DRAFT

Final Report for Project Entitled:

Investigation of Fastening of Wood Structural Panels for Opening Protection
PO Number A95F33

Performance Period: 1/6/2014 – 6/30/2014

Submitted on

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Presented to the

Florida Building Commission
State of Florida Department of Business and Professional Regulation

by

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1. Disclaimer

- The Structural Technical Advisory Committee must approve this report before it is final
- This document applies to Section 1609 of the Florida Building Code 2010: Building.
- Although not explicitly addressed herein, the findings should apply to Section R301.2.1.2 of Florida Building Code 2010: Residential

2. Applicable Sections of the Code

- 1609.1.2, Exception 1, Florida Building Code 2010: Building
- Table 1609.1.2, Florida Building Code 2010: Building
- R301.2.1.2, Exception, Florida Building Code 2010: Residential
- Table R301.2.1.2, Florida Building Code 2010: Residential

3. Executive Summary

a. Description of Issues

The letter from Joe Belcher on behalf of the International Hurricane Protection Association (IHPA) describes the project (included in **Appendix A**). FBC Staff requested that we provide third-party technical input, witness testing, and provide a final review of the report.

b. Major Findings and Recommendations for the Code

The authors' suggested modifications to the 5th Ed. of the Florida Building Code-Building are provided in **Appendix B**. The modifications address the following items:

1. Determination of wind loads for labeling and product approval of impact resistant coverings should be streamlined and made consistent with ASCE 7-10 Components and Cladding (C&C) load calculations. The current approach yields an ultimate load that is 90% of the ASCE 7-10 C&C counterpart. Further, the Code should explicitly define the relationship between ASD and LRFD (ultimate) pressures and the terminology incorporated in the testing application standards, which vary. Appendix B contains the authors' suggestion modifications to the 5th Ed. of the Florida Building Code-Building. We suggest that the Code allows a single prescriptive design (proposed herein) and simplified guidance for designers seeking alternative fastening solutions
2. The wind-borne debris protection fastening schedule (Table 1609.1.2) for wood structural panels is not conservative, e.g. an 8 ft unsupported span of 7/16 OSB with 1 inch of spacing between the fastener and the panel edge will fail in strong winds
3. Structural wood panels are a good choice for a low-cost storm shutters outside of the HVHZ if the fastening schedule is adequate. A one-approach-fits-all, low-cost design was developed and tested for Group R-3 or R-4 occupancy buildings with a mean roof height of 45 feet or less in locations where Vult is 180 mph or less. The system did not exhibit failure during static and cyclic pressure tests derived from ASTM E330 and ASTM E1996. We believe this design reasonably complements the options for metal shutter products, which are generally rated for higher pressures with the tradeoff of increased cost

The following items merit additional study:

4. Predicting the catenary forces is not straightforward given the current knowledge base. The flexibility of 7/16 OSB causes large deflections (~L/15) that cause in-plane forces that combine

with the withdrawal force induced by the out-of-plane wind loading. The lateral (shear) forces are dependent on a combination of factors, including flexural bending of the fasteners or other yield modes (crushing, rotating, hinging) and the free translation of the panel caused by oversizing of the holes that receive the fasteners

5. Designers need conservative yet realistic closed-form solutions to calculate catenary loads in a rational engineering analysis, however the standard equations most likely to be used by a designer are expected to significantly overpredict the lateral forces. Additional experimental research is required to validate closed-form solutions and to establish baseline parameters (e.g. load/slip) for typical panel materials, thicknesses and physical properties (e.g., moisture content). These data can be readily incorporated into existing standards published by APA and AWC that are referenced by the Code
6. Other combinations of hardware and wall types should be studied to determine if the one-approach-fits-all approach proposed in the study is acceptable or requires modifications to achieve suitability. Time and budget precluded the investigators from evaluating other combinations that are prevalent in Florida, however the experimental configuration required to perform this testing is now in place
7. Developing recommendations for larger openings is warranted, especially given the widespread use of sliding glass doors in one- and two-story residential buildings. Additional research is required to develop a prescriptive design solution for large openings that require more than one panel. The APA T460 *Hurricane Shutter Design Considerations for Florida* provides a logical starting point for designing multi-panel configurations

4. Scope of Work

a. Original

- Provide consultation to IHPA on the experimental design
- Witness testing at the certified product testing laboratory
- Interpret results, determine whether the problem requires action, and produce a report that explains the results and implications for the Code

b. Modified

- Carry out tests in-house using a pressure loading actuator that was specifically designed to apply time-varying load conditions
- Perform finite element analysis to estimate the catenary forces
- Interpret results, determine whether the problem requires action, and produce a report that explains the results and implications for the Code
- These modifications were approved on April 18, 2014 by the Program Manager

5. Deliverables

- A report providing technical information on the problem background, results and implications to the Code submitted to the Program Manager by June 15, 2014
- A breakdown of the number of hours or partial hours, in increments of fifteen (15) minutes, of work performed and a brief description of the work performed. The Contractor agrees to provide any additional documentation requested by the Department to satisfy audit requirements

6. Detailed Project Description

The investigators convened an oversight committee formed by members of APA, the American Wood Council (AWC) and the International Hurricane Protection Association (IHPA) to discuss issues related to use of structural wood panels for opening protection. Two teleconferences were held.

During the first meeting (January 15, 2014), the IHPA raised four issues:

1. Limited availability in Florida to find fasteners that satisfy requirements for embedment and permanent installation
2. Inadequate guidance for large openings, such as sliding glass doors
3. Inadequate resistance to catenary forces caused by out-of-plane deflection of the structural wood panel
4. Inadequate/missing edge distance requirements

Missile impact resistance was deemed to not be an issue, thus the discussion mainly centered on resistance to out-of-plane wind loads. APA, with input from AWC, presented a proposal to revise language based on *APA T460, Hurricane Shutter Design Considerations in Florida*. Major changes included the addition of stiffeners and removal of 7/16 in thick structural wood panels. The group concluded the discussion with the origin of the original provisions. Ultimately, it was decided to research the background further.

In the second meeting (February 24, 2014), APA and AWC presented a revised version of the proposed code modification that excluded earlier recommendations for panel stiffeners. Basis research conducted by the Institute for Business & Home Safety (IBHS) and Applied Research Associates was discussed. The APA contacted several individuals responsible for the current language in the code and determined that they "…were based on a code change proposal submitted by Institute for Business and Home Safety (IBHS). In a discussion with IBHS, it was evident that the current code provisions are originally intended to provide a level of opening protection against possible wind-born debris impacts during a high wind event … The 7/16 inch wood structural panels were tested by IBHS at … Clemson University … in accordance with ASTM E1996 and SBCCI SSTD-12. The test results, including impact tests and cyclic pressure tests, supported the current code provisions and adopted by the IBC, IRC, and FBC committees. Note that the intent of the exception in Sections R301.2.1.2 and 1609.1.2 was intended to address the debris impact…"

The group did not reach consensus on how to quantify wind loads on impact resistant coverings, specifically as it related to catenary forces caused by in-plane loading. One consideration was that structural wood panel opening protection was sufficiently air permeable to reduce cladding loads. Later, the stakeholders agreed that the issue of air permeability was moot. In the US, the glazing is considered sacrificial, thus the possibility that the air cavity between the panel and the window would equalize with the external load was irrelevant. The group also did not reach consensus on revising the specifications for fasteners.

Ultimately, the committee supported Dr. Masters' proposal to conduct a series of experiments to study panel behavior. FBC staff approved this testing on April 18, with planning beginning shortly thereafter. Testing commenced May 27 using the Pressure Loading Actuator (PLA) system at UF. The PLA is a robust simulation tool to recreate time-varying wind load conditions in a pressure chamber. More details about the system may be found in Kopp et al. (2010). Nearly 70 tests were performed on different configurations of full and strip panel coverings under monotonically increasing, static and cyclic pressure loading. Concurrently, theoretical and computational modeling of the panel behavior was performed.

The remainder of this section discusses the cumulative findings of this research and makes recommendations for future work.

Finding #1. Determination of wind loads for labeling and product approval of impact resistant coverings should be streamlined and made consistent with ASCE 7-10 Components and Cladding (C&C) load calculations. The current approach yields an ultimate load that is 90% of the ASCE 7-10 C&C counterpart. Further, the Code should explicitly define the relationship between ASD and LRFD (ultimate) pressures and the terminology incorporated in the testing application standards, which vary.

This issue primarily addresses how C&C loads are calculated for impact resistant coverings, which has direct bearing on how to refine the Code currently addresses the structural wood panel option. We conclude that the back-and-forth conversion between ASD and LRFD unnecessarily complicates the calculation of wind loading and ultimately results in an unjustified reduction in wind loading. Consider how a designer should interpret the current Code:

1. Section 1609.1 Exception 1 gives an ASCE 7-05 basic wind speed = 140 mph. This value must be converted to an ultimate (i.e. ASCE 7-10) wind speed via Equation 16-32:

$$V_{asd} = V_{ult}\sqrt{0.6}$$

   where V_{asd} = nominal design wind speed and V_{ult} = strength design wind speeds determined from Figures 1609A, 1609B, or 1609C.

2. The designer must select from one of two methods in ASCE 7-10 (Part 1 or 2) to calculate the ultimate positive and negative pressures

3. The resultant pressures are back converted to ASD by manipulating Equation 16-32 as follows

$$p_{asd} \propto V_{asd}^2$$

$$p_{ult} \propto V_{ult}^2$$

$$V_{asd} = V_{ult}\sqrt{0.6}$$

$$(V_{asd})^2 = \left(V_{ult}\sqrt{0.6}\right)^2$$

$$V_{asd}^2 = 0.6V_{ult}^2$$

$$p_{asd} = 0.6p_{ult}$$

4. The ASD pressure is then multiplied by a 1.5 factor of safety

5. The designer must then determine which of these loads (i.e. ASD or ultimate) corresponds to the terminology in the test standard. The language in these standards varies between “design”, “test” and “proof”

Appendix C contains the load calculations for the basis of the static and cyclic pressure tests to evaluate the “one-approach-fits-all” system described subsequently. It is evident that the more straightforward approach would be to define the ultimate wind speed and use the C&C loads determined through ASCE 7-10 as the requirements for the testing protocol. This eliminates steps 1 and 4 above. The ASD load can be computed directly from Step 3 (i.e. multiply the loads from ASCE 7-10 by 0.6).

Streamlining this approach will also make the testing application standards consistent with ASCE 7-10 C&C loads. The current approach results in an unjustified reduction in load (compare the blue shaded cells to the orange shaded cells). The current product approval requirements are 90% of the ASCE 7-10 counterparts. This inconsistency should be reconciled as soon as possible. We also suggest providing a simple and easy-to-use lookup table that explicitly defines which loads should be applied in each testing application standard. It is not reasonable to expect contractors, most product manufacturers, or others who are not hands-on with everyday engineering to readily interpret the Code in its current form despite its relatively straightforward function.

