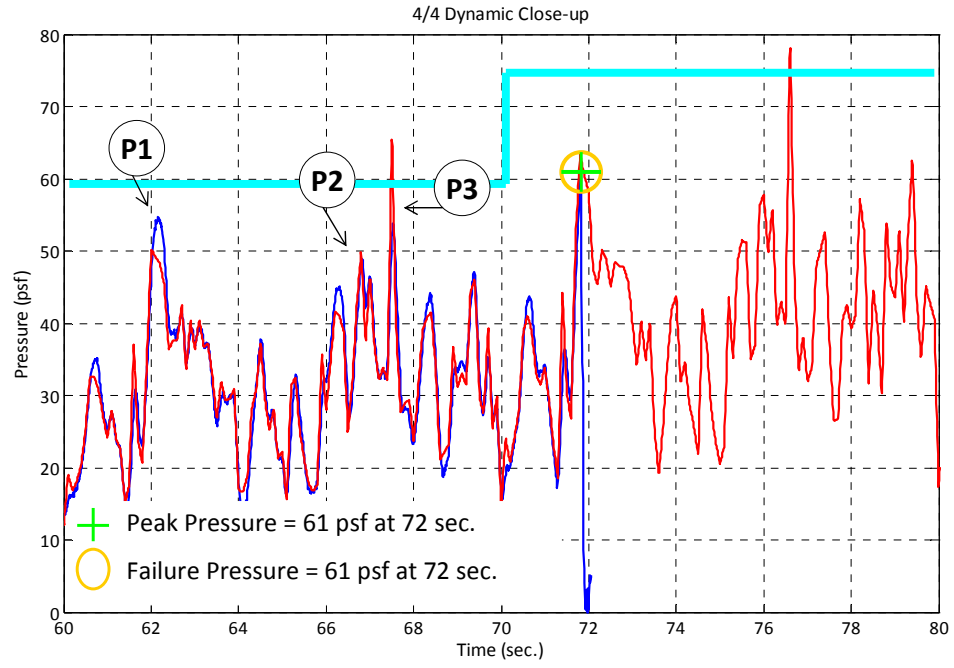
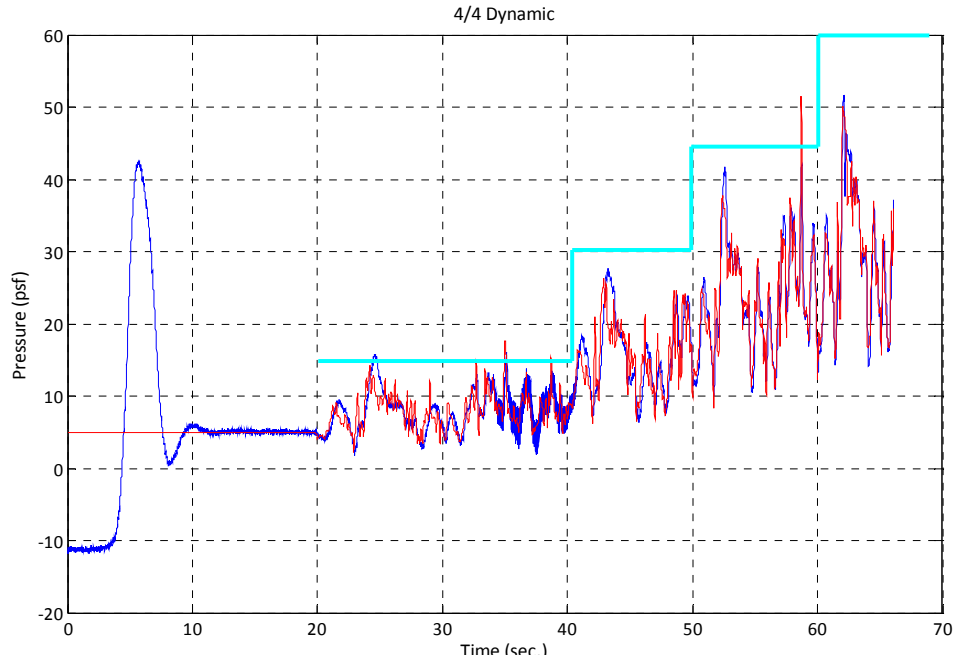


B

Figure E-28. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 (1.5 in. Staple-10) A) full time-history and B) close up of failure



