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STRUCTURAL MODELLING AND VERIFICATION METHODS TO DEVELOP A CABLE ROOF HARNESS RETROFIT SYSTEM

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ABSTRACT

Load paths in light-frame wood structures have historically been nailed connections between the sheathing and rafters, and toenail connections between the rafters and stud walls. However, these connections have poor resistance to uplifting forces, as occurs in high wind speed events, causing sheathing or roof-to-wall-connection (RTWC) failures. The improvements made to building codes after Hurricane Andrew affected only new construction, and the economic losses caused by roof failures in homes built prior to 1993 from Hurricane Katrina pointed to a need to retrofit older structures. This paper will investigate the design, analysis, and testing of a temporary cable-netting roof harness as an alternative to relatively expensive and invasive retrofitting options. To do this, a non-linear finite element analysis (FEA) is performed to model a typical light-frame wood structure with the roof harness, which is then validated through test results. After, as a comparative study, scaled down versions of the structure with and without the roof harness are created and tested using real wind load until failure at the WindEEE facility. This is done to assess the efficacy of the retrofit system. Corresponding FEA and computational fluid dynamics (CFD) models are then created to simulate the test.

Keywords: Destructive Testing; CFD; FEA; Wood Structures; Wind Engineering; Non-Linear Analysis

1. INTRODUCTION

The vertical load path of typical light-frame wood structures allows the sheathing to transfer vertical force imposed on the roof to the trusses beneath, which is then transferred to the stud walls and through to the foundation. After Hurricane Andrew in 1993 caused considerable economic losses, largely due to the failure of roofing system components (Baskaran and Dutt, 1997), building codes across North America began to reflect the need for adequately continuous vertical load paths in light-frame wood structures. Historically, the sheathing connection to the roof trusses and the roof truss connection to the stud wall were typically nailed and have poor withdrawal resistance (FEMA, 1993; van de Lindt et al., 2007). Since the building code changes only affected new construction, it left older structures prone to damage during high speed wind events. This became evident in the aftermath of Hurricane Katrina, where older structures experienced considerable damage (van de Lindt et al., 2007).

Though there exist retrofitting options for older structures such as adhesives, straps, hurricane ties, and the installation of additional studs (FEMA, 2010), these options are relatively invasive and are approximately 15-50 % of the cost of the structure to implement (Stewart et al, 2003). Therefore, a need is established to seek an alternative, low-cost method and easily applied method of securing a continuous load path.

The proposed roof harness retrofit system is comprised of a series of cables that extend over the roof and are attached to horizontal rigid bars. These bars are then connected to larger cables which are attached to permanently installed piles around the perimeter of the structure. The system as a whole prevents uplift failure of the roof-to-wall connections (RTWCs), while the finer cable spacing over the roof is designed to prevent sheathing-to-truss connection (STTC) failure. Though there exist various patents for roof harness systems (Bimberg 1997; Luzzi 1999), the proposed system seeks to address issues apparent in their design, chiefly ease of application, extensibility of cable material, and possible sheathing damage during application.

This paper will describe the development process of designing, analyzing, and testing the retrofit system. This process began with the assessment of the structural behaviour of a simple development of the finite element analysis (FEA) model of a light-frame wood structure under and its verification with full scale test results. Currently, a comparative study of a scale model structure is being carried out to assess the efficacy of a scale model structure with and without the retrofit system. The model without the retrofit system will be tested until failure using real wind loading, so that the performance enhancement of the retrofit system can be established in terms of resistance to higher wind speeds. In future tests, various angles of attack, different roof shapes, and overhangs will be considered to better understand the combined structural response of the structure and retrofit system.

2. REVIEW OF THE DEVELOPMENT AND VALIDATION OF FULL SCALE FEA MODELS

Previous work underlying this research has been focused on the FEA modelling of a light-frame wood house that had been tested as a part of the 'Three Little Pigs' project (Kopp et al., 2010). Once researchers developed a model that was able to simulate the wind induced structural response of this building, it was augmented to include a retrofit system, which was assessed through parametric studies in order to determine the best design. A portion of the full scale roof with the retrofit system applied was then tested to determine the validity of the augmented FEA model.

