Wind Uplift Reactions at Roof-to-Wall Connections of Wood-Framed Gable Roof Assembly

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ABSTRACT

The extent of roof failures that occur in wood-framed residential structures during strong winds and hurricanes has continued to raise the debate whether current design methods are sufficiently conservative or whether wind forces in hurricanes exceed our current estimates. In an effort to address these concerns, this research investigated a novel design method for determining the roof-to-wall wind uplift load on wood-framed roof trusses in residential construction. The rationale for this study hinges on the fact that roof-to-wall connections are critical to the vertical load path and they are shown to be a weak link with premature failures during hurricanes.

This report presents the methods, test setup, results and conclusions of an investigation of the structural load transfer mechanisms in roof-to-wall connections of light, wood-framed low-rise, residential structures. The work uses boundary layer wind tunnel simulated wind pressures and results of a structural model of a gable roof structure to model roof truss reaction loads. The overall objective of this project is to determine why roof-to-wall connections continue to fail prematurely in high wind events and to investigate if the design loads prescribed in ASCE 7 are sufficient for these connections.

Experimentally derived influence functions are determined from a one-third scale model of a gable roof truss assembly. Using time histories of wind uplift pressures from xxx pressure taps distributed on the roof of a 1:50 scale wind tunnel model, the time history of the roof-to-wall reaction loads are determined using a database-assisted design methodology, first proposed by scientists at the National Institute of Standards and Technology, Gaithersburg, MD. Wind tunnel tests were conducted at the Wind Load Test Facility (WLTF) boundary layer wind tunnel. Reaction load time histories were developed using: (1) simple tributary areas of the trusses and (2) influence functions of the truss reactions. A comparison of these results showed that the influence function approach (DAD method) produces larger and more variable loads than using the tributary area method.

These promising results suggest that the current design philosophy of using tributary areas may not be conservative. The DAD procedure will lead to more risk-consistent designs for wood-framed residential structures leading to reduced structural damage, cost savings, and safer buildings.

The author wishes to acknowledge the contribution of the Clemson University graduate and undergraduate students who participated at various levels in this research. Civil engineering graduate student, Mr. Peter L. Datin handled the day-to-day management of research and testing. The author acknowledges Mr. Datin’s extensive contributions that included conducting wind tunnel tests, structural modeling and overseeing the work of undergraduate student assistants. The report in large measure work reflects the results of Mr. Datin’s CE 889 semester project supervised by the Principal Investigator.

KEYWORDS: Wind tunnel tests, Aerodynamics, boundary layer, buildings, wind engineering, full-scale, wind tunnels, suburban terrain, residential, structural tests, database-assisted design.

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1. INTRODUCTION

1.1 Scope of Current Work

The purpose of this report is to study the structural load path phenomena in light-frame wood roof assemblies subjected to wind uplift loads. The study has as its motivation to understand the structural behavior of these systems that make them susceptible to premature failure during hurricanes and extreme wind events. While the literature has several reports of previous studies on roof assemblies subjected to gravity loads, there is insufficient existing knowledge on the structural behavior of wood-framed roof assemblies that are subjected to wind uplift loads. Experimental work was conducted to quantify the reaction loads at the roof structure-to-wall connections for a structure under wind uplift loads. The research utilizes a typical residential wood-frame roof assembly as the structural model.

Strong wind events continue to wreak havoc on wood-framed residential construction, and they cause severe financial and economic losses to the owner, occupants, and to the society. Post-hurricane damage investigations have shown that wood structures tend to significantly less damage when the roof system remains intact under extreme wind loading, while major damage occurs when the roof system is partially or completely damaged (Reed et al. 1997).

It is estimated that wood-frame buildings account for approximately 90% of all residential buildings constructed in the United States (Li 2005). In addition, approximately 50% of the United States population now lives within 100 miles of a hurricane-prone coastline (Alvarez 2000). Yet little research has been conducted to develop an analytical model of the structural performance of wood-frame structures under high wind events.

1.2 Overview of Experimental Study

This project is accomplished using experimentally derived influence (or load-transfer) functions measured from a 1:3 scale model of a gable, wood-truss roof assembly at the truss reactions and wind loading data derived from wind tunnel tests on scale model residential buildings. A time-history of reaction loads at the roof-to-wall connection is determined using influence functions for the reaction loads due to vertical uplift loads on a gable-roof and then combined with spatially distributed wind-tunnel derived pressures determined for a 1:50 scale model house. Wind tunnel tests were conducted in the atmospheric boundary layer wind tunnel of the Wind Load Test Facility (WLTF) at Clemson University under simulated suburban terrain conditions.

The wind tunnel model was tested for five wind directions (0º, 45º, 90º, 135º, 180º with 0º as North and 90º as East). Using the time histories of the pressure taps from these wind directions, wind load time histories were developed for the roof-to-wall connections using two different methodologies: (1) simple tributary areas of the trusses and (2) influence functions of the truss reactions. Results of the two different reaction time histories reveal that using influence functions produces larger and more variable loads than just using the tributary areas utilizing the same pressure coefficient time histories. Results show that the current design philosophy of using tributary areas may not be conservative for all wind directions.

This methodology of using influence functions and wind tunnel pressure coefficient time histories to determine roof reaction loads to wind forces is based on the database-assisted design (DAD) methodology first proposed by the National Institute of Standards and Technology (NIST), and adapted to the wood-framed residential structural system. It is expected that widespread use of the

DAD procedure will lead to more risk-consistent designs for wood-framed residential and other structures, leading to reduced structural damage, cost savings, and safer buildings.

1.3 Layout of Report

Chapter 2 focuses on the experimental setup of both the wind tunnel study and the structural testing, including a description of the instrumentation and data collection. Chapter 3 presents the research results, and Chapter 4 summarizes the study and discusses the conclusions and recommendations for future research needs.

2. METHODS AND MATERIALS

This chapter focuses on the experimental setup for both the wind tunnel testing and the structural testing.

2.1 Wind Tunnel Testing

The wind tunnel housed in the Wind Load Test Facility (WLTF) at Clemson University is a large atmospheric boundary layer wind tunnel (see Figure 2.1) meaning that develops wind flow characteristics that model the variability in near-ground natural air flow.

Figure 2.1 – Boundary layer wind tunnel at Clemson University

2.1.1 *WLTF Wind Tunnel Arrangement*

The wind tunnel at the WLTF has cross-sectional dimensions of approximately 10 ft (3 m) wide by 6.5 ft (2 m) tall and an overall working section of approximately 48 ft (14.6 m) in length. The wind flow is generated by two six-foot diameter AC-powered axial fans. After air is drawn into the tunnel through the fans, it immediately passes through a series of screens and a honeycomb-like grid to establish uniform flow across the section before entering a contraction section. The contraction section serves to increase the wind speed in the working test section. At the entrance to the test section, a series of spires and trip boards are placed over the opening to generate the wind turbulence variation in velocity with height that simulates the natural turbulence of air near the ground (i.e. the boundary layer).

After passing the spires, the air then passes over a series of blocks and/or slant boards of different sizes and shapes. These obstructions along the tunnel floor serve to reduce the wind velocity near ground and increase the turbulence of the air. By using several arrangements of the roughness elements and combinations of spire sizes and trip boards, the wind tunnel can be used to simulate different terrain exposures at different geometrical scales. This study utilized a 1:50 scale suburban terrain (see Figures 2.2 and 2.3).

Figure 2.2 – Wind tunnel arrangement for 1:50 suburban terrain

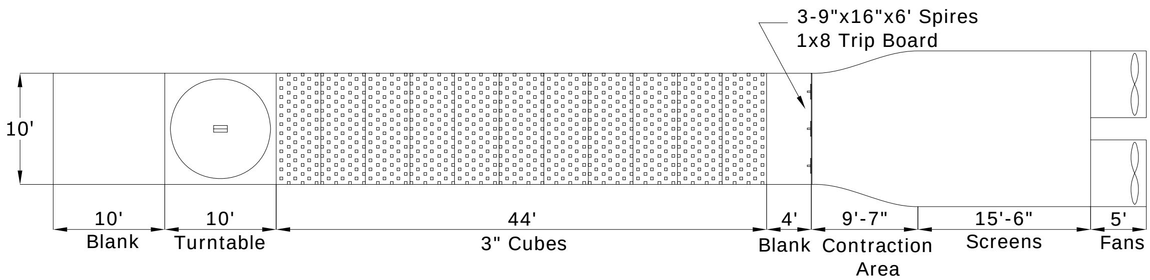


Figure 2.3 – 1:50 suburban terrain wind tunnel arrangement at the WLTF at Clemson University

2.1.2 *WLTF Wind Tunnel Testing Procedure*

Pressure tap models are constructed from plexi-glass so as to be rigid. Metal taps 0.063 in. (1.6 mm) in outside diameter are glued into the plexi-glass and are attached to miniature electronic pressure scanners by 12 in. (304 mm) long vinyl tubes. The pressure scanners are manufactured by Scanivalve®. The current setup uses ZOC33 electronic pressure scanners which have 64 channels capable of scanning pressures up to 500 Hz, allowing for near simultaneous measurements of pressures on the model. A RAD3200 digital remote A/D converter base unit allows for up to eight of the ZOC33 scanners to be connected at the same time allowing up to 512 channels to be used simultaneously. This allows for closely spaced pressure taps to cover an entire model roof.

The wind tunnel test model for this project consisted of a simple gable roof building having a 18.4^{o} (4 in 12) roof slope of with scaled dimensions of 7.2 in. (183 mm) by 14.4 in. (387 mm) in plan and a mean roof height of 3.3 in. (84 mm) (see Figure 2.4a). This model is one of the five gable roof models of the Clemson Standard Models set used at the WLTF (see Figure 2.4b). 387 pressure taps are installed on the roof as shown in Figure 2.5. The pressure data was sampled at 300 Hz and recorded for 120 seconds. Eight of these 120-second runs were recorded for each wind direction. In the analysis, these eight time histories were each divided into two sub-time histories resulting in sixteen one-minute pressure time histories for each wind direction.