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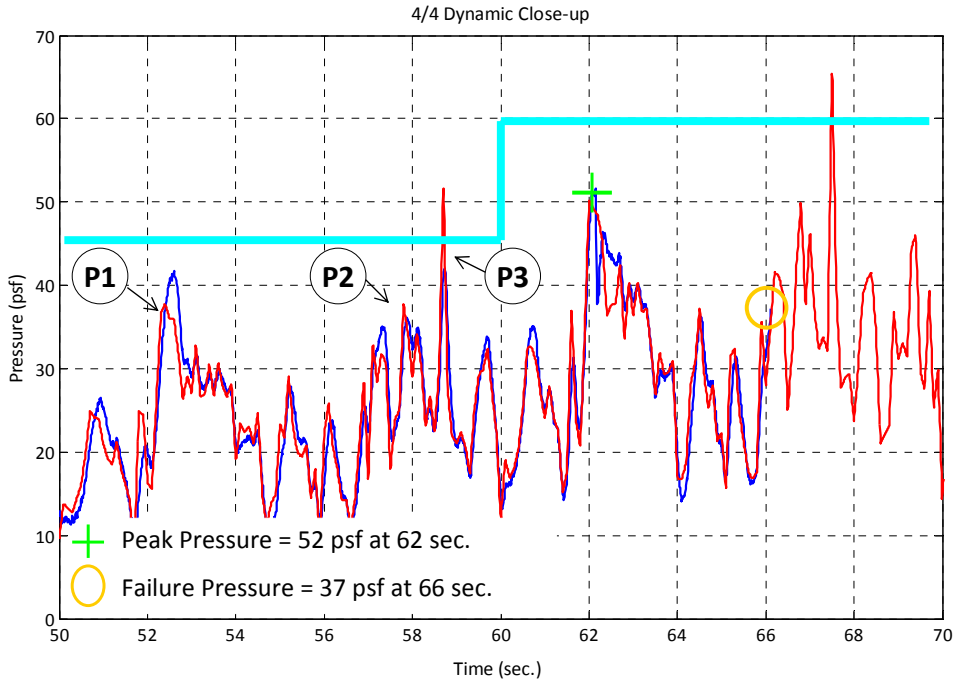
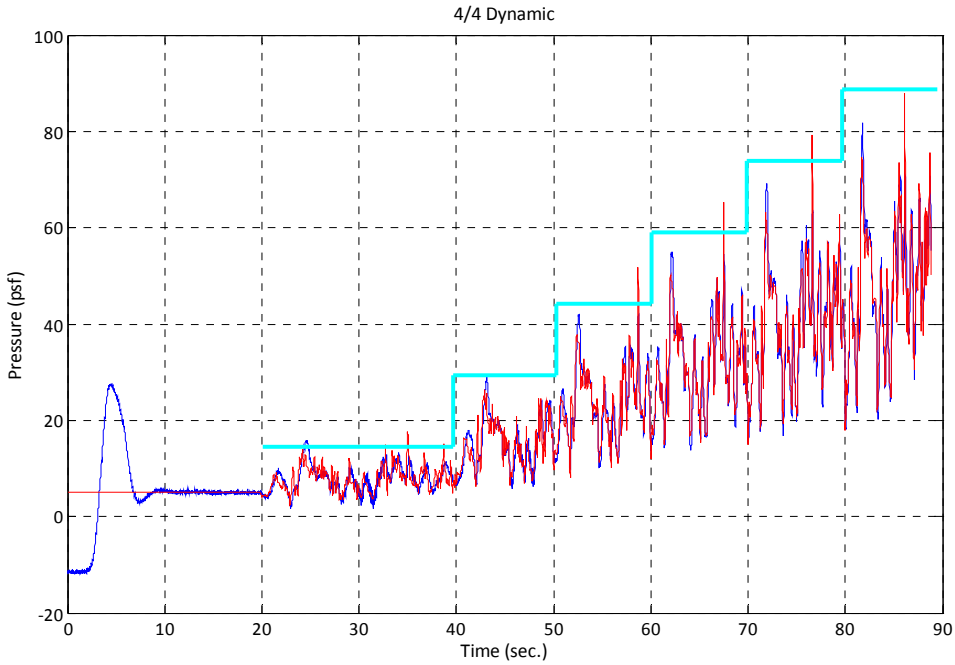