2.1 Description of the full scale structure

The full scale structure that was originally modelled by Dessouki and El Damatty (2010) was a 10 m by 10 m simple gable roof structure with a 4:12 slope. The structure was previously tested at the Insurance Research Lab for Better Homes at the University of Western Ontario under a simulated wind load provided by a wind tunnel study of a similar structure. For this experiment, the wind load was varied spatially and temporally through the utilization of 58 pressure bags that covered the entire roof. The structure was tested under mean wind speeds starting at 20 m/s and increasing by 5 m/s increments to the point of complete roof failure at 45 m/s. Full load-displacement time histories were recorded for every RTWC (Kopp et al., 2010).

2.2 Development and validation of the FEA model of the structure and proposal of a roof harness system

Dessouki and El Damatty (2010) developed a three-dimensional FEA model for simulating the response of the subject structure in the 'Three Little Pigs' project. Three-dimensional frame elements were used for modelling the truss members, stud walls, top plates, and connecting beams between the trusses, while shell elements were used for modelling the sheathing. Based on the results of 'Three Little Pigs' project (Kopp et al., 2010), the critical weak joints with respect to resisting the uplift wind load were found to be the toenailed RTWCs. In the FEA model, these were modelled as nonlinear spring elements with a load-displacement curve adapted from Reed et al. (1997). To perform a quasi-static wind load analysis, 29 random samples of the 20 m/s applied wind load time history were recorded. Based on the samples selected, good agreement was observed between the predicted displacements at critical RTWCs in the FEA model and the experimental results, with a maximum difference of 20 %. This research was extended to include an investigation of the accuracy of the RTWC reactions computed by the tributary area method. The findings revealed that the tributary area method was less accurate than the FEA model, as the difference in the results reached 50 % at some locations. This because the load attraction in comparatively stiff gable end trusses could not be accounted for by the tributary area method.

Addressing the problem of roof uplift, Dessouki and El Damatty (2010) developed a new retrofit system for mitigating roof uplift damage from high wind speed events for simple gable roofs, and incorporated it into the FEA model. Their system consisted of a two-dimensional steel wire net installed on the top of the house, with carbon fiber rods connected to the wire net on four sides to provide a uniform distributed load on the wire net, and reducing the number of external cables connecting the rods and piles anchored in the foundation, as shown in Figure 1. The augmented FEA model was then subjected to a uniform suction pressure of 2 kN/m^2 over the area of the roof. The wire net and the external cables were modelled as a nonlinear cable element under their own weight and strain loading. The rods were modelled as a three-dimensional frame element. The numerical conclusions indicated a reduction of 40 % in the average deflection of the house compared to a non-retrofitted house. This research also included a parametric study of the retrofit system using a FEA model for performing a system assessment in order to determine a suitable design. The study included an examination of the following parameters: the diameter of the rods, the diameter of the net wires, the inclination of the external cables, and the post-tensioning forces associated with the external cables.

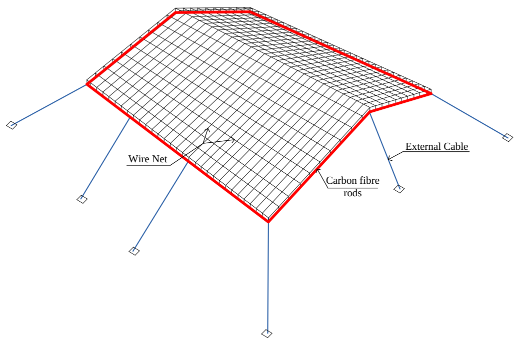

Figure 1: Retrofit system proposed by Dessouki and El Damatty

2.3 Further development of the roof harness system and validation of the augmented FEA model

Following the research of Dessouki and El Damatty (2010), Jacklin and El Damatty (2013) further refined the FEA model of the full scale structure, and proposed a simpler retrofitting system for light-frame wood houses. The FEA model of the structure and retrofitting system were then validated by testing a portion of the structures roof with the retrofit system applied. The refined FEA model simulated the RTWC as a multi-linear plastic element using 1000 samples of the time history of the wind loading recorded by Kopp et al. (2010), representing the maximum and minimum actual pressure. It was concluded that the numerical model was able to predict the RTWC displacement of with a maximum difference in results of 10 %, improving the platform with which to model the retrofit system.

