AN ABSTRACT OF THE THESIS OF

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 Evaluation of System Effects and Structural Load Paths in a Wood-Framed

 Structure.

Abstract approved:

Rakesh Gupta

The objective of this project was to develop an analytical model of a lightframed wood structure using a prevalent structural analysis computer program in order to evaluate system effects and define load paths within the structure, especially under extreme wind events. Simplified modeling techniques and material definitions were developed and used throughout the analysis.

A three dimensional 30-ft by 40-ft building was modeled using SAP2000. The building had a gable roof system comprised of Fink trusses. Wall and roof sheathing was modeled using SAP's built-in thick shell element. Conventional light-frame construction practices were assumed, and the model was linear with all joints considered to be either pinned or rigid. Also, the effect of edge nail spacing of the wall sheathing was incorporated by way of a novel correlation procedure which eliminates the need to represent each nail individually. Instead, a single sheathing element represented each wall and property modifiers were assigned to that wall element based on the nailing schedule. The NDS 3-term shear wall equation was used to derive the correlation procedure and the correlated model was compared to full-scale testing results with good agreement.

The computer model was validated against both two and three dimensional experimental studies (in-plane and out-of-plane). Once validated it was subjected to uniform loads to gain insight into its uplift behavior. Uniform uplift pressure was applied to the roof, and vertical foundation reactions were evaluated. In this phase of the investigation, the building geometry was altered in several different ways to explore the effect of these variations. Next, the model was subjected to several uplift loading scenarios corresponding to worst-case simulated hurricane events. With these inputs, the same uplift reaction profiles were generated. Finally, for comparison the model was loaded using the "Component and Cladding" pressures determined at a comparable wind speed, as given by ASCE 7-05 (lateral and uplift).

The ASCE 7-05 uplift pressures were found to adequately encompass the range of uplift reactions that can be expected from a severe wind event such as a hurricane. Also, the analytical model developed in this study inherently takes into account system effects. Consequently, it was observed that ASCE 7-05 "Component and Cladding" pressures satisfactorily captured the building's uplift response at the foundation level without the use of "Main Wind Force-Resisting System" loads. Additionally, it was noted that the manner in which the walls of the

structure distribute roof-level loads to the foundation depends on the edge nailing of the wall sheathing. Finally, the effects of variations in the building geometry were explored and notable results include the presence of a door in one of the walls. It was revealed that the addition of a door to any wall results in a loss of loadcarrying capacity for the entire wall. Moreover, the wall opposite the one with the door can also be significantly affected depending on the orientation of the trusses.

In general, it was determined that complex, three-dimensional building responses can be adequately characterized using the practical and effective modeling procedures developed in this study. The same modeling process can be readily applied in industry for similar light-framed wood structures. ©Copyright by Kenneth G. Martin January 29, 2010 All Rights Reserved

EVALUATION OF SYSTEM EFFECTS AND STRUCTURAL LOAD PATHS IN A WOOD-FRAMED STRUCTURE

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I understand that my thesis will become part of the permanent collection of Oregon State University libraries. My signature below authorizes release of my thesis to any reader upon request.

Kenneth G. Martin, Author

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EVALUATION OF SYSTEM EFFECTS AND STRUCTURAL LOAD PATHS IN A WOOD-FRAMED STRUCTURE

INTRODUCTION

A successful structural design, in its most basic form, must ensure that buildings are capable of supporting loads and performing their intended functions. To do so, engineers employ a process that is typically considered to include only two major phases: 1) the determination of loads acting on a structure, followed by 2) an analysis of the individual members to ensure they can withstand the loads. Too often it is assumed that these phases can be performed separately so long as the end result shows that member capacity exceeds the demand. However, there are two very fundamental concepts that must also be integrated into structural design, yet are often overlooked. The first concept is the need for a continuous load path. Forces originating at any point in the structure must have a route by which they can be transmitted through the structure and safely to the ground. In this sense, it is best to consider buildings not as mere assemblies that simply *support* loads, but rather as complex systems that *transmit* loads. Second, designers must consider system effects that exist within the structure. Today's buildings are so complex that individual members inherently share load with their neighbors, yet these interactions are seldom incorporated into structural evaluations. This is perhaps due to the fact that there is generally no practical manner by which to address system effects. In fact, present convention simply addresses load sharing by way of conservative factors applied to design values, such as the repetitive member factor

used in the NDS code (AF&PA 2005a) which accounts for both load sharing and partial composite action.

LOAD PATHS

In so far as load paths are concerned, it has been documented that although the basic topic is covered in most structural engineering texts, the discussions provided are usually not comprehensive enough to provide an understanding of the principles involved (Taly 2003). In fact, one of the nation's preeminent manuals on wood frame construction makes the following statement in its general provisions:

A continuous load path shall be provided to transfer all lateral and vertical loads from the roof, wall, and floor systems to the foundation. (AF&PA 2001)

However, it does not provide any additional details on how to do so. The commentary to the manual is of no help either – it actually skips over this subsection, addressing the topics before and after it but providing no further clarification on the topic of load paths. To complicate matters, engineers and designers must account for the fact that load paths in a structure are different for vertical loads compared to lateral loads, and they vary from one structure to another. Yet, despite these hurdles, it is nonetheless essential that load paths be well understood and evaluated in performing any structural analysis. Experience has shown that failure to do so leads to significant damage and even collapse (Taly 2003). History validates this notion. Albeit unfortunate, devastating events such as natural disasters tend to highlight both the gross oversights as well as the subtle misunderstandings of load paths. For example, in the aftermath of hurricane

Katrina, damage assessment teams observed widespread damage and significant patterns of structural failure. Above all else, these teams emphasized a lack of load path, especially due to uplift, as one of the prevalent failure mechanisms observed (van de Lindt et al. 2007).

TYPE OF LOADING

The need to identify and understand load paths is clear. This much is known. Implicit in this statement though is the assumption that the loads are also known, which is not always the case. As mentioned, the type of loading can vary. Loads can originate from any number of sources including snow, wind, earthquakes, personnel, and even the weight of the structure itself. Each of these load sources has the potential to cause a structural failure or, at the very least, render the building unfit to perform its intended purpose. Alongside this failure, there is a cost that is incurred to repair or replace the structure. If the prevalence for failure and the associated costs are considered as criteria to rank the aforementioned loads, wind tops the list. Wolfe (1998) gives further details about this fact, stating:

In the U.S., wind is the most common – and the most costly – cause of damage to buildings. Over a seven year period from 1986 to 1993 extreme wind damage cost \$41 billion in insured catastrophe losses as compared to \$6.8 billion for all other natural hazards combined.

This precedent was further endorsed with Hurricane Katrina, which made landfall towards the middle of the hurricane season in late August of 2005. Katrina was by far the mot costly hurricane – and disaster – in U.S. history (van de Lindt et al. 2007).

The term "extreme wind events" is not limited to hurricanes though. Included in this subset of natural hazard events are also tornadoes and severe storms. Taken as a whole, the losses from these extreme wind events continue to increase, doubling approximately every 5-10 years (Davenport 2002). Interestingly, hurricanes wreak the most havoc. They normally cause twice the damage of tornadoes in any one year and over 160 times the damage of severe winds (Wolfe 1998). One reason for this fact is that tornadoes usually affect a smaller land area than hurricanes. More critical though is the fact that winds associated with hurricanes are accompanied by large amounts of rain. Winddriven rain saturates insulation and ceiling drywall, causing it to collapse. Damage in these cases is extensive and costly to repair.

CONSTRUCTION PRACTICES

Improvements to light-frame residential construction practices were made and subsequently adopted following Hurricane Andrew in 1994. However, 90% of the homes in the U.S. were built prior to the adoption of these provisions (US Census Bureau 2003). As a result, these structures remain vulnerable to the damaging effects of hurricane winds. Also, newly constructed buildings can still find themselves at the mercy of wind loads, despite being constructed after the new code provisions. Although evidence has proven that recently built homes fair better than older homes, this only holds true when the design codes and guidelines are followed (van de Lindt et al. 2007). Regrettably, many of the prescriptive recommendations are either misunderstood or incorrectly applied (APA 1997). The damage assessment teams after Hurricane Katrina observed this firsthand, noting:

Builders and inspectors in the Mississippi Gulf Coast region appear to be familiar with <u>conventional</u> construction provisions. However, these provisions were used erroneously in a high wind region. (van de Lindt et al. 2007)

Consequently, buildings both old and new are susceptible to failures as a result of exposure to high wind loads. Therefore, it is critical to gain a deeper understanding of hurricane wind loads and their effect on light-frame wood structures.

SYSTEM EFFECTS

Previous research has been conducted in the realm of wood-frame structures exposed to wind loads, but it has stopped short of fully addressing all of the mechanisms that are at play within these complex systems, especially in uplift scenarios. For example, much of the research has concentrated towards specific components within the structure, such as roof or wall sheathing (Sutt 2000, Hill et al. 2009), or towards one particular type of connection, e.g. roof-to-wall (Reed et al. 1997, Riley and Sadek 2003). Very little work has been done to address system effects as a whole in full-size buildings. Fortunately, this shortfall can be overcome by using an analytical tool, such as a modern structural analysis computer program, which directly incorporates system interactions.

ANALYTICAL MODELING

Naturally, the use of modern computers has made the bookkeeping and computational aspects of structural analyses much easier; however, the major market for these tools has been in the realm of steel and concrete design. Wood structures have received far less attention in so far as modern computer programs are concerned. Yet tools that are widely used in academia and in practice, such as SAP2000, possess significant potential to the wood industry if manipulated to predict the response of wood-framed systems. With this in mind, the major thrust of this research has focused on the development of simple, yet accurate material assignments and property correlations to customize SAP2000 for use in three-dimensional wood structures. Considerable efforts were made to use built-in features of SAP2000 in conjunction with simple modeling techniques to capture complex structural responses (e.g. system effects, effect of nailing schedule, etc.).

OBJECTIVES

The following tasks represent the overall objectives of this study. It is worth noting that throughout this research, one goal has been kept at the forefront as each task was confronted: the initiative to address very complex structural behaviors using only the most pragmatic modeling techniques possible. In this fashion, the hope is that wood engineers and designers can use these same methods in industry to readily and accurately predict the behavior of similar wood structures. Specific goals were:

- 1. Develop a practical 3D computer model of a full-size light-frame wood structure.
- Develop a practical representation of the sheathing nailing schedule to be incorporated into the computer model.

3. Evaluate critical load paths and system effects for different building

geometries under various loading scenarios.

MATERIALS AND METHODS

GENERAL

An analytical model of a light-frame wood structure was developed and validated. Then, to better understand the behavior of the model in the presence of uplift loads, a uniform uplift pressure was applied to the roof sheathing. Several scenarios (e.g. changing the anchor bolt spacing, adding a door to one of the walls, etc.) were considered while the structure was subjected to this uniform pressure. Next, simulated hurricane uplift loads were applied to the model. Finally, the structure was subjected to ASCE 7-05 pressures. For direct comparison, the reaction profile of the structure under these code assigned loads was compared to the response of the building under the simulated hurricane loads.

The analytical model of the index building was developed using SAP2000 (Figure 1). This commercial software package – developed by Computers and Structures, Inc. based in Berkeley, California – is widely used in academia and industry. The model is comprised entirely of pinned or rigid connections, and all materials are assumed to behave within the elastic range. Non-linearity is not incorporated into this study. Studs and truss members are represented using frame elements with isotropic material properties. Wall and roof sheathing are modeled using the thick shell element with orthotropic material properties. Anchorage devices are represented by grounded springs (Computers and Structures, 2008).

The footprint of the index building is approximately 30-ft x 40-ft with overhangs on all sides. The gable roof has a 4:12 slope. Studs are spaced 16-

inches on center and trusses are 24-inches on center. There are no interior partitions (see Figure 1). Specific construction features and detailed framing plans of the index building can be found in Appendix A.



Figure 1. SAP model of the index building (exterior sheathing not shown for clarity).

MODELING

Shell Element Behavior

The roof sheathing (1/2" plywood) and wall sheathing (7/16" OSB) were modeled using SAP's thick shell element. Each wall and roof area was defined in the modeling environment using one shell element. That is, individual sheets of 4ft x 8-ft plywood/OSB were not modeled. For example, the entire side wall is represented by just one shell element in SAP. Likewise, each side of the roof is comprised of a single shell element as well. It should be noted that SAP ultimately divides each single *modeling* shell element into multiple *analysis* shell elements in a process known as meshing. The user can choose how the mesh is defined (Appendix B), and SAP also allows the user to inspect the mesh of each shell element by viewing the internal "analysis model" (Figure 2). However, in the modeling environment – where the walls and roof structure are defined and manipulated – only individual shell elements were used to represented each wall/roof section. By choosing to model the sheathing in this manner, it is assumed that the actual 4-ft x 8-ft sheathing panels transfer load and moment continuously across their joints. This practical assumption can be judged by examining the wall and roof systems in greater detail. In wall systems, blocking along all panel edges and high nailing density contribute to the validity of the assumption. In roof systems, the assumption of continuity across the joints is drawn from four sources: (1) staggered joints along truss lines, (2) edge nail spacing of 6-inch or less along truss lines, (3) unblocked panel edges have nails within 3/8-inch from the edge, and (4) "H-clips" are located in the bays between trusses. Appendix B provides further details and accompanying figures related to this discussion (Figures B-2 and B-3).

Once the shell element has been defined in the model, it is meshed into multiple *analysis* elements to ensure proper interaction with the framing members. For simplicity, the automatic meshing option feature was used as shown in Figure 2. Additional details related to the meshing of the shell element are provided in Appendix B.



Figure 2. Meshing of the wall sheathing in the gable ends. "General divide" was used for the triangular region above the top plate of the wall.

Connectivity

All joints in the SAP model are either pinned or rigid. Doing so provides convenience and ease of modeling by eliminating the need for semi-rigid connections, non-linear "link" elements, and complicated spring systems to represent joint behavior. For additional details, see Appendix B.

Stiffness of Hold-downs and Anchor Bolts

The axial stiffness of the hold-down device is listed by the manufacturer (Simpson Strong-Tie, 2008). The most common type of connector, the HDU2, was selected. The published axial stiffness of this connector is 35,000 lb/in. This value takes into account fastener slip, hold-down elongation, and bolt elongation. The axial stiffness of the anchor bolts, on the other hand, was determined using data from a previous research effort (Seaders 2004). The shear stiffness of the anchor bolts in the X and Y-direction was determined using a procedure recommended by

the American Wood Council (AF&PA 2007). Table 1 summarizes the spring stiffness values used in this study. Additional details relating to the derivation of these values are given in Appendix B.

Table 1. Spring stiffness used to model the anchor bolts and hold-downs.

Item	X- direction (shear) lb/in	Y- direction (shear) lb/in	Z- direction (axial) lb/in	Source
Hold-downs	-	-	35,000	Simpson Strong-Tie (2008)
Anchor bolts	65,000	65,000	35,000	NDS (AF&PA 2005) and Seaders (2004)

Material Properties

Frame elements, which represent the wall and truss members, were modeled using elastic, isotropic material properties. The NDS code (AF&PA 2005a) and the Wood Handbook (USDA 1999) were used to assign values. Additional details are provided in Appendix B.

Table 2. Elastic isotropic material properties used in the SAP model.

Item	Description	MOE (10 ⁶ psi)		Poisson's Ratio	
		Value	Source	Value	Source
Wall members	SPF, stud grade	1.2	NDS (AF&PA 2005a)	0.40	Wood Handbook (USDA 1999)
Truss members	SYP, No.3 and stud	1.4		0.36	

Wall and roof sheathing were each modeled using SAP's thick shell element. Orthotropic, elastic material properties were then assigned. Nine constants are needed to describe the behavior of these materials, although only the values shown in bold (Table 3) affect the response of the model. Further explanation of this point is provided in Appendix B. The values given in Tables 2 and 3 were used for all wall and roof sections in the index building with one exception. The shear modulus, G₁₂, of the wall sheathing was modified using the correlation procedure as described in the "Results and Discussion" section and in Appendix E. Table 4 provides the correlated shear modulus values for the wall sheathing.

