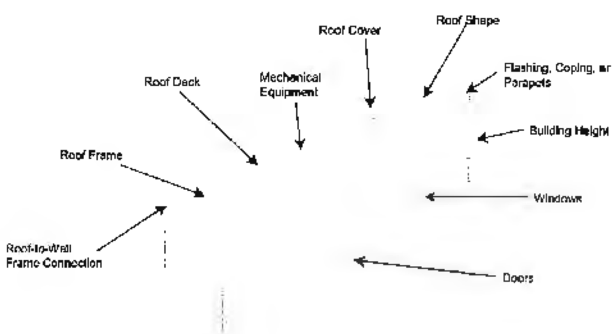
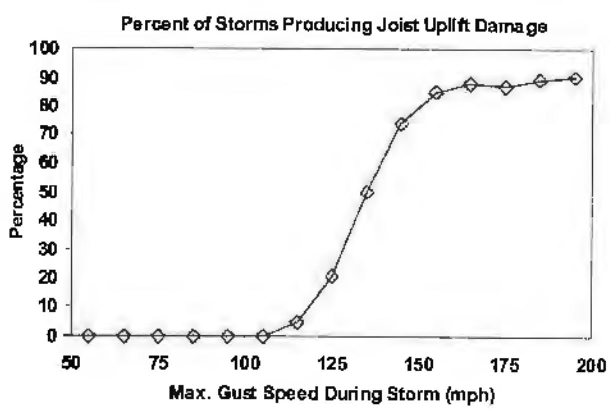
INTRARISK
Applied Research Associates, Inc.

**Version 1.3**
**August 2002**

**Florida Department of Community Affairs**

# Development of Loss Relativities for Wind Resistive Features for Residential Buildings with Five or More Units


**Prepared for:**


**Florida Department of Community Affairs**
**2555 Shumard Oak Blvd.**
**Tallahassee, Florida 32399**


**Prepared by:**


**Applied Research Associates, Inc.**
**IntraRisk Division**
**8540 Colonnade Center Drive, Suite 307**
**Raleigh, North Carolina 27615**



**DCA Contract: 02-RC-11-14-00-22-026**

**ARA Project: 1078**

## PREFACE

*(Version 1.3)*

The Florida Department of Community Affairs contracted with Applied Research Associates, Inc. to evaluate the effectiveness of wind resistance features in reducing hurricane damage and loss to residential buildings with five or more units. The project was begun in April 2002 and completed in June 2002. The scope of the project has dealt with both existing construction and new construction built to the new Florida Building Code 2001. The Florida Building Code (FBC) became effective on March 1, 2002.

The scope of this study was limited to multi-family residential buildings. An earlier project, "Development of Loss Relativities of Wind Resistive Features of Residential Structures", analyzed the loss reductions associated with wind mitigation features for single-family residences in Florida.

The DCA, DOI, and ARA make no representations on the possible interpretations in the use of this document by any insurance company. The use of information in this document is left solely to the discretion of each insurance company.

Comments on this document should be sent to:


Mr. Keith Delhomme
Florida Department of Community Affairs
2555 Shumard Oak Boulevard
Tallahassee, FL 32399-2100
Ph: (850) 488-8466; Fax: (850) 410-1582
Email: keith.delhomme@dca.state.fl.us

Mr. Howard Eagelfeld
Florida Department of Insurance
200 E Gaines Street, Larson Building
Tallahassee, FL 32399-0326
Ph: (850) 413-5319; Fax: (850) 992-3865
Email: eagelfeldh@doi.state.fl.us

Dr. Lawrence Twisdale
Applied Research Associates, Inc.
8540 Colonnade Center Drive, Ste 307
Raleigh, NC 27615
Ph: (919) 582-3336; Fax: (919) 582-3401
Email: ltwisdale@ara.com


Distribution of this document is handled by the Florida Department of Community Affairs. A pdf version of the document is available on the DCA website for downloading. A printed copy can also be obtained.

# EXECUTIVE SUMMARY

A project has been conducted to estimate the effects of wind resistive building features in reducing hurricane damage and loss for residential occupancies in condominium and tenant buildings with five or more units. The scope of this project has included both new construction to the Florida Building Code 2001 and existing construction.

