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Performance Based Wind Engineering (PBWE): Interaction of Hurricanes with Residential Structures

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Abstract: The objective of this research project is to establish a relationship between spatially varying wind loads and structural load paths in wood-framed residential buildings. By combining wind tunnel pressure data, experimentally-derived influence surfaces from a 1/3-scale wood-framed house, and a database-assisted design methodology, researchers are predicting the load time-history on roof truss-to-wall plate connections. The predicted connection loads are compared with actual wind uplift truss load time histories measured on the scale-model home which was subjected to simulated hurricane wind flows generated using a 2,800 hp, 8-fan array Hurricane Simulator. The research results are used in concurrent projects aimed at establishing a preliminary basis for performance-based wind engineering of residential structures.

1. Introduction: The vast majority of hurricane damage losses are the result of failure of single-family wood-framed residential structures [1]. Several studies have shown these costs are inordinately borne by the US treasury, taxpayers, and the insured in post-hurricane repairs and emergency disbursements, rather than on developing improved structural systems to mitigate hurricane damage.

Hurricane losses in the United States have seen astronomical annual increases, driven by hurricane-damage to the single-family residential homes. Much of this loss is attributable to building envelope failures and roof damage (i.e. roof or wall cover, roof decking, failure of roof trusses), which is initiated by the failure of nailed wood connections. The disruption to coastal communities in Florida and elsewhere from hurricanes is quite severe. A survey of 11,559 residents of 13 central Florida counties that were affected by the 2004 hurricanes found almost half the respondents evacuated prior to at least one hurricane during that year and nearly one-third of the study-area homes sustained damage with a median loss of $11,000 per home [2].

The situation is of pressing concern to the engineering community because well over 80% of single-family residential buildings were built before improved building codes were developed after Hurricane Andrew in 1992, and many areas only recently adopted hurricane-mitigation guidelines. A grey area remains regarding the structural resistance of existing homes, as the structural load paths and distribution of forces in wood-framed residential buildings are not well understood. Thus, for the majority of existing homes, (95% of which are wood-framed construction) there is insufficient knowledge to evaluate their hurricane-resistance, which is a necessary first step to developing cost-effective retrofits for the inventory of existing homes.

Engineers have known for decades that wood roof structures are vulnerable to wind damage, as reported in a National Bureau of Science report on Hurricane Camille (1969) damage [3], and in several post-hurricane reports since that time. (Table 1). Loss of roof sheathing and failure of load transfer at joints and connections were most common. Despite the improvements in hurricane resistance of residential buildings included in current building codes, insufficient progress has been made on comprehensive retrofit strategies needed to reduce the vulnerability of coastal communities. Perhaps this is due to the fact that limited tools exist to accurately evaluate the structural capacity of these structures and assess the benefits of any retrofit option.

It is estimated that over 50% of the US population lives within 100 miles of hurricane-prone coastal areas [4] mostly in single-family, wood-framed residential structures. Conservatively, 20 to 30 million wood-framed housing units are within the

Table 1: Main finding and recommendation of various post-hurricane assessment teams (1969 -2005)

Hurricane/ Year	Main Finding/ Recommendation	Report
Camille (1969)	• Most common failures were roofs. • "… (lack of) proper anchorage**……..**"	Dikkers, Marshall, and Thom
Alicia (1983)	• Most structural damage was due to loss of roof sheathing. • "Total collapse of timber-framed houses was a common scene."	Kareem
Gilbert (1988)	• Most of the damage due to anchorage deficiencies • Continuous load path is needed	Adams; Allen
Hugo (1989)	• Roof loss with subsequent collapse of walls. • Most damage was roof and wall cladding failures resulting in extensive rain damage.	Sparks
Andrew (1992)	• Excessive negative pressure and/or induced internal pressure • Correct methods for load transfer is needed	FEMA
Iniki (1992)	• Overload on roof systems due to uplift forces. • Load path must be continuous from the roof to the foundation.	FEMA
Georges (1998)	• The shingle damage resulted in extensive water penetration and subsequent damage.	FEMA 338
Charley (2004)	• High internal pressure due to window failure was the major cause of roof loss. • Load path needs to be continuous.	FEMA 488; FEMA 490
Ivan (2004)	• Structural damage was due to sheathing loss. • ….ensure a complete load path for uplift loads.	FEMA 489; FEMA 490
Katrina (2005)	• Structural failures limited to roof sheathing loss & roof-to-wall connection failure. • A continuous load path must be present.	FEMA 548