(a)

(b)

Figure 2.4 – (a) 4 in 12 sloped gable roof model used for this study and (b) The entire Clemson Standard Model (CSM) set

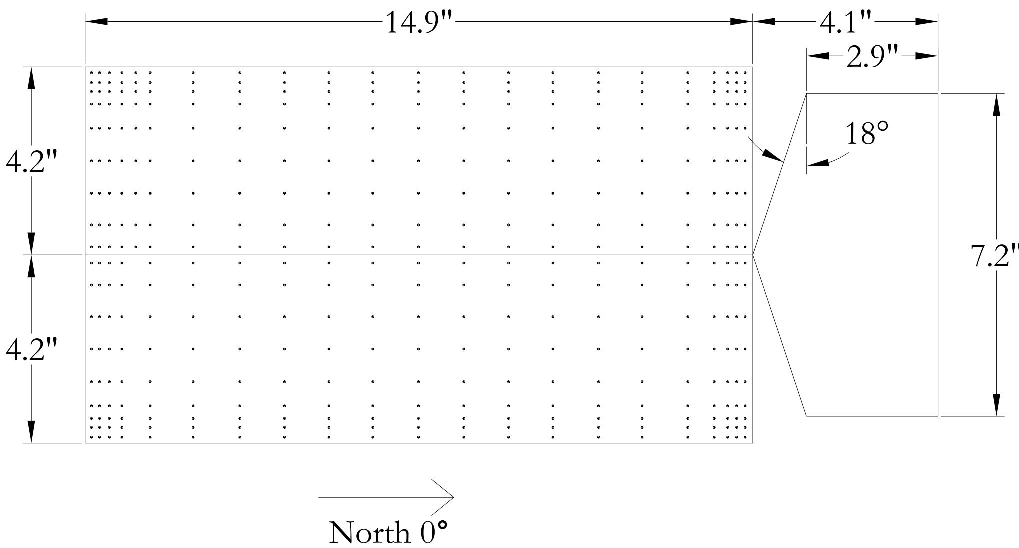


Figure 2.5 – Scale model dimensions and tap locations for 4 in 12 roof slope model

2.2 Structural Model Roof Setup

To conduct the influence function tests, a one-third scale wood-truss roof was fabricated following methods proposed by Gupta et al. (2005). A one-third scale model of a typical wood-framed gable roof structure is feasible and relatively easy to construct and test. The highlights of the model roof are the inexpensive materials needed, the ability to construct a much larger and more complex roof inside a confined laboratory space, and the need for less expensive and lower capacity data acquisition equipment (i.e. load cells, etc.).

The nine scaled trusses used were constructed using No. 2 Grade Southern Yellow Pine (SYP) 2x4 dimensional lumber. The 1:3 scale model truss members were fabricated from 2x4s with scale dimensions of 0.5 in. (12.7 mm) and 1.17 in. (29.6 mm). These dimensions were achieved by first ripping the 2x4s into four pieces and then using a planer to shave the rest of the excess wood until the desired thicknesses were achieved. The scaled truss dimensions are shown in Figure 2.6 along with the truss loading points.

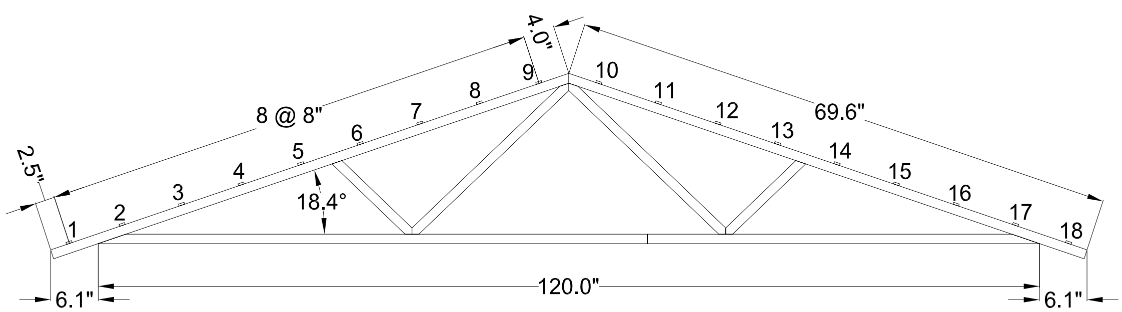


Figure 2.6 – Truss layout and numbering

Roof sheathing is typically 4 by 8 ft oriented strand board (OSB) or plywood. The roof sheathing for the one-third scale roof was modeled using strips of red oak spaced at 8 in. on center to provide the scaled flexural stiffness (EI) of the tributary area of the strip. This 8 in. spacing corresponds to a 24 in. wide section of the full-scale sheathing or half of a sheet width. Red oak strips were fabricated from 1x4s obtained at a local lumber supplier in a similar manner to the scaled 2x4s. The full-scale roof sheathing modeled was 15/32 in. OSB which is the typically used material for residential construction in coastal South Carolina. More work is still needed for further validation of modeled roof sheathing.

Figure 2.7 shows the completed 1:3 scale roof in the reaction frame with the loading mechanism, and Figure 2.8 shows the schematic of the roof truss layout and numbering system. The slat boards used to model the sheathing were attached to the trusses using metal eye hooks with an overall length of 1.8125 in. (46 mm) and a diameter of 0.191 in. (4.85 mm). A tension/compression load cell was attached to each reaction through a setup designed to represent the top plate width with the trusses attached using a strap (see Figure 2.9). The actual details of the roof-to-wall connector were not modeled since the focus of this experiment was to determine the load distribution of the roof through the roof-to-wall connections in order to quantify the loads that need to be transferred through this connection. In other words, actual hurricane clips (for example the Simpson Strong Tie H2.5A connector) were not modeled since we were not interested in the actual performance and capacity of these connectors. Much testing has been done on these types of connectors and their failure capacity is well documented. The focus was only to determine the actual loads transferred through these connections, and therefore, the structural capacity of the connector was designed so that it would perform without failure under these test conditions.

Figure 2.7 – Completed 1:3 scale model roof

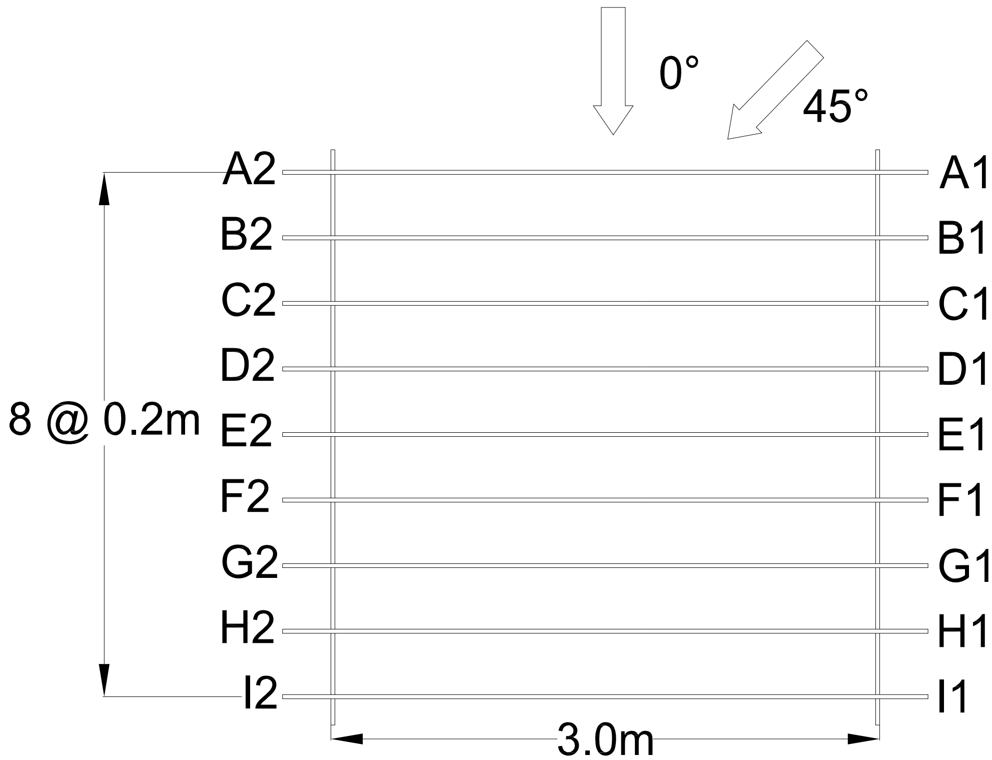
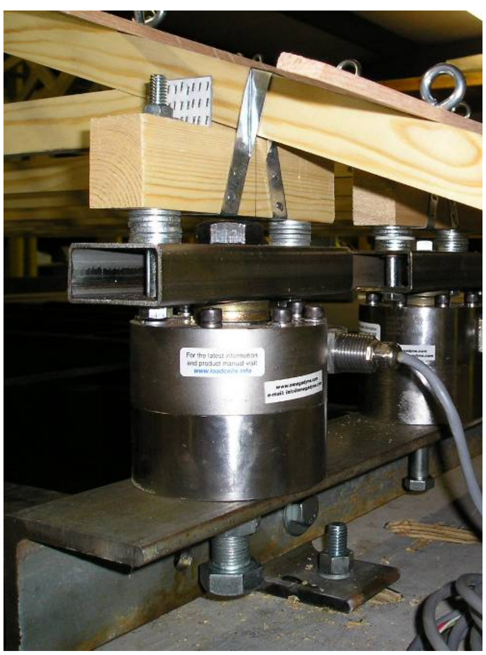


Figure 2.8 – 1:3 scale model truss dimensions and loading points



(a)

(b)

Figure 2.9 – Roof-to-wall connections: (a) Load cell configuration and (b) Dummy spacer

Data acquisition (DAQ) was accomplished through the use of a USB based DAQ system manufactured by National Instruments called Compact DAQ. This is a mobile and versatile system with multiple types of "modules" that can be inserted in the Compact DAQ chassis to perform a number of different functions. The DAQ measurements were controlled through LabVIEW, a software package developed by National Instruments that allows users to build programs to acquire a desired signal, perform preliminary calculations, and record the data in a variety of formats for further analysis later.