The basic approach used in this study to develop the loss relativities is identical to the methodology used in a companion report for single-family houses, "Development of Loss Relativities for Wind Resistive Features of Residential Structures." This methodology has involved the analyses of individually modeled buildings at numerous locations in Florida. Each building has been modeled with a specific set of wind resistive features. The features considered in this project include: roof shape, roof covering, secondary water resistance, roof-to-wall connection, roof deck material/attachment, opening protection, and wall construction.

Due to differences in construction method and applicable building codes and standards, condominium and tenant buildings have been divided into three categories based on building height and frame construction. Group I Buildings include one to three story buildings with wood or masonry walls. These buildings typically have a wood truss roof system with plywood sheathing. Group II Buildings are 60 ft tall or less and have steel, concrete, or reinforced masonry frames. They generally have steel or concrete roof decks, although wood decks may also be used. Group III Buildings include all buildings over 60 ft tall.

The FBC 2001 specifies design wind speed, wind-borne debris region design options, and definitions of Terrain Category. In the wind-borne debris region, designs for both enclosed and partially enclosed structures have been evaluated, per the FBC and ASCE 7-98.

The loss relativities in this report are based on total loss (structure, contents, and loss of use). The possible allocations of loss to building owner, tenant, condominium association, and condominium unit owner have not been evaluated. The relativities based on total loss provide a reasonable and simple approach for this initial study.

The loss cost relativities for existing construction are developed in the form of a set of tables for each Building Group. Two main tables are provided, one set for Terrain B and one set for Terrain C. Each of these tables are normalized to a "central" building, which is a representative building as opposed to the weakest building. The relativity for the central building is one, with the relativities for stronger buildings less than one and weaker buildings more than one. The basic relativity tables were all computed for 2% deductible. The Terrain B results are primarily for inland locations and the Terrain C results are primarily for barrier islands and locations within 1500 feet of the coastline. The range of loss relativity is comparable to the previous study when comparisons to hip and gable roof shapes are made. The ranges of loss relativity are slightly larger in this study since flat roof shapes are included. It was also observed that the difference between hip and gable roofs is less than the single family residential study because the hip "structure" is a smaller percentage of roof framing for condominium buildings.

For new construction to the Florida Building Code (FBC), the loss relativities have been computed and reduced to three sets of tables for minimal design loads. The loss relativities for minimal design construction to the FBC show strong loss reductions over the typical building in the existing construction tables. Separate entries are included for the High Velocity Hurricane Zone (HVHZ), which corresponds to Broward and Miami-Dade Counties. In the HVHZ, opening protection is required for all new construction. Because new construction may be designed for higher loads than the FBC 2001 minimums, a separate table of adjustments is provided for these cases. In addition, this table can also be used for new buildings that are later mitigated beyond the code minimums.

The analysis results for new construction clearly indicate that the Florida Building Code 2001 will improve the design and construction of new buildings in the state.

Further improvement and refinement of the work performed in this project may lead to improved estimates of relativities in the future. The report discusses areas where more data is needed as well as building features that have not been explicitly modeled.

# TABLE OF CONTENTS

## TABLE OF CONTENTS (Continued)

## TABLE OF CONTENTS (Continued)

# 1.0 INTRODUCTION

## 1.1 Objective

Florida Statute 627.0629 reads, in part, as follows:

> *A rate filing for residential property insurance must include actuarially reasonable discounts, credits, or other rate differentials, or appropriate reductions in deductibles, for properties on which fixtures or construction techniques demonstrated to reduce the amount of loss in a windstorm have been installed or implemented. The fixtures or construction techniques shall include, but not be limited to, fixtures or construction techniques which enhance roof strength, roof covering performance, roof-to-wall strength, wall-to-floor-to-foundation strength, opening protection and window, door, and skylight strength. Credits, discounts, or other rate differentials for fixtures and construction techniques which meet the minimum requirements of the Florida Building Code must be included in the rate filing. ...*

The purpose of this study is to produce a public domain document that provides data and information on the estimated reduction in loss for wind resistive building features for condominium and renter occupancies in buildings with five or more units.