B

Figure E-29. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 (1.5 in. Staple-11) A) full time-history and B) close up of failure



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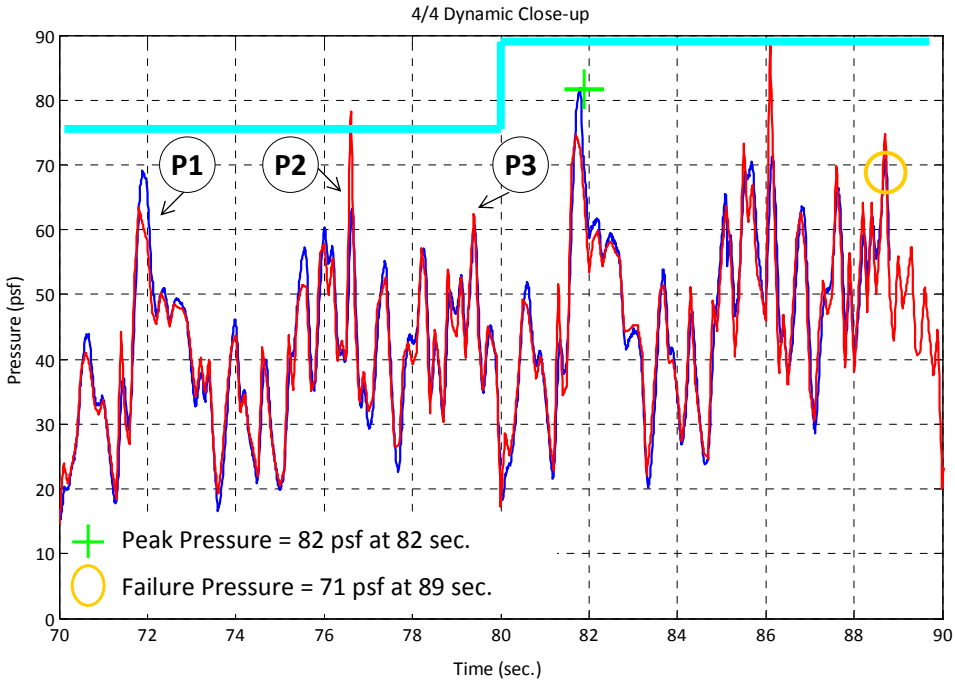


B

Figure E-30. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 (1.5 in. Staple-12) A) full time-history and B) close up of failure

- Harvested 1.5 in. Staple at 4 in. / 4 in. Ret. with A-2 Static (2 panels)

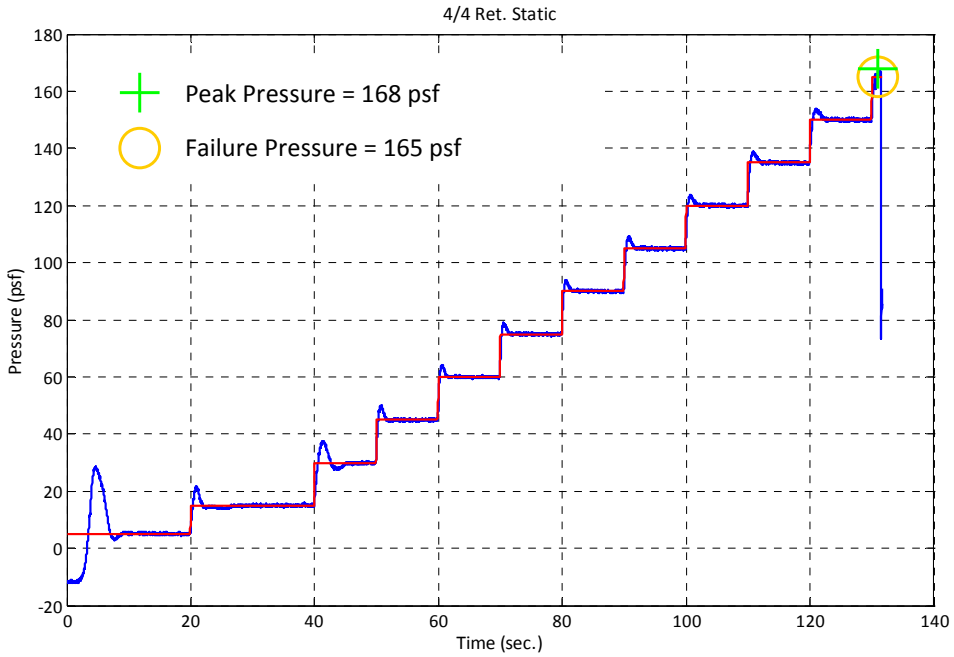


Figure E-31. Summary of pressure time-history for static 1.5 in. Staple at 4/4 with Ret. A-2 (1.5 in. Staple-13)