Shear MOE Poisson's Ratio⁴ Description Modulus (10^5 psi) (10^5 psi) Item Type E_1 \mathbf{E}_2 E₃ **G**₁₂ G_{13} G₂₃ μ_{12} μ_{13} μ_{23} Wall 7/16" OSB 7.4 2.3 2.3 1.2 1.2 1.2 0.08 0.08 0.08 sheathing^{1,2} Roof 1/2" 19 2.9 2.9 1.5 1.5 1.5 0.08 0.08 0.08 sheathing³ Plywood

Table 3. Elastic orthotropic material properties used in the SAP model.

¹ MOE and shear modulus values from Doudak (2005)

² Shear modulus values subject to the correlation procedure (Table 4)

³ MOE and shear modulus values from Wolfe and McCarthy (1989) and Kasal (1992)

⁴ Poisson's ratio from Kasal (1992)

RESEARCH METHODS

The following steps were used to validate and load the model. The geometry modifications listed were explored for the uniform uplift pressure load case only.

Verification / Validation

The following validation procedures were explored in support of this research effort. The studies from literature which were used for comparison are noted. Further information pertaining to each scenario is provided in the appendices also noted.

- Two-dimensional *individual* truss behavior Wolfe et al. (1986) Appendix C
- Three-dimensional roof *assembly* behavior Wolfe and McCarthy (1989) –
 Appendices C and D
- Two-dimensional shear wall behavior Langlois (2002), Lebeda (2002), and Sinha (2007) – Appendices C and E
- 4. Three-dimensional influence functions of the *entire building* Datin (2009)
 Appendices C and F

Load Cases

The following load cases were explored in support of this research effort. Further information pertaining to each scenario is provided in the appendices noted.

- 1. Uniform uplift pressure Appendix G
- 2. Simulated hurricane uplift pressures Appendix H

- a. Load case 1 Absolute maximum uplift at the corner of the roof
- b. Load case 2 Local maxima over entire roof
- c. Load case 3 Absolute maximum uplift at the ridge of the roof
- 3. ASCE 7-05 pressures Appendix I
 - a. Uplift acting alone
 - b. Lateral forces acting alone
 - c. Combination of uplift and lateral forces

Geometry Scenarios

The following geometry variations were explored for the first load case noted above (uniform uplift pressure). The standard building geometry was used for the simulated hurricane uplift and the ASCE 7-05 pressures.

- 1. Standard building (control case)
- 2. Changing the edge nailing of the wall sheathing
- 3. Adding length to the building
- 4. Presence of doors in each wall
- 5. Gable wall missing (three-sided structure)
- 6. Presence of roof blocking
- 7. Different overhang construction (ladder vs. outlooker)
- 8. Varying the anchor bolt spacing
- 9. Removing anchor bolts at key locations

RESULTS AND DISCUSSION

MODEL VALIDATION

A four-step validation procedure, incorporating both 2D and 3D behavior, was used to ensure the accuracy of the SAP2000 modeling techniques. First, a 2D individual truss comparison was conducted against Wolfe et al. (1986) in order to verify the assumptions of pinned/rigid joint connectivity within the truss. Next, a 3D roof assembly (Wolfe and McCarthy, 1989) verified the load sharing response of the model. Third, a 2D investigation using multiple sources – Langlois (2002), Lebeda (2002), and Sinha (2007) – was performed to establish the validity of the shear wall behavior. Finally, the model of the index building itself was validated against a 1/3 scale prototype tested by researchers at the University of Florida (Datin 2009). The results of this multipart verification process showed that the SAP2000 computer model and the simplified techniques used in its creation adequately characterize the structural responses witnessed by physical testing. Details pertaining to each verification step are provided in Appendix C.

CORRELATION MODEL FOR NAILING SCHEDULE OF SHEATHING

One of the primary objectives of the present study was to develop a practical means to incorporate the effect of edge nailing into the SAP model. Previous researchers have modeled fasteners individually using a set of "zerolength link elements" for each nail. If the nailing schedule is changed, the model must be revised one nail at a time. Although this arrangement directly takes into
account the actual number of nails in the system, the process can be laborious. This may seem reasonable for sub-assembly models like segments of a shear wall, but for full-size 3D complex structures it is simply not feasible.

Table 4 presents the results of the correlation study. These values relate the change in stiffness resulting from a variation in the edge nailing to the shear modulus for the shell element, G_{12} , of the wall sheathing in SAP. It should be noted that the extent to which edge nailing affects diaphragm or shear wall stiffness is dependent on the presence of blocking. Unblocked systems, such as residential *roof* systems, are relatively unaffected by changes in the edge nailing. On the other hand, blocked systems, such as residential *wall* systems (assuming the typical practice of placing OSB panels vertically), do respond to changes in the nailing schedule. Therefore, this study focuses on the effect of edge nailing in the *wall* sheathing (i.e. not the *roof* sheathing).

Table 4. Correlation between nailing schedule and the shear modulus G_{12} of the
shell element in SAP.

Sheathing	Stud Spacing	MOE of Members	Required G ₁₂ in SAP (10 ⁴ psi) for each Edge Nail Spacing (inches)				
	(in)	(10° psi)	2-in	3-in	4-in	6-in	12-in
7/16" OSB	16 or 24	1.2 to 1.6	9.43	6.38	4.86	3.34	1.81

Appendix E offers a detailed explanation of how these values were determined. In addition, comparisons between the correlated sheathing model and physical shear wall tests are also provided in Appendix E.

UNIFORM UPLIFT PRESSURE

All output plots for the uniform pressure load cases are provided in Appendix G. These plots represent vertical reactions at the hold-downs and anchor bolts. Positive values represent uplift (tension) while negative values represent downward forces (compression). Unless otherwise noted, the edge nailing for the wall sheathing used for all output results is 6-inches on center.

Standard Geometry (Control Case)

Before altering the geometry the standard index building was loaded with a uniform uplift pressure to establish a control case to which all other arrangements could be compared. As expected, the building response was symmetric. The gable walls, or end walls, show a load intensity (i.e. spike) directly beneath the peak of the roof (see Figure 3). This results from load accumulating in the roof structure, delivered via the ridgeline to the anchor bolt directly below (see Figure 4). In Figure 4, the von Mises stresses¹ in the shell element are displayed. Doing so highlights the accumulation of load at the ridge and the subsequent concentration in the gable wall directly beneath it. For the edge nailing shown (6-inches), the load is not evenly distributed by the gable wall, and a spike in load intensity is witnessed at the anchor bolt directly below the ridge. As shown later, reducing the spacing between nails along the panel edges (e.g. 2-inches) can dramatically minimize the magnitude of this load intensity (Figure 6).

¹ The von Mises stress is a convenient method of combining the stresses (normal and shear) which act in all three directions (X, Y, and Z) into a single parameter, called the equivalent stress or "von Mises stress."



Figure 3. Reaction profile for the gable wall, uniform uplift pressure.



Figure 4. Load accumulation in the gable end below the ridge of the roof.

The side walls, or eave walls, display a parabolic reaction profile (see Figure 5). The side wall experiences the highest reactions of all locations in the building, with the maximum occurring in the middle. In this location, load originating in the roof structure is not effectively transferred to the end walls and is, in essence, forced to the side walls. The practical implication of this finding is that an anchor bolt located in the side wall carries more load than one located in the end wall – even below the ridge line (of course, only for the load scenario and geometry described).



Figure 5. Reaction profile for the side wall, uniform uplift pressure.

Effect of Edge Nailing

Unless otherwise noted, the edge nailing of the wall sheathing panels in this study is 6-inches on center. However, the effect of edge nailing can be observed using the correlation procedure described in Appendix E. As the edge nailing gets denser, the wall becomes stiffer and capable of distributing the roof loads more evenly to the foundation. In looking at the 2-inch edge nailing reaction profile for the gable end (Figure 6), it can be seen that the seven interior anchor bolts each carry approximately the same vertical load (about 1400 to 1500 lbs). The greatest margin between these anchorages is 105 lbs from joint I.D. 52 to 58. In comparison to the 12-inch nailing option, it is noted that the load varies much more significantly. In other words, the less rigid wall is incapable of evenly distributing the roof loads with the 12-inch nailing option is 594 lbs from joint I.D. 58 to 70, more than five times the margin which was witnessed for the 2-inch nailing schedule.

This trend is observed in the side walls, too (Figure 7). That is, the 2-inch nailing pattern is a muted version of the 12-inch nailing schedule. The more rigid 2-inch wall distributes load evenly, minimizing load variation among foundation fasteners.



Figure 6. Effect of edge nailing for the gable wall, uniform uplift pressure.



Figure 7. Effect of edge nailing for the side wall, uniform uplift pressure.

Extended Building (30-ft x 92-ft)

The index building used for this study has a footprint of 30-ft by 40-ft. However, to explore the effect of adding length to the structure, a longer version was modeled. This extended building had an extra length of 52-feet, yielding an overall footprint of 30-ft by 92-ft. The reaction profiles of the gable end for both buildings are similar (Appendix G, Figure G-4). That is, the gable end anchorage devices witness similar loads regardless of the change in length of the building.

The noteworthy difference appears in the side wall, where the effect of building length becomes apparent. In the side wall, it can be seen (Appendix G, Figure G-4) that the reaction profile no longer takes on the parabolic shape as with the standard building, but is trapezoidal instead. This trapezoidal loading behavior is expected based on theoretical 2-way slab behavior. The shape of this loading profile is a result of load sharing within the structure (see Figure 8). Within the first and last 25% of the building, roof loads are shared with the end walls. In the middle half of the building, however, load does not make it to the end walls and is carried by the side walls alone. Thus, the reaction profile of the side wall shows a steady increase throughout the first quarter of the building length, as load is carried more and more by the side walls and less by the end wall. This continues until the middle half of the building is reached, whereupon the load levels off. In this region, roof loads are carried solely by the side walls. The remainder of the building is symmetric with the first half.



Figure 8. Load distribution within the roof of the extended building when subjected to uniform uplift.

Also, the reaction profile for the side walls of both buildings is nearly identical for the first four anchorage devices (Appendix G, Figure G-4). That is, for a distance of approximately 16-feet (since the anchor bolts are spaced 4-ft apart), the side wall reactions are independent of length. This is also in agreement with theoretical 2-way slab behavior, which predicts that the building responses should be similar for a distance of approximately half the total width, or 15-ft.

Effect of Door Openings

Door in End Wall

An opening 16-ft long representing a typical overhead garage door was located in the center of the end wall, near side, to examine its effect. In these plots (Figures 9 and 10), the solid blue line represents the reaction profile for the near wall. The dashed blue line represents the profile for the far wall, and the purple line is the reaction for the walls if no door were present whatsoever. In the gable wall (Figure 9), the anchorage devices on either side of the door carry more load, as expected. However, taken as a whole they do not carry the same load as if there were no door at all. This is presumed to result from the general loss of stiffness introduced by the door opening. When no door is present, the sum of the nine reactions in the end wall is 12,987 lbs. With the door, the remaining six anchorages only carry 12,063 lbs – a difference of 924 lbs. The 924 lbs goes into the side wall over the first half of the building, as can be seen in the Figure 10. Also, it is particularly important to note that the opposite gable wall, the one without the door, has the same reaction profile as if no door were present. That is, it does not recognize the presence of the door. The significance of this finding is made clear when the door is placed in the side wall instead of the end wall (see Figure 11).



Figure 9. Reaction profile for the end walls with door in center of near-side gable wall.



Figure 10. Reaction profile for the side walls with door in the center of near-side gable wall.

Door in Side Wall (centered)

A similar door 16-ft long was located in the center of the side wall, instead of the end wall. In this scenario, the presence of the door creates very large load amplifications in the reaction profile of the wall containing the door (Figure 11). In fact, the reactions at the columns that frame the openings witness nearly twice the uplift reaction. Despite seeing this jump in magnitude, the wall as a whole does not carry as much load as if the door were not present (represented by the dashed pink line in Figure 11).



Figure 11. Reaction profile for the side walls when a door is centered in the near side wall.

This same behavior was noted for the scenario in which the door was placed in the gable wall. With no door in the building, the side walls each carry 26,208 lbs over 11 anchorage devices. When the door is present, the remaining eight anchorages carry only 25,300 lbs, representing a reduction of 908 lbs. The balance of the load is not carried by the opposite wall, as might be expected. Instead, the back side wall (dashed blue line in Figure 11) actually carries less load now that the door is present. This is an interesting system effect resulting from the orientation of the trusses. Since the trusses are oriented perpendicular to the side walls, a reduction in stiffness in the front side wall (i.e. the presence of a door) presents itself as a corresponding reduction in stiffness in the back side wall. The flexibility

introduced by the presence of the door affects the trusses directly atop the opening. The reduced stiffness of these trusses is noticeable at the near side *and* at the far side of each truss in the vicinity of the door, thus the back side carries less load even though there is no door in that wall. For example, the *back* side wall carries 25,338 lbs, a net loss of 870 lbs compared to the case of no door. So, in summary, the *back* side wall loses 870 lbs and the *front* side wall (with the door) loses 908 lbs, a total of 1778 lbs. This load is evenly shared by each gable end, resulting in an increase of 889 lbs distributed evenly over the anchorage devices there. Slightly more load is carried by the corners of the gable end closest to the door than on the opposite side (1889 lbs vs. 1857 lbs), but for the most part the balance of the load is carried evenly, resulting in a symmetric load distribution in the end walls.

It is worth noting that the load carrying capacity of the back side wall, the one without the door, is highly dependent on the size of the header used to span the door opening and the presence of a ceiling (Appendix G, Figure G-7). As the header depth increases, the opening becomes more rigid and, thus, is capable of carrying more load. However, very little of this additional load-carrying capacity is realized in the front side wall where the door is. For example, in comparing the use of a 12-in deep header (realistic) to a 24-in deep beam (unrealistic), the sum of the reactions of the front side wall only increases from 25,300 lbs to 25,763 lbs (+2% difference). The greatest individual increase in the reaction occurs on either side of the door opening, where the load there increases by only 5% when the 24-inch header is used. However, a different story unfolds on the back side wall. This is

where the bulk of the load-carrying capacity reveals itself, particularly in the vicinity of the door opening. Across the five anchor bolts corresponding to the door's location, the sum of the reactions increases from 11,654 lbs to 13,248 lbs (+14% difference).

The presence of a ceiling (1/2-inch GWB in this case) also increases the load carrying capacity in this building geometry (Appendix G, Figure G-7). Interestingly, a similar trend is observed compared to the header tests. In particular, the front side wall with the door witnesses very little change while the back side wall, especially in the vicinity of the door, experiences a significant increase in ability to transmit load to the foundation. In fact, the ½-inch GWB ceiling is technically more effective in attracting loads to the back side than the extremely deep header (24-inch). As in the case of the deep header, the additional load-carrying capability comes from the increase in stiffness that the ceiling provides.

Finally, it should be noted that the stiffness across the opening also affects the load carried by the gable ends. With a flexible header (12-inch deep), the gable walls carry more load than with a stiff header (24-inches deep). In any case, the presence of a door centered in the side wall always results in an increase in the load carried by the end walls.