The retrofit system proposed by Jacklin and El Damatty (2013) consisted of steel bearing cables installed at the roof of the house. These bearing cables were connected to rigid aluminum bars in order to create a uniform load at the bearing cables and to reduce the number of external steel cables that were connected between the rigid bars and the foundation piles, as shown in Figure 2. The numerical model was also designed to include the retrofitting system in the analysis. The bearing and external cables were modelled using a nonlinear cable element, and the rigid bars were modelled as a three-dimensional frame element. The system was tested numerically through the application of incremental uplift pressure in order to investigate the performance of the retrofitting system with respect to reducing the displacement at the RTWCs. The numerical model was also used for predicting the RTWC displacement at the plastic range (i.e., mean wind velocity exceeding 25 m/s), a point at which an RTWC is subject to permanent withdrawal. This research was then extended to include a parametric study of the retrofit system, whereby numerous cross sections of rigid bars, bearing cables, and external cables were tested numerically with the goal of minimizing the weight of the system.

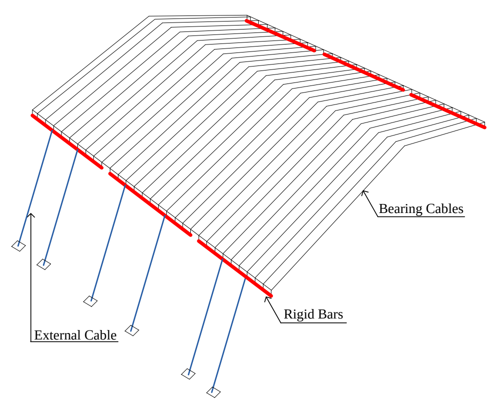

Figure 2: Retrofit system proposed by Jacklin and El Damatty (2013)

Jacklin and El Damatty (2013) then conducted a comparative full scale test under static load. The structure tested was a strip of the full scale roof, consisting of three typical trusses spanning 9 m, spaced 0.6 m apart and with slope of 4/12. There were six RTWCs using three hand driven 16d spiral shank nails each. The retrofit system consisted of five 3/16 in diameter interior roof cables, and two 0.5 in. diameter external cables at each side of the rigid aluminum beam. The aluminum beam was a hollow rectangular section 102 mm deep by 51 mm wide and 6 mm thick. The uplift wind load was simulated by using a hydraulic jack connected beneath the prototype at six points. LVDTs were installed at each RTWC in order to capture the load-displacement time history. Four strain gages were installed at each external cable in order to determine the axial force. It was concluded that the retrofit system increased the capacity of the prototype by 40 %. This value was previously predicted by the FEA models of both Dessouki and El Damatty (2010) and Jacklin and El Damatty (2013), indicating accuracy of the numerical results.

Figure 3: Sample roof strip and retrofit tested by Jacklin and El Damatty (2013)

3. DEVELOPMENT OF DESTRUCTIVE TESTING METHODS AND SCALE MODEL STRUCTURE

The next step in the development process is to refine the goal of the comparative study is to destructively test the model structure without the roof harness, then apply the roof harness to an identical model and compare the structural behavior of the two test models. The WindEEE facility at Western University was proposed to test the structure, by exhausting wind from within the test chamber outside the facility to the model structure at an angle of attack of 0º, as seen in Figure 4. Though an oblique angle would have had more critical local effects on sheathing failure, the greatest suction attributed to the RTWCs to induce failure there was produced at 0º.

To achieve failure with the existing fan speed capabilities, the length scale of the model structure as well as the structural resistance of its members and connections needed to be reduced, while the wind speed exiting the facility needed to be increased by funneling the wind flow through a contraction. To simplify the experiment, only RTWC failure was allowed to occur, by securing the sheathing to the roof trusses such that it would not fail.

Figure 4: Contraction and model setup

An initial test of the developed model structure and contraction did not achieve the desired RTWC failure since the contraction itself failed when the wind speed reached approximately 28 m/s. However, according to an FEA model of the structure and previous wind tunnel studies (Dagnew et. al., 2012) the predicted RTWC failure wind speed was also approximately 28 m/s, taking into account the average nail withdrawal capacity. This increase in uplift resistance is attributed to the higher than expected moisture content of the plywood sheathing that had previously been exposed to rain. Correctional measures for these sources of failure are outlined in the following sections.