The load cells used were manufactured by Omega Engineering, Inc. The load cells used for the truss reactions consisted of 1,000 lb and 5,000 lb capacity load cells capable of measuring loads in compression and tension. The load cell used for the load application was an S-type load cell with a capacity of 100 lbs.

3. ANALYSIS AND RESULTS

This chapter presents the results of the current study including the determination and selection of the model boards used to construct the 1:3 scale roof trusses and roof sheathing, as well as the results of the experimentally derived influence functions and the resulting truss reaction wind load time histories using both simple tributary area and influence function methodologies.

3.1 Wind Tunnel Testing of Clemson Standard Model

In order to calculate the wind loads on the roof, the design wind speed needs to be converted to mean roof height. The design wind speed used in this study was obtained from the reference standard *Minimum Design Loads for Buildings and Other Structures* (ASCE 2006) also known as ASCE 7-05 published by the American Society of Civil Engineers. Along the South Carolina coast, ASCE 7-05 recommends a three-second gust design wind speed of 130 mph (58 m/s), at a 33 ft (10 m) reference height in open terrain exposure. The house analyzed is located in suburban terrain so the equivalent design wind speed was determined for suburban terrain and is equal to 80.3 mph referenced to mean roof height in suburban terrain.

The wind load, F (in lbs), at each pressure tap on the wind tunnel model is calculated by the following equation:

$$F = 0.00256 C_p C_a A V^2_{3\text{sec},mrh,design,sub} \tag{3.1}$$

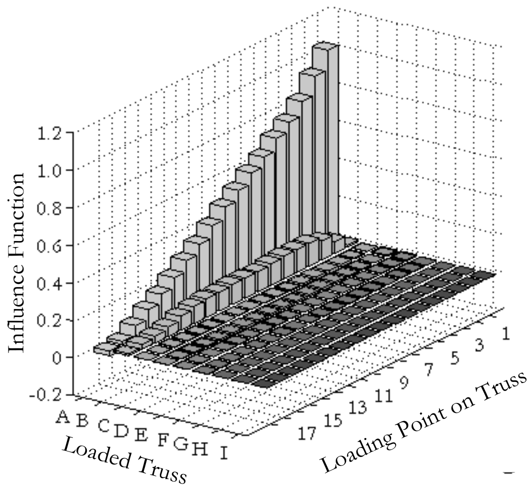
where: C_p is the wind pressure coefficient referenced to the pitot height measured in the wind tunnel; C_a is an adjustment factor to convert wind pressure coefficients to the mean roof height of the building; A is the tributary area in ft^2 of each pressure tap; $V_{3\sec,mrh,design,sub}$ is the 3-sec design wind speed at mean roof height in suburban terrain (80.3 mph for this experiment); and 0.00256 is one-half the density of air ($\frac{1}{2}\rho$).

3.2 Influence Functions

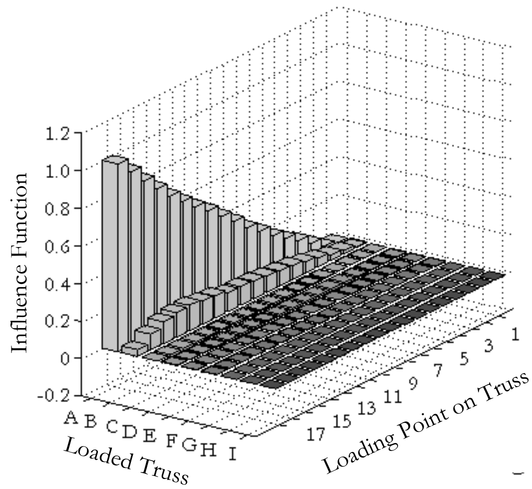
Influence functions were determined for the reactions of the first three trusses (A-C) (see Figure 2.8). Eighteen loading points per truss (corresponding to the intersection of the oak strips and the truss) were used to develop the influence surface. Figure 2.6 shows the dimensions of the model truss and the loading points.

A fixed load of approximately 100 pounds was applied at each loading point and the corresponding truss reactions were measured. This procedure was repeated for all 18 load points on each truss. The influence function for a reaction due to a specific loading point is determined by calculating the ratio of the applied load (approximately 100 lbs) and the truss reaction load measured by the load cell. This ratio normalizes the applied load and presents the results in a coefficient form. The results provide an influence surface for the reactions representing the load sharing between the trusses through the two-way action of the sheathing.

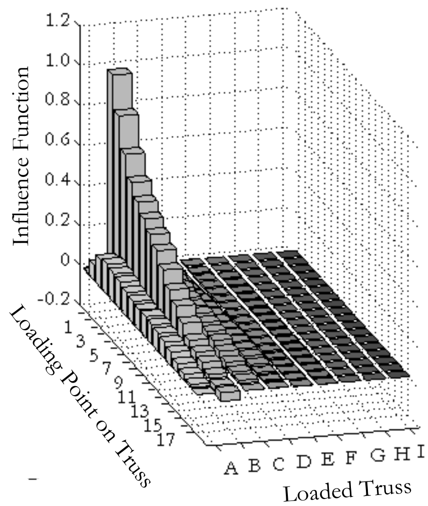
Figure 3.1 shows the influence surfaces for the six reactions. The shape of these influence surfaces is important. The influence surface is largest at the reaction location, for example Reaction A1 has the largest influence functions when the roof is loaded near that reaction. As the load is applied further from the reaction, the load decreases. If the load is applied to the same truss as the reaction, the influence functions are much larger than for adjacent trusses. It is of interest to note that the influence surfaces normalize to approximately zero once the load is applied to trusses that are at least two or three trusses away from the reaction truss. This means that a reaction is only being affected by loads applied to the two or three trusses most adjacent to the reaction truss on either side. These influence surfaces are also only for the 15/32 in. (11.9 mm) OSB sheathing.

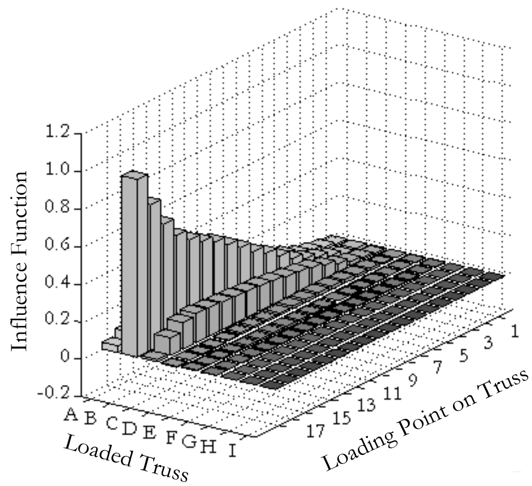


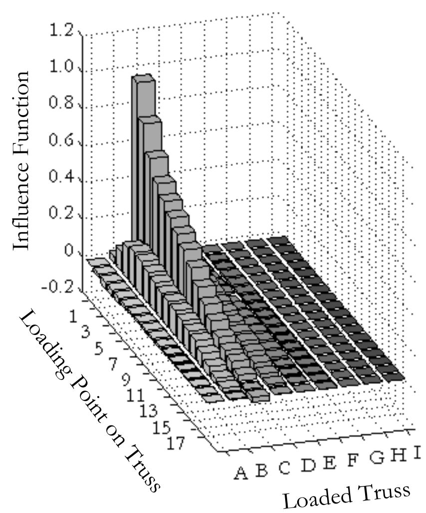
Reaction A1



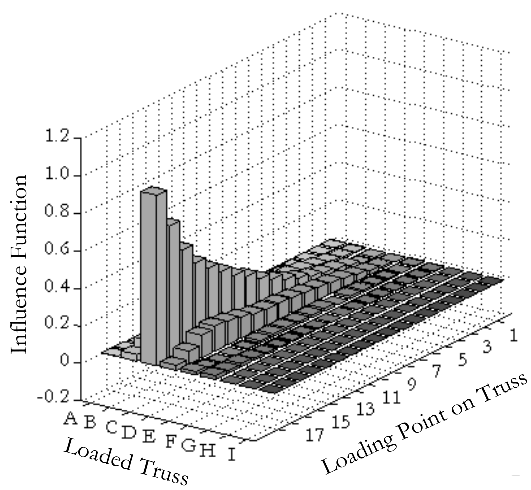
Reaction A2







Reaction C1



Reaction C2

Figure 3.1 – Influence surfaces for truss reactions

3.3 Roof-to-Wall Connection Wind Load Time Histories

Once the influence functions were generated and the wind tunnel pressure time histories were collected, reaction (roof-to-wall connection) load time histories could be computed. This provides a different way to analyze the loads from previous research. In the past, generally the most important load was only the peak load and calculating a few basic statistics. A time-history of the load provides a more realistic look at the loads and is the main input into a database assisted design (DAD) procedure to provide more a risk consistent loading procedure and subsequent design loads. A series of MATLAB programs were written to combine the pressure time histories and the influence functions to determine the reaction loads.

3.3.1 *Calculation of Reaction Load Time Histories*

The first step in determining the reaction wind load time histories was determining the tributary areas of both the wind tunnel pressure taps and the influence function load points. The tributary areas for pressure taps were then overlaid on the tributary areas for the loading points. The intersection of the pressure tap tributary areas with influence function load points were then determined. Figure 3.2 shows how this process works. Two assumptions were made in determining these areas.

1. The wind pressure coefficients for each pressure tap represent the uniform pressure over the pressure tap tributary area.
2. The influence function for each load point represents the behavior of the structure over the tributary area of that load point.