Door in Side Wall (not centered)

In this scenario, the door was placed in the side wall but closer to one gable end (i.e. not centered in the side wall). As noted with the previous scenarios, the remaining anchorage devices in the wall with the door experience an increased uplift reaction (Figure 12). This scenario creates a slightly greater spike in the reaction profile than with the door centered, as the uplift reaction in the anchor bolt adjacent to the doorway jumps from 2,898 lbs to 5,501 lbs (nearly double). With the door centered, the reaction increased to only 4,960 lbs (540 lbs less). As before, the wall with the door – taken as a whole – does not carry as much load as if there were no door (812 lbs less). Also, the opposite side wall experiences the same loss of load-carrying capability, especially in the vicinity of the doorway. The anchorages in the back side wall collectively carry less load than if the door were not present (1,115 lbs less). Therefore, the back side wall – without the door – actually carries less load than the front wall with the door (303 lbs less).



Figure 12. Reaction profile for the side walls when a door is located off-center in the near side wall.

The significant difference in this scenario is present in the end walls (Figure 13). When the door was centered, each gable wall helped carry the balance of the load lost in the side walls and, thus, saw an increase – equal for both (Appendix G, Figure G-6). When the door is offset to one side, the response is obviously no longer symmetric. The gable wall closest to the door carries most of the difference in load (Figure 13). The other end, farthest from the door, actually carries less load than if the door was not present. Simply stated, the gable wall closest to the door carries more load than the one farther away. This highlights the load path for this scenario: load that is not transmitted in the vicinity of the doorway is transferred to the closest gable wall, leaving the other gable wall with a net reduction in overall load transfer.



Figure 13. Reaction profile for the gable walls when a door is located off-center in the near side wall.

Doors Centered in Both Walls

This is a combination of the two previous scenarios, with 16-ft doors centered in one gable wall and one side wall (Appendix G, Figure G-9). In this combined arrangement, the reaction profiles are not significantly different than for the individual cases alone. This scenario highlights the system effects arising from the orientation of the trusses. When a door is located in the gable wall, the opposite end does not significantly behave any different. However, when a door is located in the side wall, the opposite side witnesses a dramatic reduction in load transfer in the region of the door.

Gable Wall Missing (Three-sided structure)

Here the *lower* section of one gable end has been omitted (below the top plate), leaving only the triangular portion of wall sheathing *above* the top plate intact (Appendix G, Figure G-10). Unexpectedly, the back gable wall does not compensate for the opening and carry more load. Instead, it actually carries less (1,164 lbs less, -9% difference). The missing gable wall provides the required flexibility in the structure to overcome the directionality effect of the trusses, and it reduces the load-carrying capability in the back gable wall – akin to the observations made when adding a door in the side wall. The side walls are forced to make up for this reduced load transfer in the gable ends. Thus, each side collectively carries more load than before (+5,225 lbs, +20% difference). This increase is primarily witnessed over the first half of the building, closest to the opening.

Effect of Roof Blocking

Blocking was added to the roof structure, spaced at 4-ft intervals – or the width of the roof panels (Appendix G, Figure G-11). This practice is rarely used in modern residential construction, but its effect on the structure was of interest. In general, the effect of blocking in the roof structure was found to be negligible when subjected to uplift loads. This is expected as the primary purpose for adding blocking to a diaphragm is to resist lateral loads, not vertical.

Effect of Overhang Construction (Ladder vs. Outlooker)

In standard construction practice, the gable overhangs are framed predominantly with one of two different options (Appendix G, Figure G-12). The outlooker style overhang is used for overhangs that extend 1-ft or more beyond the walls of the building. For shorter overhangs (i.e. less than 1-ft), the ladder style is used. In the presence of uniform uplift, the framing style of the gable overhang does not affect the reaction profiles significantly. As expected, the ladder style yields slightly lower uplift reactions than the outlooker style. This occurs because the outlooker acts like a lever with its pivot point at the connection to the first interior truss. In this fashion, there is amplification – from prying action – at the midpoint of the outlooker directly atop the gable wall and anchor bolts. There is no difference in the side wall reaction profile regardless of the gable overhang framing choice.

Effect of Anchor Bolt Spacing (4-ft vs. 6-ft)

High wind scenarios recommend an anchor bolt spacing of 4-ft, which is the default spacing used in this study. However, in other geographic regions the typical anchor bolt spacing is 6-ft o.c. As expected, the result of increasing the spacing from 4-ft to 6-ft is higher load at each anchorage device since there are a fewer number of devices to resist the same amount of applied uplift (Appendix G, Figure G-13). The general shape of the reaction profiles is the same; only the magnitude of the individual reactions is affected. The maximum reaction, located at the midpoint of the side wall, increases from 2,898 lbs to 4,636 lbs (+60%). The reaction in the center of the gable wall increases from 1,642 lbs to 2,551 lbs (+55%).

Effect of Anchor Bolts Missing

Two scenarios were considered: (1) an anchor bolt missing from the center of side wall – where the overall maximum reaction occurs, and (2) an anchor bolt missing from the center of the gable wall – where the localized load amplification for that wall occurs. When the anchorage is missing in the *side* wall (Appendix G, Figure G-14), the reactions for the neighboring anchor bolts increase. In the opposite wall, a decrease in load transfer takes place in the vicinity of the missing bolt. Taken as a whole, the side walls no longer carry the same amount of load, and the balance is carried by the end walls. When the anchorage is missing in the *end* wall (Appendix G, Figure G-15), the neighboring anchor bolts once again see an increase. However, the opposite gable wall does not witness a decrease in the load that it carries. This behavior is wholly unlike the case for the anchor bolt missing in the side wall. The direction of the trusses isolates the back wall response from minor geometry changes that occur in the front gable wall. Also, the front gable wall – taken as a whole – does not carry the same load as before. The remainder goes into the side wall over the first half of the building. This same behavior was observed with the scenarios involving the overhead door opening when it was located in the gable wall. In fact, these cases involving missing anchor bolts can be described as having the same response as those associated with openings/doors in the walls – just muted or less severe.

SIMULATED HURRICANE UPLIFT PRESSURES

The results from the simulated hurricane events (Datin and Prevatt 2007) are provided in Figures 14 through 16. The plots represent the vertical foundation reactions for each wall within the structure. The edge nailing of the wall sheathing is 6-inches on center. Positive values represent uplift, for both applied pressure and observed reactions.

Load Case 1 – Absolute Maximum Uplift at the Corner of the Roof

Load case 1 behaves as expected with the highest uplift coinciding over the corner of the building where the maximum uplift pressure is present (Figure 14). The gable wall on the leeward side of building (dashed line) displays a more symmetrical reaction profile, similar to the profile observed when subjected to a uniform pressure (Figure 3). The side walls indicate that the uplift occurring at the corner only affects the reaction profile over the windward half of the building. The



Figure 14. Wind tunnel pressures, load case 1 – maximum uplift at the corner of the roof.

profile for the leeward half of the building behaves just as one that is subjected to a uniform uplift pressure (Figure 5).

Load Case 2 – Local Maxima Over Entire Roof

Figure 15 shows the reaction profile for load case 2 of the simulated hurricane loading. Again, the gable profiles behave as expected, with the windward end wall experiencing more uplift than the leeward end wall. The leeward side wall experiences the highest uplift since the leeward roof is loaded with more pressure. There is a significant drop in the uplift reaction in the leeward side wall (dashed line) at joint I.D. 92. This occurs because there is a net lateral force that "racks" the structure towards this corner of the building. The lateral force arises because the uplift pressures do not act purely vertical. Instead, they are oriented normal to the roof, giving rise to both a horizontal and a vertical component of force. Because there is more uplift pressure on the leeward roof than the windward side, there is a net horizontal force acting on the structure, creating the same effect as a lateral load.

Load Case 3 – Absolute Maximum Uplift at the Ridge of the Roof

The reaction profile for load case 3 is presented in Figure 16. This case is unique because the applied loads include a pressure acting downward (shown as a negative value in Figure 16). The reaction profile for the windward gable end shows a load intensity directly below the ridgeline of the roof where the maximum uplift occurs. The leeward gable end shows greater uplift at joint I.D. 5 than at joint number 4, indicative of the net lateral load being applied perpendicular to the



Figure 15. Wind tunnel pressures, load case 2 – local maxima over entire roof.





View looking

this way



Figure 16. Wind tunnel pressures, load case 3 – maximum pressure at the ridge.

Joint ID (for windward side)

side walls. As in load case 2, the net lateral load is a result of an imbalance of the applied wind pressures acting normal to the roof. This imbalance also gives rise to the reaction profile observed in the side walls. The leeward side of the roof is subjected to greater applied uplift pressures, resulting in higher reaction forces.

ASCE 7-05 PRESSURES

Three scenarios were considered with the ASCE 7-05 pressures: (1) uplift acting alone, (2) lateral forces acting alone, and (3) a combination of both – lateral in conjunction with uplift. Output plots as well as applied pressure values for the three load cases are provided in Appendix I.

Uplift Loads Acting Alone

The general shape of the reaction profiles (Appendix I, Figure I-1) is similar to that which was witnessed for the uniform pressure scenarios (Figures 3 and 5). The magnitude of the reaction values is slightly different, though, because the applied load is not identical to the uniform pressure cases.

Lateral Loads Acting Alone

This load scenario highlights an observation that was not expected. The interior six anchor bolts on the windward side wall experience *less* uplift than the corresponding anchor bolts on the leeward side (Figure 17). This results from system effects within the roof assembly. When the index building shifts or "racks" toward the back side wall due to lateral loads applied perpendicular to the ridgeline, the overhang plunges downward in the front and lifts up in the back (Figure 18). If



Figure 17. Side wall reaction profile with ASCE 7-05 lateral pressures acting alone.



Figure 18. System effects within the truss assembly due to lateral loads.

the system were rigid, the lateral loads would create an overturning mechanism with uplift on the windward side. However, when acting in conjunction with the aforementioned roof assembly system effects, the net outcome is a downward force on the windward side. An equal and opposite response is observed on the leeward side.

It should be noted that the ends of the building behave as expected based on rigid-body motion. The gable end reactions show that uplift is observed at the windward corners of the building while compression is experienced at the leeward corners (Figure 19). Thus, the building behaves as a rigid body near its ends where the presence of the gable wall provides the required lateral stiffness. However, near the middle of the building where lateral rigidity is not provided, system effects within the roof assembly dominate the building's response and those artifacts generally associated with rigid-body motion (i.e. overturning reactions) are not witnessed. In essence, the roof system in plan view acts as a beam subjected to flexure. The gable ends provide restraint while the lateral forces acting perpendicular to the ridgeline induce bending. In the presence of this bending force, the roof assembly experiences torsional system effects which cause it to deflect out-of-plane, resulting in the upward pitch in the rear and the downturn in the front (Figure 18).

A final note about this load case can be made in relation to the general shape of the reaction curves. It was observed that the reaction plots are doublysymmetric about one another. To be precise, the reaction profile for the windward gable wall can be mirrored about both its vertical and horizontal axis to yield the profile for the leeward gable wall (Figure 19). The same is true for the side wall profiles (Figure 17). This characteristic is expected for lateral loading applied to a symmetrical building geometry.



Figure 19. Gable wall reaction profile with ASCE 7-05 lateral pressures acting alone.

Lateral + Uplift Loads

This scenario is a superposition of the previous two, and the reaction profiles behave accordingly (Appendix I, Figure I-3). At each anchorage device, the uplift reaction is the algebraic sum of the forces induced by the individual cases acting alone.

COMPARISONS

Simulated Hurricane vs. ASCE 7-05

It is of particular importance to determine whether or not the code-based design loads (ASCE 7-05) adequately address the sustained effects from extreme wind events such as hurricanes. The current study offers a unique insight into this area of interest. Thus, the uplift reactions predicted by the SAP model are compared between the three simulated hurricane load scenarios and the ASCE 7-05 uplift-only scenario (see Figure 20 and Appendix I, Figures I-4 and I-5). It can be seen that the code procedure satisfactorily encompasses the loads witnessed during the three hurricane simulations.



Figure 20. Comparison between uplift reactions using ASCE 7-05 and those predicted by the simulated hurricane events. The solid line represents the windward end wall while the dashed line signifies the opposite, leeward end.

SAP vs. Wood Frame Construction Manual

In addition to the numerous validations already noted, the results from the SAP model can also be compared against values tabulated in the Wood Frame Construction Manual (WFCM) published by the American Forest and Paper Association (2001). The most noteworthy comparison comes from Table 2.2A in the WFCM, wherein uplift connection loads are tabulated at different wind speeds. In order to make a direct comparison to SAP, no dead load is assumed to act within the building. Thus, the WFCM gives an uplift connection load of 548.5 lb/ft. With anchor bolts spaced at 4-ft intervals, this translates into an individual uplift load of 2,194 lbs. This value is derived using the "Main Wind Force-Resisting System" (MWFRS) pressures given by ASCE (considering only uplift). For comparison, the maximum individual uplift reaction predicted by the SAP model is 2,244 lbs (+2% difference), which is a result of applying "Component and Cladding" (C&C) loads to the model (see Appendix I, Figure I-1).

Although the two values show good agreement, the most significant ramification of this comparison is that the Wood Frame Construction Manual uses MWFRS pressures to derive their tabulated values, while the SAP model uses C&C pressures. Appendix I provides a more thorough explanation of the difference between these two types of wind loads, but for the present discussion it is important to realize that the C&C pressures represent localized peak loads acting directly on specific structural elements. On the other hand, MWFRS pressures were developed for members which do not receive wind loads directly. Therefore, MWFRS pressures are generally lower than their C&C counterparts because the localized effects that cause the higher pressure coefficients for components and cladding are effectively averaged by the time these forces make their way into the MWFRS elements (AF&PA 2001). In other words, ASCE 7-05 and the WFCM acknowledge that there are system effects at play within the structure that reduce the intensity of the wind loads as they are transmitted throughout the building. However, they have no way of accounting for these system effects directly. Instead, the code compensates by using two completely different sets of wind loads.

The advantage of using a computerized 3D analytical tool like the present SAP model lies in the fact that system effects are inherently incorporated. Thus, the need for two different sets of wind loads may perhaps be alleviated. The present study emphasizes this potential benefit, observing that "Component and Cladding" pressures can be applied to the outermost surface (i.e. sheathing), yet the program output for *foundation-level* forces are within 2% of those predicted using the "Main Wind Force-Resisting System" pressures. It must be clarified, however, that this observation is of course limited only to the uplift loads and building geometry explored within this research. The ASCE 7-05 wind loading procedure, including the distinction between C&C and MWFRS pressures, has a proven track record of success in practice and is applicable over a wide range of building geometries and loading scenarios not considered in the present study.

ROOF SHEATHING UPLIFT

As already mentioned in the "Introduction and Background" section, damage assessment teams in the aftermath of Hurricane Katrina observed widespread damage and significant patterns of structural failure. A significant amount of these failures were related to the uplift of roof sheathing panels around the perimeter of the roof and near the ridgeline. This type of failure can be potentially costly since wind-driven rain can enter the building once the roof sheathing panels are detached. Efforts were made in the present study to use the SAP model for the prediction of uplift forces between the roof shell element and the framing members below – in essence to predict sheathing uplift failures. However, this undertaking proved to be inconclusive and was not developed further (see explanation in Appendix J).

Alternatively, a simple hand calculation was used to readily show that 12inch *field* nailing is not adequate to secure the sheathing panels located in the critical regions along the perimeter of the roof and near the ridgeline, for a basic wind speed of 130 mph (Appendix J). It was noted by the damage assessment teams that *conventional* construction provisions were erroneously used in the high wind region of the Gulf Coast (van de Lindt et al. 2007). A nailing schedule of 6inches along supported panel edges and 12-inches over intermediate supports (field nailing) is considered to be "conventional" for most regions of the United States (not high wind locations), and this fastening schedule is normally included under the prescriptive sections of most building codes (APA 2006). Thus, it is presumed that builders and inspectors in the Gulf Coast region incorrectly employed and approved this *conventional* 6-inch edge/12-inch field nailing schedule in the new residential construction of the Gulf Coast region (high wind). Instead, a *minimum* nailing schedule of 6-inch edge/6-inch field should have been used for the critical zones around the perimeter of the roof and near the ridgeline.