A previous study, "Development of Loss Relativities for Wind Resistive Features of Residential Structures," has focused on estimating loss relativities for single-family residential occupancies. This report makes reference to the single-family report and the user will need a copy of that report for additional discussion.

## 1.2 Scope

The scope of this study must include, as a minimum, the wind resistive features called out in the statute, namely:

1. Enhanced Roof Strength
   a. Roof deck connection-to-roof framing
   b. Roof deck material and strength
2. Roof Covering Performance
3. Roof-to-Wall Strength
4. Wall-to-Floor-to-Foundation Strength
   a. Wall-to-floor strength
   b. Floor-to-foundation strength
5. Opening Protection
   a. Windows
   b. Doors
   c. Skylights

In addition, the study addresses some other features that have been demonstrated to reduce the amount of loss in windstorms.

The scope of this study is limited to multi-family residential occupancies in buildings with five or more units. The scope of this project includes both existing and new construction to the Florida Building Code, 2001 (FBC).

This project uses hurricanes as the windstorm to produce the loss relativities. Hurricanes dominate the severe wind climate in Florida and, hence, are the primary contributors to windstorm loss costs.

The features for which discounts are provided must be practically verifiable so insurers can be reasonably confident a particular building qualifies for the discounts.

Due to schedule and budget limitations, the scope of work does not include analysis of the building stock distribution for existing construction.

## 1.3 Technical Approach and Limitations

The basic approach used herein to estimate how loss costs change with wind resistive fixtures and construction techniques relies primarily on engineering models and loss analysis for individual buildings. The buildings are modeled with and without specific wind resistive fixtures. These buildings are then analyzed for hurricane damage and loss using Applied Research Associates, Inc.'s, HURLOSS methodology. The HURLOSS methodology has been reviewed and accepted by the Florida Commission on Hurricane Loss Projection Methodology. The public domain documents on HURLOSS are available from the Commission. In addition, this report provides further information on the model and its validation. Technical papers are also referenced.

An advantage of the individual building modeling approach used for this study is that it is based on a detailed engineering model that replicates how engineers design and analyze real structures. A similar approach has been adopted by the Federal Emergency Management Agency (FEMA) in the development of a National Wind Loss Estimation Methodology. The engineering load and resistance modeling methodology used in this approach has been reviewed by the Wind Committee of the National Institute for Building Science. This committee includes national experts in wind engineering and meteorology.

The estimation of losses for buildings with specific engineering details is an emerging technology and has many limitations. The treatment of uncertainties and randomness in the hurricane wind field, wind boundary layer, the built environment, building loads, resistances, and loss adjustment are an important part of the modeling process. The data sources include: historical data, wind tunnel test information, building code information, post-hurricane damage surveys, laboratory tests, full-scale tests, insurance claim folders, and insurance company portfolio exposure and loss data.

Judgments are used to supplement this modeling process. The HURLOSS computed relativities have been compressed using a judgment factor. The resulting loss relativities, while reasonable estimates at this time, are likely to evolve with more data and further model improvements. There is clearly room for refinement and improvement and a strong need for more data.

## 1.4 Florida Building Code

The State of Florida first mandated statewide building codes during the 1970s, requiring local jurisdictions to adopt one of the model codes. The damage produced by Hurricane Andrew and other disasters in the 1990s revealed fundamental building code weaknesses and also that building code adoption and enforcement was inconsistent throughout the state. The state has attempted to respond to this situation by reforming the state building construction system with emphasis on uniformity and accountability. The Florida Building Code (FBC) is the central piece of the new building code system. The single statewide code is developed and maintained by the Florida Building Commission.

The FBC supersedes all local codes and is automatically effective on the date established by state law. The new building code system requires building code education requirements for all licensees and uniform procedures and quality control in a product approval system.

The FBC is compiled in four volumes: Building, Plumbing, Mechanical, and Fuel Gas.

The National Electrical Code© is adopted by reference. The scope of this project has been limited to wind resistive construction features, which are in the Building volume.

Section 4 and Appendix B provide additional discussion on specific requirements of the FBC with respect to wind mitigation features.