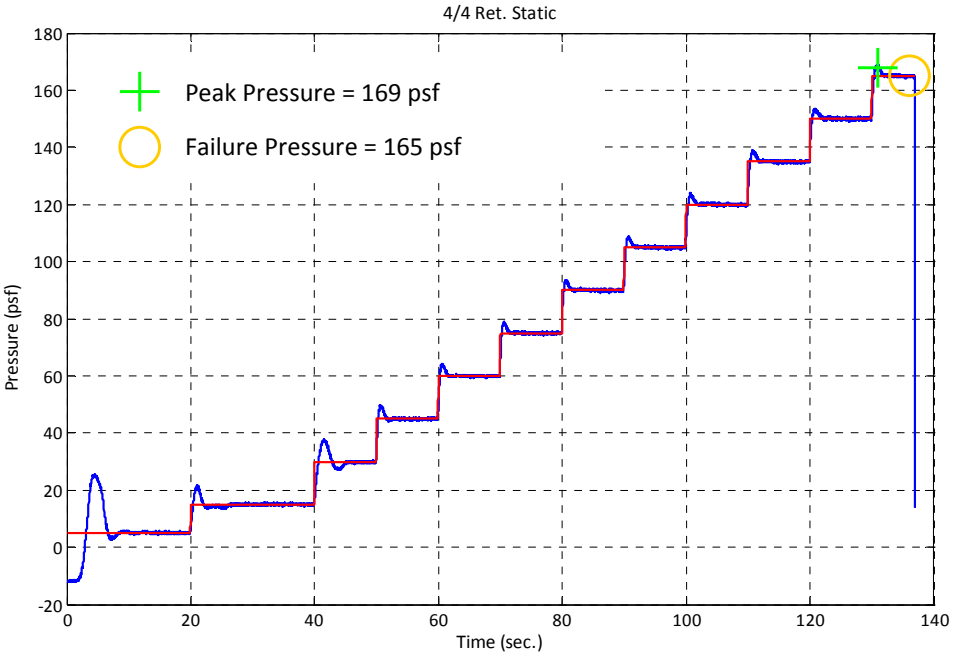
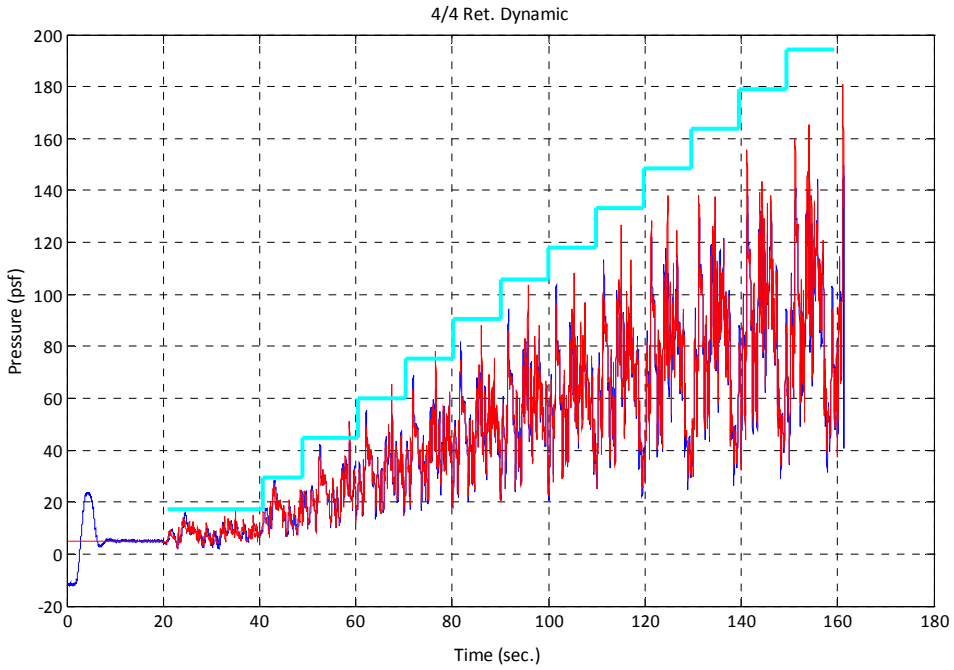