Finding #2. The wind-borne debris protection fastening schedule (Table 1609.1.2) for wood structural panels is not conservative, e.g. an 8 ft unsupported span of 7/16 OSB with 1 inch of spacing between the fastener and the panel edge will fail in strong winds

IHPA sponsored a certified product approval testing laboratory to test panels built to the minimum allowable fastener schedules in Table 1609.1.2 and found that the panels failed at 25-35 psf, which is well below the allowable limits. The proposed guidelines (described in the next section) reduce the span to a maximum of 42 inches and incorporate a 2 in distance from the fastener to the edge of the panel to improve the resistance to wind pressure.

Finding #3. Structural wood panels are a good choice for a low-cost storm shutters outside of the HVHZ if the fastening schedule is adequate. A one-approach-fits-all, low-cost design was developed and tested for Group R-3 or R-4 occupancy buildings with a mean roof height of 45 feet or less in locations where Vult is 180 mph or less. The system did not exhibit failure during static and cyclic pressure tests derived from ASTM E330 and ASTM E1996. We believe this design reasonably complements the options for metal shutter products, which are generally rated for higher pressures with the tradeoff of increased cost

The technical direction of research focused on adapting the prescriptive guidance in Exception 1 of Section/Table 1609.1.2 to withstand the out-of-plane wind loading tabulated in **Appendix C**. We opted to continue using the code minimum panel thickness (7/16 in) because the stakeholder oversight committee deemed impact resistance to not be an issue. Applied Research Associates (2003, prepared by T.R. Reinhold) found that 7/16 OSB panel meets impact resistance for regions outside the high velocity hurricane zone, i.e. the shutters can resist ASTM E1996 Missile C (a 4.5-pound 2x4 traveling at 27 mph).

Multiple changes to the basis specification set forth in Table 1609.1.2 are proposed. They include:

- Installing the fasteners on the panel edges (long sides) to reduce the unsupported span from ~8 ft to ~4 ft. The Code currently allows fasteners to be installed on the ends of the panel (short sides)
- Choosing a single fastener configuration. Hanger bolts were chosen for ease of installation, assuming that a homeowner or contractor would choose this option over a No. 8 and 10 wood-screw-based anchor system. The fastening system that is the basis of the recommendations consists of:
    - 1/4 X 3-7/16 PanelMate Plus 305SS
    - 1/4 X 1 in Fender Washer
    - 1/4-20 Hexnut
- Increasing the distance from the fastener to the edge of the panel from 1 in to 2 in to prevent tear-out
- Requiring slightly oversized holes to reduce the catenary force (and to make it easy to hang)
- Requiring 3 in wall overlap to limit inward deflection
- Using large (1 in) washers or washered wingnuts to prevent pull-through

Two fastener embedment lengths were studied to evaluate their performance under load and to perform an indirect comparison of the change in lateral stiffness caused by the panel hole eccentrically bearing on the hanger bolt.

- Case 1: 2 in of embedment, i.e. full penetration of the lag into the wood buck

– Case 2: 1.25 in of embedment plus a 0.75 in “spacer” to simulate the offset created by an exterior finish such as fiber cement board or stucco

Case 1 and Case 2 configurations were evaluated under static and cyclic pressure tests derived from ASTM E330 and ASTM E1996. Strip and full panel tests were performed. **Appendix D** contains drawings of the test frame and the panel as tested. **Appendix E** contains records or pressure and deflection for both tests. The cyclic test was subdivided into a positive sequence and negative sequence to provide a break for the polyethylene sheeting to be switched from one side of the panel to the next.

The systems did not fail and the observed deflections did not significantly vary between Case 1 and 2, which suggests that any reduction in the catenary force in the connection is due to (a) the gap between the hole and the fastener or (2) the material crushing or reorienting.

Finding #4. Predicting the catenary forces is not straightforward given the current knowledge base. The flexibility of 7/16 OSB causes large deflections (~L/15) that cause in-plane forces that combine with the withdrawal force induced by the out-of-plane wind loading. The lateral (shear) forces are dependent on a combination of factors, including flexural bending of the fasteners or other yield modes (crushing, rotating, hinging) and the free translation of the panel caused by oversizing of the holes that receive the fasteners

Finite element models were developed to assess the expected lateral force on panel fasteners and validated against the deflection measured during for experimental strip test. The system that was modeled consisted of a single tributary width of panel, with a single bolt centered on each end (Figure 1). In this configuration, the panel acts undergoes beam action and catenary forces are developed, resulting in both a withdrawal force and a lateral force on the fastener (Figure 2). Each model was tested under two load configurations: toward the building and away from the building.

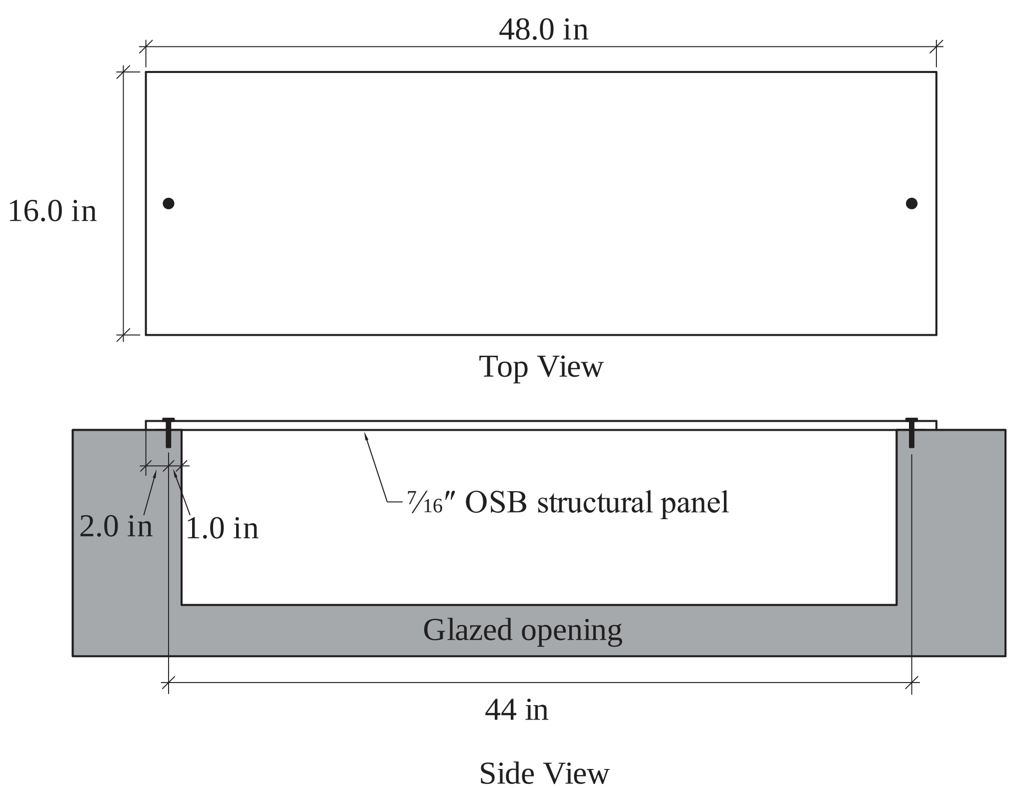

Figure 1. Panel used in experimental testing

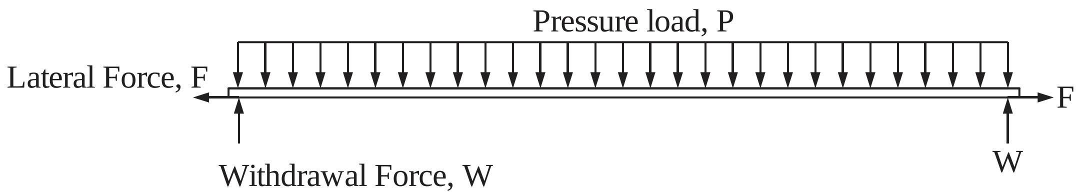

Figure 2. Free body diagram of panel undergoing beam action with catenary effects.

The panel was modeled with 4-node shell elements, using a large-displacement formulation to capture catenary effects. Based on results from the 4-point bending tests, a linear-elastic constitutive model was used for the OSB, with a Young's modulus of 215 ksi. To represent the surface of the building, contact springs—effectively rigid in compression but with no stiffness in tension—were installed at both ends, on every node located between the edge of the panel and the edge of the glazed opening, preventing the ends from moving towards the building in the z-direction. The contact springs used a small-displacement formulation so that the direction of the contact force would not change as the panel deformed.

One node on each end represented the fastener, with a boundary condition applied to prevent movement in the y- and z-directions. Acting together, the z boundary condition and the contact springs held the ends in place, so that the panel acted like a beam with near-complete rotational fixity at the supports. Additionally, lateral spring elements were installed at the fastener nodes to allow for partial restraint of the panel ends in the x-direction, representing the load-slip relationship of the fastener connection. Equation 10.3-1 in the NDS (citation) includes a design equation for the load-slip modulus, λ:

$$\lambda = 180{,}000 D^{1.5}$$

where D is the diameter of the fastener. For the ¼ -in. hanger bolts used in the experimental apparatus, the NDS predicts a load-slip modulus of λ = 22,500 lbs./in. However, this equation is based on tests of Douglas Fir timbers (rather than OSB) with bolts loaded in pure shear (Zahn, 1991). The actual load-slip modulus of the experimental apparatus is likely to have been much smaller. To assess the effect of the load-slip modulus on the panel response, four different lateral stiffness cases were modeled in each load direction:

- <u>Laterally rigid</u>: The lateral springs were effectively rigid, providing total lateral fixity.
- <u>High lateral stiffness</u>: λ = 22,500 lbs./in. (the NDS value).
- <u>Medium lateral stiffness</u>: : λ = 5625 lbs./in. (one quarter of the NDS value)
- <u>Low lateral stiffness</u>: λ = 1125 lbs./in. (one twentieth of the NDS value)

The finite element results show that the lateral stiffness at the supports has a large influence on the panel deflection. For the toward-the-building load direction, the low lateral stiffness case appears to be the best fit to the measured deflection data (Figure 3). The low load-slip relationship in the experimental apparatus (relative to the Equation above) can be attributed to a combination of factors, including flexural response in the fasteners, the use of oversized bolt holes, and the use of OSB panels with a Young's modulus much lower than natural timber.