Research conducted in support of this project concurs with this presumption. Comparisons proved that the ASCE 7-05 pressures adequately encompass the forces experienced during simulated hurricane events (Figure 20 and Appendix I, Figures I-4 and I-5). Using these code-developed pressures, a straightforward hand calculation (Appendix J) shows that a minimum of 6-inch edge nailing with 6-inch field nailing is required for basic wind speeds of 130 mph (near the corner and perimeter zones of the roof). This calculation aligns with the recommendations of the APA (2006) and the Wood Frame Construction Manual (AF&PA 2001), both of which offer published nailing schedules tabulated at different wind speeds. These sources recommend a *minimum* of 6-inch edge/6-inch field nailing for locations that experience basic wind speeds (3-second gust) greater than or equal to 90 mph, including all coastal regions of the Gulf of Mexico and the entire Atlantic seaboard (APA 2006, WFCM 2001, ASCE 2005). Thus, all sources are in agreement: 6-inch edge/12-inch field nailing schedules are not adequate for roof perimeter zones (including both sides of the roof peak) in the high wind regions of the Gulf Coast.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on research conducted in support of this project and, therefore, pertain only to the specific load cases previously described.

- 1. The 3D computer model and techniques developed within this research effort successfully predicted the behavior of complex, three-dimensional, wood-framed structures.
- 2. The correlation procedure and modeling techniques developed within this study eliminate the need to represent individual fasteners in the model and provide a simple means by which to revise the nailing schedule if desired. The changes in stiffness associated with variations of the nailing schedule are tied to a single material property in SAP2000, which can be easily modified.
- 3. Near the ends of the building, load accumulates at the ridgeline of the roof and is transferred to the gable walls directly below the roof peak. The anchor bolts in the middle of the gable wall consequently carry a significant portion of the uplift loads contained within the end walls. Also, since attic vents are often placed directly below the roof peak, special care should be taken to ensure that the roof-level loads can be successfully transmitted from the ridge through the gable wall and ultimately to the foundation.

- 4. When subjected to uniform uplift loads, anchor bolts located in the side walls experience the highest uplift reactions. In the middle of the building, loads originating in the roof structure are not shared with the end walls. Instead the side walls carry the uplift forces alone. The presence of lateral loads or extremely non-uniform uplift scenarios (i.e. loading only one corner of the building) repudiates the validity of this observation.
- 5. The edge nailing density strongly influences the ability of the walls to share roof-level loads. In uplift scenarios, the walls of the building distribute roof-level loads to the foundation. The extent to which they distribute these loads is dependent on their stiffness, which is in turn dependent on the edge nailing of the wall sheathing.
- 6. The addition of a door to *any* wall results in a loss of load carried by the entire wall. The remaining individual anchorages, especially those directly adjacent to the opening, experience greater uplift. However, taken as a whole, the wall collectively carries less load than if the door were not present. The balance of the uplift forces is shared with the other walls in the structure.
- 7. The extent to which a variation in the geometry (e.g. missing anchor bolt or the presence of an opening) on one side of the building affects the opposite side of the building is highly dependent on the orientation of the trusses. Modifications to one gable end do not significantly affect the opposite gable end. The orientation of the trusses parallel to the gable walls isolates the two

ends from one another. However, those same modifications to a side wall have a considerable affect in the opposite side wall.

- 8. ASCE 7-05 "Component and Cladding" pressures adequately address the expected uplift loads from extreme wind events such as hurricanes. The uplift reactions predicted by the SAP model when loaded with the ASCE 7-05 C&C pressures (uplift only) fully encompass those of the simulated hurricane events at the same basic wind speed.
- 9. The analytical model developed in this study accurately predicted uplift forces at the foundation level using ASCE 7-05 "Component and Cladding" pressures applied to the roof. To account for system effects, foundation-level forces are conventionally computed using the MWFRS set of wind loads from ASCE 7-05.

SUGGESTIONS FOR FURTHER RESEARCH

- The SAP model developed in the present study represents a four-sided structure with minimal penetrations. Future research could make use of the modeling techniques use herein to develop the model into a more complex residential structure. Suggestions include the following:
 - a. The addition of doors and windows
 - b. Gypsum wallboard could be attached to the interior side of the studs using the layered shell option built into SAP

- c. Interior walls load-bearing or merely partitions could be included
- 2. Additional load cases could be considered. The loads in the project were limited primarily to uplift. For example, wind tunnel testing could provide the pressures experienced on the walls of the index building. These could then be included in the model in the form of lateral loads applied separately or in conjunction with the existing uplift loads.
- 3. More complex roof geometries could be investigated. The present study made use of the traditional gable roof style. With the same building footprint, a hipped roof could easily be researched. Alternatively, the footprint could be modified, in which case more complex roof assemblies might be in order.
- 4. Foundation level uplift reactions were the exclusive output with the current effort. Future research could investigate member forces (axial force, bending moment, torsion, etc.) which, in turn, could be used to evaluate stresses based on member sizes. This could be done with no modifications whatsoever to the present SAP model.
- 5. Alternative framing styles could be explored. For example, steel framing can easily be substituted for the existing wood studs by changing the material properties and frame assignments in SAP. Heavy timber construction can similarly be modeled.
- 6. The use of the SAP model for predicting roof sheathing uplift can be explored in greater detail. Efforts within this project proved to be inconclusive, however,
with more time this option might prove to be very powerful for examining panel-to-frame forces.

7. Full-scale shear wall tests could be conducted at various edge nailing schedules to further validate the correlation procedure used in this project. Most shear wall studies are carried out to failure loads (i.e. ultimate capacity), providing very little "load vs. deflection" information within the lower load ranges.

PRACTICAL ADVICE

The suggestions that follow pertain to the building dimensions and load scenarios examined in support of this research effort only.

- 1. When a door is placed in the side wall of the building, the uplift reactions at the columns that frame the opening nearly double. Locating a door in the end wall of the building similarly amplifies the reaction profile, but the overall *magnitude* of the uplift in the end wall is far less than in the side wall. In fact, the maximum uplift experienced when one entire gable wall was missing (30 feet) was *less* than if a typical overhead door (16 feet) were located in the side wall. Thus, it is recommended that an overhead door be placed in the end wall of the building rather than in the side wall. If a door must be placed in the side wall, care should be taken to ensure that the foundation connections are designed for the increased loads predicted by this study.
- 2. In uplift scenarios, the presence of blocking in the roof assembly and the framing style of the gable overhang do not significantly affect the *foundation* reactions.

- 3. Roof sheathing uplift, on the other hand, is affected by the presence of blocking (i.e. blocking in these regions reduces the chance of panel separation). Since the outlooker-style gable overhang is, in essence, equivalent to roof blocking for the first two bays of roof framing, it is recommended over the ladder style to help resist sheathing uplift in high wind regions.
- 4. Conventional edge/field nailing of 6-inches/12-inches is not adequate for roof perimeter zones (and on either side of the roof peak) in the high wind regions of the Gulf Coast. A *minimum* of 6-inch edge/6-inch field nailing is recommended for these critical roof zones.

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APPENDICES

APPENDIX A

CONSTRUCTION DETAILS OF THE INDEX BUILDING

GENERAL OVERVIEW

Pictures of the SAP model are provided in Figures A-1 to A-6 for reference. The approximate footprint of the index building is 30-ft x 40-ft. The width was altered slightly in order to provide a convenient meshing distance in SAP. Thus, the actual footprint dimensions in SAP are 29-ft 4-in wide by 40-ft long. A framing plan was developed to guide the modeling process as well as to be used by researchers at the University of Florida in developing their 1/3 scale model prototype. The framing plan is provided in Figures A-7 to A-10 for reference.

WALL FRAMING

Wall framing members – studs and plates – are Spruce-Pine-Fir (SPF), studgrade. Interior studs are 2-in by 4-in nominal dimension (1.5-in x 3.5-in actual), referred to simply as "2x4's" for the remainder of this section. End and corner studs are double 2x4's. Thus, the blocked corner detail shown in the framing plan (Figures A-7 and A-8) was not employed in SAP for convenience. Interior studs are spaced 16-inches apart on center. The top plates of the walls are comprised of two 2x4's oriented flatwise and nailed together. The bottom plate is a single 2x4 oriented flatwise. The bottom plate is anchored to the foundation using ½-inch diameter anchor bolts spaced at 4-ft intervals. At each corner of the building there is a Simpson Strong-Tie HDU2 hold-down attached to the double-stud (not shown in the framing plan).

ROOF FRAMING

Truss members are comprised entirely of 2x4 dimension lumber of Southern Yellow Pine (SYP), No. 3 and stud grade. Interior trusses are Fink style spaced 24-inches apart. End trusses have vertical web members also spaced 24inches on center. The roof pitch is 4:12. Two gable overhang details were considered in the study: the outlooker style and the ladder style. The outlooker style is used for overhangs that extend 1-foot or more beyond the building. Ladder styles are used for shorter overhangs. In SAP both overhangs were considered to be 2-feet long for direct comparison of loads. Overhang details are not provided in the framing plan.

SHEATHING

The roof sheathing is ¹/₂-inch plywood while the wall sheathing is 7/16-inch oriented strand board (OSB). Roof sheathing is oriented with its strong axis parallel to the ridgeline of the roof. In this fashion, the 8-foot panel length is parallel to the ridgeline while the 4-foot panel width is parallel to the truss lines. Panel joints are staggered to eliminate continuous edge lines. Conversely, the OSB panels are fastened with their strong axis oriented vertically (the long dimension of the panel is placed vertically).



Figure A-1. SAP model of the index building – frame members shown with sheathing.



Figure A-2. SAP model of the index building showing the meshing of the sheathing (represented by shell elements).



Figure A-3. SAP model of the index building showing the springs which represent anchor bolts and hold-downs.



Figure A-4. SAP model of the index building – extruded view of framing members (sheathing not shown for clarity).



Figure A-5. SAP model of the index building – view looking down.



Figure A-6. Interior section view of the index building in SAP.



Figure A-7. Framing plan, sheet 1 of 4 – Overall plot plan.



Figure A-8. Framing plan, sheet 2 of 4 – Gable wall details.



Figure A-9. Framing plan, sheet 3 of 4 – Side wall details.



Figure A-10. Framing plan, sheet 4 of 4 – Truss detail.

APPENDIX B

MODELING TECHNIQUES

DESIGN ANALOG

To make a computer model of any building, the real structure must be idealized. This theoretical representation of the actual building geometry is called the design analog. Studs, trusses, and other framing members are symbolized simply as line segments in SAP2000, and a variety of design analogs are possible.

In this study, the line segments representing the *wall framing* members are drawn through the centerlines of the actual building geometry. On the other hand, the *roof system* (i.e. truss analog) does not use the centerlines, and instead follows standard industry convention (Callahan 1993) – see Figure B-1. With this in mind, as well as considering automatic meshing options, the actual footprint dimensions of the SAP2000 structure are 29-ft 4-in wide by 40-ft long.



Figure B-1. Analog used in SAP2000.

SHELL ELEMENT BEHAVIOR

As noted in the SAP reference manual (Computers and Structures 2008), the shell element is a type of area object that is used to model (1) membrane, (2) plate, and (3) shell behavior. Membrane behavior only includes in-plane forces (no out-of-plane). Plate behavior is the opposite – out-of-plane bending and transverse shear are included but in-plane forces are not. The shell behavior is a combination of the two. Thus, all forces and moments are supported. Further, two options exist once the shell element is selected: thick or thin, both of which affect the out-ofplane bending behavior. The thin shell neglects the effects of transverse shearing deformation, while the thick shell includes them. To fully capture the behavior of the roof and wall sheathing, the thick shell element, with full shell behavior, was selected for use in this study.

Because each wall and roof area was defined using one shell element, it is assumed that the actual 4-ft x 8-ft sheathing panels transfer load and moment continuously across their joints. To explore this assumption, the wall and roof systems are examined in greater detail. In a wall system, the panels are typically placed vertically; therefore all edges of the panel are blocked (see Figure B-2). Also, the nails are located within 3/8-inch of the panel edges, providing fixity directly up to the joint line. In addition, nail spacing is at most 6-inches on center along the edges. This continuous support system and high nail density give credibility to the assumption of continuity across the joints for wall sheathing.



Figure B-2. Typical wall detail. Blocking along all edges of each panel and high nailing density at edges contribute to the assumption of continuity across individual panel joints.

In the roof structure, however, blocking is rarely used in residential structures. Nevertheless, the edges of the plywood that fall upon truss lines are nailed every 6-inches at the most, oftentimes less (i.e. 3 or 4-inches on center), and the joints in this direction are staggered so as not to create a continuous line. Along the unblocked edges, field nailing requires each nail to be located within 3/8-inch on either side of the edge, so there are two nails within a distance of approximately 1-inch at these edges (see Figure B-3).



Figure B-3. Typical roof sheathing detail. The assumption of continuity across the joints is drawn from four sources: (1) staggered joints along truss lines, (2) 6-inch or less edge nail spacing along truss lines, (3) unblocked edges have nails within 3/8-inch from edge, and (4) "H-clips" at intermediate locations.

In addition, "H-clips" are recommended between adjacent panels and are located midway between each pair of trusses. In lieu of these clips, tongue-and-groove edges are recommended, also encouraging continuity across the joints (APA 2007). With these considerations in mind, it is plausible that the roof panels, like the wall panels, behave less as individual sheets and more as one cohesive unit.

MESHING OF THE SHELL ELEMENT

The automatic meshing option feature was used exclusively in this study. The shell elements were meshed using either the "maximum size" option or the

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"general divide" tool from the "Assign automatic area mesh" menu. Figure B-4 shows the inputs that are required to mesh a shell element using the "maximum size" option.

		Meshing Group		Units			
C No Auto Meshing		ALL	•	lb, in, F	•		
C Mesh Area Into This Number of Objects (Quads and Tri Along Edge from Point 1 to 2 Along Edge from Point 1 to 3	angles Only)	Sub Mesh Option Sub Mesh As Required To Obtain Elements No Larger Than The Specified Maximum Size					
 Mesh Area Into Objects of This Maximum Size (Quads a Along Edge from Point 1 to 2 Along Edge from Point 1 to 3 	und Triangles Only) 16 16	Maximum Sub Me	shed Size				
Mesh Area Based On Points On Area Edges (Quads an Points Determined From: Intersections of Straight Line Objects In Meshing	d Triangles Only) Group With Area Edges						
Point Objects In Meshing Group That Are On Are							
Mesh Area Using Cookie Cut Based On Straight Line Objection Extend All Lines To Intersect Area Edges							
C Mesh Area Using Cookie Cut Based On Point Objects In 1							
Rotation of Cut Lines From Area Local Axes (Deg)							
C Mesh Area Using General Divide Tool Based On Points a Maximum Size of Divided Object							
Local Axes For Added Points							
🥅 Make same on Edge if adjacent corners have same local	axes definition						
🦳 Make same on Face if all corners have same local axes d							
Restraints and Constraints For Added Points		ПК	1				
Add on Edge when restraints/constraints exist at adjacen (Applies if added edge point and adjacent corner points had		Cancel	1				
Add on Face when restraints/constraints exist at all corne		-					

Figure B-4. Meshing of shell elements based on maximum size. This option was used for the walls (between the top and bottom plate) and for the roof.