3.1 Development of the contraction

3.1.1 Design of the contraction

The aim of the contraction was to increase the available wind speed to at least 45 m/s, which is 150 % of the speed at which failure of the model is predicted to occur, such that the roof harness can demonstrate a sufficient performance enhancement. This resulted in an area contraction that was 6 m wide by 1.9 m tall at the aperture, reducing the cross sectional area of the wind window by approximately 79 %. A CFD analysis demonstrated that at full fan speed, the available wind speed at the face of the structure would be approximately 48 m/s, without considering losses.

In Figure 3, the contraction consists of the inclined garage door resting on the two aluminum columns shown, using its own weight to restrain the contraction doors from blowing outward. However, it was found that at wind speeds of approximately 30 m/s the outward wind pressure is greater than the weight of the garage door and the unrestrained contraction failed. To counter this, a cable will run alongside each aluminum column establishing a pretension between the garage door and the slab on grade. The pretension force will be equal to the own weight of the door subtracted from the wind pressure generated from 100 % fan speed. This will ensure that the contraction is constantly in compression and stable at all wind speeds.

3.1.2 Assessing the wind profile downstream of the contraction

The wind profile downstream of the contraction aperture is still a work in progress. It was assessed by measuring the average wind speed and turbulence intensity factor in a three dimensional grid as shown in Figure 4. The wind profile exhibits good uniformity over the height of the wind window, and over the width of the model structure. Interaction with the ambient atmospheric conditions as the wind exits the contraction caused a marked decrease in wind speed and turbulence near the sides of the wind window.

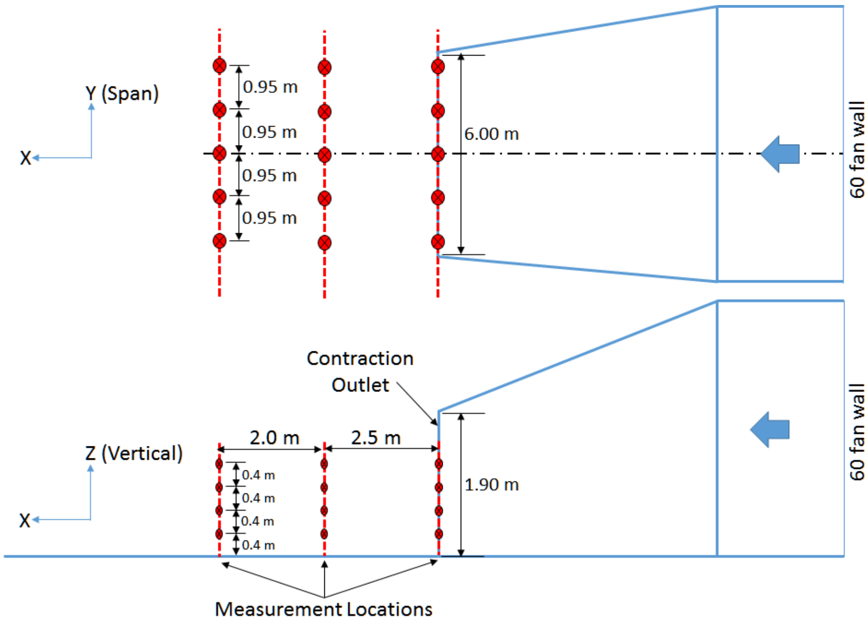

Figure 4: Velocity sampling grid (WindEEE)

The turbulence intensity profile along the height of the model at its center is approximately 6 %, indicating a highly laminar profile. For the purposes of spatially modelling uplift in the FEA model of the scaled structure, a suburban profile was assumed. Though the turbulence intensity profile is much less than that of a suburban profile which is approximately 30 % (Endo et al., 2009), it does not affect the comparative nature of the study. In future testing, to more accurately simulate a suburban wind profile, roughing blocks and mesh grids at the aperture of the contraction are being considered to introduce more turbulence.

3.2 Development of scale model structure

3.2.1 Geometric scaling of the model structure

To achieve failure in the model structure, while remaining well within the wind window produced by the contraction, the width of the square structure was scaled from 10 m to 3 m. In keeping with a 10:3 length scale ratio, the overall height of the structure is 1.2 m.

However, governed by the uplift capacity of the RTWCs, the truss spacing was increased from 0.61 m in the full scale model to 1 m in the scaled model resulting in 4 trusses. When the area reduction is taken into account, this allows each RTWC to experience approximately 50 % of the full scale uplift load, critical in achieving failure.