paths of hurricanes. Most of those structures are not engineered and/or lack systematic wind-resistant designs. This research anticipates increasing need for structural engineering tools and improved fundamental knowledge to comprehensively upgrade the performance of the population of existing wood residential buildings.

The goal of this research is to initiate a paradigm shift in wood frame design methods for wind by developing the wood-framed residential structure design problem based on a system (performance-based) approach rather than a component design, as is currently the case. Significant improvement lies not in tinkering with existing designs, but changing the methodology by which we calculate and design load path and load transfer during strong winds.

The central hypothesis is that there is insufficient information in current building codes to determine risk-consistent wind design loads on typical residential structures. Additionally, there is a dearth of knowledge regarding structural load paths in light-framed wood structures, typical of residential buildings. This research investigates an analytical technique to determine structural response to spatially and temporally varying wind pressures, using a database-assisted design (DAD) methodology.

Simiu et al. [5] contend that inherent simplifications in current design procedures (of ASCE 7) produce wind loads that can differ substantially and erratically from the peak effects induced by fluctuating loads. In 1998, researchers at the National Institute of Standards and Technology (NIST) proposed an approach utilizing wind-tunnel derived wind pressure time histories and analytically-derived structural influence functions directly in the structural analysis of a portal-framed steel building [6]. This research evaluates the applicability of the DAD method to another class of building, the wood-framed residential structure.

Substantial changes to the minimum standards for construction of residential housing took place in Florida in both 1994 and 2001. Studies of the damage resulting from Hurricane Charley (2004) confirm that these code changes improved the wind resistance of both masonry wall and timber frame residential housing [7, 8]. In these studies, far less structural damage was observed for residential housing compared with that observed from previous intense hurricanes before the code changes were made (e.g. Andrew, 1992). However, relatively minimal roof damage (less than 10%) still can result in large percentage losses (80% to 100%) relative to insured values of the houses. Thus the code change experiment in Florida can be viewed as partially successful, with a few outstanding issues remaining to be addressed.

Despite the changes made to building codes, their impact on the hurricane performance of wood-framed residential structures has not always been realized. Firstly, beyond the state of Florida, there have arguably been minimal improvements in building code and construction of residential structures. Post-storm reports after Hurricane Katrina revealed similar patterns of structural failures of roof sheathing loss (Figure 1) and connection failures, that were observed in Hurricane Andrew [9]. In addition, the building envelopes of many homes continue to fail from strong winds resulting in costly water leakage and damage to the interior that continues to drive the annual economic losses from hurricanes. While Hurricane Katrina highlighted the threat of storm surges to coastal areas and areas located along rivers and waterways, Hurricane Charley showed that damaging high winds can also extend even further inland, for example, causing damage to structures in Orlando, FL, that were nearly 100 miles away from the storm's landfall.

Figure 1: Roof sheathing failures (at ridge and eave) of houses in Hurricane Katrina [9]

Two conclusions are frequently repeated in hurricane damage assessment reports of residential structures: (1) the wind load paths are not clearly understood and (2) roof design pressures are not accurate for structures in suburban neighborhoods (see Table 1). The urgency for addressing the issue is underscored by enormous losses suffered and the increasing size and frequency of hurricanes. Seven of the ten most expensive hurricanes in US history occurred during the 14-month period from August 2004 through October 2005, resulting in $73.3 billion in insured losses [10]. There is certainty that hurricanes will continue to make landfall, and without changes to current design and construction philosophies, widespread damage to residential structures will continue. A concerted effort is needed to improve the structural performance of existing homes and the life safety of their residents so that residents may feel secure staying in their home during high wind events and minimize the volume of traveling populations contributing to gridlock and pre-storm evacuation traffic.