When the mesh is defined in this manner, SAP add points along each edge of the area object (shell element) at equally spaced intervals such that the distance between the points does not exceed the specified length, thereby creating a grid or "mesh" over the area. For example, the length has been set to 16-inches in Figure B-4. Consequently, SAP will divide the wall area into 16-in x 16-in grids. The mesh can be refined as desired. Smaller grid spacing will result in a finer mesh, but this comes at the expense of longer processing time. In this study, walls were meshed to match the stud spacing (16-inches), and roof sheathing was meshed to match the truss spacing (24-inches). In a few load cases, it was necessary to divide the roof sheathing into smaller grids (12-inch meshing) based on the manner in which the applied loads were defined. This brings up an interesting point. To ensure that the framing members interact properly with the shell elements, it is important to use a mesh size that is a multiple of the frame spacing in the area of interest (e.g. for 24-inch truss spacing, mesh the roof elements at 6, 12, or 24 inches). When odd lengths are present within the structure, such as around openings or at the very end of a wall, a separate shell element with its own unique meshing option can be created. Alternatively, different meshing options such as the "general divide" tool, described in the following paragraph, could be explored in these cases.

The wall sheathing in the gable ends above the top plate was meshed using the "general divide" option (see Figure B-5). The meshing group in this case consisted of the framing members in the gable wall.



Figure B-5. Meshing of shell elements based on points and lines in a specified meshing group. This "general divide" option was used for the gable walls in the region above the top plate.

CONNECTIVITY

The members within the truss are connected using a mixture of pinned and rigid connections (see Figure B-6). This configuration was used successfully in the research effort conducted by Gupta and Limkatanyoo (2008). It is worth noting that the joint representing the heel of the truss is *not* coincident with the connection to the top plate of the wall. Since the analog of the wall members is drawn through the centerlines, these two joints are offset by a distance of 1.75 inches (i.e. half the width of the nominal 2-in x 4-in top plate). Both of these connections are rigid. See Figures B-6 and B-7. Vertical web members in the gable end trusses and

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Figure B-6. Connectivity of the trusses – mixture of pinned and rigid joints.



Figure B-7. Detail of the truss heel showing the offset between heel joint and connection to the top plate (1.75-inches).

overhang framing members were considered to be pinned at each end.

All members in the walls are pinned, including stud-to-plate connections (at both ends) as well as plate-to-plate connections at the corners of the building (see Figure B-8). In this configuration, the wall framing provides no lateral stiffness unless sheathing is present.



Figure B-8. All connections within the wall are considered to be pinned.

STIFFNESS OF HOLD-DOWNS AND ANCHOR BOLTS

Established practices in the design of wood structures (Breyer 2007) specify that hold-down devices carry vertical loads (i.e. axial) but no lateral loads (i.e. shear). A separate set of anchorage devices, called anchor bolts, carry all of the shear forces as well as resist wind uplift loads. Therefore, each hold-down device in this study is modeled as *one* grounded spring having a stiffness in the Z-direction alone (i.e. axial) while the anchor bolts are modeled as *three* grounded springs – providing a stiffness in each of the X, Y, and Z-directions. The X and Y-directions correspond to shear forces parallel and perpendicular to the wall, respectively, while the Z-direction corresponds to vertical forces (i.e. axial stiffness). See Figure B-9.



Figure B-9. Orientation of axes for anchor bolts and hold-down devices.

The axial stiffness of the hold-down device is listed by the manufacturer (Simpson Strong-Tie, 2008). The most common type of connector, the HDU2, was selected. The published axial stiffness of this connector is 35,000 lb/in. This value takes into account fastener slip, hold-down elongation, and bolt elongation. The axial stiffness of the anchor bolts, on the other hand, was determined using research data from a previous study. Seaders (2004) subjected partially anchored walls to lateral loads. These walls contained no hold-down devices whatsoever, isolating

the behavior of the anchor bolts. Vertical deflection was measured at both the tension (inboard) and the compression (outboard) side of the walls as lateral load was applied (Figure B-10). When plotted against the applied load, this data provided axial stiffness values for the anchor bolt assembly in both tension and compression. As it turned out, the stiffness values in compression were similar to those in tension (Figure B-11). Because these tests were conducted on a full-scale wall assembly and not on the anchor bolts themselves, this stiffness not only takes into account bolt elongation, but also wood crushing, washer deformation, and other similar real-world effects that take place when a structure is loaded. Values were averaged for five different walls, resulting in a vertical stiffness of 33,000 lb/in. For practical purposes this value was rounded up to match the value of the hold-down stiffness, so that both the anchor bolts and the hold-downs have the same axial stiffness of 35,000 lb/in.

The shear stiffness of the anchor bolts in the X and Y-direction was determined using a procedure recommended by the NDS (AF&PA 2005). It suggests using the load-slip modulus value for wood-to-wood connections. This may seem counterintuitive considering the anchor bolts connect wood to concrete (i.e. bottom plate to foundation); however, the NDS value for dowel-type fasteners (e.g. anchor bolts) is governed by the MOE of the main member and the side member. Relatively speaking, the MOE of concrete is much closer to wood than it is to steel, the other available load-slip value. In fact, the MOE of steel is one order of magnitude greater than the MOE of wood and concrete (see Table B-1).



Figure B-10. Inboard and outboard vertical deflection measurements for a typical partially anchored wall. (Seaders 2004)



Figure B-11. Stiffness in the vertical direction for a typical partially anchored wall. Five walls in total were averaged. (Seaders 2004)

Material	Modulus of Elasticity (10 ⁶ psi)	Source
Wood	1-2	Askeland 1994
Concrete	4	Lardner 1994
Steel	29	Lardner 1994

Table B-1. Comparison of MOE values for determining the load-slip modulus.

For this reason, the American Wood Council recommends "for fasteners into concrete, wood-to-wood values are used as a reasonably conservative approach" (AF&PA 2007). Accordingly, the shear stiffness for the anchor bolts was calculated using the load-slip modulus equation given in the NDS code as follows:

 $\gamma = 180,000 \cdot D^{1.5}$ (lb/in) NDS 10.3.6

"D" in this equation is the diameter of the anchor bolt (1/2-inch). Therefore, a value of 65,000 lb/in was used for the shear stiffness of the anchor bolts in the X and Y-directions. Table B-2 summarizes the spring stiffness values used in this study.

Item	X- direction (shear) lb/in	Y- direction (shear) lb/in	Z- direction (axial) lb/in	Source
Hold-downs	-	-	35,000	Simpson Strong-Tie (2008)
Anchor bolts	65,000	65,000	35,000	NDS (AF&PA 2005) and Seaders (2004)

Table B-2. Spring stiffness used to model the anchor bolts and hold-downs.

MATERIAL PROPERTIES

Frame Elements

Frame elements, which represent the wall and truss members, were modeled using elastic, isotropic material properties. Two parameters are required to define these materials: the modulus of elasticity (MOE) and Poisson's ratio. The shear modulus is computed as a function of these two values using the following equation.

$$G = \frac{E}{2\left(1+\mu\right)}$$

MOE values were assigned using the NDS code (AF&PA 2005). Poisson's ratio for Southern Pine (SYP) was calculated using the average of the radial and tangential values of the four species that comprise the group. For the Spruce-Pine-Fir (SPF) group, Poisson's ratio was based on three of the eight species that make up the group. Data was not readily available for the remaining five species (USDA 1999).

Table B-3. Elastic isotropic material properties used in the SAP model.

Item	Description	N (10	IOE 0 ⁶ psi)	Poisson's Ratio		
		Value	Source	Value	Source	
Wall	SPF, stud	1.2	NDS	0.40		
members	grade	1.2		0.40	Wood Handbook	
Truss	SYP, No.3	1.4	(AF & F A)	0.26	(USDA 1999)	
members	and stud	1.4	2003a)	0.30		

Shell Elements

Wall and roof sheathing were modeled using SAP's thick shell element. Orthotropic, elastic material properties were then assigned. Nine constants (six of them independent) are needed to describe the behavior of these materials: three moduli of elasticity, three shear moduli, and three Poisson's ratio. The SAP reference manual states that E_3 , μ_{13} , and μ_{23} are "condensed out of the material matrix" (Computers and Structures, 2008). In other words, they are dependent properties. Additionally, the values of G_{13} and G_{23} , which are used to compute the transverse shearing stiffness, were found to have a negligible effect on the behavior of the model. Thus, the only values that are truly of importance are shown in bold in Table B-4. These represent the strong-axis MOE (E_1), weak-axis MOE (E_2), inplane shear modulus (G_{12}), and in-plane Poisson's ratio (μ_{12}). SAP however requires input values for the remaining parameters, so all values must be provided per Table B-4.

Description		$\begin{array}{c c} \mathbf{MOE} & \mathbf{S} \\ \mathbf{MOE} \\ (10^5 \text{ psi}) & \mathbf{MOE} \\ (10) \end{array}$		Shear Iodulus 10 ⁵ psi)		Poisson's Ratio ⁴				
Item	Туре	E ₁	$\mathbf{E_2}$	E ₃	G ₁₂	G ₁₃	G ₂₃	μ_{12}	μ_{13}	μ ₂₃
Wall sheathing ^{1,2}	7/16" OSB	7.4	2.3	2.3	1.2	1.2	1.2	0.08	0.08	0.08
Roof sheathing ³	¹ /2" Plywood	19	2.9	2.9	1.5	1.5	1.5	0.08	0.08	0.08

Table B-4. Elastic orthotropic material properties used in the SAP model.

 $\frac{1}{2}$ MOE and shear modulus values from Doudak (2005)

² Shear modulus values subject to the correlation procedure (Table 4)

³ MOE and shear modulus values from Wolfe and McCarthy (1989) and Kasal (1992)

⁴ Poisson's ratio from Kasal (1992)

DISCUSSION OF ASSUMPTIONS

Although the assumptions used in the modeling process have been introduced and explained, their individual effect on the behavior of the model needs to be discussed. For example, the assumption that the peak of each truss behaves as a pinned connection, while the heel behaves as a rigid one, leads to slightly more deflection in the upper portion of the top chords compared to the lower portion. Also, because the heel joint is fixed, the angle between the top chord and the bottom chord remains fixed as the truss deflects and the members experience inplane rotation. Thus, the rotation of the lower portion of the top chord is coupled with a similar rotation in the bottom chord in the vicinity of the heel joint. This behavior is obviously different than what takes place at the peak where the joint is pinned. Near the ridge, any rotation of the top chord on one side of the roof is essentially isolated from the other side of the roof by the pinned connection.

In addition to the assumptions of connectivity, the correlation model for the nailing schedule of the sheathing has limitations. The correlation procedure, described in the "Results and Discussion" section and in Appendix E, was developed for shear walls 8-feet in length by 8-fee in height. However, the walls of the index building are much longer than this. An explanation of the error introduced by extending the correlation to these longer walls is provided in Appendix E, but for the present discussion it is sufficient to point out that the error increases as the shear walls increase in length and as the edge nail spacing decreases (Figure E-8).

As previously discussed in this section, individual 4-ft x 8-ft plywood/OSB panels were not modeled, and continuity was assumed across their joints. If the panel joints actually behave more like hinges, or something between a pinned or rigid connection, the load sharing behavior of the sheathing would be affected. Specifically, the panels would not distribute load as effectively across their joints, isolating behavior at one end of the building from the other end. In the case of the roof assembly, however, the load sharing characteristics of the model (along with the assumption of continuity across the panel joints) were compared to published data with good agreement (Appendices C and D).

Finally, it should be noted that the assumption of linearity in the model implies that the material properties of the constitutive elements remain in the "linear-elastic" realm. That is, any elongation, rotation, or other type of deflection experienced by the model would theoretically be recovered when the load is removed. Implicit in this assumption is the fact that the load levels are assumed to be within the linear realm as well. In other words, if load levels induce stresses greater than the known proportional limit of the materials, the assumption of linearity is no longer valid and the response of the model loses accuracy. Consequently, it is worth pointing out that the loads encountered in the present research study were sufficiently within the design load range such that the assumption of linearity is valid. Although perhaps a limitation in one sense, the assumption of linearity is essential in maintaining practicality in the modeling process.

APPENDIX C

VALIDATING THE MODEL

VALIDATION

Two-Dimensional Truss Model Behavior

The truss model used in this study is comprised of either rigid or pinned connections (see Appendix B, Figure B-6). These assumptions about connectivity were validated against previous literature and experimental results. Wolfe et al. (1986) subjected forty-two full-size trusses to vertical loads acting over the top chord. Some of the trusses were tested to design loads, while others were loaded to failure. The study included two different roof pitches (3:12 and 6:12) and three different MOE categories (low, medium, and high). Li (1996) used Wolfe's research to validate his own analytical truss model. In his ETABS model, Li made use of spring elements to represent the metal-plate-connected joints at the heel and at the tension splice of the bottom chord. All other connections were assumed to be pinned. Thus, Li's model incorporated semi-rigid behavior that has purposely been eliminated in the present effort. Consequently, these two previously published studies serve as an excellent opportunity to validate the current SAP model.

The 6:12 roof pitch was selected, and a load of 66 lb/ft was applied along the sloped length of the top chord (acting vertically). This load represents the total design load for the truss. Four trusses from each MOE category were used for comparison, and MOE values of the individual members were assigned based on the values provided by Wolfe et al. 1986. In order to directly compare to the reported results, deflections were averaged at the five interior panel point locations (i.e. where the web members intersect the chord members) – see Figure C-1. Deflections were compared at load levels within the design load range to ensure that the response behavior remained within the linear-elastic realm. Table C-1 shows the comparison between the full-scale testing performed by Wolfe et al. (1986), the spring-element ETABS model developed by Li (1996), and the simplified SAP model used in the present study.



Figure C-1. SAP model for the 2D verification.

Table C-1. 2D verification – deflection comparison for 6:12 slope trusses tested
to their design load. Percent difference values are included,
comparing the two analytical models (the present effort and Li's
previous research) to the physical testing performed by Wolfe.

	Deflection (in)							
Truss No.	Wolfe et al. 1986	Present SAP model ¹		Li, 1996 ²				
6L2	0.172	0.179 4%		0.179	4%			
6L3	0.180	0.186	3%	0.186	3%			
6L5	0.194	0.176	-9%	0.176	-9%			
6L7	0.198	0.175	-12%	0.175	-12%			
Average for Low MOE	0.186	0.179	-4%	0.179	-4%			
6M1	0.123	0.118	-4%	0.121	-2%			
6M2	0.136	0.127	-7%	0.129	-5%			
6M4	0.121	0.126	4%	0.127	5%			
6M7	0.117	0.132	13%	0.133	14%			
Average for Medium MOE	0.124	0.126	1%	0.128	3%			
6H1	0.107	0.094	-12%	0.099	-7%			
6H2	0.107	0.097	-9%	0.101	-6%			
6H6	0.086	0.101	17%	0.104	21%			
6H7	0.107	0.102	-5%	0.104	-3%			
Average for High MOE	0.102	0.099	-3%	0.102	0%			

¹ Percent difference values compare the current SAP model to Wolfe et al.

² Percent difference values compare Li's ETABS model to Wolfe et al.

As highlighted in Table C-1, there is – on average – no more than 4% difference between the present SAP model and the full-scale test results, which is the same level of accuracy that Li was able to achieve with his spring-element model. In fact, there is only 3% difference between the present simplified SAP model and the more complicated ETABS spring-element model developed by Li. Therefore, the simplified assumption of connectivity based on rigid and pinned

connections is deemed adequate for use in this study. This conclusion concurs with the findings of Limkatanyoo (2003), who used the same arrangement of rigid/pinned connections in his research. He compared a similar two-dimensional SAP model to an industry-accepted program called VIEW widely used by truss designers. Limkatanyoo checked both deflections and stresses (using the combined stress index, CSI) and concluded that the simplified design analog was in agreement with the more complicated, semi-rigid models employed by the truss industry (i.e. the results from VIEW).