To further increase the wind load experienced over the roof surface, a slope of 3:12 was used. Though this differs slightly from the 4:12 slope used in the full scale model, it has only a small effect on the structural response of the truss system, while allowing for much greater anticipated uplift pressure from flow separation.

Since there are a limited number of trusses in the scaled model, the gable trusses were eliminated to prevent the effects of load sharing from significantly reducing the load experienced by the two interior trusses. To do this, the two gable end stud walls were disconnected from the trusses above them by the incorporation of a gap, rendering them simply supported. Further, to seal the building envelope and conserve the internal pressure of the structure, reinforced plastic fabric (RPF) was used to cover the exterior of the end trusses. Rather than using sheathing, the RPF did not provide the truss with additional in-plane stiffness. The scale model structure then effectively resembles an interior section of the full scale model structure. The structural drawing of the model is shown in Figure 6.

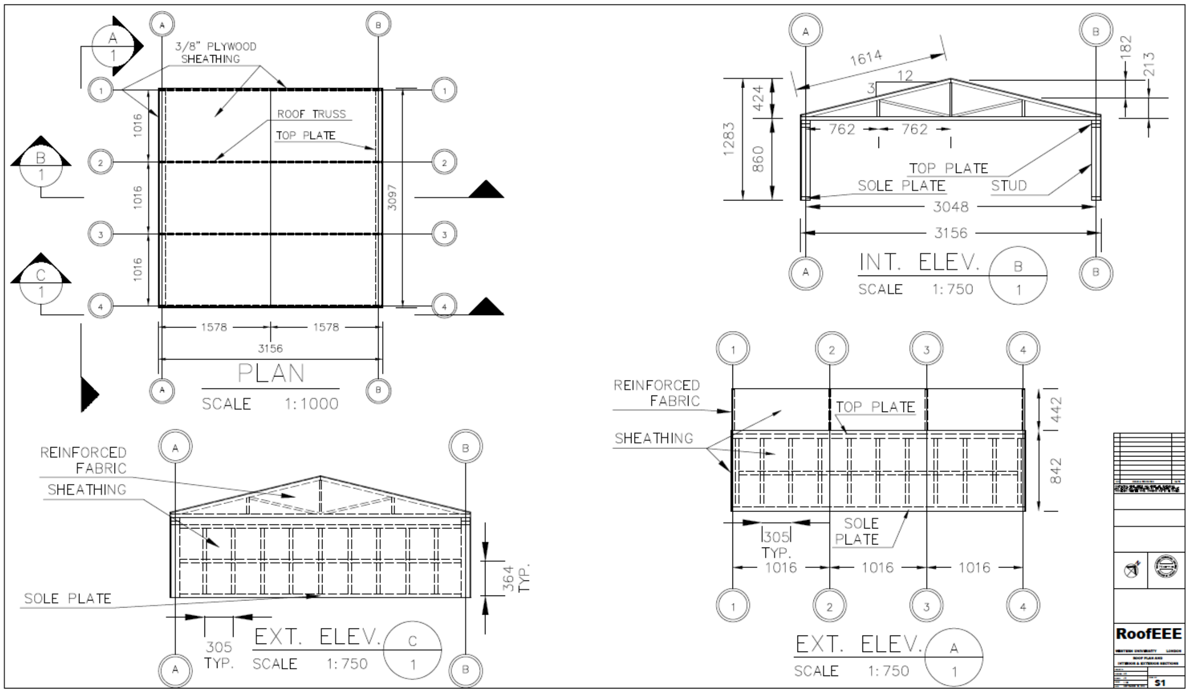

Figure 6: Structural drawing of the model structure

3.2.2 Reduction of toenail connection capacity

To achieve failure at the target velocity of 30 m/s required the RTWC capacity be reduced to at least approximately 400 N. To maintain consistent non-linear behaviour, a variety of smaller common nails in a standard toenail configuration (AWC, 2007) were tested using the apparatus developed by Morrison et al. (2011). A pressure loading actuator was connected to an airbag generating uplift to a sample specimen fixed to the bottom of the apparatus. A ramp load of 8 kN/m was applied to the specimen and time histories of both load and displacement were generated. The 2D common nail was selected for the model structure as it demonstrated an average maximum withdrawal capacity of 335 N over 12 trials, while maintaining qualitatively similar non-linear behaviour to the 12D common nails, used in the full scale RTWCs. The 12D common nail was also tested with 12 trials similarly and the results of the withdrawal behaviour for toenails using both 12D and 2D common nails are seen in Figure 7.