2. Literature Review

2.1 Load Distribution: Experimental studies for structural load path in wood-frame structures are not common. In the past few decades, a few investigators [11-13] have studied the structural behavior of wood truss assemblies. Recently, Waltz [14] used a series of 3-D finite element models to define load paths and behavior of a hip roof under uniform vertical loads. He showed that "many of the load paths utilize members and connections that are not always considered to share the burden." It was further concluded that "much of the disparity between historical performance and a component-based, two-dimensional analysis can be attributed to the three-dimensional behavior of the roof that is not considered in the simplifying assumptions." Cramer and Wolfe [11] suggested that tributary-width load assignment and weakest link failure assumptions are unrealistic for designing assemblies where load-sharing is prevalent, as is the case in residential structures. However, those earlier tests focused on gravity load distributions rather than wind loads, and it is not obvious that roof systems will have symmetric behavior in response to wind loads.

The load distribution in light-frame wood roof assemblies was investigated [11, 15] using three-dimensional analytical and experimental models of a wood truss roof under gravity loads. The studies verified that an analytical model could predict member forces. A natural extension of their research relevant to this study is the investigation of load sharing effects on structures subjected to wind uplift loads (discussed above) using a 3D system [16].

Other researchers have quantified system factors for load-sharing factors for roof-truss systems [15, 17], similar to the load-sharing factor (or repetitive member factor) of 1.15 for bending [18]. Although the use of system factors is appropriate when designing a single truss, which is always a part of an assembly, the same factor may not be needed if the whole assembly is analyzed and designed as a system. After almost a decade of debate, an ASTM consensus-based guide [19] was approved in 2001. The guide "identifies variables to consider when evaluating repetitive-member assembly performance for parallel framing systems. It also discusses general approaches to quantifying an assembly adjustment including limitations of methods and materials when evaluating repetitive-member assembly performance." However, no studies are yet available showing the use of this standard in evaluating system effects in repetitive-member wood assemblies.

Roof truss assemblies used in residential buildings are usually more complicated than previously tested assemblies which may not include all of the system effects that can occur in more complex geometries. Typical residential structures include intersecting roof gables, hip-roof portions, dormers etc., which all require trusses of varying overall stiffnesses, and sizes. While full-scale laboratory experimental testing of these systems would be ideal, cost and space considerations have led many researchers to rely upon computer simulations, especially for parametric studies. However, to predict the structural behavior of an entire roof system, appropriate and reasonable assumptions are required to develop the computational model, and these assumptions must eventually be verified using physical testing.

2.2 Influence Functions: Structural influence functions are determined by applying point uplift loads over a grid of locations on the roof sand measuring the reaction loads within the structure, (in this case, at roof truss to wall plate connections and at the foundation). The influence function for a point of interest is calculated by dividing the measured load by the load applied at that point. The resulting fraction (or percentage) of the applied load provides a relative measure of the forces transferred to the reaction point of interest.

2.3 Wood Modeling: In 1997, Mani [20] used a 1/8-scale wood structural model of a roof to develop influence functions for heel joint reactions to vertically upward loads applied to the roof. Mani found favorable agreement between the gravity load influence functions he developed with those developed previously [13, 21]. He also developed influence functions for wind uplift loads applied as point loads on the roof surfaces. However, the small size of Mani's model (1 to 8) resulted in overly stiff truss plate connections.

More recent studies by Gupta et al. [22] developed a 1/3-scale model of an 8.5 m (28 ft) span Fink wood truss using metal plate and staple connectors. Gupta compared the gravity load behavior of his model truss with full-scale prototype trusses, showing that the 1/3 scale model with appropriately scaled wood members and metal truss plates can predict the behavior of a full-scale roof truss.