Figure E-32. Summary of pressure time-history for static 1.5 in. Staple at 4/4 with Ret. A-2 (1.5 in. Staple-14)

- Harvested 1.5 in. Staple at 4 in. / 4 in. with Ret. A-2 Dynamic (2 panels)

Table E-5. Summary of dynamic pressure trace for 1.5 in. Staples at 4”/4” with Ret. A-2 panels (blue – actual and red target)

	Peak 1	Peak 2	Peak 3	Peak Pressure	Failure Pressure
1.5 in. Staple-15	141 psf @ 151 sec. 160 psf @ 151 sec.	140 psf @ 154 sec. 164 psf @ 154 sec.	144 psf @ 156 sec. 141 psf @ 156 sec.	150 psf @ 161 sec. 181 psf @ 161 sec.	150 psf @ 161 sec. 181 psf @ 161 sec.
1.5 in. Staple-16	95 psf @ 92 sec. 86 psf @ 92 sec.	85 psf @ 96 sec. 105 psf @ 96 sec.	82 psf @ 98 sec. 89 psf @ 98 sec.	102 psf @ 102 sec. 98 psf @ 102 sec.	90 psf @ 105 sec. 94 psf @ 105 sec.



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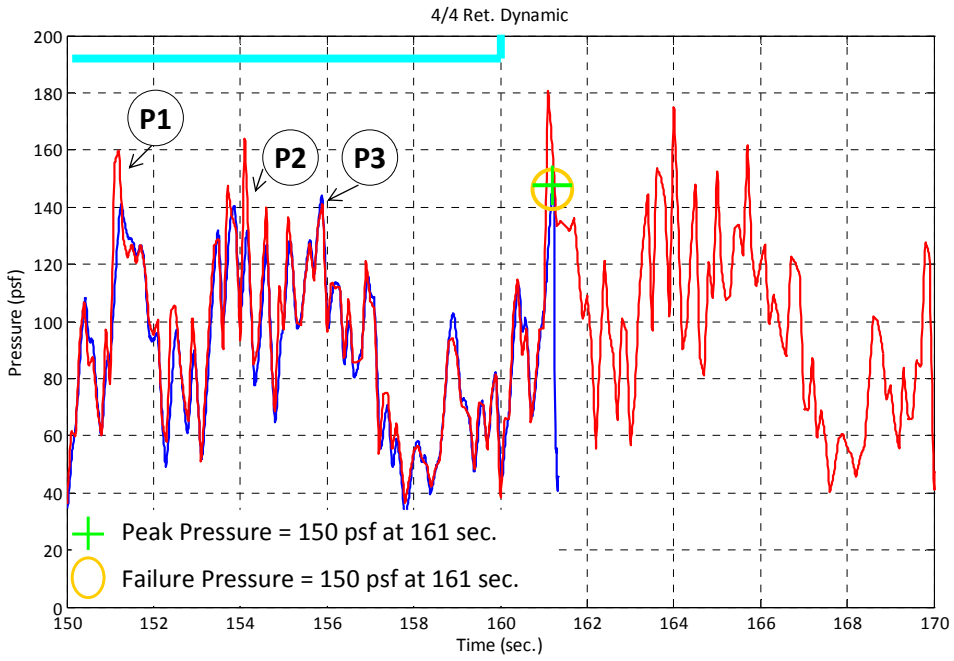
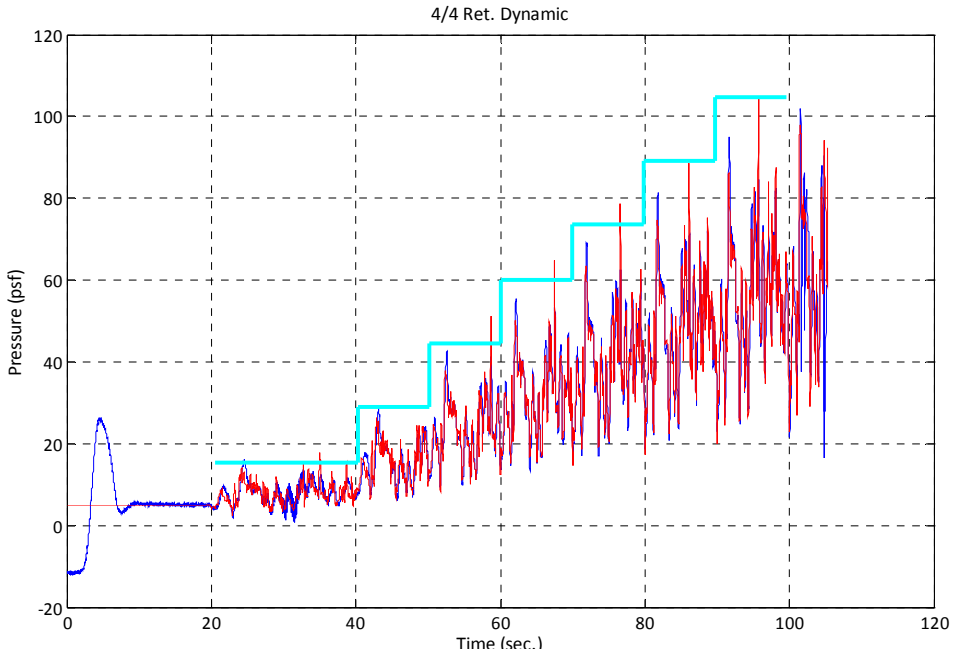