## 1.5 State-of-the-Art in the Classification of Buildings for Wind

The commonly used insurance construction classes are based on the ISO classes, which were originally developed primarily for fire risk classification. The ratings with respect to masonry, semi-wind resistive and superior frame, while capturing some of the differences in the performance of the main structural system with respect to wind loads, do not address the key causes of wind damage and loss associated with roof covering, window and door performance, roof deck, roof-to-wall performance, and building aerodynamics. These ISO classes are still commonly used by the insurance industry, but it is widely recognized that these classes are not ideal for wind ratings.

Several developments have taken place in the past few years that focus on an emerging fundamental change in the classification of buildings for wind damage and loss.

First, FEMA has begun the development of a national wind loss estimation methodology. This methodology includes the development of a detailed classification system for buildings based on the wind damage and loss characteristics. While this work is not publicly available at this time, the initial version will be published in early 2003.

Second, the Residential Construction Mitigation Program (RCMP) initiated by the state of Florida in 1997, has provided unique information on single-family building construction features, mitigation options and costs for existing buildings, and the expected loss reduction benefits from mitigation. Detailed inspections were performed for over 2,000 houses in selected coastal counties in Florida between 1998-2000. The resulting data provides a unique source of information to help characterize the current building stock in the state.

Third, the Florida Windstorm Underwriting Association (FWUA) recognized the need for wind-based insurance classes and in 1998-1999 developed a first generation Class Plan aimed at classifying buildings by their wind risk characteristics rather than the ISO fire based characteristics. The FWUA Class Plan has been in effect since July 2000 and residential occupancies (single-family and 1-4 unit occupancy/buildings) are being rated according to the construction features in their Class Plan. The loss relativities in their Class Plan were based on actuarial judgment coupled with model calculations of the type used in this study.

The FWUA residential rating factors for renter contents and condominium units were based on only two variables: opening protection and wall construction. These factors were based simply on actuarial judgment with an eye toward the results of the single-family building analysis. Clearly, the FWUA classification for condominium and renters did not represent a state-of-the-art classification.

The classification produced in this project provides the first basic step in the rating of residential construction for buildings with multi-family occupancies. There are many potential complexities for these types of buildings and this study will limit the number of building and insurance parameters considered.

## 1.6 Review of Building Features that Influence Hurricane Damage and Loss

For many years, engineers have focused on the structural frame and load-path issues in designing buildings for wind loads. However, beginning in the 1970's, engineers began to document the importance of the building envelope (roof deck and covering, roof-to-wall connection, windows, doors, etc.) performance in influencing the resulting financial loss experienced by buildings in windstorms. In many storms, the building frame performed adequately, but the windows and/or doors failed, often due to impact by wind-borne debris. Roof covering was almost always damaged, resulting in water penetration into the building, particularly for hurricanes.

Damage and the ensuing losses to buildings were found to be governed by the performance of the building envelope, including many non-engineered components, such as roof covering, windows and doors, roof deck, etc. The key structural frame connection for most failures was the roof-to-wall connection. Foundation failures and frame failures, other than the roof-to-wall frame connection, were found to be extremely rare for multi-unit buildings. Proximity of missile sources is also important for large buildings with glazed openings extending up the full height of the structure.

These observations stand in sharp contrast to earthquake induced damage to buildings, which is governed primarily by the building foundation and building frame performance.

The wind induced damage and the ensuing losses for multi-family structures are governed by the performance of the building envelope. Figure 1-1 shows the important wind resistant features of a typical multi-family residential building. The failure of the non-engineered components such as the roof cover, roof deck, roof-wall connections, windows and doors is responsible for most of the losses. As in single-family homes, the most likely frame failure is at the roof-wall connection.

**Roof Cover and Roof Shape.** The performance of the roof cover in a hurricane is a key to the performance of large buildings. Once the roof cover fails, water begins to enter the building damaging interior drywall, electrical and mechanical systems, floor covering and contents. For flat roof shapes, which are common for multi-story buildings, the performance of the roof cover is critical due to the potential for significant water leakage into the building.

The performance of shingles and tiles on low rise multi-family buildings is similar to that seen for single-family buildings. Figure 1-2 illustrates partial loss of roof cover on a condominium building in Hurricane Erin.