Datin and Prevatt [23] extended Gupta's study [22] to a three dimensional model using a 1/3-scale Fink truss roof model. In this preliminary study (Figure 2), the sheathing was modeled as beams having the equivalent flexural stiffness as the sheathing width it replaced. The beam strips were attached to the roof trusses with screw eye bolts to provide the loading points on the structure. Influence functions (shown in Figure 3) were determined for the three end trusses (six reaction points). However, it was found this model inadequately accounted for two-dimensional load distributions of the roof sheathing system.

Figure 2: Previous 1/3-scale roof model [23]

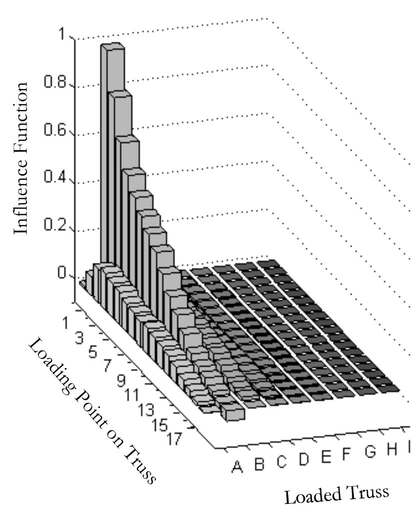

Figure 3: Influence function for one truss reaction (Datin and Prevatt 2007)

2.4 Database-Assisted Design (DAD): For their DAD study, Simiu et al. [5] developed a MATLAB-based Database Assisted Design (DAD) software program called windPRESSURE [24] that provides a graphical interface to analyze wind tunnel derived pressure time series and structural influence coefficients of their building. The wind tunnel data is stored in an hierarchical data format (HDF) file [25]. The HDF format enables the compilation and access to an extremely large dataset of wind tunnel results (e.g. exceeding 40 GB of storage for gable roof residential building). The archiving system is useful for this specific analytical study, and it has wider usage as a future contribution to the NIST database of wind tunnel test results, now in the public domain (see www.nist.gov/wind). Typical HDF files of wind tunnel data would include photographs of the model and tunnel arrangements, details on test wind speed, wind flow characteristics, pressure coefficient time history for tested wind directions at all pressure taps, and ancillary information deemed necessary to define the tests [25].

The DAD program incorporates an interpolation algorithm to enable use of the database for building structures that do not match with the dimensions of the tested building model. Using this approach the building's structural response at critical locations can be used to evaluate the level of risk-consistency in current wind load design provisions. For typical residential structures that have folded roof geometries and irregular floor plans, there is little likelihood that the simplified pressure zones in ASCE 7 [26] accurately represent the spatial pressure distributions for all wind directions. Datin and Prevatt [23] used wind tunnel results and the database assisted design approach to predict peak wind uplift loads at roof-to-wall joints within a 3D model by combining: (a) wind tunnel pressure data and (b) experimentally-determined influence functions.

3. Test Approach: Figure 4 shows a flowchart identifying the main components of this research. Wind tunnel derived pressure time histories developed at Clemson University [23] were used to determine the wind load time histories for 386 pressure taps and five wind directions. Next, a 1/3-scale wood-framed residential structure (exterior walls and roof only) was constructed and instrumented using 21 load cells. Structural influence surfaces were determined for structural reactions at (1) 8 roof truss to wall plate connections, (2) 9 wall to foundation connections, and (3) 3 connections between the wall plate and bottom chord of the end truss. The reaction load time histories are derived for these locations by analytically combining the wind tunnel pressure data, influence surface values, and an assumed tributary area of each pressure tap.