Three-Dimensional Truss Assembly Behavior

Once the model's two-dimensional behavior was validated, the next step involved the verification of its 3D response. To do this, a nine-truss roof assembly was modeled in SAP for comparison to a full-scale test conducted at the Forest Products Laboratory (Wolfe and McCarthy 1989). In this experiment, researchers represented a 16-foot section from the middle of a conventional gable style roof in order to quantify the effects of load sharing within an assembly of trusses. The trusses were built with members from one of three MOE categories, resulting in a nine-truss assembly comprised of three trusses from each stiffness category – low, medium, and high. The variable stiffness trusses were located randomly within the assembly to accentuate the effect of load sharing, the hypothesis being that greater load would be carried by the stiffer trusses regardless of location within the assembly.
Load sharing was quantified using influence matrices for both reactions and deflections. In the case of the reactions, a "load influence matrix" was defined as the sum of the vertical reactions at each truss expressed as a fraction of the total applied load. For deflections, a similar "deflection influence matrix" was defined as the deflections observed at each truss expressed as a fraction of the total deflection for all trusses. At each truss, deflections were measured at four different locations along the top chord and then averaged. First, load was applied consecutively to each individual truss. Next, load was applied to all trusses at once (i.e. full assembly load). In both cases, the load per truss was 66 lb/ft, as in the 2D verification. It's worth noting, though, that the magnitude of the load has little effect on either the load sharing or the deflection sharing results within the 3D assembly. This rather significant finding was observed by the researchers at the FPL, noting that the load influence matrix "changed little with load level throughout the linear load range" and "normalized deflections also show little variation from one load step to the next within the linear range" (Wolfe and McCarthy 1989). In other words, the influence matrices for load and deflection are unaffected by changes in the magnitude of the applied load until the assembly begins to experience damage.

Li (1996) attempted to model this same nine-truss assembly in his research. Instead of using the shell element, whose behavior was not fully understood at the time, Li modeled the plywood sheathing using beam elements. These plywood "beams" had the same thickness as the plywood sheathing and their widths were assigned according to their tributary length along the top chord. He used three beams per roof section (six beams in total). To account for the partial composite action between the sheathing and the top chords, which tends to increase the flexural stiffness of the assembly, Li increased the bending capacity of the top chord members by increasing their moment of inertia. In the present study, partial composite action is inherently built into the model by way of the shell element. Likewise, the need for "sheathing beams" is eradicated for the same reason. Comparison plots between the present SAP model, the physical testing (Wolfe and McCarthy 1989), and the "sheathing beam" model (Li 1996) are given in Appendix D.

Insofar as load sharing is concerned when individual trusses are loaded, there is 14% difference (on average) *at the loaded truss* between the SAP model and the physical testing results. Li was able to achieve 13% difference with his sheathing beam model. However, the striking difference between the two models is not at the *loaded* truss, but rather at the *non-loaded* trusses. SAP shows an average of 8% difference at the non-loaded trusses when compared to the physical testing, while Li's model predicts 26% difference. In other words, when trusses are loaded individually, the two models predict load sharing *at the loaded truss* with the same accuracy, whereas the shell element model is able to capture the load sharing *at the remaining trusses* with much more accuracy than the sheathing beam model. This trend can be observed in Table C-2 as well as the plots provided in Appendix D (Figures D-1 to D-9). Similarly, the deflection sharing values also showed good agreement with the experimental results when each truss was loaded individually. At the loaded truss, the deflection sharing displayed 7% difference. Non-loaded trusses exhibited only 1% difference (see Table C-3 and Appendix D, Figures D-10 to D-18). Li did not include a deflection check in his research, therefore a comparison cannot be made to his work beyond that which is provided in Table C-2.

Table C-2.3D verification – Absolute percent differences in load sharing when
each truss is loaded individually (compared to Wolfe et al. 1989).

Loaded	Present SAP Model (Shell Element)		Li, 1996 (Sheathing Beams)		
Truss	At Loaded Truss	At Non-Loaded Trusses	At Loaded Truss	At Non-Loaded Trusses	
1	8	1	14	39	
2	28	9	5	29	
3	5	7	10	30	
4	9	7	22	12	
5	20	5	10	12	
6	19	5	18	31	
7	2	9	12	13	
8	28	7	12	23	
9	4	24	11	50	
Average	14 %	8 %	13 %	26 %	

Table C-3. 3D verification – **Average percent differences** in <u>deflection</u> sharing when each truss is loaded individually (compared to Wolfe et al. 1989).

Present SAP Model				
(Shell Element)				
At Loaded Truss	At Non-Loaded Trusses			
-7 %	+1 %			

The SAP model was also compared to the experimental tests when the entire assembly was loaded simultaneously. Again, the results showed good agreement (see Table C-4 and plots in Appendix D, Figures D-19 and D-20). Individual deviations were limited to 12% difference in load sharing and 9% difference in deflection sharing when compared to Wolfe et al. (1989). Li did not include the full assembly load scenario in his research, so no comparison is made to his work here.

Truss Number	Present SAP Model (Shell Element)		
	Load Sharing	Deflection Sharing	
1	2	-9	
2	8	1	
3	-12	-3	
4	-10	-1	
5	1	2	
6	11	1	
7	-4	0	
8	6	0	
9	1	3	

Table C-4.3D verification –Percent differences in load and deflection sharing
when all trusses are loaded simultaneously (compared to Wolfe et al.
1989).

As a result of three-dimensional verifications (consisting of two different load scenarios), the shell element behavior was deemed adequate to predict the load sharing and partial composite action response of the sheathing.

Shear Wall Behavior

In this study, three main variables were addressed in the investigation of the shear wall behavior: (1) anchorage devices, (2) frame and shell behavior, and (3) the nailing schedule of the wall sheathing. The anchor bolt and hold-down stiffness values were determined from previous testing and from manufacturer's data, respectively (Appendix B, Table B-2). Also, the connectivity of the individual wall members was established early in the study – all vertical studs were to be pinned at each end (see Appendix B, Figure B-8). Lastly, the material properties, meshing principles, and general behavior of the shell element were investigated and documented (see Appendix B, Figure B-4 and Table B-4). Thus, the main purpose of the shear wall behavior validation process was to develop a correlation between the in-plane stiffness of the wall system in SAP and the nail spacing at the edges of the OSB sheathing. Appendix E provides a detailed description of the process by which the correlation was developed. Once established, the correlated SAP model of the shear wall was compared to the experimental findings of several different shear wall studies (see Appendix E, Figures E-2 to E-7). It should be noted that the correlation procedure is valid only for the OSB sheathing used in this study. If plywood were used as the wall sheathing material, a separate correlation would be required using the G_a values provided in SDPWS Table A.4.3A (AF&PA 2005b).

Influence Functions

Once the 2D and 3D validations were complete, the final step involved creating a SAP model of the entire building. However, this model of the index

building needed to be validated itself before proceeding. To do perform this validation, analytical influence functions due to uplift were created using SAP and were compared to experimentally determined results developed by researchers at the University of Florida (Datin 2009). Appendix F provides a detailed description of this investigation. Also presented in Appendix F is a comparison of results at four selected locations within the structure (see Figures F-8 and F-9).

The result of this effort showed that the SAP model behaved in agreement with the scaled physical testing. In general, influence functions of *roof-to-wall* connections show that loads applied more than two trusses away from a point of interest have very little effect at that point (Figure F-9, Load cells 2 and 15). Within a distance of two trusses (48 inches), the effect of the load is greater. This finding concurs with the results obtained from truss assembly tests performed at the FPL (Wolfe and McCarthy, 1989). On the other hand, foundation connections showed much less sensitivity to the location of the applied load (see Figure F-9, Load cells 6 and 10). This occurs because the walls of the structure share the load among all anchorage devices, effectively dissipating roof-level load intensities as they make their way to the foundation (see Figure 6, 2-inch nailing). This type of load sharing behavior is indicative of a stiff wall system (i.e. tighter nailing schedule at the edges of the wall sheathing). As noted in the "Results and Discussion" section of this report, the nailing schedule strongly influences the ability of the walls to share roof-level loads. As edge nailing density is reduced (i.e. nails are spaced farther apart), the walls become more flexible and are

therefore less capable of dissipating roof loads evenly at the foundation level – load intensities become more prevalent (see Figure 6, 12-inch nailing).

The few discrepancies that exist between the experimental results and the analytical ones can be attributed to a perceived difference in stiffness of the respective structures. It is believed that the 1/3 scale prototype built at the University of Florida is slightly more rigid than the SAP model due to the choice of sheathing and fasteners that were employed. OSB in the prototype was ¹/₄-inch thick and sheathing nails were represented with ³/₄-inch long, #4 screws spaced at four inches on center. The scaled OSB thickness was selected based on its out-ofplane flexural stiffness under the presumption that this property would be of the most interest for wind loads imparting uplift on the roof structure. Although this notion holds true for the roof sheathing, it does not necessarily apply to the wall sheathing, which is loaded in-plane in an uplift scenario. Further, the #4 screws were selected to ensure that connection failure in uplift would not occur. The researchers at the University of Florida found that the use of 1/3 scale nails led to premature failure by way of nail withdrawal and sheathing pull-through. This connection failure did not provide any insight into load paths or load sharing that would otherwise be observable if the connections remained intact. Thus, the decision was made to ensure that the connections between the sheathing and framing did not fail, and the use of the #4 screws served this purpose (Datin 2009). This choice of fastener, though, is assumed to contribute to the overall higher rigidity of the structure, observable by the minor differences between the influence

function contour plots – especially at the foundation level (see Figure F-9, Load cells 6 and 10).

APPENDIX D

3D VERIFICATION – LOAD SHARING STUDY

The following plots provide a comparison between the present SAP model, the physical testing (Wolfe and McCarthy 1989), and the "sheathing beam" model (Li 1996). Appendix C provides a detailed description of this validation procedure.

LOAD SHARING WHEN TRUSSES ARE LOADED INDIVIDUALLY



Figure D-1. Truss 1 loaded individually.



Figure D-2. Truss 2 loaded individually.



Figure D-3. Truss 3 loaded individually.



Figure D-4. Truss 4 loaded individually.



Figure D-5. Truss 5 loaded individually.



Figure D-6. Truss 6 loaded individually.



Figure D-7. Truss 7 loaded individually.



Figure D-8. Truss 8 loaded individually.



Figure D-9. Truss 9 loaded individually.

DEFLECTION SHARING WHEN TRUSSES ARE LOADED INDIVIDUALLY



Figure D-10. Truss 1 loaded individually.



Figure D-11. Truss 2 loaded individually.



Figure D-12. Truss 3 loaded individually.



Figure D-13. Truss 4 loaded individually.



Figure D-14. Truss 5 loaded individually.



Figure D-15. Truss 6 loaded individually.



Figure D-16. Truss 7 loaded individually.



Figure D-17. Truss 8 loaded individually.



Figure D-18. Truss 9 loaded individually.

LOAD SHARING WHEN FULL ASSEMBLY IS LOADED



Figure D-19. Load sharing when all trusses are loaded simultaneously.

DEFLECTION SHARING WHEN FULL ASSEMBLY IS LOADED



Figure D-20. Deflection sharing when all trusses are loaded simultaneously.

APPENDIX E

CORRELATION TO NAILING SCHEDULE OF SHEATHING

The extent to which edge nailing affects diaphragm or shear wall stiffness is dependent on the presence of blocking. Unblocked systems are relatively unaffected by changes in the edge nailing. As previously noted residential, lightframe *roof* systems fit into this unblocked category. On the other hand, *wall* systems are blocked (assuming the typical practice of placing OSB panels vertically). Therefore, this study focuses on the effect of edge nailing in the *wall* sheathing.

For a given load value, the shear wall deflections predicted by the NDS code were compared with those predicted by SAP (see Figure E-1). Code deflections were computed using the SDPWS 3-term equation (AF&PA 2005b). Generally, deflection in shear walls comes from *four* sources: bending of the framing members, shear within the panel, nail slip between the panel and the framing, and finally anchorage elongation. The SDPWS, a supplement to the NDS code, however provides a simplified 3-term equation for computing shear wall deflection. This 3-term equation combines the effects of panel shear and nail slip into one term by way of a parameter called the apparent shear wall stiffness, G_a. Also, the 3-term equation provides a linear relationship between load and deflection, which is of particular interest in this study since the SAP model assumes linear behavior as well. The SDPWS goes one step further and tabulates G_a values for different nailing schedules, allowing the predicted deflection of a shear wall to

be computed at each possible edge nailing scenario (Table E-1). The goal then is to match the analytical model to each of these computed deflections by iteratively changing the shear modulus, G_{12} , in SAP (Figure E-1). When a value of G_{12} in SAP is found to give the same deflection as predicted by the SDPWS equation, the correlation is complete for that particular nailing schedule. The process is repeated for each possible nailing schedule, resulting in the correlations shown in Table E-2.

 Table E-1. Apparent stiffness for 7/16" OSB and computed deflections using the SDPWS 3-term shear wall equation at a load value of 3,000 lbs.

Edge Nail Spacing	Apparent Stiffness, Ga	Computed Deflection, Δ		
(in)	(kips/in)	(in)		
2	42	0.0867		
3	28	0.1224		
4	22	0.1516		
6	15	0.2152		
12 ¹	7.9^{2}	0.3950		

¹ SDPWS does not provide tabulated values beyond 6-inch edge nailing. This value is for research purposes only.

 2 G_a at 12-inch edge nailing is extrapolated using a power function fit through the other four given values.





SAP2000 Model

Figure E-1. The depicted correlation procedure was repeated for each nail spacing.

Sheathing	Stud Spacing	MOE of Members	Required G ₁₂ in SAP (10 ⁴ psi) for each Edge Nail Spacing (inches)				
	(in)	(10^{6} psi)	2-in	3-in	4-in	6-in	12-in

9.43

6.38

4.86

3.34

1.2 to 1.6

7/16"

OSB

16 or 24

Table E-2. Correlation between nailing schedule and the shear modulus G_{12} of the shell element in SAP.

The correlation procedure eliminated the effects of anchorage by setting this term to zero in the 3-term equation and similarly by fixing the hold-downs in the SAP model. Consequently, the values given in Table E-2 are entirely independent of the anchorage system used, ensuring universal applicability in practice. The correlation did however include the effects of deflection from frame bending in order to investigate the effect of MOE and stud spacing. As it were, these two factors presented no appreciable difference in the results of the correlation, again making the values in Table E-2 universally applied over the ranges shown.

The correlated SAP model was compared to previously published experimental results. Sinha (2007) tested 16 shear walls. 11 of these walls, referred to as "Type A," were sheathed with OSB on one side and gypsum wallboard (GWB) on the other. The remaining 5 walls, referred to as "Type B," were sheathed with OSB alone (i.e. on one side only). Stud spacing for both was 24-in o.c. Lebeda (2002) tested a total of 13 walls under both monotonic and earthquake loading (i.e. CUREE protocol). Of these tests, three were directly comparable to the present study – the three monotonic tests of the control wall. The control wall in this case was constructed using standard practices with studs

1.81

spaced at 16-in o.c. Finally, Langlois (2002) tested 14 shear walls, also under both monotonic and earthquake load scenarios. Again, three tests were directly comparable to the validation of the correlation procedure used in this study – the first three monotonic wall tests. These walls had studs spaced 16-in o.c. Hold-downs and anchor bolts were similar for all walls. Also, all of these shear wall tests employed 4-in edge nailing and 12-in field nailing on the OSB sheathing. Thus, there are a total of 22 walls to which the correlated SAP model can be compared (see plots that follow, Figures E-2 to E-7). It should be noted that the inclusion of the "Type A" walls from Sinah (2007), which were sheathed with a combination of OSB and GWB, did not drastically increase the combined average stiffness (compare Figures E-6 and E-7).