A large number of multi-family dwellings are constructed with single flat roofs with a single ply membrane, modified bitumen or built-up roof. Figures 1-3 through 1-5 show some examples of failed single ply membrane roofs. In addition to being attached to the roof with mechanical attachments or being adhered to the roof, membranes can be held to the roof using ballast. The ballast is usually gravel or paving stones. Ballasted roofs make up a relatively small percent of the population of flat roofs.

Figure 1-6 shows an example of the interior damage caused by the loss of a single ply membrane roof. In most cases, the failure of a built up roof, a single ply membrane roof or a modified bitumen roof initiates when the flashing at the edges of the building fails. The building shown in Fig. 1-5 could not be occupied for more than a year following the failure of the roof cover.

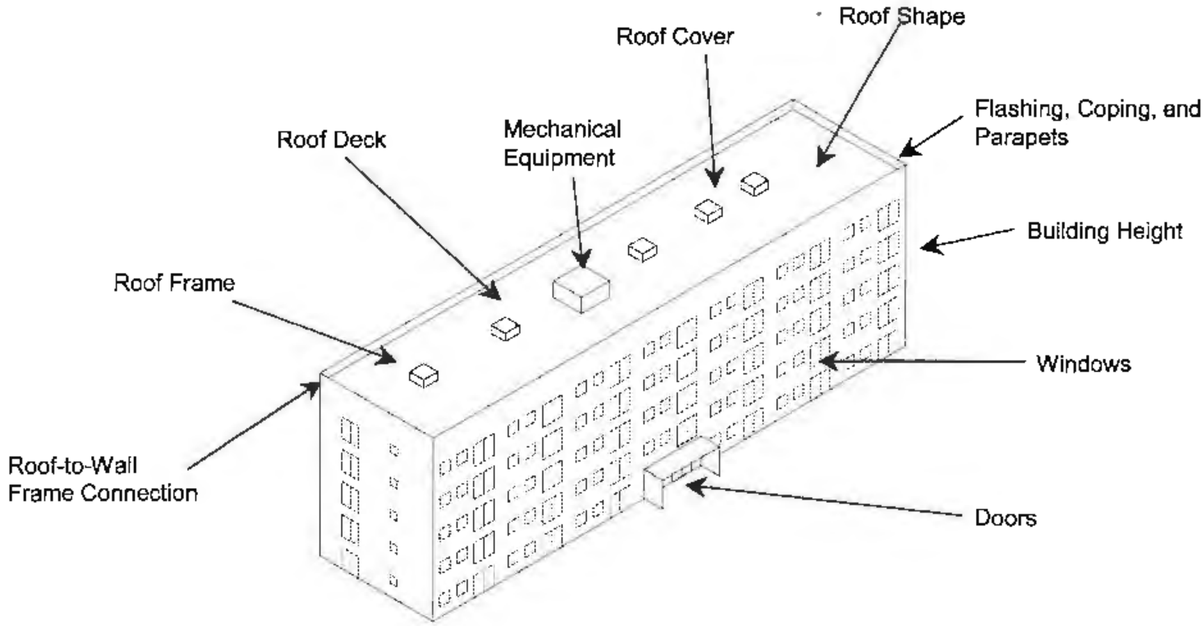



**Figure 1-1. Features that Control Hurricane Damage and Loss for Buildings with 5 or More Units**

**Figure 1-2. Shingle Damage to Condominium Building from Hurricane Erin**

**Figure 1-3. Failure of Single Ply Membrane Roof Cover for 3 Story, Flat Roof Condominium Building**

**Figure 1-4. Failure of Single Ply Membrane Roof Cover for 4 Story, Flat Roof, Concrete Frame Condominium Building**

**Figure 1-5. Failure of Single Ply Membrane Roof Cover for 4 Story, Flat Roof Condominium Building**

**Figure 1-6. Interior Damage to Condominium Caused by Water from Failed Roof Membrane**

Figure 1-7 shows an example of the failed flashing at the edge of a modified bitumen roof. Failure of Single Ply Membrane roofs on relatively low buildings can also be initiated by tearing of the membrane caused by the impact of flying debris and by tearing caused by roof top equipment becoming dislodged in high winds. On buildings with Single Ply Membrane Roofs, BUR or Modified Bitumen roofs, once the initial failure begins, a progressive failure generally follows, with the roof cover peeling away from the roof deck. This progressive failure mechanism often results in large areas of the roof deck being exposed to the rain. The large areas of exposed roof deck associated with the failure of flat roof covers is clearly evident in the photographs presented in Figures 1-3 through 1-5.