As part of the validation of the results, a second phase of the research will be to install pressure taps in the roof and walls of the building and subject it simulated (and scaled) turbulent wind flow using UF's Hurricane Simulator. In research closely related to this project, the development of a turbulent wind model is underway to ensure that reasonable flow reversal can be obtained near the leading edge of the model, and that the wind flow characterization at this model scale is representative of full-scale conditions. The comparison of structural loads with applied wind pressure distributions will provide a check against the predicted reactions developed by the analytical model/wind tunnel data/DAD approach.

4. Test Setup

4.1 Scaling Laws for 1/3 Scale Model: In order to appropriately use a structural scale model, scaling laws must be maintained. An important non-dimensional scaling law used for structural models is defined as P/EL^2, where P is the load, E is modulus of elasticity (MOE), and L is the geometric length [27]. Therefore, equating full scale to model scale:

$$\frac{P_p}{P_m} = \frac{E_p L_p^2}{E_m L_m^2} = S_E S_L^2 \qquad (1)$$

where S_E and S_L are scale factors of the full-scale (prototype) MOE and geometric length to model scale, respectively. Two important results from this scaling law relate to the applied load on the model structure and the flexural stiffness of the sheathing. Rearranging Equation 1 yields:

$$P_p = S_E S_L^2 P_m \qquad (2)$$

If the MOE is held constant between the prototype and the model, then the full-scale load (P_P) is equal to nine times the model load ($9P_m$) for a 1/3-scale structural model. Therefore, the model scale is loaded with one-ninth of the full-scale load.

With regard to the flexural stiffness (EI) of the materials, the scaling law is:

$$\left(EI\right)_p = S_E S_L^4 \left(EI\right)_m \qquad (3)$$

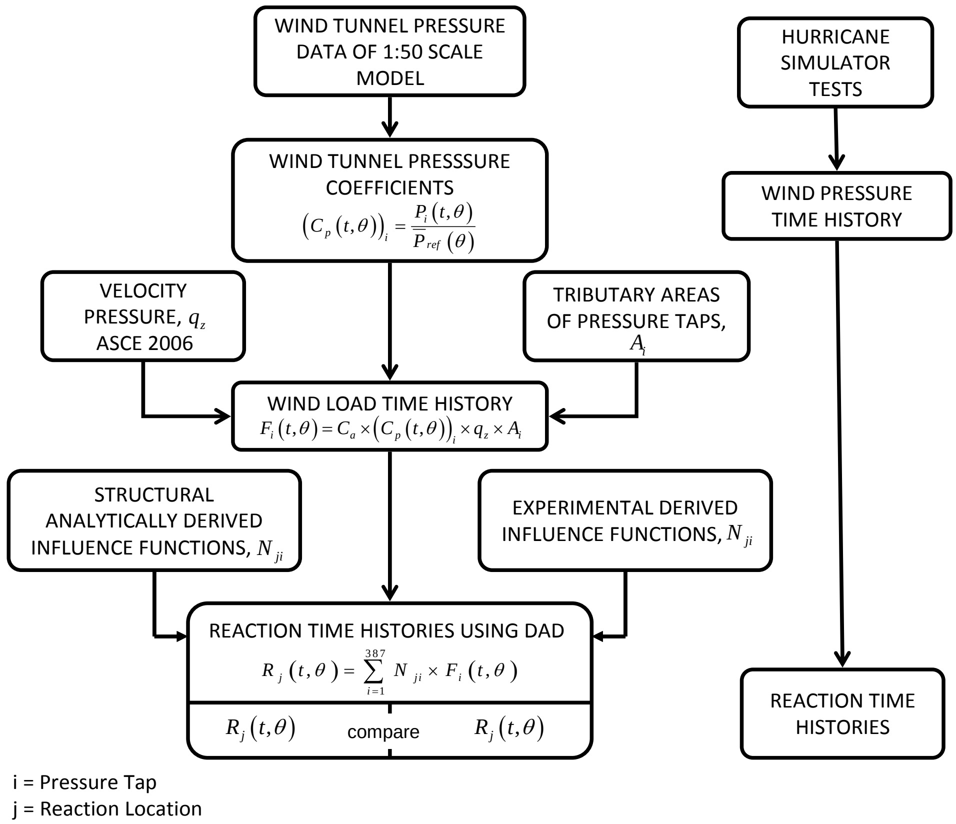

Figure 4: Flowchart for determination of reaction time histories

where I is the moment of inertia. Therefore for a 1/3-scale structure, if the MOE is held constant between prototype and the model materials, the flexural stiffness of the prototype structure would be 81 times the model's flexural stiffness.