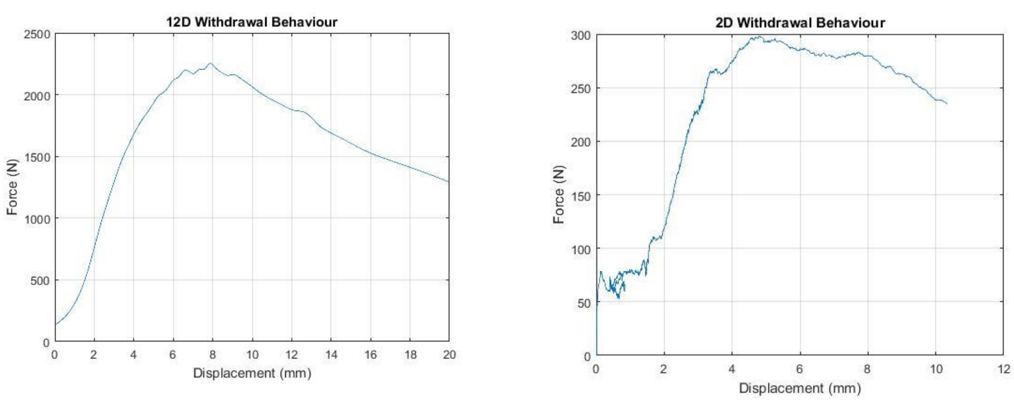

Figure 7: 12D and 2D common nail average withdrawal behaviour

3.2.3 Resizing structural members

The scale model structure was designed to isolate the structural behaviour of the roof components by ensuring that the stud walls were significantly heavier and stiffer than the roof. Further, the size of members in the roof trusses and the thickness of the sheathing were reduced such that they would exhibit similar load sharing properties and values of stress at failure as the full scale model, given the reduction in uplift load. Morrison et al. (2012) determined that the failure velocity of the full scale structure would be approximately 45 m/s. The reduction of element size was done by performing an FEA of the scale model structure using wind load derived from the target failure wind speed velocity of 30 m/s and comparing the values of stress in the structural roof elements to those in the full scale model FEA at failure. It was concluded that the roof truss members had to be reduced to 1 in. x 1.5 in. for the chords, 1 in. x 1 in. for the webbing, and 3/8 in. for the sheathing.

3.3 Scaling and design of the roof harness

Using the same design philosophy as used to resize the scale model structural members, the roof harness will be sized to exhibit the same wind load to stress relationship as the full scale model. Using the roof harness system developed by Jacklin and El Damatty (2013), the rigid aluminum bars and steel cables will reflect the optimum design achieved with respect to the own weight of the system. The external cables of the harness will be anchored to spreader beams running along the top of the model platform.

4. CONCLUSIONS AND FUTURE DIRECTION OF TESTING

The development of an FEA model capable of accurately simulating structural response to within 10 % of the displacement in the test structure was integral in understanding the potential benefit of the retrofit system. Further, the comparative testing of a sample full scale section of roof with the retrofit system proved both the effectiveness of the system as an alternative load path, and the accuracy of the FEA models that simulate it. The results of comparative testing pointed to a need to better understand the response of the structure and retrofit system by more accurately modelling the loading conditions.

This need was met with the current comparative study using real wind loading. The results of the initial test of the scale model structure indicated a need to improve the contraction stability under high wind speed, and reduce the own weight of the sheathing. However, with proposed modifications, future tests should produce failure in the structure, and be able to produce sufficient wind speed to evaluate the performance enhancement of the retrofit system. Further, a proposed pressure tap grid and application of strain gauges, load cells and LVDTs to the scale model structure and contraction will allow for validation of the FEA and CFD models used to design the structure.

Though this experiment only considers a simple gable roof structure, the development of techniques for scale model structural testing using wind load will allow for future tests involving more complex roof shapes and roof overhangs. Additionally, future tests will incorporate different angles of attack, and further optimize the retrofit system.

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