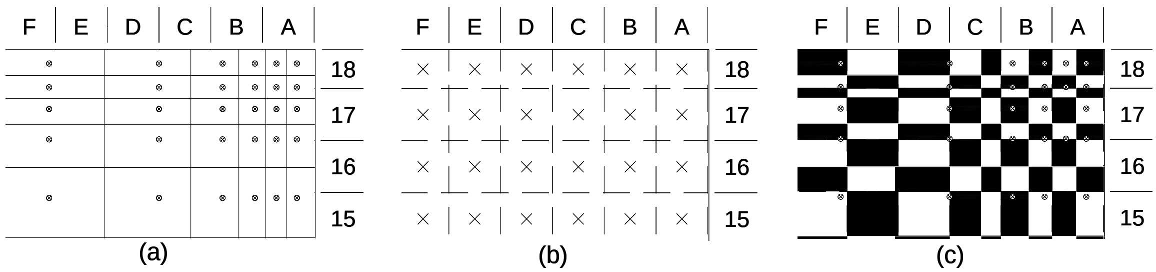


Figure 3.2 - Tributary areas to develop DAD reaction loads.: (a) Pressure tap areas , (b) Influence function loading point tributary areas , and (c) Resultant overlaid wind load areas for each truss loading point..

The influence functions and wind load time-histories were used to analytically determine the time-history of the reaction of at each roof-to-wall connector. For comparison purposes, this reaction wind load time-history determined using DAD was compared to a time-history developed using the accepted tributary area procedure of the ASCE 7-05. The tributary area of each of the wind tunnel pressure taps was overlaid on the tributary areas of the trusses (in a similar manner as described in Figure 3.2). This was done for comparison to the method currently used in design of using only the tributary areas of the individual trusses to determine the reaction loads. The truss reactions were determined by statics using the wind loads determined utilizing the experimentally collected wind pressure coefficient time histories and Equation 3.1.

3.3.2 *Analysis of Reaction Load Time Histories*

Figure 3.3 shows a typical wind load time history for reaction A1 for both the tributary area and influence function methods. It is obvious there is an increase in the peak wind load magnitude obtained using the influence function (DAD) methods over the tributary area method. Table 3.1 shows some basic statistics of the reaction time histories. There is a significant visual difference in the time histories between the two methods. Using the influence functions produces peak loads that are much higher than using only the tributary areas. The mean value for the influence function method exceeds the tributary area mean by 14 to 97%. The maximum uplift load and the 99th percentile load also increase 10 to 92% from the tributary area method to the influence function method.

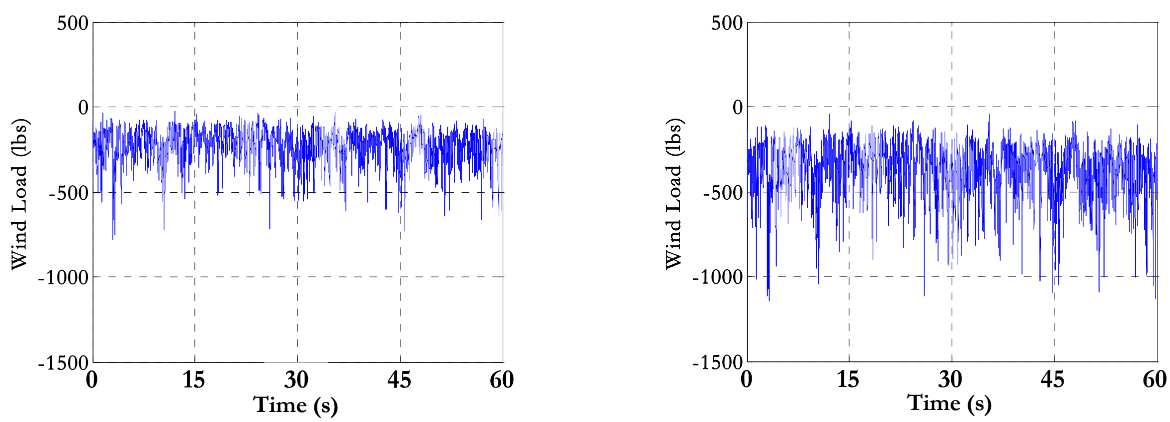


Figure 3.3 – Typical roof-to-wall reaction wind load time histories for reaction A1 using (a) Tributary area methodology and (b) Influence function methodology.

TABLE 3.1 – Wind load time histories statistics (lbs)

Reaction	Wind Direction	Tributary Area			Influence Function		
		Mean Load	Mean Max Uplift Load	99th Percentile Load	Mean Load	Mean Max Uplift Load	99th Percentile Load
A1	0º	214	768	474	351	1236	776
	45º	156	754	448	210	1215	699
	90º	67	271	166	104	454	264
	135º	70	221	147	108	361	230
	180º	30	141	85	48	232	136
A2	0º	215	734	475	346	1106	735
	45º	244	734	481	317	1097	690
	90º	113	350	227	155	473	317
	135º	93	302	197	131	397	269
	180º	33	138	89	50	206	135
B1	0º	194	671	422	301	1024	641
	45º	117	581	329	167	819	472
	90º	62	233	149	99	408	246
	135º	68	215	143	114	373	240
	180º	27	140	82	47	223	134
B2	0º	178	639	390	316	1045	670
	45º	203	602	401	298	885	596
	90º	105	309	210	157	470	317
	135º	88	303	190	145	423	291
	180º	26	133	83	51	221	142
C1	0º	171	602	369	299	1037	630
	45º	132	435	285	220	726	462
	90º	73	270	168	137	493	305
	135º	88	264	178	173	505	342
	180º	33	146	91	64	269	166
C2	0º	170	610	370	216	730	450
	45º	200	579	393	228	631	432
	90º	107	314	212	137	377	261
	135º	90	301	190	133	387	259
	180º	26	131	83	42	176	114

3.4 Extreme Value Analysis of Reaction Time Histories

One of the most important values in the reaction load time histories is the peak, or extreme, value. Since the reactions each have sixteen time histories, an extreme value analysis can be performed. A Type I Extreme Value (Gumbel) distribution is fitted to the data for each reaction. The probability density functions (pdf) and the cumulative distribution functions (cdf) are shown in Figures 3.4-3.11 for reactions A1 and B1 for wind directions 0 ° and 45° and for both the tributary area and influence function methodologies.

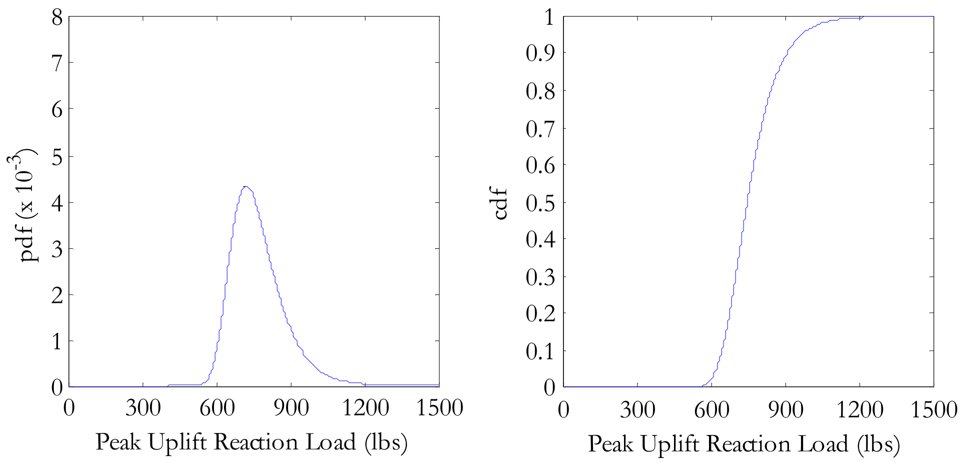


Figure 3.4 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction A1 for a wind direction of 0° for tributary area loading methodology

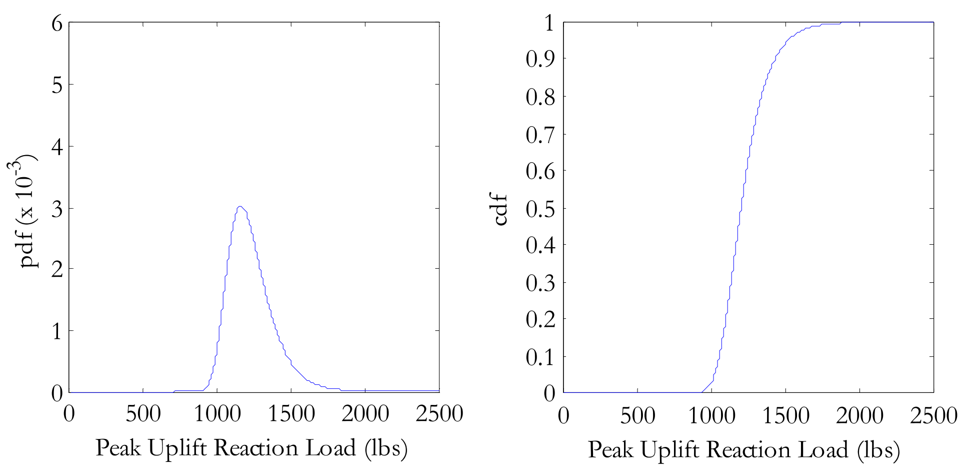


Figure 3.5 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction A1 for a wind direction of 0° for influence function loading methodology