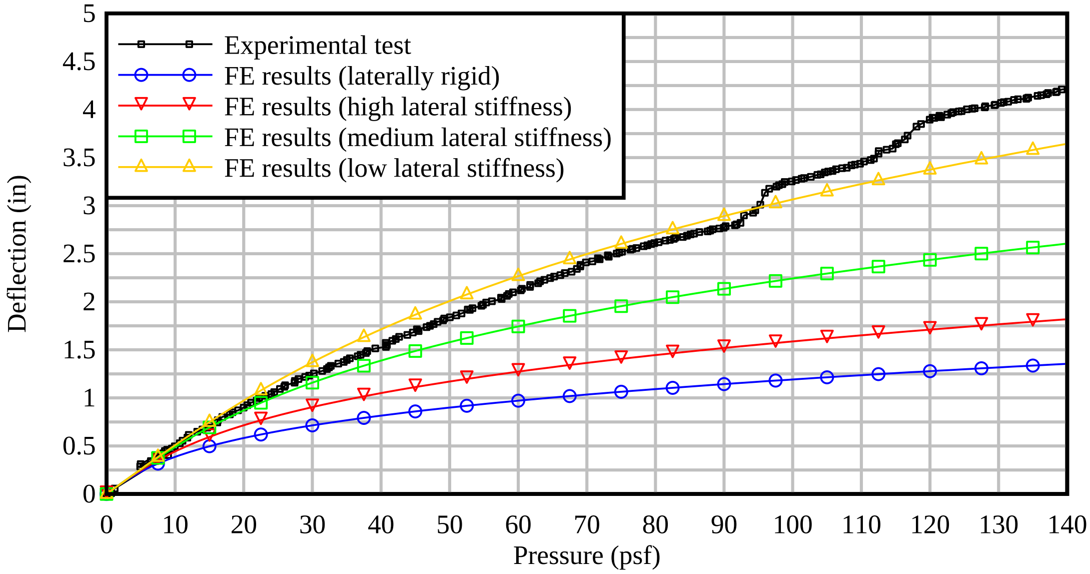

Figure 3. Comparison between experimentally-measured deflections and FEA results in the towards-the-building load direction

For the away-from-the-building load direction, none of the lateral stiffness cases provided an especially good fit to the experimental data over the full load range (Figure 4). Initially, the deflection is lower even than the laterally rigid case, and as the load increases the experimental curve crosses through each of the lower stiffness response curves in turn. The exact reason for this is unclear, but the experimental result seems to indicate a non-linear load-slip relationship in the fasteners which was not accounted for in the model. The short timeframe allotted for experimentation made a more thorough investigation unfeasible. However the low-lateral-stiffness case remains conservative over the largest load range, and is the best choice for developing short-term design recommendations.

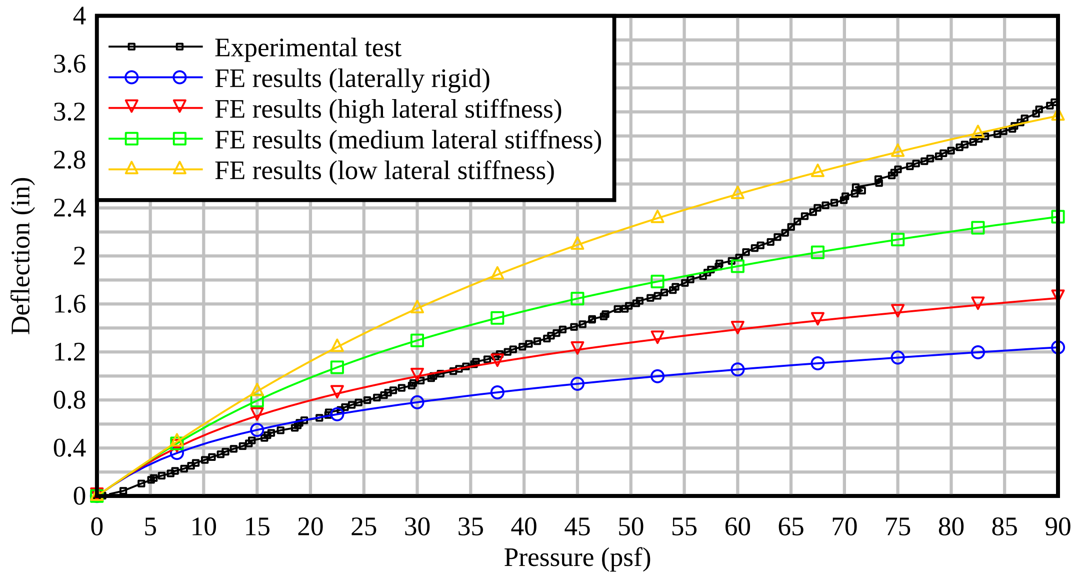

Figure 4. Comparison between experimentally-measured deflections and FEA results in the away-from-the-building load direction

The lateral stiffness was also found to have a significant influence on the lateral forces exerted on the fasteners. In the laterally rigid case, when the panel ends are completely prevented from moving inward, the panel must elongate in order to deflect, developing considerable axial tension that reacts against the fastener, producing enormous lateral loads. As the lateral stiffness decreases and the panel ends move closer together, the elongation is reduced, relaxing the axial tension and reducing the loads (Figures 5-6). Because the length scale of the elongation is very small (it is considered negligible in most structural analysis applications), a very small contraction of the supports can relax a large percentage of the axial tension, significantly reducing the lateral force. In realistic situations, it is expected that the actual lateral load is closer to the low lateral stiffness case that the laterally rigid case.

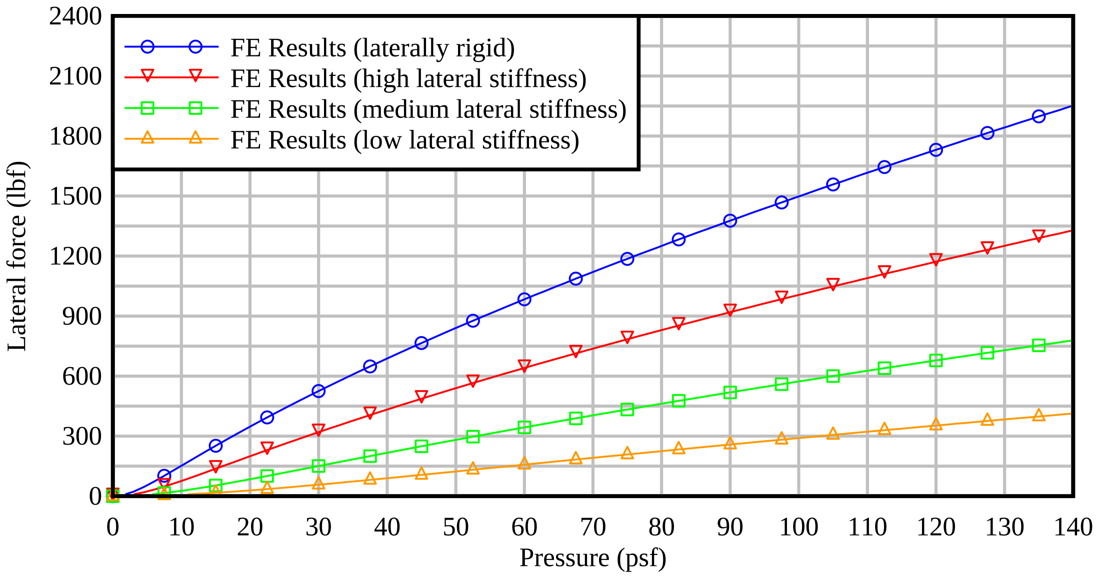

Figure 5. Comparison between FE lateral force results for different lateral stiffnesses in the toward-the-building load direction

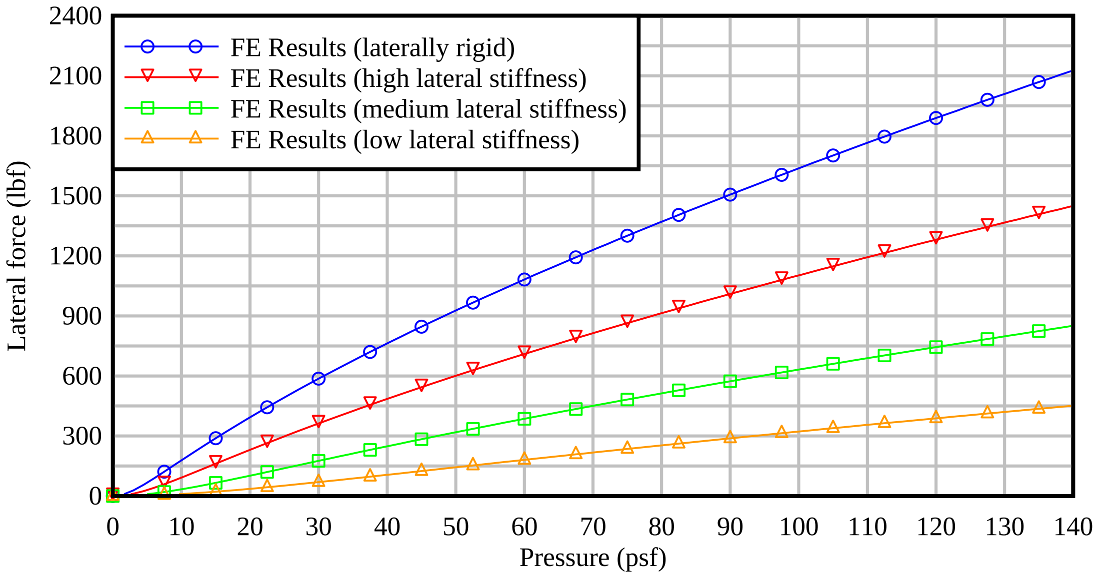

Figure 6. Comparison between FE lateral force results for different lateral stiffnesses in the away-from-the-building load direction

For deflection limit states, the standard equation for a linear beam with fixed-fixed support conditions is recommended:

$$\delta = \frac{wL^4}{384EI}$$

At high levels of load, this equation is conservative, relative to the measured deflections in both load directions (Figures 7-8). Theoretically, at low levels of load, the formula can be slightly unconservative (as observed in Figure 7) because the beam response is still largely linear and the true support conditions are somewhere between fixed-fixed and pinned-pinned. However, after non-linear effects start to influence the beam response (in the load range where most design checks take place) the ideal fixed-fixed beam deflection provides a useful upper bound.