4.2 Model Construction: The 1/3-scale wood frame house (Figure 5 and Figure 6) was constructed using trusses made from Southern Yellow Pine (SYP) lumber and truss plate connectors developed by Gupta et al. (2005). The walls were fabricated using Spruce-Pine- Fir (SPF) lumber and the wall sheathing using 0.25 in. thick oriented strand board (OSB). The nails were simulated using #4 wood screws 0.75 in. in length, installed into predrilled holes in the wood to minimize splits in the scaled 2x4 framing members.

Figure 5: 1/3-scale house framing

Figure 6: 1/3-scale house completed

The scaled framing members were cut from full size 2x4s and then planed to the desired width and thickness (0.5 in. by 1.17 in.). ASTM D 4761 (Standard Test Methods for Mechanical Properties of Lumber and Wood-Base Structural Material) was used to determine the MOE of the full-scale and model scale boards. 60 ten foot long SYP 2x4s were purchased and tested to determine the MOE based on ASTM D 4761 Method B (Flat-Wise Bending, Center-Point Loading). Since a humidity and temperature controlled room was not available to precondition the wood, the moisture content (MC) was monitored for two weeks until the readings stabilized (around approximately 11% MC). 44 of the 60 SYP boards were then tested with an average MOE of 2017 ksi and a coefficient of variation of 19.5%. The boards were then cut and planed into four scaled 2x4s which were then tested as well for MOE. 85 scaled 2x4 specimens were then tested in the same manner to determine the MOE. The average MOE was calculated as 2186 ksi with a COV of 25.3%. Using a t-test, these two values were found to be statistically equal based on a significance level of 0.05.

Several materials were tested for use as roof sheathing. Since the structure was to be loaded with simulated wind loads on the roof (i.e., out-of-plane loading on sheathing), the flexural stiffness (*EI*) was considered to be the most important property. ASTM D 3043 (Standard Test Methods for Structural Panels in Flexure) was used to determine the MOE of sheathing materials. The full-scale material selected to be modeled was ½ in. OSB. Bending tests were performed on the full-scale sheathing to set a benchmark for determining the model scale sheathing properties. Thin plywood, hardwood, and 0.25 in. thick OSB were all tested. Based on the modulus of elasticity and the thickness of the materials, the 0.25 in. thick OSB was selected as a good representation of the ½ in. full-scale OSB.

The test equipment used for the experiment includes twenty-one 300 lb tension/compression load cells (Figure 7) connected to a data acquisition device and software. A computer-controlled 200 lb pneumatic actuator (Figure 8) with a 4 in. stroke applies point loads to the roof. The applied load is monitored by an in-line tension/compression load cell that forms a feedback loop for controlling the load using a Proportional Integral Derivative (PID) control system algorithm.

4.2 Wind Tunnel Analysis: Wind tunnel tests were carried out in the boundary layer wind tunnel of the Wind Load Test Facility (WLTF, now called Wind and Structural Engineering Research Facility) at Clemson University in 2006 [23]. The WLTF wind tunnel is an open flow wind tunnel with a 14.6 m (48 ft.) long test section and a cross-section measuring 3 m (10 ft) wide by 2 m (6.7 ft) tall. Spires and roughness elements were used to simulate velocity and turbulence intensity profiles at a 1:50 geometric scale for a suburban exposure. Gradient wind speed in the tunnel was approximately 13 m/s (29 mph). Velocity profile measurements were conducted using an IFA-300 hot wire anemometer and probes, both by TSI, Inc. The 1:50 suburban terrain used had a full-scale roughness length, z_0, of 0.22 m (0.72 ft). The turbulence intensity at mean roof height was 24%. The velocity and turbulence intensity profiles are shown in Figure 9. The power spectrum of the velocity is shown in Figure 10 for a height of 10 m (33ft) as well as the well-known von Karman spectrum.