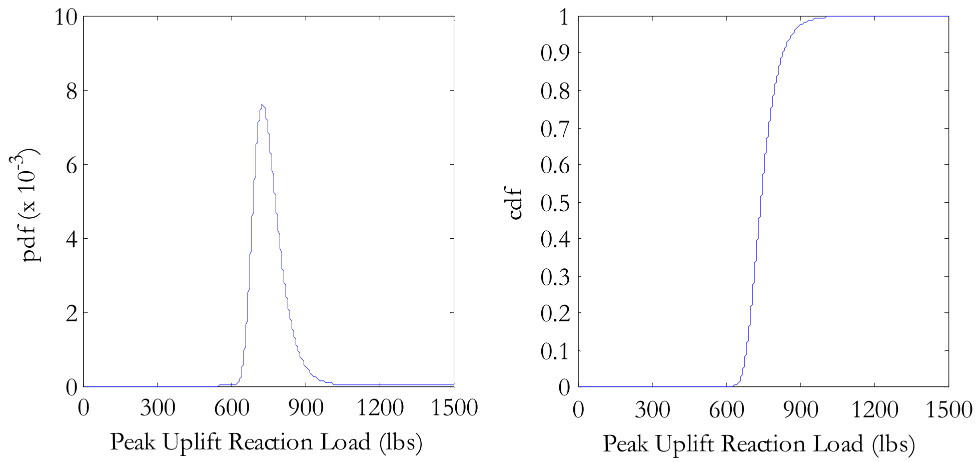


Figure 3.6 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction A1 for a wind direction of 45° for tributary area loading methodology

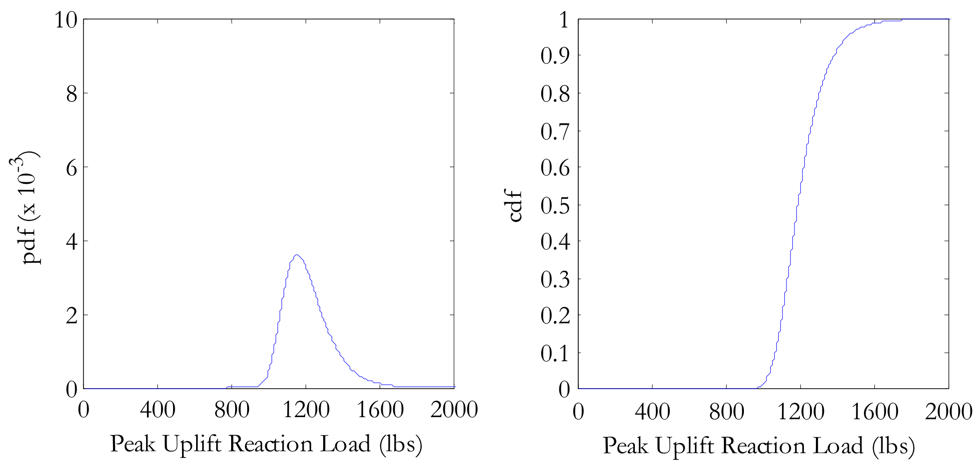


Figure 3.7 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction A1 for a wind direction of 45° for influence function loading methodology

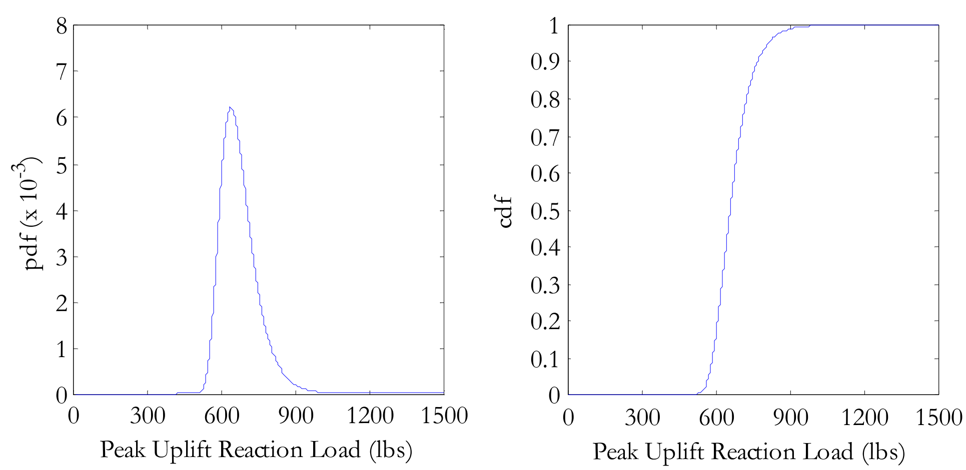


Figure 3.8 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction B1 for a wind direction of 0° for tributary area loading methodology

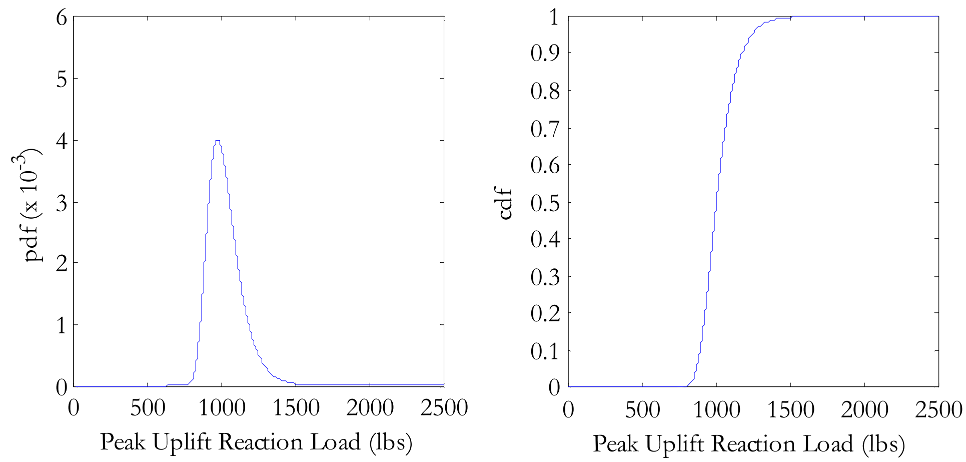


Figure 3.9 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction B1 for a wind direction of 0º for influence function loading methodology

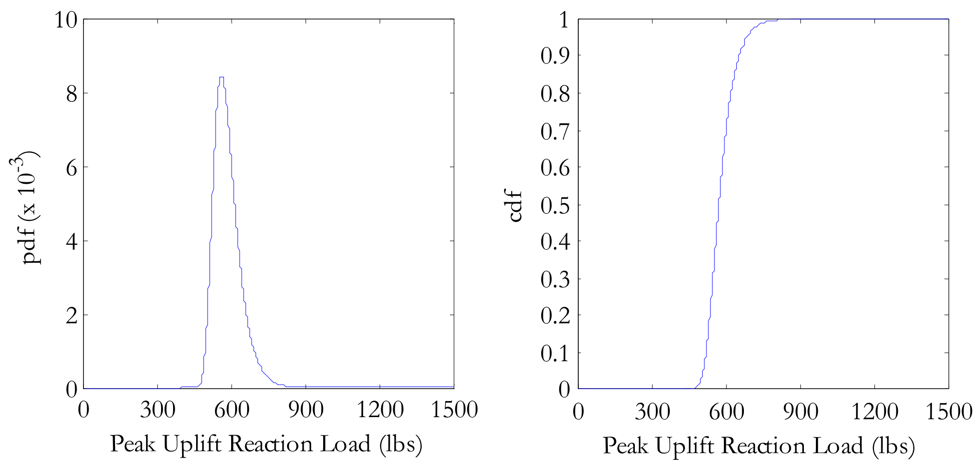


Figure 3.10 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction B1 for a wind direction of 45º for tributary area loading methodology

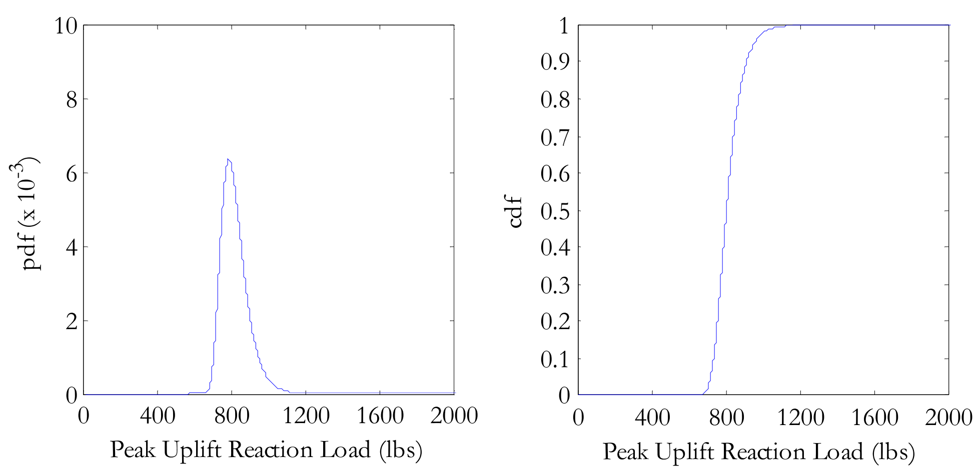


Figure 3.11 – Probability distribution function (PDF) and cumulative distribution function (CDF) for peak uplift reaction loads for Reaction B1 for a wind direction of 45º for influence function loading methodology

3.4.1 *Comparison of Tributary Area and Influence Function Generated Distributions*

Tables 3.2 and 3.3 show that there is a large discrepancy between the mean values of the distributions for the tributary area and the influence function generated reaction time histories. The influence function generated mean peak loads are 40-70% larger than the corresponding tributary area mean peak loads.