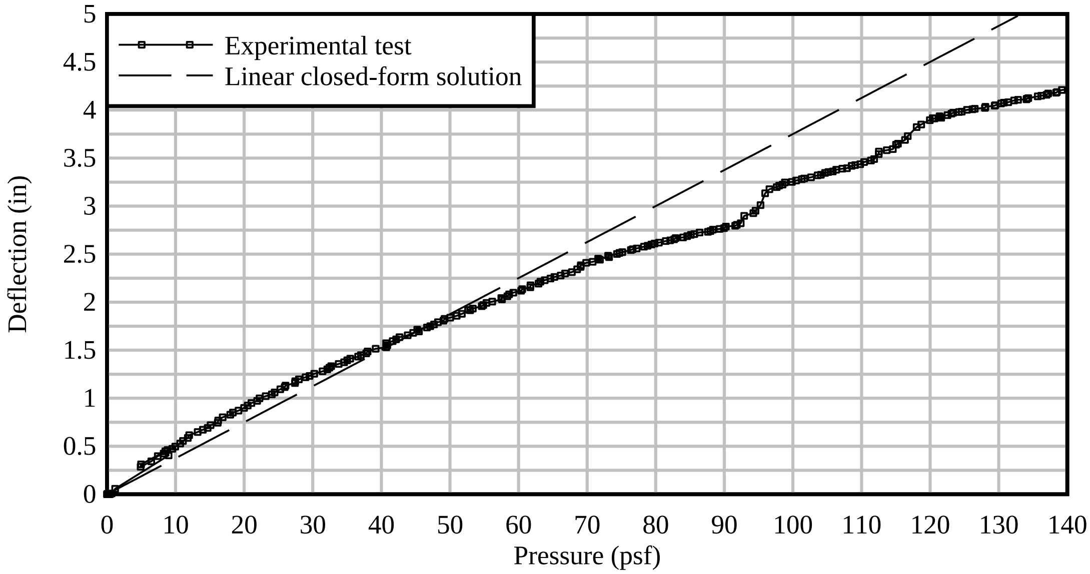

Figure 7. Validation of recommended closed form solution for deflection (load applied toward building)

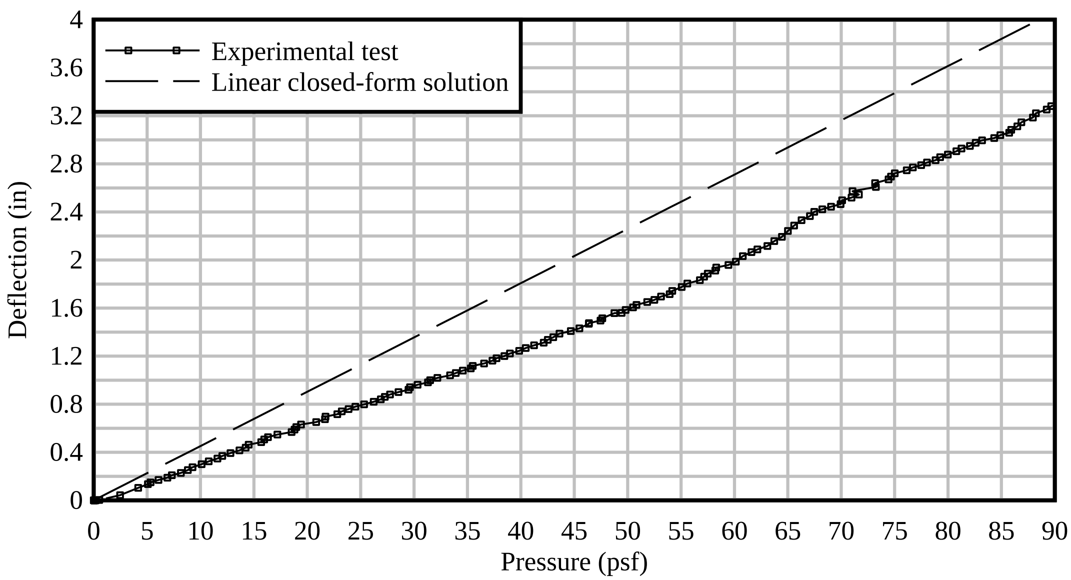

Figure 8. Validation of recommended closed form solution for deflection (load applied away from building)

7. Suggestions for Future Research

Finding #5. Designers need conservative yet realistic closed-form solutions to calculate catenary loads in a rational engineering analysis, however the standard equations most likely to be used by a designer are expected to significantly overpredict the lateral forces. Additional experimental research is required to validate equations and to establish baseline parameters (e.g. load/slip) for typical panel materials, thicknesses and physical properties (e.g., moisture content). These data can readily incorporated into existing standards published by APA and AWC that are referenced by the Code

See Figures 5-6, which shows the range of catenary forces as a function of the lateral stiffness of the connection. Attempts to apply closed-form solutions readily accessible in the public domain to model these loads were hindered by a lack of knowledge about the load/slip relationship at the panel. Preliminary results from lateral load testing on a uniform testing machine found that stiffness was on the order of 600 lb/in, which is almost 1/40th of the value for a ¼ in diameter dowel type fasteners in a wood-to-wood connection (NDS 10.3.6). We also found that applying conservative equations that assume an infinite lateral stiffness likely overestimate the actual catenary loads.

Finding #6. Other combinations of hardware and wall types should be studied to determine if the one-approach-fits-all approach proposed in the study is acceptable or requires modifications to achieve suitability. Time and budget precluded the investigators from evaluating other combinations that are prevalent in Florida, however the experimental configuration required to perform this testing is now in place

Testing of CMU wall systems is now underway, and this report will be updated as new findings become available. Many other combinations of fastener type and finished wall assemblies exist. Additional testing is warranted if comprehensive guidelines are sought.

Finding #7. Developing recommendations for larger openings is warranted, especially given the widespread use of sliding glass doors in one- and two-story residential buildings. Additional research is required to develop a prescriptive design solution for large openings that require more than one panel. The APA T460 *Hurricane Shutter Design Considerations for Florida* provides a logical starting point for designing multi-panel configurations

Sliding glass doors and field/factory mulled windows are commonly found in one- and two-story single family homes. The recommendations provided herein are not applicable to these openings in most cases because the span is larger than 4 ft. We consider this component (as well as the items described in Finding #5) to be essential missing pieces to finalizing recommendations.

8. References and Project Material

- Provided by IHPA
  - Letter to Florida Building Commission dated September 16, 2013
  - ATI Test Report dated December 1, 2011. Videos:
    - http://youtu.be/iDLzf0wF0Zc
    - http://youtu.be/2fcv5GD_qUM
    - http://youtu.be/BdSNDsScIcE
  - Letter from Engineering Express dated January 10, 2014

- Provided by APA
  - APA T460 Hurricane Shutter Design Considerations in Florida
  - Applied Research Associates. 2001. Impact and Pressure Testing of Hawaii Hurricane Relief Fund Window Protection Design.
  - Applied Research Associates. 2003. Impact and Pressure Testing of Florida Building Code Minimum Plywood and OSB Shutter Systems.
  - Institute for Business & Home Safety. 2012. Industry Perspective: Impact Resistance Standards. In: Natural Hazard Mitigation Insights No. 12
- Other resources
  - Kopp GA, Morrison MJ, Gavanski E, Henderson DJ, Hong HP. The Three Little Pigs' Project: hurricane rick mitigation by integrated wind tunnel and full-scale laboratory tests. Natural Hazards Review 2010; November, 151-161.
  - Young, W. C. and Budynas, R. G. (2002). Roark's Formulas for Stress and Strain: 7th Edition, McGraw–Hill, New York, NY.
  - Zahn, J. (1991). "Design Equation for Multiple-Fastener Wood Connection", Journal of Structural Engineering, ASCE, Vol. 117, No. 11, pp. 3477–3486.

Appendix A. Letter from the International Hurricane Protection Association

JDB CODE SERVICES, INC.

September 16, 2013

Florida Building Commission
C/O Mo Madani, DBPR
1940 North Monroe Street
Tallahassee, FL 32399

SUBJECT: IHPA Request for Funding For Research Project Related to Fastening of Wood Structural Panels for Opening Protection

Florida Building Commission:

Please consider this a request for funding for an important research project related to the fastening of wood structural panels as specified by the Florida Building Code. During the August meetings at Fort Lauderdale the Florida Building Commission (Commission) adopted a definition for the term "research" as follows:

> "An important and necessary endeavor that aimed at studying specific code related issue(s)/topics for the purpose of providing solutions to a specific problem or future code change(s) directed at improving the implementation and enforcement of the FBC. The issue to be researched must be fully understood (i.e. with clear purpose and goals); clearly defined with specific scope of work/approach; and within budget."

The International Hurricane Protection Association (IHPA) requests up to $10,000.00 be expended for testing of the fastening specified at Tables 1609.1.2 and R301.2.1.2 of the Florida Building Code. This is an important and necessary endeavor because testing conducted and previously submitted to the Commission indicates the current code is inadequate for the intended task.

Testing conducted by Architectural Testing, Inc. for IHPA indicates there is a problem with the ability of the code specified fastening schedule to resist the structural loads specified by the code for opening protection products. The failures noted were under structural loading and would undoubtedly lead to failure of the panel if subjected to the cyclical loading specified by the code for opening protection products. Additionally, it was discovered during the testing that the fasteners specified by the code are not readily available in the marketplace.

The research proposal is to review the findings of the 2003 Loss Relativities for FBC Wood Panel Shutters[1] (LRWPS or the Study). The Study was used to develop the fastening tables for wood structural panels used in the FBC. The Study conducted testing on both the wet and dry condition. The technical approach of this project will involve:

1. Engineering Analysis. The performance of engineering analysis based on a review of the LRWPS and including catenary loading based on the findings of the testing previously sponsored by IHPA[2] to develop values for a table that incorporates edge distance on the buck, edge distance on the panel, tensile strength, deflection, end failure, and yielding or over-pulling of the anchors used for attachment of wood structural panels. A test strategy will be developed based on the final calculations considering

1 Loss Relativities for FBC Wood Panel Shutters, Department of Community Affairs DCA Contract 03-RC-11-14-00-22-034, ARA IntraRisk June 30, 2003, Final Report,
2 Architectural Testing, Inc. Test Report dated December 1, 2011.

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appropriate safety factors for wood structural panels installed using common anchors that are widely available in the marketplace.

2. The engineering analysis will be contributed to the project by an IHPA member. The estimated value of the analysis is $5,000.00.
3. Testing will be to ASTM E 330-02 for structural testing and ASTM E 1886-05 and ASTM E 1996-09 for impact and cyclic testing for large missile.
4. Testing Program. The testing will involve a maximum of three tests to validate the data generated in the engineering analysis.
    a. A dry test using an OSB wood structural panel in accordance with the methodology of the LRWPS.
    b. A wet test using an OSB wood structural panel in accordance with the methodology of the LRWPS.
    c. A dry test using a plywood structural panel in accordance with the methodology of the LRWPS.
5. Testing to be performed by a Florida Building Commission approved testing laboratory.
6. Responsibilities of the testing lab include:
    a. All testing will be on a wood test buck as constructed by the testing laboratory.
    b. Mounting test specimens.
    c. Conducting tests.
    d. Writing of sealed test report.
7. IHPA will provide test specimens of commonly available materials and fasteners purchased from a retail outlet.
8. IHPA will attend and witness testing.
9. IHPA will provide installation drawings which will indicate fastener type and spacing, required shim space, and any other details pertinent to installation of wood structural panels.
10. Installation drawings shall become a referenced document in the final test report.
11. IHPA estimates the cost of Items 7, 8, and 9 at $1,000.00
12. Testing is estimated to cost $8,775.00 and shall not exceed $10,000.00. The total funding requested is to cover the testing costs only.
13. The results of the engineering analysis and testing will be used to validate the existing values or, as indicate, to develop final recommendations for new table values to replace those of Tables 1609.1.2 and R301.2.1.2 of the Florida Building Code, Building and Residential, respectively.
14. If indicated, new values will be submitted to the Florid Building Code as proposed code changes.