(a)

(b)

Figure 7: Load cells at (a) roof-to-wall connections and (b) foundation

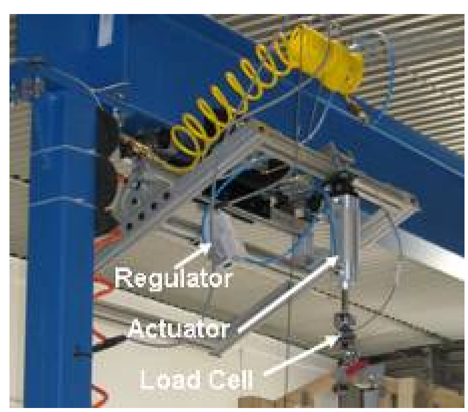

Figure 8: Load actuator setup

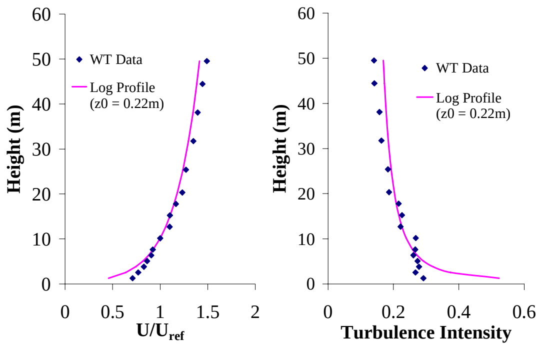

Figure 9: Mean wind velocity profile and turbulence intensity profile

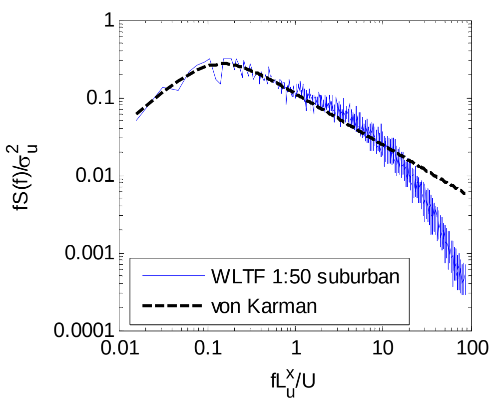

Figure 10: Power spectrum of wind speed at 10m elevation (full-scale)

The wind tunnel model, representative of a single-story residential building, has a floor plan of 30 ft wide by 60 ft long and an 18 in. roof overhang around the perimeter. The wall height is 12 ft and the roof has a 4-in-12 slope, giving a mean roof height of 14.3 ft. 387 pressure taps were installed in the roof as shown in Figure 11. The pressure taps are attached to the pressure scanners by 304 mm (12 in.) long vinyl tubes having a 1.6 mm (0.063 in.) inside diameter, and pressure data was sampled at a rate of 300 samples per second and recorded for a 120 second sample time. Surface pressure data is collected using a data collection system that consists of eight Scanivalve® ZOC 33 electronic pressure scanning modules connected to a RAD3200 digital remote A/D converter. This system provides high frequency and near-simultaneous measurements of pressures on the model. Pressure data were measured in the wind tunnel for five wind directions. Eight samples were recorded for each wind direction.