TABLE 3.2 – Type I Extreme Value Distributions Mean and Standard Deviation for Tributary Area Peak Negative Reaction Loads

Reaction	0º		45º		90º		135º		180º	
	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)
A1	767	108.8	753	62.0	269	34.0	221	18.8	140	19.8
A2	739	69.5	731	69.8	350	44.3	304	39.4	137	17.0
B1	672	75.7	583	55.6	233	32.2	215	20.7	139	21.1
B2	635	73.4	604	53.9	309	34.6	304	46.4	133	16.8
C1	601	81.0	435	49.1	271	31.7	263	27.8	146	15.9
C2	606	69.1	582	55.2	315	32.8	300	40.5	131	15.9

TABLE 3.3 – Type I Extreme Value Distributions Mean and Standard Deviation for Influence Function Peak Negative Reaction Loads

Reaction	0º		45º		90º		135º		180º	
	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)	Mean (lbs)	Std (lbs)
A1	1232	156.4	1214	130.4	450	58.0	362	36.8	231	39.9
A2	1105	84.0	1089	108.6	472	57.4	397	41.5	206	24.5
B1	1026	117.6	818	73.9	408	46.8	373	36.6	223	32.8
B2	1046	114.9	884	80.2	473	62.5	424	49.8	221	26.2
C1	1038	138.7	728	85.2	492	49.8	502	49.2	271	33.3
C2	726	82.9	631	54.1	379	45.5	388	47.1	176	18.7

Figure 3.12 shows the cumulative distribution functions (CDF) of both the tributary area and influence function generated peak values for all six reactions for a wind direction of 0º. It is interesting to note the large difference between the distributions. These graphs help to reinforce the no-

tion that the commonly used and accepted tributary area methodology is non-conservative and careful consideration needs to be given to the assumptions made in using this methodology in design, specifically of wood-frame roof assemblies.

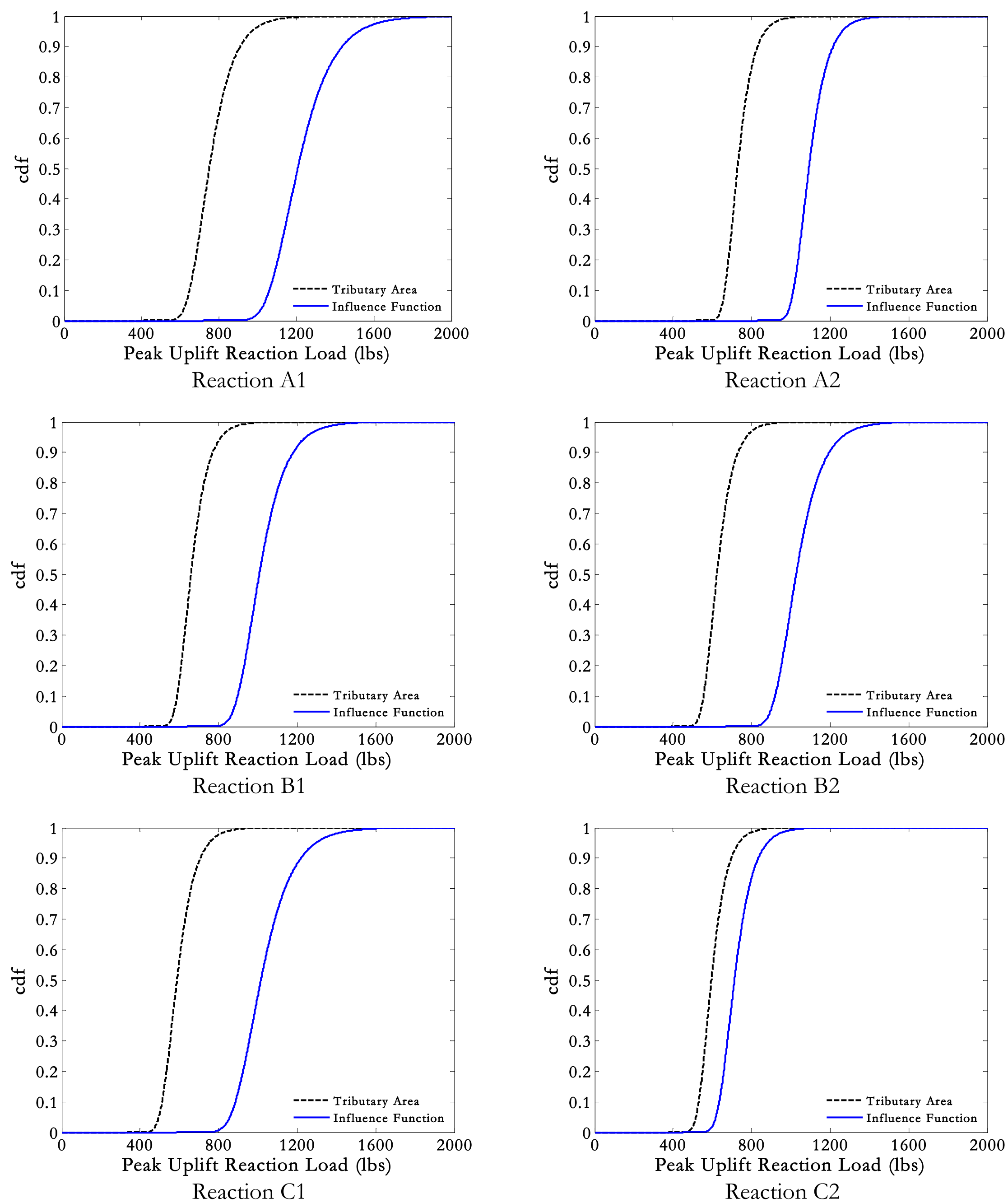


Figure 3.12 – CDF for wind direction 0º showing both tributary area and influence function generated peak uplift loads

3.4.2 *Comparison of Extreme Values with ASCE 7-05*

Table 3.4 shows five wind directions compared with the ASCE 7-05 derived wind loads for MWFRS design loads. Of interest to note, the MWFRS procedure only considers loading in the longitudinal (0º and 180º) and transverse (90º and 270º) directions.

TABLE 3.4 – Comparison of Tributary Area and Influence Function Peak Negative Reaction Distributions with ASCE 7-05 MWFRS Reaction Loads

Reaction	Wind Direction	Tributary Area		Influence Function		ASCE 7-05 MWRFS
		Mean	Std	Mean	Std	
A1	0º	**-767**	108.8	**-1232**	156.4	**-532**
	45º	-753	62.0	-1214	130.4	—
	90º	-269	34.0	-450	58.0	-640
	135º	-221	18.8	-362	36.8	—
	180º	-140	19.8	-231	39.9	-348
A2	0º	**-739**	69.5	**-1105**	84	**-532**
	45º	-731	69.8	-1089	108.6	—
	90º	-350	44.3	-472	57.4	-640
	135º	-304	39.4	-397	41.5	—
	180º	-137	17	-206	24.5	-348
B1	0º	**-672**	75.7	**-1026**	117.6	**-532**
	45º	-583	55.6	-818	73.9	—
	90º	-233	32.2	-408	46.8	-640
	135º	-215	20.7	-373	36.6	—
	180º	-139	21.1	-223	32.8	-348
B2	0º	**-635**	73.4	**-1046**	114.9	**-532**
	45º	-604	53.9	-884	80.2	—
	90º	-309	34.6	-473	62.5	-640
	135º	-304	46.4	-424	49.8	—
	180º	-133	16.8	-221	26.2	-348
C1	0º	**-601**	81	**-1038**	138.7	**-532**
	45º	-435	49.1	-728	85.2	—
	90º	-271	31.7	-492	49.8	-640
	135º	-263	27.8	-502	49.2	—
	180º	-146	15.9	-271	33.3	-348
C2	0º	**-606**	69.1	**-726**	82.9	**-532**
	45º	-582	55.2	-631	54.1	—
	90º	-315	32.8	-379	45.5	-640
	135º	-300	40.5	-388	47.1	—
	180º	-131	15.9	-176	18.7	-348

Figures 3.13 through 3.18 show the extreme value distribution means and other significant values plotted against ASCE 7-05 calculated reaction loads for MWFRS. Most startling of this visual is the discrepancy for a wind direction of 0º. The mean peak wind load is 25-45% higher than the ASCE 7-05 value if tributary areas are used. For the influence function generated peak values, the mean is 90-135% higher than the ASCE 7-05 prescribed load.

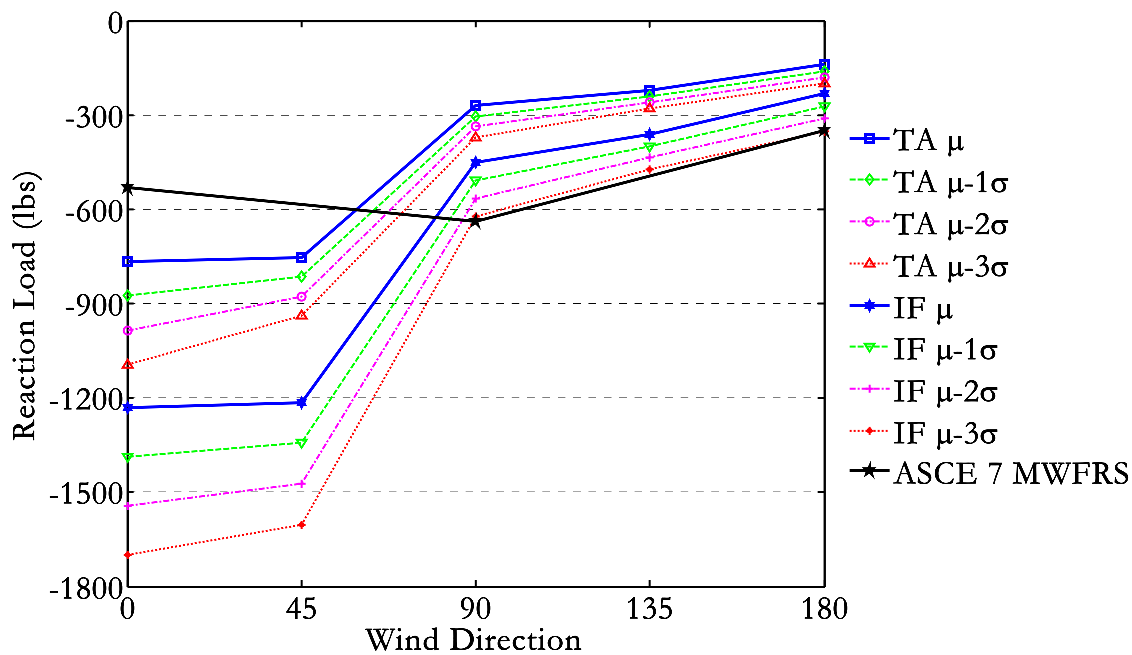


Figure 3.13 – Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction A1 with tributary area and influence function generated loads