B

Figure E-33. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 with Ret. A-2 (1.5 in. Staple-15) A) full time-history and B) close up of failure



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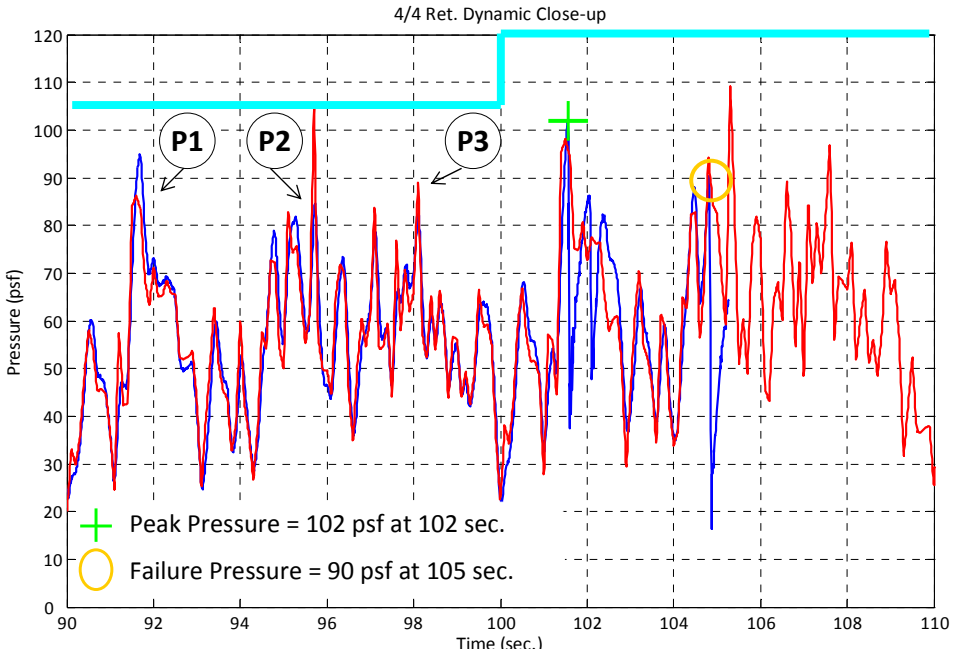

B

Figure E-34. Summary of pressure time-history for dynamic 1.5 in. Staple at 4/4 with Ret. A-2 (1.5 in. Staple-16) A) full time-history and B) close up of failure

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BIOGRAPHICAL SKETCH

The author was born in Davis, California, in 1983. From there he moved with his family to Nigeria then Tanzania. He relocated to the Washington DC area where he played football and skied at every chance. In 2002 he began attending Clemson University where he received his Bachelors of Science in civil engineering. While at Clemson he worked at the Wind Load Test Facility conducting research with the wind tunnel. In 2007 he began attending the University of Florida where he received his Master of Engineering degree in 2009.