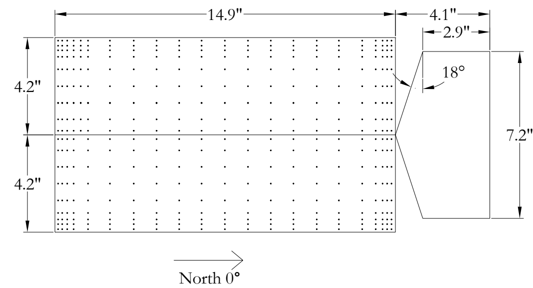

Figure 11: Wind tunnel model dimensions and pressure tap locations

The pressure time histories recorded at each pressure tap for each test run were normalized by the mean pressure measured at a reference height to generate pressure coefficient time histories using a MATLAB program. The pressure coefficient calculation is given by:

$$\left(C_p(t,\theta)\right)_i = \frac{P_i(t,\theta)}{\overline{P}_{ref}(\theta)} \qquad (4)$$

where $\left(C_p(t,\theta)\right)_i$ is the pressure coefficient of the i-th tap at time t for wind direction θ. $P_i(t,\theta)$ is the pressure recorded at the i-th tap at time t for wind direction θ. $\overline{P}_{ref}(\theta)$ is the mean pressure recorded at the reference height for direction θ.

The tributary areas of the pressure taps were evaluated to develop the load time series at each tap. The developed pressure coefficient time series were converted to equivalent 3-second gust coefficients referenced to mean roof height by developing an adjustment factor, C_a, using the method outlined by Simiu and Scanlan [28].

The velocity pressure, q_z, given in ASCE 7-05 [26] is computed as:

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \ \text{(psf)} \qquad (5)$$

where K_z is the exposure coefficient, K_{zt} is the topographic factor, K_d is the directionality factor, V is the wind speed in mph, and I is the importance factor. However, in the current analysis, Equation 5 is modified to:

$$q_z = 0.00256 V_{3sec,14.3ft,sub}^2 \qquad (6)$$

where $V_{3sec,14.3ft,sub}$ is the design 3-second gust wind speed at mean roof height (14.3ft) in suburban terrain. $V_{3sec,14.3\,ft,sub} = 80.3mph$ for a 130mph design wind from ASCE 7 for Exposure B. K_z and K_d are accounted for in the wind tunnel pressure coefficients, and I is taken as unity.

The wind load time history of each pressure tap is then calculated as:

$$F_i\left(t,\theta\right) = C_a \times \left(C_p\left(t,\theta\right)\right)_i \times q_z \times A_i \quad (7)$$

Where $F_i\left(t,\theta\right)$ is the load in pounds on the *i*-th pressure tap at time t for wind direction θ, and A_i is the tributary area of the *i*-th pressure tap.

Using the DAD methodology, the influence functions measured experimentally with the 1/3-scale house are then combined with the wind load time histories to determine the dynamic load time histories at the roof-to-wall connections as follows:

$$R_j\left(t,\theta\right) = \sum_{i=1}^{n} N_{ji} \times F_i\left(t,\theta\right) \qquad (8)$$

Where $R_j\left(t,\theta\right)$ is the load time history at the *j*-th reaction for wind direction θ and N_{ij} is the influence function at the *i*-th pressure tap for the *j*-th reaction.

7. Future Work: After developing the wind load time histories using the DAD methodology, the 1/3-scale house will be subjected to actual winds produced by the Hurricane Simulator at the University of Florida (Figure 12). The Hurricane Simulator generates hurricane force winds by using an 8-fan array consisting of 4.5 ft diameter fans hydraulically powered by four marine diesel engines with a combined 2800 hp. The Simulator has the capability of maintaining a 120 mph wind out of an opening approximately 10 ft by 10 ft. Dynamic effects are introduced by fins that oscillate side to side at the exit of the Simulator. The house will be instrumented with pressure taps to simultaneously record actual wind pressures and structural loads. The locations of the pressure taps will coincide with some of the loading points for the influence functions and the DAD methodology will be used to determine the roof-to-wall connection time histories. These will then be compared with the actual time histories at the roof-to-wall connections measured during the simulator testing.

Figure 12: Hurricane Simulator at University of Florida

8. Acknowledgements

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