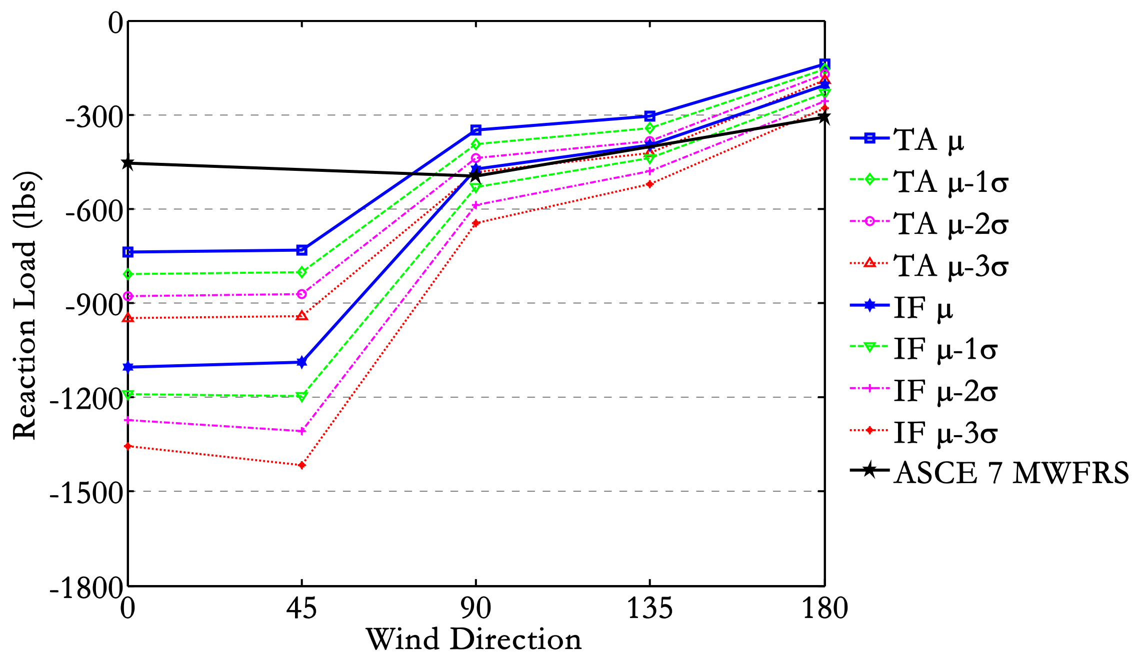


Figure 3.14 – Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction A2 with tributary area and influence function generated loads

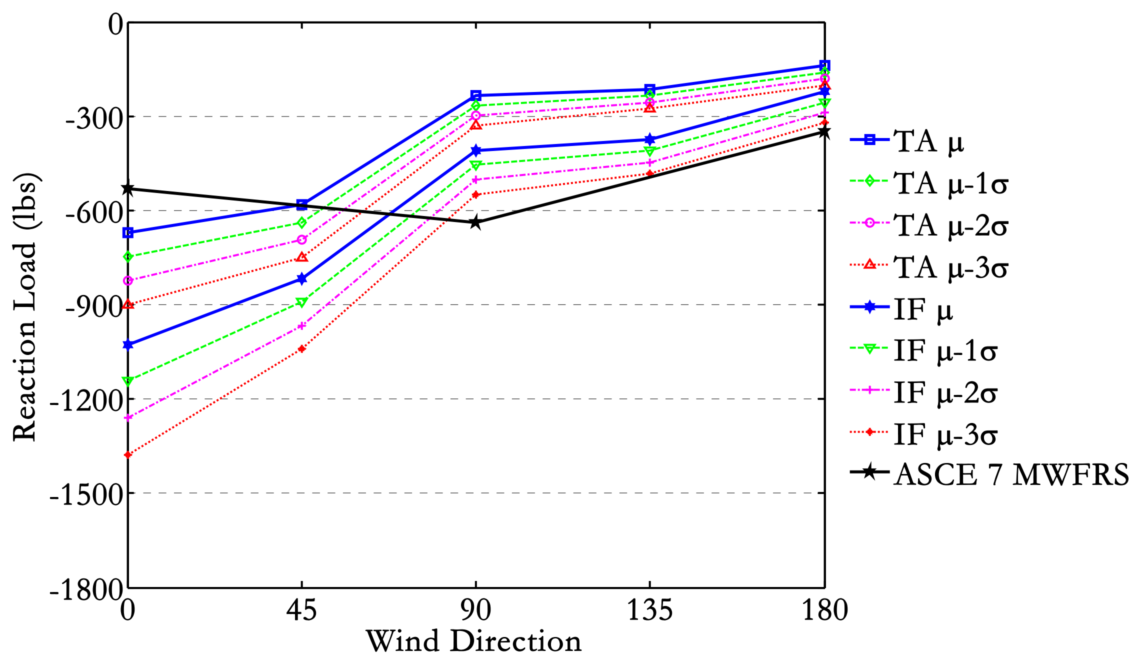


Figure 3.15 - Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction B1 with tributary area and influence function generated loads

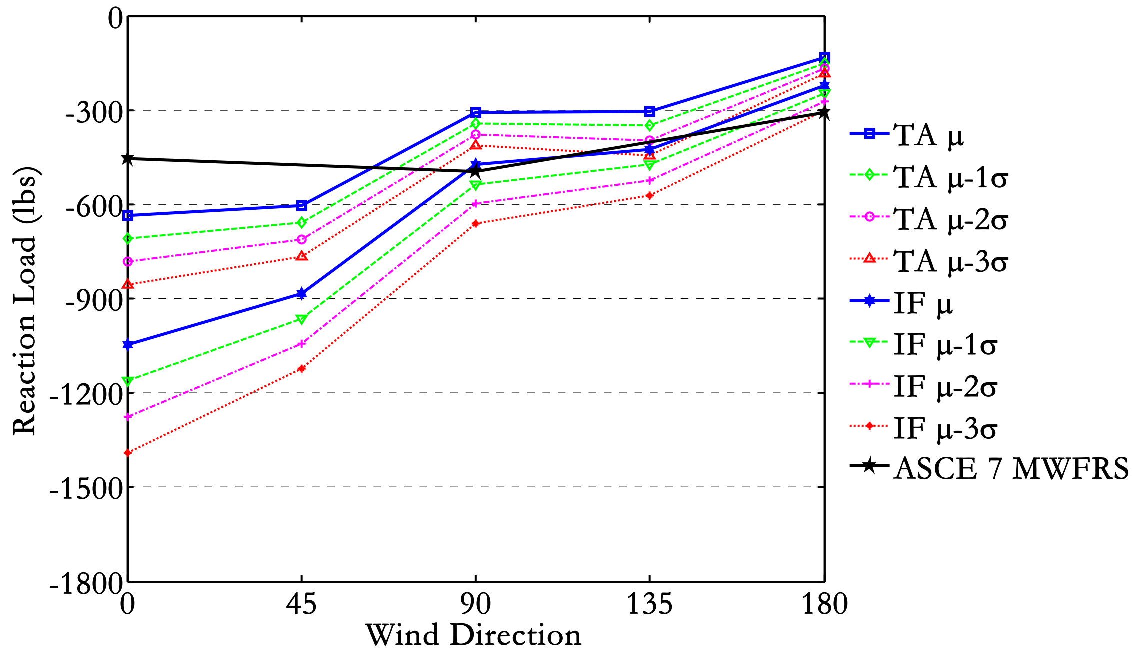


Figure 3.16 – Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction B2 with tributary area and influence function generated loads

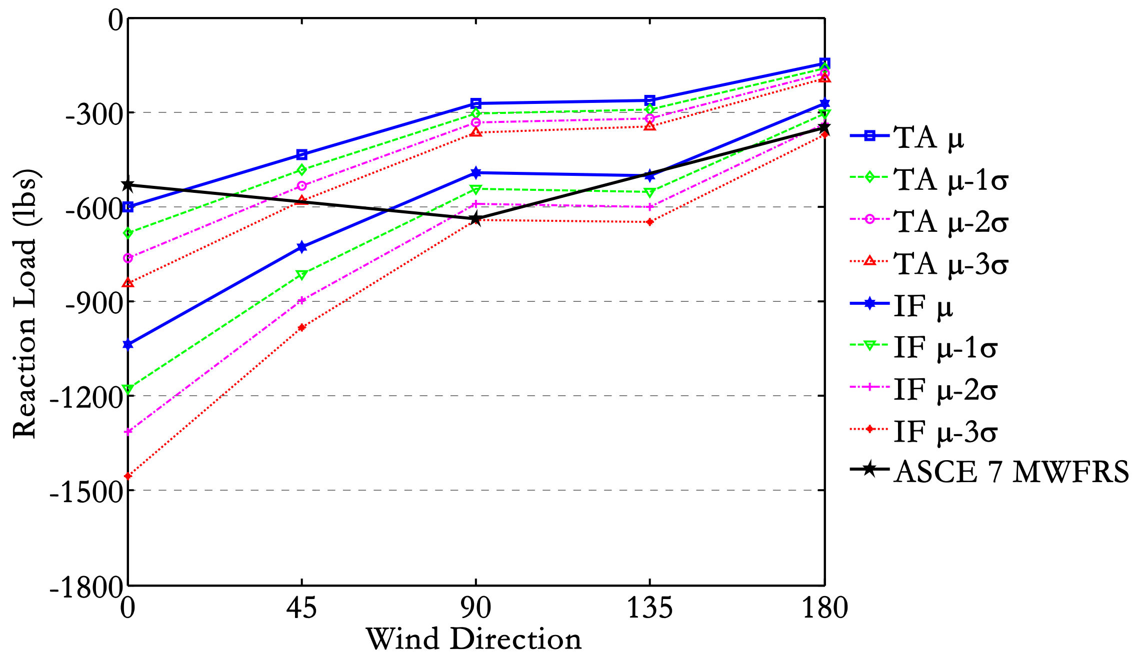


Figure 3.17 – Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction C1 with tributary area and influence function generated loads