Respectfully submitted,

Joseph D. Belcher

Joseph D. Belcher, Code Consultant

Cc. Frank Browning, IHPA President
Tom Johnston, Immediate Past President

Appendix B. Recommendation for changes to the 5th Edition (2014) Florida Building Code

Red text = edits made by project investigators

Note 1: The 5th Edition (2014) Florida Building Code - Post Commission Post Glitch revisions call out Section 1609.1.2.3. That section (now edited) appears as 1609.1.2.4

Note 2: Corresponding changes will need to be made to FBCR R301.2.1.2.

Note 3: The version corresponds to the Post Commission Post Glitch document

CHAPTER 16
STRUCTURAL DESIGN

SECTION 1609
WIND LOADS

1609.1.2 Protection of openings. In *wind-borne debris regions*, ~~glazing~~ glazed openings in buildings shall be impact resistant or protected with an impact-resistant covering meeting the requirements of ~~, an *approved* impact-resistant standard or ASTM E 1996 and ASTM E 1886 referenced herein as follows:~~ , SSTD 12, ANSI/DASMA 115 (for garage doors and rolling doors) or TAS 201, 202 and 203, AAMA 506. ASTM E 1996 and ASTM E 1886 referenced herein, or an *approved* impact-resistant standard as follows:

1. Glazed openings located within 30 feet (9.1 m) of grade shall meet the requirements of the Large Missile Test of ASTM E 1996.

2. Glazed openings located more than 30 feet (9.1 m) above grade shall meet the provisions of the Small Missile Test of ASTM E 1996.

3. Storage sheds that are not designed for human habitation and that have a floor area of 720 square feet (67 m2) or less are not required to comply with the mandatory windborne debris impact standards of this code.

4. Openings in sunrooms, balconies or enclosed porches constructed under existing roofs or decks are not required to be protected provided the spaces are separated from the building interior by a wall and all openings in the separating wall are protected in accordance with Section 1609.1.2 above. Such spaces shall be permitted to be designed as either partially enclosed or enclosed structures.

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) ~~and maximum panel span of 8 feet (2438 mm)~~ shall be permitted for opening protection in Group R-3 or R-4 occupancy buildings with a mean roof height of 45 feet (13 716 mm) or less in locations where Vult is 180 mph (80 m/s) or less ~~as Group R-3 or R-4 occupancy~~. The opening shall not exceed 42 inches (1 067 mm) X 90 inches (2 286 mm). Panels shall be precut to overlap the wall by 3 inches (76.2 mm) on all sides and ~~so that they shall be~~ attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the ~~anchorage~~ attachment method and ~~shall be~~ secured with ~~the~~ corrosion-resistant attachment hardware permanently installed on the building ~~provided~~. ~~Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building.~~ At a minimum, panels shall be fastened at 16 inches (406.4 mm) o.c. along the edges of the opposing long sides of the panel. Fasteners shall be located 1 inch (25.4 mm) from the opening and 2 inches (50.8 mm) inward from the panel edge. The hardware shall consist of ¼-inch hanger bolts and either (a) 1/4 inch (6.3 mm) washer with a 1 inch (25.4 mm) flange and a

¼-20 hexnut or (b) a ¼-20 washered wingnut with a minimum of a 1 inch (25.4 mm) flange. Fasteners shall penetrate through the external wall covering with sufficient embedment length to provide a minimum of 300 lbs of withdrawal resistance. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1,500 pounds. Alternatively, attachments may be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7. These systems shall meet the requirements of Section 1609.1.2.4 below. ~~Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where Vult does not exceed 180 mph (80 m/s) Vasd, determined in accordance with Section 1609.3.1 does not exceed 140 mph (63 m/s).~~

2. Glazing in Risk Category I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in Risk Category II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.
4. Exterior balconies or porches under existing roofs or decks enclosed with screen or removable vinyl and acrylic panels complying with Section 2002.3.3 shall not be required to be protected and openings in the wall separating the unit from the balcony or porch shall not be required to be protected unless required by other provisions of this code.

TABLE 1609.1.2
WIND-BORNE DEBRIS PROTECTION FASTENING SCHEDULE FOR WOOD STRUCTURAL PANELS[a, b, c, d]

FASTENER TYPE	FASTENER SPACING (inches)		
	Panel Span ≤ 4 feet	4 feet < Panel Span ≤ 6 feet	6 feet < Panel Span ≤ 8 feet
No. 8 wood-screw-based anchor with 2-inch embedment length	16	10	8
No. 10 wood-screw-based anchor with 2-inch embedment length	16	12	9
$^{1}/_{4}$-inch diameter lag-screw-based anchor with 2-inch embedment length	16	16	16

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448 N, 1 mile per hour = 0.447 m/s.

a. This table is based on a V_{asd}, determined in accordance with Section 1609.3.1 of 140 mph and a 45-foot mean roof height.

b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.

c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. Fasteners shall be located a minimum of $2^{1}/_{2}$ inches from the edge of concrete block or concrete.

d. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1,500 pounds.

1609.1.2.1 Louvers. Louvers protecting intake and exhaust ventilation ducts not assumed to be open that are located within 30 feet (9144 mm) of grade shall meet requirements of ANSI/AMCA 540 or shall be protected by an impact resistant cover complying with the large missile test of ASTM E 1996 or an approved impact-resistance standard. Louvers required to be open for life safety purposes such as providing a breathable atmosphere shall meet the requirements of AMCA 540.

1609.1.2.2. Application of ASTM E 1996. The text of Section 6.2.2 of ASTM E 1996 shall be substituted as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the strength design wind speed, *Vult*, as follows:

6.2.2.1 *Wind Zone 1*—130 mph ≤ ultimate design wind speed, *Vult* < 140 mph.

6.2.2.2 *Wind Zone 2*—140 mph ≤ ultimate design wind speed, *Vult* < 150 mph at greater than one mile (1.6 km) from the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.3 *Wind Zone 3*—150 mph (58 m/s) ≤ultimate design wind speed, *Vult* ≤ ~~160~~ 170 mph (63 m/s), or 140 mph (54 m/s) ≤ ultimate design wind speed, *Vult* ≤ ~~160~~ 170 mph (63 m/s) and within one mile(1.6 km) of the coastline. The coastline shall be measured from the mean high water mark.

6.2.2.4 *Wind Zone 4*— ultimate design wind speed, *Vult* > ~~160~~ 170 mph (63 m/s)

~~R~~1609.1.2.2.1 Modifications to ASTM E 1886 and ASTM E 1996.

Table 1 of ASTM E 1886 and ASTM E 1996 – revise the third column to read as follows:

Air Pressure Cycles

0.2 to 0.5 P_{pos}[1]

0.0 to 0.6 P_{pos}

0.5 to 0.8 P_{pos}

0.3 to 1.0 P_{pos}

0.3 to 1.0 P_{neg}[2]

0.5 to 0.8 P_{neg}

0.0 to 0.6 P_{neg}

0.2 to 0.5 P_{neg}

Notes:

1. P_{pos}= 0.6 x positive ultimate design load in accordance with ASCE 7.
2. P_{neg} = 0.6 x negative ultimate design load in accordance with ASCE 7.

1609.1.2.4 Impact resistant coverings.

1609.1.2.4.1 ~~Impact resistant coverings shall be tested at 1.5 times the design pressure (positive or negative) expressed in pounds per square feet as determined by the Florida Building Code, Building Section 1609 or ASCE 7, for which the specimen is to be tested. The design pressures, as determined from ASCE 7, are permitted to be multiplied by 0.6.~~

Impact resistant coverings shall be tested for resistance to uniform static air pressure using ASTM E330 or TAS 202 and resistance to uniform cyclic air pressure using ASTM E1996, TAS 202 or TAS 203 at the pressures defined in Table 1609.1.4.X. These pressures are defined for Vult = 181 mph (80.9 m/s) or equivalently Vasd = 140 mph (62.6 m/s). For Vult larger than 181 mph, the pressures in the table shall be multiplied by the squared ratio of the wind speeds:

$$p_{Vult} = p_{181\,mph}\left(\frac{V_{ult}}{181}\right)^2$$ **(Equation 16-X)**

The loads shown in the table are based on an Effective Wind Area of 10 square feet (0.93 square meters). For larger Effective Wind Areas, the values in Table 1609.1.X may be adjusted to consider the area-dependent external pressure coefficients shown in Figure 30.4-1 in ASCE 7. Topographic effects may also be considered following the guidelines set forth in ASCE 7.

Table 1609.1.4.X. WIND LOAD REQUIREMENTS FOR IMPACT RESISTANT COVERINGS (Vult = 181 mph)

Height	Ultimate Neg. Pressure[A]			Ultimate Pos. Pressure[A]			ASD Neg. Pressure[B]			ASD Pos. Pressure[B]			Height
(ft)	B	C	D	B	C	D	B	C	D	B	C	D	(ft)
15	-65	-95	-116	+48	+71	+86	-39	-57	-69	+29	+43	+52	15
20	-70	-101	-122	+52	+76	+91	-42	-61	-73	+31	+45	+54	20
25	-75	-106	-126	+56	+79	+94	-45	-64	-76	+33	+47	+57	25
30	-79	-110	-131	+59	+82	+97	-47	-66	-78	+35	+49	+58	30
40	-85	-117	-137	+64	+88	+102	-51	-70	-82	+38	+52	+61	40
45	-88	-120	-140	+66	+90	+105	-53	-72	-84	+40	+54	+63	45
50	-91	-123	-143	+68	+92	+107	-55	-74	-85	+41	+55	+64	50
60	-96	-128	-147	+72	+95	+110	-57	-76	-88	+43	+57	+66	60
70	-100	-132	-151	+75	+98	+113	-60	-79	-91	+45	+59	+68	70
80	-104	-136	-155	+78	+101	+116	-62	-81	-93	+47	+61	+69	80
90	-108	-139	-158	+80	+104	+118	-64	-83	-95	+48	+62	+71	90
100	-111	-142	-161	+83	+106	+120	-66	-85	-96	+50	+64	+72	100
120	-117	-148	-166	+87	+110	+124	-70	-88	-99	+52	+66	+74	120
140	-122	-153	-171	+91	+114	+127	-73	-91	-102	+55	+68	+76	140
160	-127	-157	-175	+95	+117	+130	-76	-94	-105	+57	+70	+78	160
180	-131	-161	-178	+98	+120	+133	-79	-96	-107	+59	+72	+80	180
200	-135	-164	-182	+101	+123	+136	-81	-98	-109	+61	+74	+81	200
250	-144	-172	-189	+108	+129	+141	-86	-103	-113	+64	+77	+84	250
300	-152	-179	-195	+113	+134	+145	-91	-107	-117	+68	+80	+87	300
350	-159	-185	-200	+119	+138	+149	-95	-111	-120	+71	+83	+89	350
400	-165	-190	-205	+123	+142	+153	-99	-114	-123	+74	+85	+92	400
450	-171	-195	-209	+127	+146	+156	-102	-117	-125	+76	+87	+93	450
500	-176	-199	-213	+131	+149	+159	-105	-119	-127	+79	+89	+95	500

[A]Proof load in ASTM E330, Test load in TAS 202-94
[B]Test load in ASTM E330, Design Pressure in TAS 202-94, P_{pos} and P_{neg} in ASTM E1996 and Design Wind Load in TAS 203-94

1609.1.2.4.2 Impact resistant coverings. Impact resistant coverings shall be labeled in accordance with the provisions of Section 1710.8.