As can be seen in the plots (Figures E-3, E-5 to E-7), the correlated SAP model reasonably predicts the deflection of the shear walls for the load range shown. This load range corresponds to one-third the ultimate capacity of the walls, or the practical linear response realm of the walls. The Type B, OSB-only walls tested by Sinha (2007) display less stiffness than those tested by Lebeda (2002) and Langlois (2002). It is presumed that this difference is attributed to the stud spacing. Sinha's walls used framing members spaced at 24-inches on center, while Lebeda and Langlois employed 16-inch spacing. Conversely, the deflections predicted by the SAP model and by the SDPWS 3-term equation do not depend on stud spacing. Therefore, it is desirable that the correlated SAP model fall within the range of the 16-in and 24-in spacing tests, which it does (Figure E-5 to E-7). Figure E-7 shows

the SAP model to be within 0.07 inches (~1/16-inch) of the combined average at one-third the ultimate capacity, translating into an error of approximately 30% difference. Although not perfect, it should be pointed out that shear wall behavior in this load range is difficult to measure and predict due to minor variations in workmanship such as uneven nail spacing, tightness of bolts, etc. (Doudak 2005). Thus, the resulting correlation method was deemed appropriate for use in this study.

The range of applicability of the resulting correlation should be mentioned. That is, the correlation procedure was performed considering shear walls with a length and height of 8-feet by 8-feet in order to directly compare to published data (Sinha 2007, Lebeda 2002, Langlois 2002). However, the walls in the index building are significantly longer than this, i.e. 30-feet end walls and 40-feet side walls. Since shear wall test data for walls of this length was not readily available, the range of applicability was determined by comparing the predicted deflection using the SDPWS 3-term equation against the deflection output of SAP. Thus, for all walls of the index building, the shear wall deflection was computed at each nailing schedule using the SDPWS method. Then similar length shear walls were modeled in SAP and the deflection at all nailing schedules was determined using the correlated G_{12} values provided in Table E-2. The error observed between the two methods, in terms of predicted deflection, was then plotted as a function of shear wall length (Figure E-8).

The two methods predict the same deflection for the 8-foot length walls since this represents the original correlation. However, it can be seen that the correlated G_{12} values (Table E-2) begin to over predict deflection both as the shear wall length increases and as the edge nail spacing decreases. For the 6-inch edge nail spacing used predominantly throughout this research effort, the error introduced (in terms of predicted deflection) in the 30-ft end walls of the index building is 5%, while 7% is shown in the 40-ft side walls (Figure E-8). To see the effect that this error has on the correlated G₁₂ value, the correlation procedure was repeated using the 6-inch edge nailing for both the 30-ft end wall and the 40-ft side wall. The result of this investigation showed that the 30-ft end wall would require a G_{12} value of 3.51×10^4 psi, compared to 3.34×10^4 psi determined from the original correlation. The percent difference between these two values is 5%, the same error predicted by the deflection comparisons. In the case of the 40-ft side wall, the required G₁₂ value in order to match the predicted SDPWS deflection value is 3.58x10⁴ psi. This value represents a 7% increase over the original correlated value, again in line with what the deflection comparison revealed. In retrospect, it is expected that the two comparisons (deflection and stiffness) should be tied to one another, as they are. This results from the fact that the SAP model is linear, meaning the assembly deflection is inversely proportional to the assembly stiffness. In other words, if it is desired to *decrease* the deflection by 5%, the stiffness must be *increased* by a similar amount.



Figure E-2. OSB-only shear wall tests – 11 total walls.



Figure E-3. Comparison to correlated SAP model at 4-in o.c. edge nailing.



Figure E-4. Average stiffness for each type of wall study.



Figure E-5. Comparison of the correlated SAP model to the individual averages.



Figure E-6. Combined average of the OSB-only walls compared to the correlated SAP model.



Figure E-7. Include the OSB+GWB walls for a total comparison (22 walls).



Figure E-8. Percent error in predicted deflection as the length of the shear wall increases.

APPENDIX F

INFLUENCE FUNCTIONS – CONTOUR PLOTS

Researchers at the University of Florida built a 1/3 scale model of the index building (Figures F-1 to F-5) according to the framing plan shown in Appendix A (Figures A-7 to A-10). The scale model followed proper structural scaling laws and incorporated every last construction detail, including scaled metal plate connectors designed by Gupta et al. (2005). Datin (2009) determined influence functions for twenty load cells: 11 roof-to-wall and 9 wall-to-foundation (Figure F-8). Of these twenty, four were selected for comparison – two roof-to-wall and two foundation connections. The locations for comparison are highlighted in red in Figure F-8. Uplift loads were applied to a dense grid on the roof using a pneumatic actuator (Figure F-6). This grid is represented by the green dots in Figure F-7. Hence, at each location corresponding to a green dot in Figure F-7, the pneumatic actuator lifted up on the roof, and the reactions at all twenty load cells were recorded. Loads were oriented normal to the roof plane and were applied in a stepwise fashion, increasing from 10 lbs to 50 lbs at 10-lb increments. Similar to the conclusion drawn by researchers at the FPL with their nine-truss roof assembly (Wolfe et al., 1989), Datin also noted that load magnitude did not affect the influence function results. Therefore the comparison to the SAP model used only the highest load level of 50 lbs. Influence functions were determined by dividing the measured reaction at the point of interest by the magnitude of the load applied to the structure. This "influence value" was mapped to the location of the applied

load. For example, if the influence function for load cell no. 10 is desired, load would be applied to the first green dot shown in Figure F-7 and the corresponding reaction at load cell no. 10 would be recorded. This reaction would then be divided by the magnitude of the applied load and mapped to the location on the roof where the load was applied (i.e. the green dot). The process is repeated for all green dots, and the resulting contour plot represents the influence function for load cell no. 10. The entire procedure is repeated for all load cells.

Using SAP, a similar process was employed to develop the influence function contour plots for the index building, with a couple of exceptions. First, the load was scaled by a factor of nine (or the square of the length scale) to bring the magnitude of the applied load from a 1/3 scale level to the full-scale level of the SAP model. Thus, the uplift load magnitude was 450 lbs applied normal to the roof surface. In SAP, this force was resolved into its vector components in the horizontal and vertical direction since point loads are not readily applied normal to a sloping surface in SAP. Second, the dense grid that Datin (2009) used (Figure F-7) was modified slightly in SAP for simplification. Instead, a grid was assigned based on 24-inch spacing. Load was iteratively applied, reactions were tabulated, and comparable contour plots were generated (see Figure F-9). A discussion of the results is provided in Appendix C.



Figure F-1. Construction of the 1/3 scale model at the University of Florida (Datin, 2009).



Figure F-2. Construction of the 1/3 scale model at the University of Florida (Datin, 2009).



Figure F-3. Construction of the 1/3 scale model at the University of Florida (Datin, 2009).



Figure F-4. Construction of the 1/3 scale model at the University of Florida (Datin, 2009). Location of load cells is highlighted.



Figure F-5. The completed 1/3 scale model at the University of Florida (Datin, 2009).



Figure F-6. The pneumatic actuator applied uplift loads normal to the roof.



Figure F-7. Uplift loads were applied consecutively at each green dot using a pneumatic actuator.



Figure F-8. Load cell placement in the 1/3 scale wood frame house. Locations of comparison are highlighted in red.


(a) Experimental Influence Functions (1/3 scale model tests)



Figure F-9. Comparison between influence functions determined (a) experimentally and (b) analytically using SAP.

APPENDIX G

UNIFORM UPLIFT PRESSURE LOAD CASES

UNIFORM UPLIFT PRESSURE

With the validation steps complete, the index building was loaded in SAP with a uniform pressure to study the general behavior of the structure over a range of different geometry layouts. The uplift pressure was 50 psf applied normal to the surface of the roof. Output plots (Figures G-1 to G-15) and deflected shapes (Figures G-16 to G-24) are provided in the following figures. Deflections are reported in inches. See the "Results and Discussion" section within the main body of the thesis for further discussion.



Uniform pressure – Standard building 30' x 40'

Figure G-1. Uniform pressure, standard building geometry.

Joint ID (for near side)

Uniform pressure – Effect of nailing schedule





Figure G-2. Uniform pressure, standard building – effect of nailing schedule.



Uniform pressure – Extended building 30' x 92'

Figure G-3. Uniform pressure, extended building.



Uniform pressure – Standard vs. long building

Figure G-4. Comparison between the standard and the extended building.



Uniform pressure – Door in gable wall (end wall)

Figure G-5. Uniform pressure, door in end wall.



Figure G-6. Uniform pressure, door in side wall.



Figure G-7. Uniform pressure, door in side wall – effect of header depth and ceiling.

Uniform pressure – Door in side wall (not centered)



Figure G-8. Uniform pressure, door in side wall – not centered.



Uniform pressure – Doors in center of both walls

Figure G-9. Uniform pressure, doors centered in both walls.



Figure G-10. Uniform pressure, gable wall missing.



Figure G-11. Uniform pressure, blocked vs. unblocked roof assembly.



Figure G-12. Uniform pressure, effect of changing the overhang framing style.

Uniform pressure – Effect of anchor bolt spacing





Figure G-13. Uniform pressure, effect of anchor bolt spacing (4-ft vs. 6-ft).

Uniform pressure – Anchor bolt missing in side wall





Figure G-14. Uniform pressure, effect of an anchor bolt missing in the side wall.

Uniform pressure – Anchor bolt missing in gable wall





Figure G-15. Uniform pressure, effect of an anchor bolt missing in the gable wall.



Figure G-16. Deflected shape – standard geometry (control case). Deflections reported in 10^{-3} inches.



Figure G-17. Deflected shape – extended building. Deflections reported in 10^{-3} inches.



Figure G-18. Deflected shape – door in gable wall. Deflections reported in 10^{-3} inches.



Figure G-19. Deflected shape – door in side wall (centered). Deflections reported in inches.



Figure G-20. Deflected shape – door in side wall (not centered). Deflections reported in inches.



Figure G-21. Deflected shape – doors centered in both walls. Deflections reported in inches.



Figure G-22. Deflected shape – gable wall missing. Deflections reported in 10^{-3} inches.



Figure G-23. Deflected shape – anchor bolt missing in gable wall. Deflections reported in 10^{-3} inches.



Figure G-24. Deflected shape – anchor bolt missing in side wall. Deflections reported in 10^{-3} inches.

APPENDIX H

SIMULATED HURRICANE PRESSURES

BACKGROUND

Datin and Prevatt (2007) subjected a 1:50 scale model of the index building (Figures H-1 and H-2) to simulated hurricane force winds, equivalent to a 3-second gust wind speed of 130 mph. The tests were conducted in the atmospheric boundary layer wind tunnel of the Wind Load Test Facility (WLTF, now called the Wind and Structural Engineering Research Facility) at Clemson University. The model was outfitted with 387 pressure taps installed on its roof, and the testing was conducted at each of the five following wind directions: 0°, 45°, 90°, 135°, and 180°. Pressure was sampled at 300 Hz and recorded for 2 minutes, resulting in 36,000 pressure readings at each tap for each wind direction.



Figure H-1. Wind tunnel arrangement for 1:50 scale model of index building in suburban terrain. (Datin and Prevatt, 2007)



Figure H-2. 1:50 scale model of the index building (left). Dots indicate pressure taps (right). (Datin and Prevatt, 2007)

LOAD CASES

From these pressure time history files, three load cases were selected as input cases for the SAP model in order to observe the response of the structure to the simulated hurricane wind loads:

- 1. Absolute maximum uplift at the corner of the roof
- 2. Local maxima for all pressure taps
- 3. Absolute maximum uplift at the ridge of the roof

Load case 1 represents the time step associated with the maximum uplift pressure experienced at the corner of the roof over all time steps for all wind directions. Therefore, load case 1 signifies the instantaneous moment while the *absolute* maximum uplift is occurring at the corner of the roof. Load case 2 is a compilation of the *local* maxima for all pressure taps at the 45° wind direction, regardless of where they occur in the history file. These maxima are combined and applied to the roof structure simultaneously. Load case 3 is similar to case 1 except the point of interest is at the ridge, not at the corner. Thus, load case 3 represents the time step in the history files associated with the *absolute* maximum uplift pressure observed at the ridge, which also happens to correspond to the 45° wind direction.

MODELING

In order to input the pressures into SAP, a 2-ft by 2-ft grid was superimposed over the actual roof pressure contours. For all three load cases, each individual grid region was assigned the prevalent pressure within its perimeter. Then the roof structure in the SAP model was divided in the same fashion and loaded with the discrete pressures. In this way, the "analog" pressures provided by the wind tunnel tests were "digitized" for input into the analytical model (see Figures H-3 to H-5). The following plots represent the vertical foundation reactions for each wall within the structure. The edge nailing of the wall sheathing is 6-inches on center. Positive values represent uplift, for both applied pressure and observed reactions. A discussion of the results is provided in the "Results and Discussion" section of the main thesis. Figures H-6 to H-11 show the deflected shapes for each load case. Deflections are reported in inches (positive values represent uplift).



Load Case 1 – Maximum pressure at the corner

Figure H-3. Wind tunnel pressures, load case 1.



Figure H-4. Wind tunnel pressures, load case 2.



Load Case 3 – Maximum pressure at ridge

Figure H-5. Wind tunnel pressures, load case 3.



Figure H-6. Deflected shape – load case 1. Deflections reported in 10^{-3} inches.



Figure H-7. Deflected shape – load case 1 (view looking down). Deflections reported in 10^{-3} inches.



Figure H-8. Deflected shape - load case 2. Deflections reported in 10^{-3} inches.



Figure H-9. Deflected shape – load case 2 (view looking down). Deflections reported in 10^{-3} inches.



Figure H-10. Deflected shape – load case 3. Deflections reported in 10^{-3} inches.



Figure H-11. Deflected shape – load case 3 (view looking down). Deflections reported in 10^{-3} inches.

APPENDIX I

ASCE 7-05 LOAD CASES

"C&C" vs. "MWFRS" PRESSURES

ASCE 7-05 provides separate provisions for wind design using loads for either the "Main Wind Force-Resisting System" (MWFRS) or for "Component and Cladding" (C&C) members. C&C loads were created by ASCE to represent peak gusts which occur over small areas as a result of localized funneling and turbulence. The localized loads can cause failures which in turn can affect the overall "Main Wind Force-Resisting System" (MWFRS). Elements of the building which are either "loaded directly by the wind or receive wind loads originating at relatively close locations" are categorized as components and cladding (Douglas and Weeks, 2003). On the other hand, members which make up the MWFRS are considered to be assemblages of major structural elements that provide support and stability for the overall structure. MWFRS members are generally *not* loaded directly by the wind, but rather receive wind loads by way of the components and cladding. Some elements can be identified in both systems (e.g. structural sheathing), making the interpretation of the code intentions complicated. One suggested interpretation is to design these components independently for each load scenario, requiring two separate analyses. However, it has been noted that the C&C load case attempts to address the "worst case" scenario (Douglas and Weeks, 2003). Also, roof sheathing uplift is a primary concern in major wind events, and it is presumed that this failure results from localized effects acting on the roof

structure. Thus, for this project, component and cladding (C&C) pressures were deemed the appropriate choice for determining the loads acting upon the index building.

APPLIED LOADS

"Component and Cladding" (C&C) pressures were computed using the analytical procedure (Method 2, ASCE 7-05). The simplified procedure (ASCE Method 1) was employed as well, but the more comprehensive analytical approach (ASCE Method 2) was selected to ensure accuracy and minimize generalizations. After using both methods and comparing the results side by side, though, it should be noted that the ASCE simplified procedure provides identical wind loads to the analytical approach for the index building used in this study.