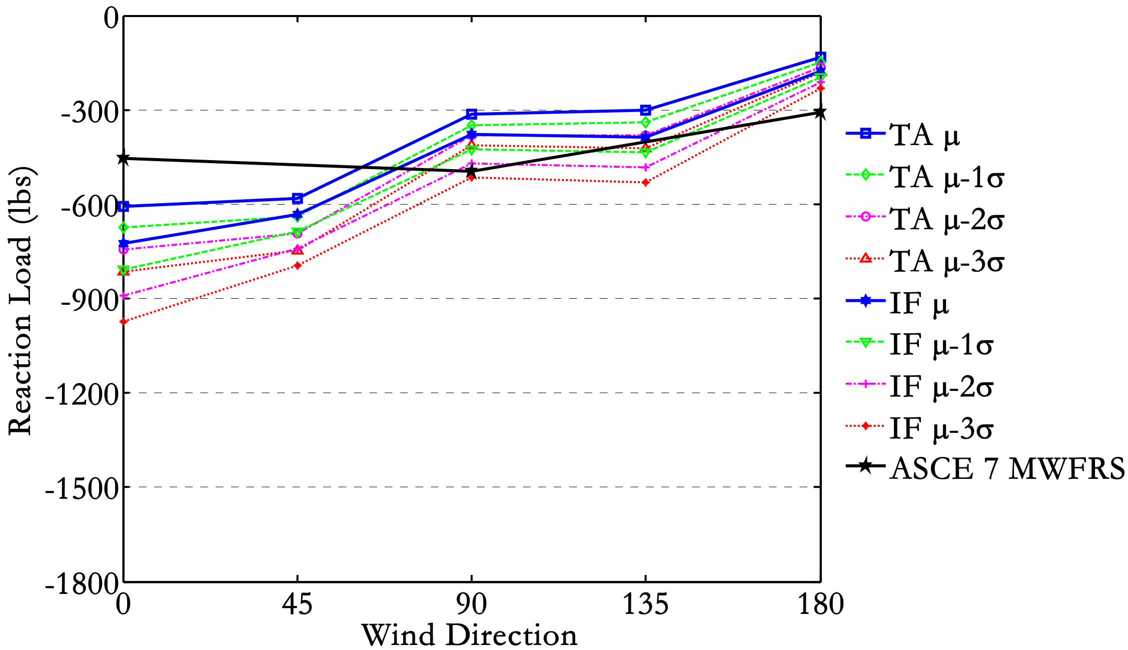


Figure 3.18 – Comparison of ASCE 7-05 MWFRS Reaction Loads for Reaction C2 with tributary area and influence function generated loads

4. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This chapter provides a summary of the work performed and the most significant conclusions from the research results. Recommendations for further research will also be discussed.

4.1 Summary

The research study presented in this report used experimental and analytical results from structural tests and wind tunnel model studies to quantify the dynamic loads transferred through the roof-to-wall connections of a wood-framed gable roof assembly. The experiment focused on the roof-to-wall connection which is a critical connection in the wind uplift vertical load path, and it has been shown to be a weak link in the structural systems of wood-framed residential structures.. The work documented results to further our understanding of actual load transfer forces and determine reasons for the susceptibility of the connection to premature failures during hurricanes.

The project used experimentally derived influence (or load-transfer) functions measured from a 1:3 scale model of a gable, wood-truss roof assembly at the truss reactions and wind loading data derived from wind tunnel tests on a scale model residential building. Time-histories of reaction loads at the roof-to-wall connections were determined using influence functions for the reaction loads due to vertical uplift loads on a gable-roof and combined with spatially distributed wind-tunnel derived pressures determined for a 1:50 scale model house. The tests were conducted in the atmospheric boundary layer wind tunnel of the Wind Load Test Facility (WLTF) at Clemson University under simulated suburban terrain conditions.

The wind tunnel model was tested for five wind directions (0º, 45º, 90º, 135º, 180º). Using the time histories of the pressure taps from these wind directions, wind load time histories were developed for the roof-to-wall connections using two different methodologies: (1) simple tributary areas of the trusses and (2) influence functions of the truss reactions.

This methodology of using influence functions and wind tunnel pressure coefficient time histories is a step in the direction of a database-assisted design (DAD) procedure for wood-framed residential structures currently under development at the National Institute of Standards and Technology (NIST). The DAD procedure will lead to more risk-consistent designs for wood-framed residential structures leading to reduced structural damage, cost savings, and safer buildings.

4.2 Limitations of the Study

The results of this study have several important limitations that need to be considered first before any conclusions can be made.

1. The building studied is a simple rectangular gable roof structure. Most typical residential structures are much more complex and the resulting wind pressure distributions on such a roof may be much different than for the simple gable roof.
2. The one-third scale roof assembly did not have a typical gable end truss. The gable end truss usually consists of several vertical web members at about a 24 in. spacing so that exterior wall sheathing can be attached easily. All of the trusses used in this study including the gable end were Fink style (see Figure 2.6). A typical gable end truss is much stiffer than the interior trusses, so it can be expected that the load sharing and influence functions will be different near the gable truss.
3. This study only looked at 15/32 in. (11.9 mm) thick oriented strand board (OSB). While this is a common roof material, other roof materials (i.e. plywood) and thicknesses

are also common. Therefore, the influence functions and the resulting roof-to-wall connection loads are only for this 15/32 in. OSB.

4. This study used a simulated sheathing not actual scaled sheathing. The wood strips are a first attempt to model the sheathing and do not provide a complete model of the sheathing. For instance, point connections were used to attach the strips to the trusses as opposed to nailing at a specified interval around the edges and in the middle. An actual modeled sheathing would be sheets of some material (i.e., plywood) scaled appropriately. More work is needed in this area in order to properly develop one-third scaled sheathing.
5. Only one design wind speed was investigated (130 mph) as well as only one terrain exposure (suburban).
6. Internal pressures were not included in the analysis of the connection loads.

4.3 Conclusions

1. Load sharing does occur in wood-framed roof assemblies for uplift loads. This load sharing is accomplished through the sheathing transferring the loads to adjacent trusses. Figure 3.1 shows that the load sharing can be significant with trusses transferring as much as 20% of their load to a reaction on a neighboring adjacent truss.
2. The peak roof-to-wall connection loads tend to follow a Type I Extreme Value distribution for both tributary area loading and influence function loading.
3. The peak roof-to-wall connection loads derived using the DAD approach are 40-70% larger than the loads derived using simple tributary area loading. This large difference is startling and may be the reason for continued roof-to-wall connection failure during extreme wind events. The results suggest that the tributary area loading philosophy is not conservative for repetitive member roof systems.

4. Comparison with ASCE 7-05 derived roof-to-wall connection design loads (see Figures 4.21 through 4.26) show that the mean peak wind (uplift) load for a wind direction of 0º is 25-45% higher than the ASCE 7-05 MWFRS load if tributary areas are used. For the influence function generated peak values, the mean peak wind load is 90-135% higher than the ASCE 7-05 prescribed load for a wind direction of 0º.
5. ASCE 7-05 only considers designing the MWFRS for winds that are parallel (0º and 180º) and perpendicular (90º and 270º) to the structure. The results of this study suggest that cornering winds at 45º to the building walls create just as large of forces as do winds occurring from 0º. It is still unknown whether 0º or 45º even constitute the directions that produce the largest loads. Other wind directions on a finer scale (i.e. every 5º-10º) should be tested and the resulting loads developed to see which wind direction actually produces the highest loads. Other published research suggests that possibly 30º or 60º may produce the largest loads. Therefore, it is recommended that ASCE 7 considers how to properly account for this discrepancy and include a method to ensure that the worse cases scenario is accounted for in the design.
6. The Database-Assisted Design (DAD) methodology is a feasible method to determine the wind loads on typical residential roofs. As the roof systems become more complex and the wind loads become more uncertain, the DAD procedure will be a valuable asset in determining more risk-consistent design loads that will increase the probability of successful damage mitigation in low-rise wood-framed buildings in extreme wind events.

4.4 Recommendations for Further Research

Based on the research presented in this report, several recommendations for future research are made.

1. Because of the large difference in the loads developed by the tributary area and influence function methods, verification of this difference is needed before proceeding with future work in this area. A simple statically loaded roof could be sued to tell if the difference between the two methods is as pronounced as shown in this study. Using wind tunnel data from another wind tunnel laboratory would also be a good check of this large difference.
2. Comparison of the influence functions developed in this study experimentally with analytically derived influence functions would help us to understand whether some of the current modeling programs account accurately for load sharing and re-distribution. If a difference does exist, a method to properly correct the analytically derived influence functions could possibly be determined.
3. Full-scale testing of a gable roof structure should be conducted to verify that the one-third scale model adequately represents the full-scale system.
4. Different sheathing thicknesses should be tested to investigate their effects on the load sharing and influence functions.
5. Different types of roof assemblies should also be tested (i.e. intersecting roofs, complex roofs, etc.) to investigate how the wind loads are distributed and transferred to the roof-to-wall connections.
6. A reliability study should be conducted to identify all of the parameters and the significance of each parameter to the load sharing ability of a wood-framed roof assembly as well as the load transfer through roof-to-wall connections.

5. REFERENCES

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