**Performance of Roof Deck.** As in single-family construction, the performance of the roof deck is critical to the overall performance of the entire building in a hurricane. Once a portion of the roof deck fails, significant quantities of water begin to enter the building causing rapid and extensive damage to the interior of the building and its contents. Figure 1-8 shows an example of a complex of multi-family dwellings built with a flat wood roof, which experienced extensive roof deck damage. Figure 1-9 presents an example of a gable roof building which experienced minor roof deck damage. Figure 1-10 presents an example of roof deck damage to a three story building, having a roof constructed from a combination of mono-slopes and gables.

**Roof-Top Equipment.** Flat-roofed buildings often have air conditioning and other equipment on the roof deck. The tie-down connections and water proofing details around this equipment are important to the roof cover and roof deck performance (see Fig. 1-11). Figure 1-12 illustrates the failure of poorly attached AC units on a condominium building.

**Figure 1-7. Example of Edge Flashing Failure on a Modified Bitumen Roof**

**Figure 1-8. Extensive Roof Deck Damage to a Multi-Family Building**

**Figure 1-9. Minor Roof Deck Damage to a Multi-Family Building**

**Figure 1-10. Example of Roof Sheathing Damage Experienced by a Three Story Multi-Family Building**

**Figure 1-11. Multi-Level Flat Roof with Numerous Equipment and Architectural Frame Penetrations**

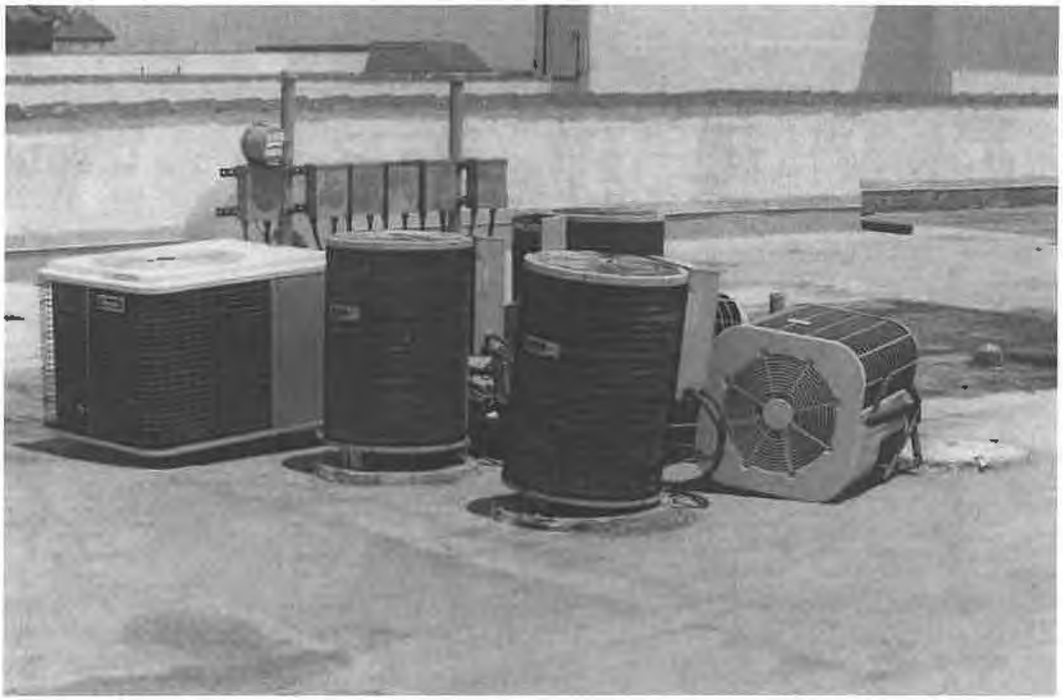


**Figure 1-12. Example of Failed Roof-Top AC Units on High Rise Condominium Tower**

**Roof-to-Wall Connections.** The failure of roof-to-wall connections results in enormous damage to a building, in most cases causing a loss that approaches the full insured value of the building and its contents. In the case of wood roofs, the construction characteristics of multi-family buildings are often the same as those used in single-family construction. Such characteristics include the use of toe-nail connections to connect the roof truss to the wall. Figure 1-13 shows an example of a roof-wall connection failure on a two story condominium unit that occurred during hurricane Erin in 1995.