The structure was considered to be enclosed, and the maximum uplift pressures were determined by considering the ASCE case of positive internal pressure. For direct comparison to Datin and Prevatt's research (2007), a basic wind speed of 130 mph was used in the calculations. The following assumptions and classifications were made in order to determine the appropriate wind loads for the index building:

•	Exposure category B	[ASCE 7-05, 6.5.6.3]
•	Topographic factor, $K_{zt} = 1.0$	[ASCE 7-05, 6.5.7]
•	Occupancy category II	[ASCE 7-05, Table 1-1]
•	Importance factor, $I = 1.0$	[ASCE 7-05, Table 6-1]

Thus, ASCE 7-05 Figure 6-3 provides net design wind pressures at this basic wind speed for the index building. These applied pressures are given in Table I-1. Roof and overhang pressures represent uplift while wall pressures represent lateral loads. The zones identified are shown in ASCE 7-05 Figure 6-3, and they are also provided here in Figures I-1 and I-2.

Description	Zone	Pressure (psf)
	1	27.8
Roof	2	48.4
	3	71.6
Overhenge	2A	56.7
Overhangs	3A	95.3
Walls	4	31.6
vv alls	5	38.0

 Table I-1.
 Applied pressures using ASCE 7-05 for "Components and Cladding."

LOAD CASES

Three scenarios were considered with the ASCE 7-05 pressures: (1) uplift acting alone, (2) lateral forces acting alone, and (3) a combination of both – lateral in conjunction with uplift. Output plots as well as applied pressure values for the three load cases are provided in the following plots – Figures I-1 to I-5. The deflected shape for each scenario is shown in Figures I-6 to I-8. Deflections are reported in inches.



Figure I-1. ASCE 7-05 pressures, uplift loads acting alone.

ASCE 7-05 Pressures – Lateral loads acting alone



Figure I-2. ASCE 7-05 pressures, lateral loads acting alone.


ASCE 7-05 Pressures – Lateral + Uplift

Figure I-3. ASCE 7-05 pressures, lateral plus uplift loads.



Figure I-4. ASCE 7-05 uplift response compared to the wind tunnel simulations.



Figure I-5. ASCE 7-05 uplift response compared to the wind tunnel simulations.



Figure I-6. Deflected shape - ASCE 7-05 uplift loads acting alone. Deflections reported in 10^{-3} inches.



Figure I-7. Deflected shape - ASCE 7-05 lateral loads acting alone. Deflections reported in inches.



Figure I-8. Deflected shape - ASCE 7-05 lateral + uplift loads. Deflections reported in inches.

APPENDIX J

ROOF SHEATHING UPLIFT CALCULATION

HAND CALCULATION

Assumptions

- 1. 8d common nails are used to secure the $\frac{1}{2}$ -inch thick roof sheathing panels.
 - a. This is *required* by the IBC as a minimum for wood structural panels.
 - b. This is also *recommended* by the APA <u>Roof Sheathing Fastening</u> Schedules for Wind Uplift (APA 2006).
- 2. The specific gravity of the framing members (i.e. truss top chords) is 0.49 or greater.
 - a. This assumption is valid for common species such as Douglas-Fir-Larch (North) and Southern Pine.
- 3. ASCE 7-05 "Component and Cladding" (C&C) pressures are used with a basic wind speed of 130 mph, exposure B, and mean roof height less than 15-ft.
 - a. The maximum uplift pressure is 95 psf (see Appendix I, Table I-1 and Figure I-1).

Computation

.

Multiply the maximum applied C&C pressure by the tributary length of the roof • sheathing:

$$\left(\frac{95 \ lb}{ft^2}\right)(2 \ ft) = 190 \ \frac{lb}{ft} = 15.8 \ \frac{lb}{in}$$
 (refer to Figure J-1)

This represents the amount of force pulling up along the truss line (Figure J-1).

Divide this value by the allowable nail withdrawal value for wind design. •

Allowable nail withdrawal value for wind design ($C_D = 1.6$):

 $W' = 97 \ lbs$ [WFCM, Table 7A]

(See note on the following page regarding this value)

$$\frac{\left(15.8\frac{lb}{in}\right)}{\left(97\frac{lb}{nail}\right)} = 0.163\frac{nails}{in}$$

• Invert this result to determine the number of inches per nail:

$$\frac{1}{0.163 \frac{nails}{in}} = \underbrace{6.13 \quad \frac{inches}{nail}}_{nail}$$

• If the distance between nails is greater than this value, the uplift pressure will exceed the capacity of the nail to prohibit withdrawal. Thus, the provided

field nailing must be 6-inches or less.

• For the conditions shown, 12-inch field nailing is not adequate to prevent sheathing uplift.

Note: The allowable nail withdrawal value, W, can also be determined using the following NDS procedure. The convenience of the WFCM becomes readily apparent in comparing the number of steps that follow to the single tabulated value provided by the WFCM.

8d common nail, D = 0.131" and L = 2.5" [NDS Table L4]

This translates into 2-inches of nail penetration with $\frac{1}{2}$ -inch roof sheathing. Nail withdrawal, $W = 1380 G^{5/2} D$ [NDS Eq.(11.2-3)]

$$W = (1380)(0.49)^{5/2}(0.131) = 30.4 \text{ lbs per inch of penetration}$$
$$W = \left(30.4 \frac{lbs}{in}\right)(2 \text{ inch penetration}) = 60.8 \text{ lbs}$$

Adjusted withdrawal for wind design,

$$W' = \left(60.8 \frac{lb}{nail}\right) (C_D = 1.6) = 97.2$$
 lbs per nail

Compare this value to the one conveniently tabulated by the WFCM, (Table 7A).



Figure J-1. Typical roof sheathing panel with 6-inch edge nailing and 12-inch field nailing. The magnitude of the force per lineal inch is only applicable for the applied pressure (95 psf) given in the example calculation.

ANALYTICAL PREDICTIONS – SHEATHING UPLIFT

SAP provides an output feature which allows the user to display "element joint forces" for the shell element. The user can tabulate these forces for every structural element used in the analytical model (frames, shells, etc.), essentially providing a free-body-diagram for all the individual components. For example, output is provided at the corners of meshed regions for the shell element. Specifically, the roof shell element is subdivided into 2-ft by 2-ft squares, and joint forces are provided at the joints representing corners of these squares. So the user must pick a location of interest and manually combine the joint forces, considering the contribution from each of the surrounding meshing regions. This process is not only laborious, but also ineffective without further study. It is unclear at this time how SAP allocates the internal forces between these components. Efforts to gain insight into this topic were not readily understood using simplified models, so no attempt was made to develop this undertaking further, given the complexity of the model at hand. Further investigation is recommended.

APPENDIX K

LITERATURE REVIEW

ANALYTICAL MODELING

Joint Connectivity

When building an analytical model with a structural analysis program, the designer is immediately confronted with the decision of how to connect the structural members to one another. Since the actual behavior of most joints often lies in the realm somewhere between purely pinned and purely rigid, a simplified model must be proposed unless more advanced, nonlinear semi-rigidity will be incorporated. Mtenga (1991) developed a two-dimensional truss model with semirigid, nonlinear connections at all the joints. Although his model proved to be an accurate predictor of member forces and moments when compared to experimental results, Mtenga concluded that the model might be unnecessarily complicated. Thus, Li (1996) proposed a more simplified approach. Spring elements (i.e. semirigidity) were only used in two locations within the truss, representing (1) the metal-plate-connected joints at the heel and (2) the tension splice in the bottom chord. Also, these spring elements were linear. All other connections were assumed to be pinned. Li's model showed good agreement with experimental results. More importantly, Li verified his model not only against 2D truss studies, but he also represented entire truss assemblies and successfully captured the 3D response. Dung (1999) used similar truss joint connections as Li, although with a

different computer program. He also modeled a three-dimensional truss assembly and, likewise, achieved satisfactory results. Limkatanyoo (2003) offered yet an even more practical approach to modeling the joints in the truss. He considered all the joints to be either rigid or pinned and consequently eliminated the need for nonlinear, semi-rigid connections altogether. He compared his two-dimensional SAP model to an industry-accepted program called VIEW, widely used by truss designers, and checked both deflections and stresses (using the combined stress index, CSI). Limkatanyoo concluded that the simplified design analog was in agreement with the more complicated, semi-rigid models employed by the truss industry (i.e. the results from VIEW).

Modeling of Sheathing and Fastening

Sheathing offers two primary structural benefits: load sharing and composite action. Load sharing involves the redistribution of forces between individual members within an assembly. No two members are identical in their material makeup, connections are similarly variable, and differential deflections resulting from installation can exist (i.e. uneven surfaces). Thus, there are inherent differences in stiffness between individual members and their assemblages. Sheathing lends itself to distributing load away from limber members toward stiffer ones. Composite action, on the other hand, involves the increase in bending capacity that is afforded to a framing member when a panel product such as sheathing is fastened to it. That is, the framing member and a portion of the sheathing behave like a "T-beam", and their combined stiffness is greater than if separate. Composite action is sometimes referred to as "partial" composite action to address the fact that there is some amount of slip in the nailed connection between the two elements. Partial composite action and load sharing are separate mechanisms independently affecting the structural response of a building in which sheathing is present. Consequently, the goal of any analytical model is to address both of these effects when incorporating sheathing products into the model.

Many previous studies have used beam elements to model the behavior of sheathing. For example, Li (1996) used three beam elements per side of his ninetruss roof assembly to represent the roof sheathing. These "beams" were assigned a thickness and MOE of actual plywood, and their width corresponded to the tributary length of the sheathing as measured along the top chord. To incorporate composite action, Li increased the moment of inertia of the truss top chords. Li's model was found to be in good agreement with experimental results. Dung (1999) and Limkatanyoo (2003) also used frame elements in a similar fashion to simulate roof sheathing. However, partial composite action was not included in these studies. Limkatanyoo noted that a plate element would be "more suitable and appropriate" to represent sheathing and he suggested that "future research should focus on finding a way to model sheathing panels."

To this end, some researchers have explored the use of "area" or "shell" elements built in to modern structural analysis programs such as SAP. Doudak (2005) represented OSB wall sheathing in 2D shear wall tests, using the shell element with elastic orthotropic material properties. In fact, some of the material properties defined in Doudak's research were employed in the present research effort (see Table 3). Doudak fastened the sheathing to the framing members by way of nonlinear "link" elements which exhibited strength degradation as the connection approached failure. This modeling procedure required the use of individual link elements to model each and every fastener in the shear wall. Although accurate, such "meticulous detailing" – as Doudak noted – can be quite laborious to implement. Results from the analytical model were compared to similar full-scale tests, and it was found that the two were in good agreement. In particular, it was noted that the analytical model predicted the ultimate capacity quite well, while the prediction of initial stiffness was not as accurate in some cases. The latter was attributed to "shake down" effects such as variations in workmanship (e.g. nail spacing, alignment, tightness of bolts, etc.) which are "notoriously difficult to measure and predict."

The goal of Doudak's 2D shear wall model was to verify its behavior for incorporation into a much more complex 3D model subjected to lateral loads. The full-scale structure that Doudak tried to represent contained various types of openings such as windows, multiple standard doors, and one large overhead door. The construction materials used in the full-scale building were different than what he validated in his shear wall model, however. For example, plywood and gypsum wallboard were present in the full-scale building while Doudak verified his shear wall models using OSB alone. As a result, some material properties were approximated based on generic values. Nonetheless, the predictions of the finite element model proved to be within 20% error, with the primary source of this error relating to uncertainty about the true material properties. All in all, the research concluded that the model is "capable of predicting the distribution of the applied load through the house structure with reasonable accuracy."

Zisis (2006) also made use of SAP's shell element while investigating wind effects on low-rise wooden buildings. In this study, the area of interest was "environmental" wind loads as opposed to extreme wind events. Thus, the maximum wind speeds were relatively low, resulting in correspondingly low load levels. With such small forces being considered, Zisis opted for a simplified modeling approach that excluded "plate" behavior (out-of-plane bending). He instead chose to use only the shell's membrane behavior, which accounts for inplane forces alone. As a result, the sheathing response in this research was limited to in-plane behavior. Also, isotropic material properties were assigned to the membrane, which may not necessarily capture the orthotropic nature of plywood and OSB sheathing. Finally, Zisis noted that a linear model was used; however, no additional details were provided. Thus, in transitioning to a linear model – by eliminating the nonlinear "link" element of Doudak's effort – it is unclear how Zisis modeled the connection between the shell element and the framing members (i.e. sheathing nails).

LOAD SHARING AND SYSTEM EFFECTS

The Forest Products Laboratory (FPL), based in Madison, Wisconsin, has pioneered several full-scale load sharing studies in the past 30 years. McCutcheon (1977) initiated the effort with an investigation into the reduced deflection observed among floor joists when a layer of sheathing is fastened to them, known today as partial composite action. Seven floors were constructed and tested, including effects such as glue versus nailed connections and tongue-and-groove joints versus standard 1/8-inch gaps at panel edges. McCutcheon developed a computational method to quantify the extent of composite action based on composite beam theory and load-slip characteristics. Then he compared this approach to the full-scale experimental tests with good agreement. In general, he showed that there is an interaction between the sheathing and the joists which tends to increase the stiffness of the floor system as a whole, but the two components do not act as if they were rigidly connected together. Instead, they display "partial" composite action resulting from the sheathing fasteners and the panel edges.

Wolfe and McCarthy (1989) then investigated load sharing within an assembly of roof trusses. In this experiment, researchers represented a 16-foot section from the middle of a conventional gable style roof in order to quantify the effects of load sharing within the assembly. The trusses were built with members from one of three MOE categories, resulting in a nine-truss assembly comprised of three trusses from each stiffness category – low, medium, and high. Trusses were tested individually outside the assembly and then again within the assembly. In the assembly, the variable stiffness trusses were located randomly within the assembly to accentuate the effect of load sharing. The researchers found that stiffer trusses carry a greater share of the load and truss deflections were far less in the assembly

than outside. Likewise, failure loads were higher in the assembly than individual trusses displayed alone outside the assembly. Wolfe and LaBissoniere (1991) continued this research by testing two more truss assemblies. In this effort, however, the trusses were not sorted into different MOE categories. Instead, the construction was meant to be "representative of conventional truss fabrication practice," noting that the previous study (Wolfe and McCarthy, 1989) was more of an "extreme condition" in terms of truss stiffness variation. Thus, the trusses were all relatively the same stiffness. In addition, no bias toward failure in the wood members was given. Wolfe and McCarthy had previously used heavier connector plates at critical joints to encourage failure in the wood rather than in the connections. Also, Wolfe and LaBissonierre constructed their roofs with a gable end. Results showed that 40-70% of the load that is applied to an individual truss is distributed to adjacent unloaded trusses by the plywood sheathing. They also noted that composite action between the sheathing and the framing members diminished as loads approached the capacity of the assembly. In this range, joint slip caused a reduction in the composite action. This is in stark contrast to load sharing, which the researchers showed increased as load approaches assembly capacity. Finally, the study showed that load sharing increased the assembly load capacity 13-49% (compared to individually loaded trusses outside the assembly), leading to presumption that the repetitive member factor of 1.15 (15% increase) used in the NDS code is conservative. Moreover, this repetitive member factor does not address the mechanism that distributes load around areas of local weakness (i.e.

load sharing). Instead, it applies only to individual members like rafters rather than to the allowable load on the truss as a whole.

Additional load sharing and system interaction studies were also conducted by LaFave and Itani (1992), Percival and Comus (1980) and many others. A detailed literature review of these studies is presented by Gupta (2005) and is, therefore, not included here.