1609.1.3 Optional exterior door component testing. Exterior side-hinged door assemblies shall have the option to have the components of the assembly tested and rated for impact resistance in accordance with the following specification: SDI 250.13.

1609.1.4 The wind-borne debris regions requirements shall not apply landward of the designated contour line in Figure 1609A or 1609B. A geographical boundary that coincides with the contour line shall be established.

1609.1.5 Testing to allowable or nominal loads. Where testing for wind load resistance is based on allowable or nominal wind loads, the design wind loads determined in accordance with ASCE 7 or Section 1609 are permitted to be multiplied by 0.6 for the purposes of the wind load resistance testing.

1609.2 Definitions. The following words and terms shall, for the purposes of Section 1609, have the meanings shown herein.

HURRICANE-PRONE REGIONS. Areas vulnerable to hurricanes defined as:

1. The U. S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed for Risk Category II buildings is greater than 115 mph (40 m/s) and
2. Hawaii, Puerto Rico, Guam, Virgin Islands and American Samoa.

WIND-BORNE DEBRIS REGION. Areas within hurricane-prone regions located:

1. Within 1 mile (1.61 km) of the coastal mean high water line where the ultimate design wind speed Vult is 130 (48 m/s) or greater; or
2. In areas where the ultimate design wind speed Vult is 140 mph (53 m/s) or greater.

For Risk Category II buildings and structures and Risk Category III buildings and structures, except health care facilities, the windborne debris region shall be based on Figure 1609A. For Risk Category IV buildings and structures and Risk Category III health care facilities, the windborne debris region shall be based on Figure 1609B.

WIND SPEED, V_{ult}. Ultimate design wind speeds.

WIND SPEED, V_{asd}. Nominal design wind speeds.

1609.3 Basic wind speed. The ultimate design wind speed Vult, in miles per hour, for the development of the wind loads shall be determined by Figures 1609A, 1609B and 1609C. The ultimate design wind speed Vult for use in the design of Risk Category II buildings and structures shall be obtained from Figure 1609A. The ultimate design wind speed Vult for use in the design of Risk Category III and IV buildings and structures shall be obtained from Figure 1609B. The ultimate design wind speed Vult for use in the design of Risk Category I buildings and structures shall be obtained from Figure 1609C. The exact location of wind speed lines shall be established by local ordinance using recognized physical landmarks such as major roads, canals, rivers and lake shores wherever possible.

1609.3.1 Wind speed conversion. When required, ultimate design wind speeds of Figure 1609A, B and C shall be converted to nominal design wind speeds, Vasd, using Table 1609.3.1 or Equation 16-32.

$$V_{asd} = V_{ult}\sqrt{0.6} \qquad \textbf{(Equation 16-32)}$$

where:

V_{asd} = nominal design wind speed
V_{ult} = strength design wind speeds determined from Figures 1609A, 1609B, or 1609C.

Appendix C. Load calculations for impact resistant coverings

ASCE 7-10 Components & Cladding Loads
Chapter 30. Part 1: Low-Rise Buildings

V_{asd}	140	mph	Specified in FBC-Building 1609.1.2, Exceptions
V_{ult}	181	mph	FBC-Building Equation 16-32
h	45	ft	Specified in FBC-Building 1609.1.2, Exceptions
Exposure	D	unitless	Assumed
z_g	700	ft	ASCE 7-10 Table 26.9-1
α	11.5	unitless	ASCE 7-10 Table 26.9-1
K_z	1.25	unitless	ASCE 7-10 Table 30.3-1
K_{zt}	1.00	unitless	ASCE 7-10 Section 26.8
K_d	0.85	unitless	ASCE 7-10 Section 26.8
$q_{z=h}$	89	psf	ASCE 7-10 Equation 30.3-1
GC_{pi}	0.18	unitless	ASCE 7-10 Table 26.11-1
EWA	10.00	sq. ft	Assumed (panel is 32 square feet)
GC_p (-)	-1.40	unitless	ASCE 7-10 Figure 30.4-1. Zone 5 Negative Pressure
GC_p (+)	1.00	unitless	ASCE 7-10 Figure 30.4-1. Zone 5 Positive Pressure
	-140	psf	ASCE 7-10 Equation 30.4-1 -> Ultimate Design Load
	105	psf	ASCE 7-10 Equation 30.4-1 -> Ultimate Design Load
ASD Conversion Factor	0.6	unitless	FBC-Building 1609.1.2.3.1
	-84	psf	ASD Load
	63	psf	ASD Load
Load Factor	1.5	unitless	FBC-Building 1609.1.2.3.1
	-126	psf	ASD Load w/ Factor of Safety
	94	psf	ASD Load w/ Factor of Safety

Figure 9. ASCE 7-10 Components & Cladding Loads (Part 1 Method)

ASCE 7-10 Components & Cladding Loads
Chapter 30. Part 2: Low-Rise Buildings (Simplified)

V_{asd}			
V_{ult}	180	mph	
λ	1.78	unitless	ASCE 7-10 Figure 30.5-1
K_{zt}	1.00	unitless	ASCE 7-10 Section 26.8
p_{net30}	-78	psf	ASCE 7-10 Figure 30.5-1. Zone 5 Negative Pressure for EWA <= 10 sq. ft
	58	psf	ASCE 7-10 Figure 30.5-1. Zone 5 Positive Pressure for EWA <= 10 sq. ft
	-139	psf	Ultimate Design Load -> ASCE 7-10 Equation 30.5-1
	104	psf	Ultimate Design Load -> ASCE 7-10 Equation 30.5-1
ASD Conversion Factor	0.6		FBC-Building 1609.1.2.3.1
	-83	psf	ASD Load
	62	psf	ASD Load
Load Factor	1.5	unitless	FBC-Building 1609.1.2.3.1
	-125	psf	ASD Load w/ Factor of Safety
	93	psf	ASD Load w/ Factor of Safety

Figure 10. ASCE 7-10 Components & Cladding Loads (Part 2 Method)

Appendix D. Test Specimens

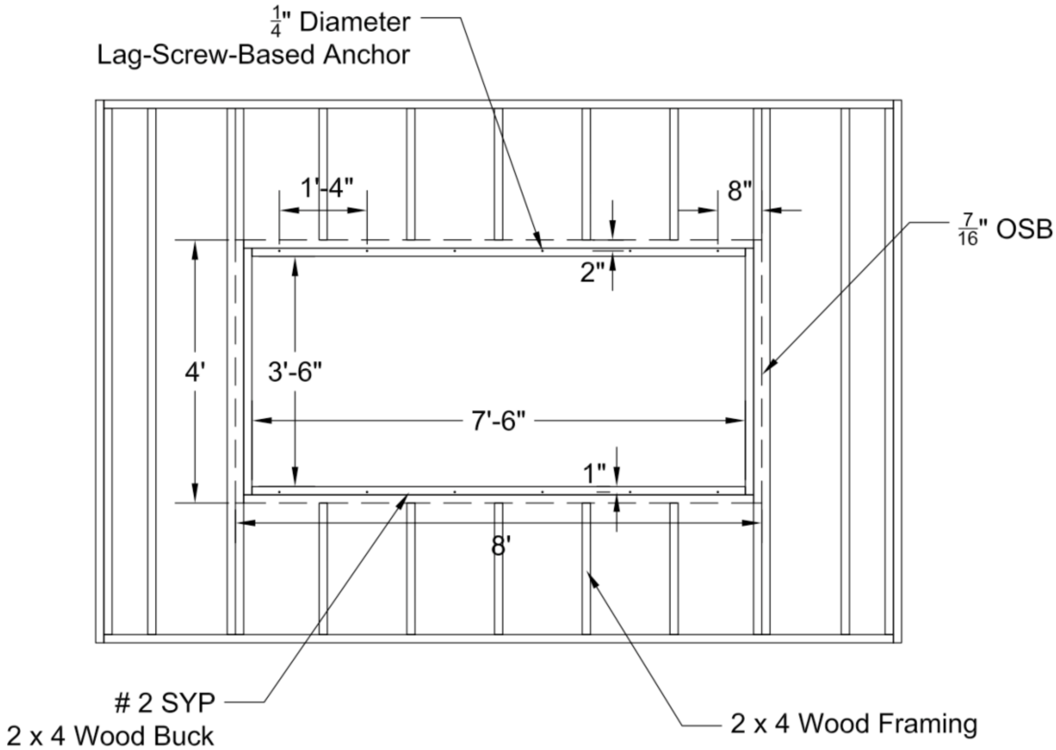

Figure 11. Wood frame wall configuration

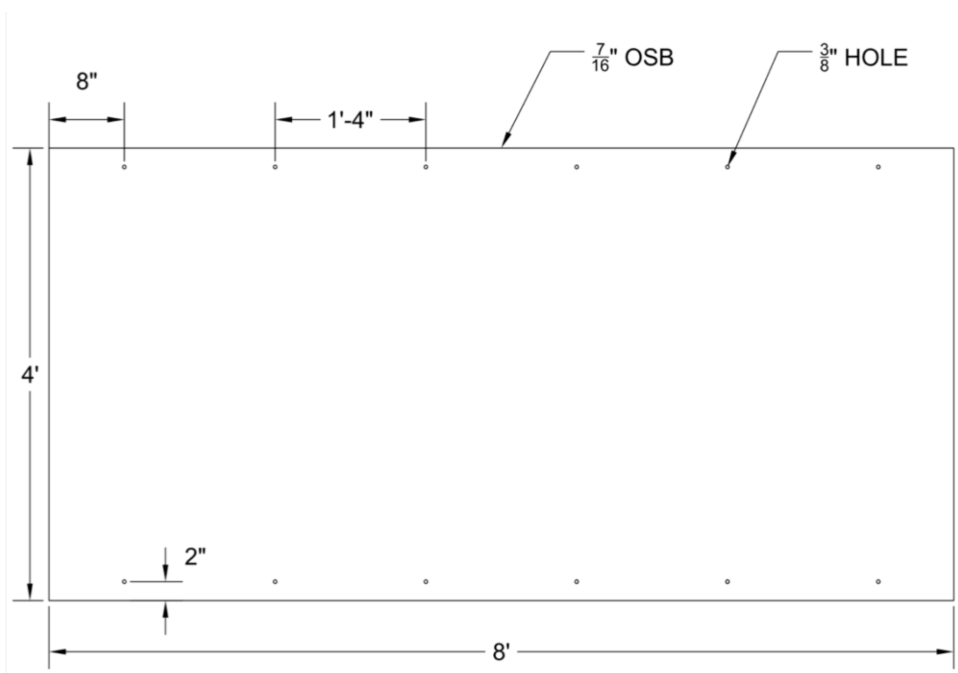

Figure 12. Shutter configuration

Appendix E. Results from uniform static and cyclic loading tests

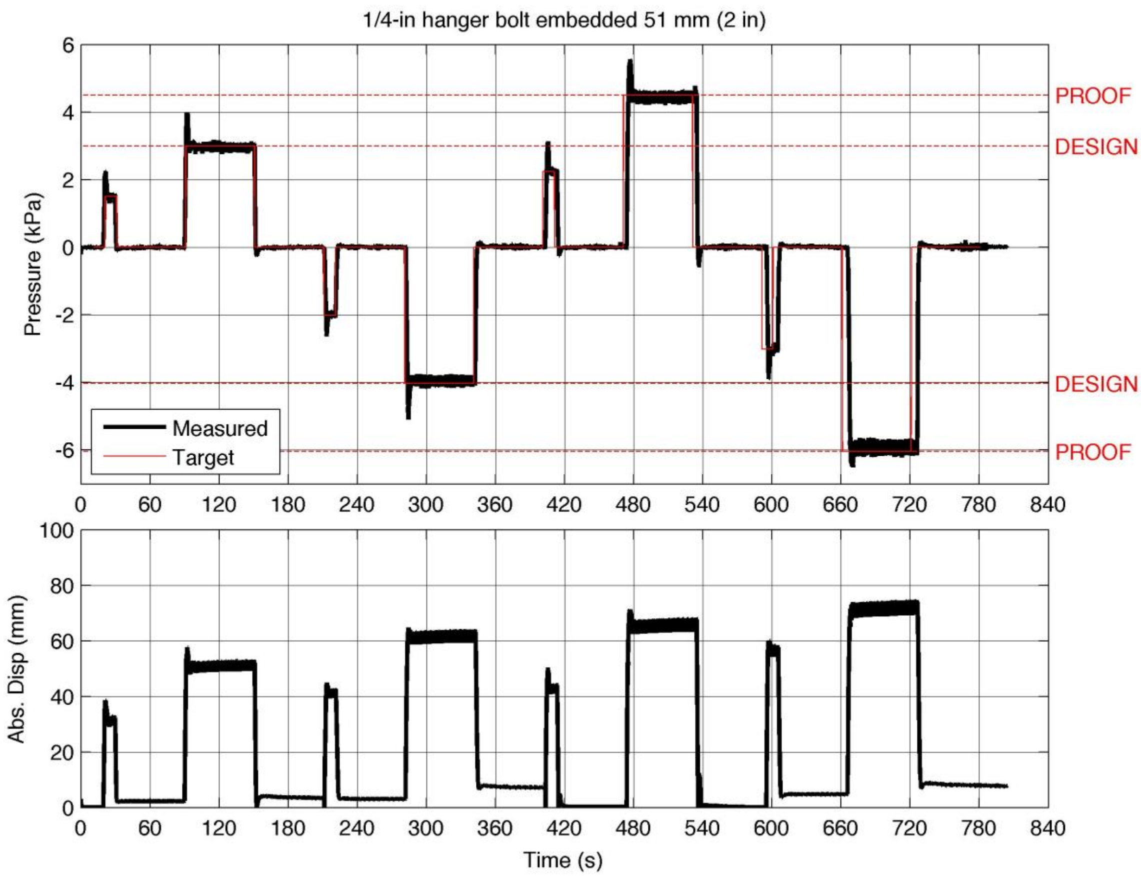

Figure 13. Static pressure loading sequence derived from ASTM E 330 for Case 1 (2.00 inch embedment)

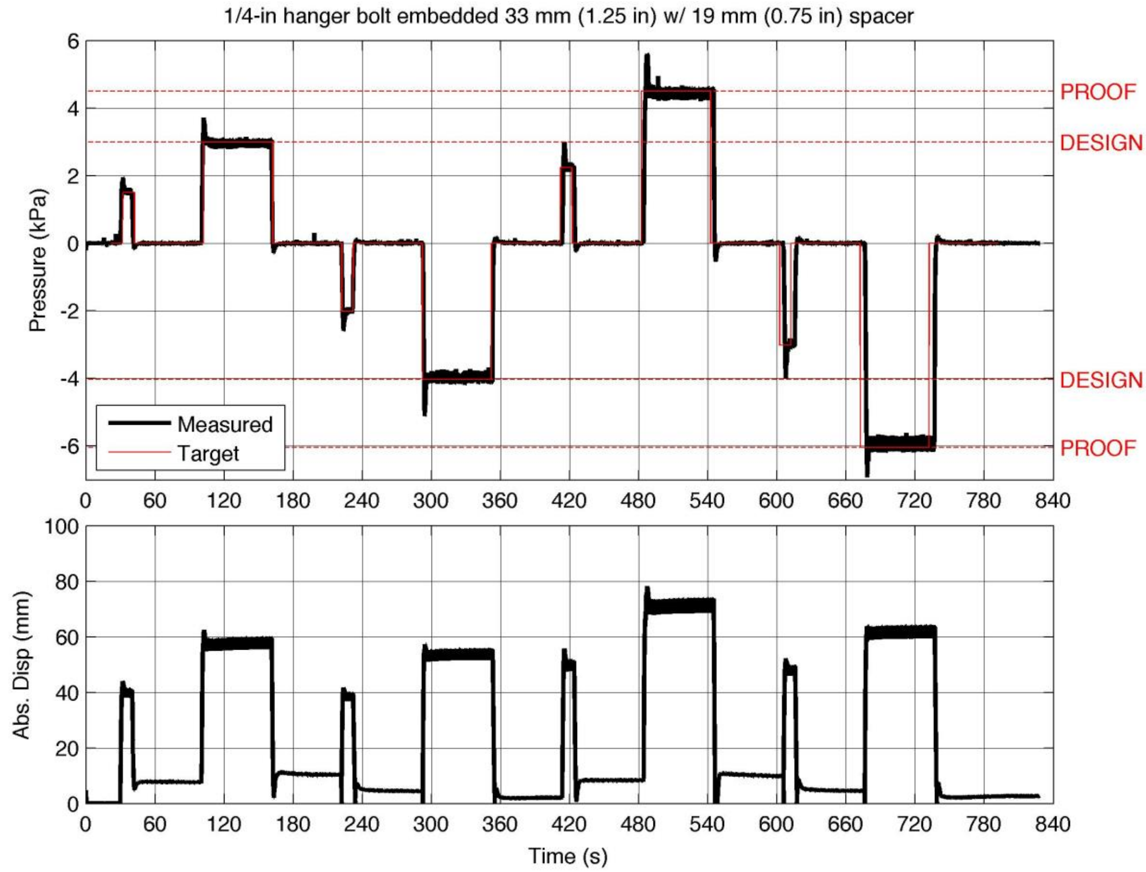

Figure 14. Static pressure loading sequence derived from ASTM E 330 for Case 2 (1.25 inch embedment)

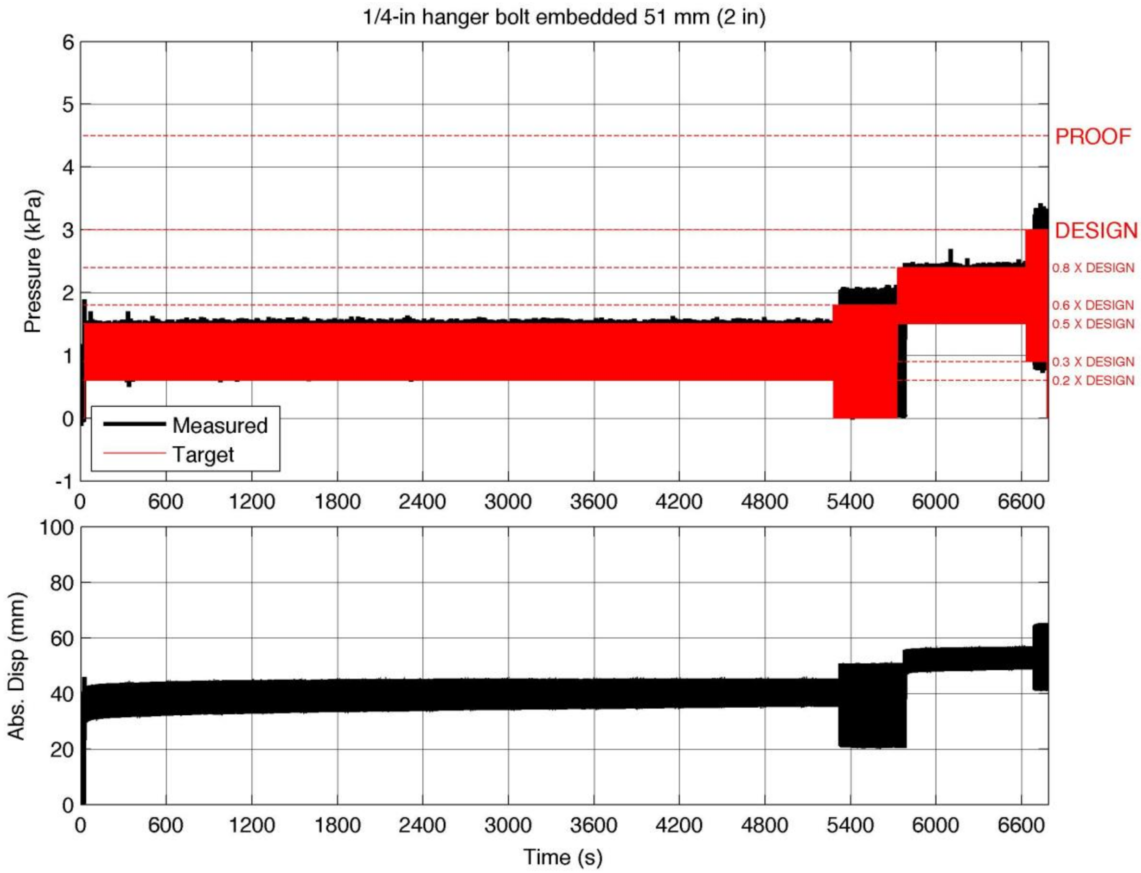

Figure 15. Cyclic pressure loading sequence derived from ASTM E1996 for Case 1 (2.00 inch embedment) for the positive pressure case

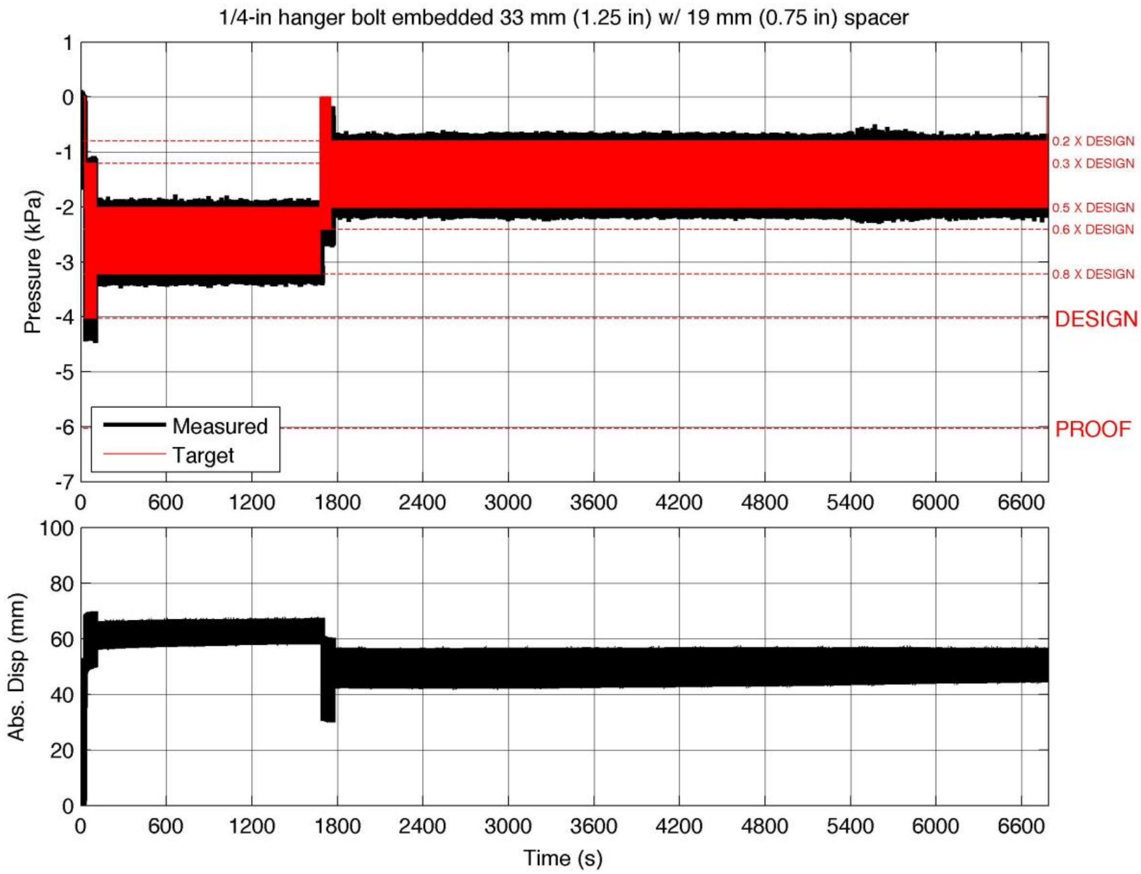

Figure 16. Cyclic pressure loading sequence derived from ASTM E1996 for Case 1 (2.00 inch embedment) for the negative pressure case

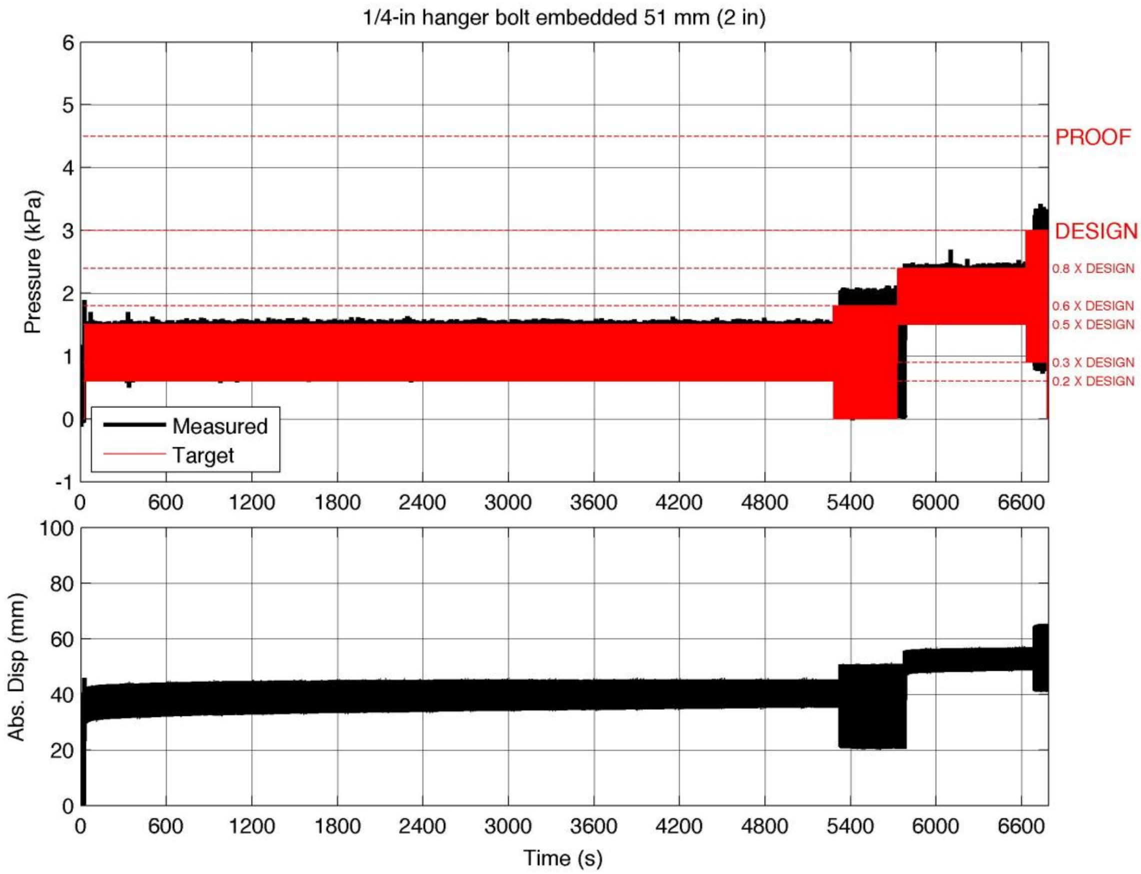

Figure 17. Cyclic pressure loading sequence derived from ASTM E1996 for Case 2 (1.25 inch embedment) for the positive pressure case

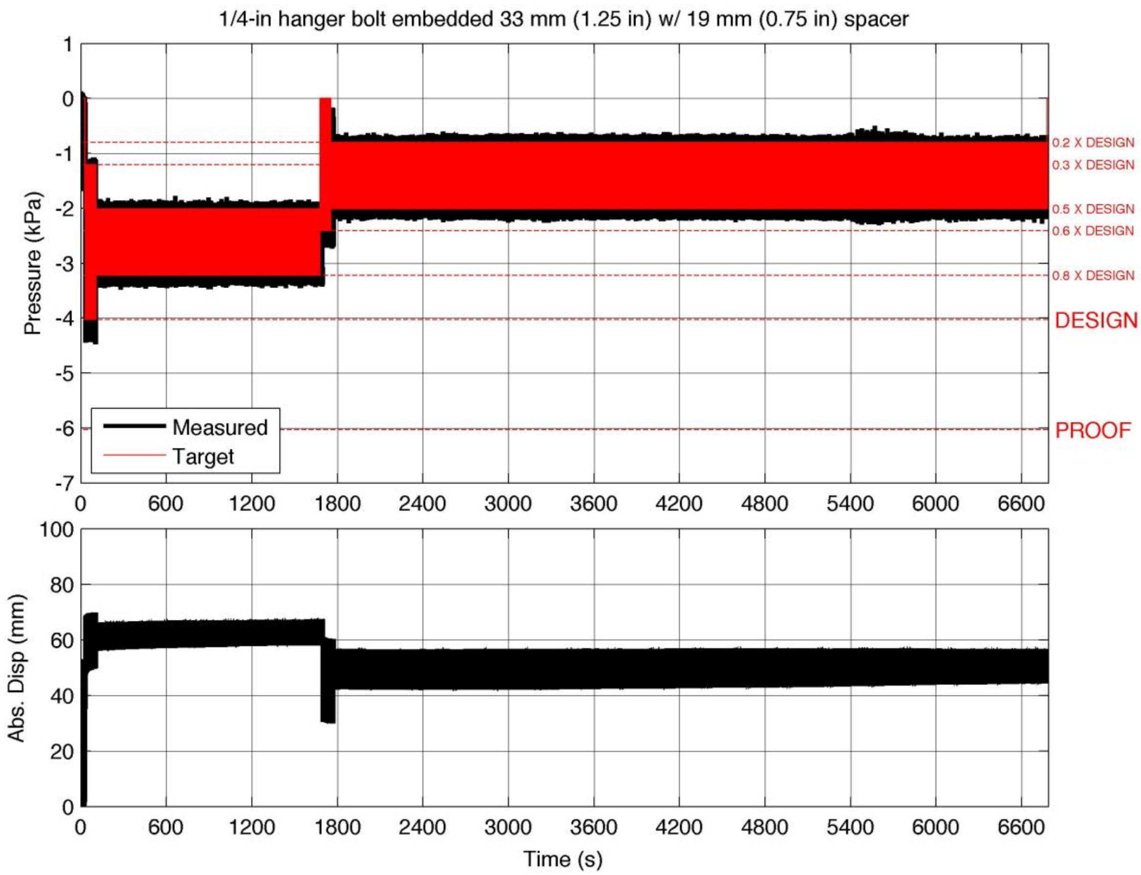

Figure 18. Cyclic pressure loading sequence derived from ASTM E1996 for Case 2 (1.25 inch embedment) for the negative pressure case