For buildings with steel roofs, the roof is usually constructed using open web steel joists, with a welded connection to the wall frame. Open web steel joist roof systems generally fail under wind loads either through buckling of the lower chord of the joist or through an uplift failure of the welded connection attaching the joist to the wall.

**Opening Failures.** Opening failures in multi-family buildings occur often in hurricanes. The failures result from a combination of breakage associated with being impacted by wind-borne debris as well as pressure induced failures, either inward or outward. Once a window has failed, damage to the interior of the building is caused through the introduction of wind and water into the building. Figure 1-14 shows windows on the corners of a high rise condominium tower that failed due to the action of wind pressures during Hurricane Opal in 1995. Figure 1-15 presents examples of windows that failed due to wind pressures on a three story condominium building. Sliding glass door failures are also common (see Fig, 1-16). Improved designs of the sliding glass door framing can dramatically improve performance (see Fig. 1-17) to both wind and water.

**Figure 1-13. Example of Roof-Wall Connection Failure**

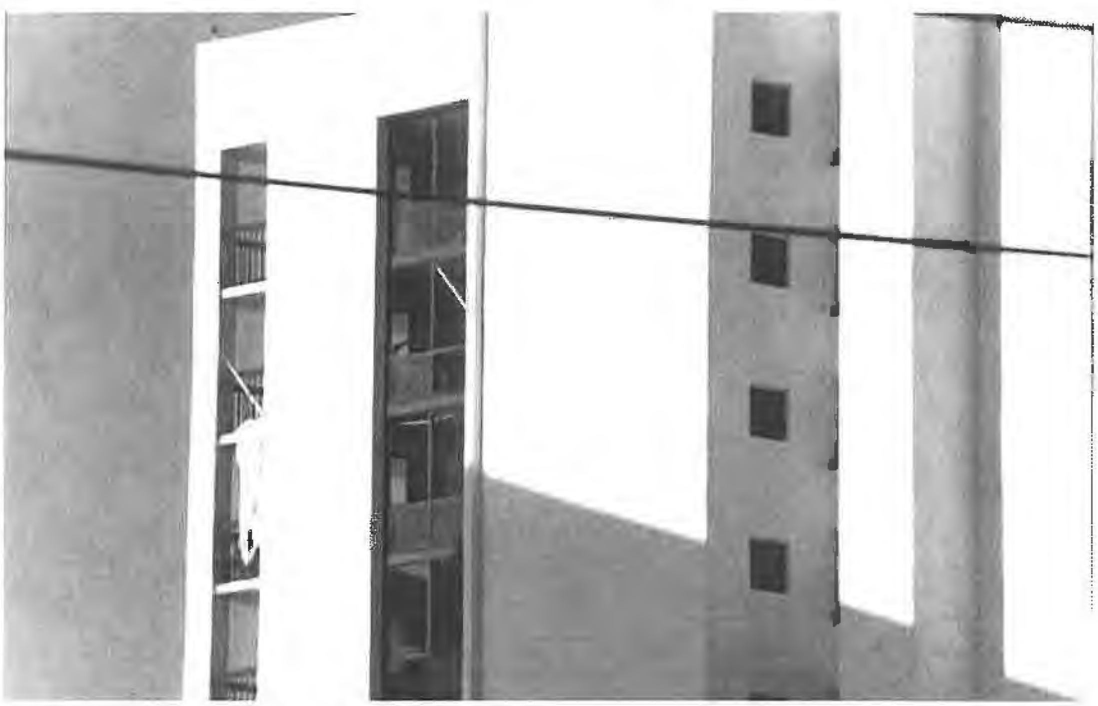


**Figure 1-14. Window Failures on High Rise Condominium Tower**

**Figure 1-15. Window Failures on Three Story Condominium**

**Figure 1-16. Sliding Glass Door Failure in Condominium Building**

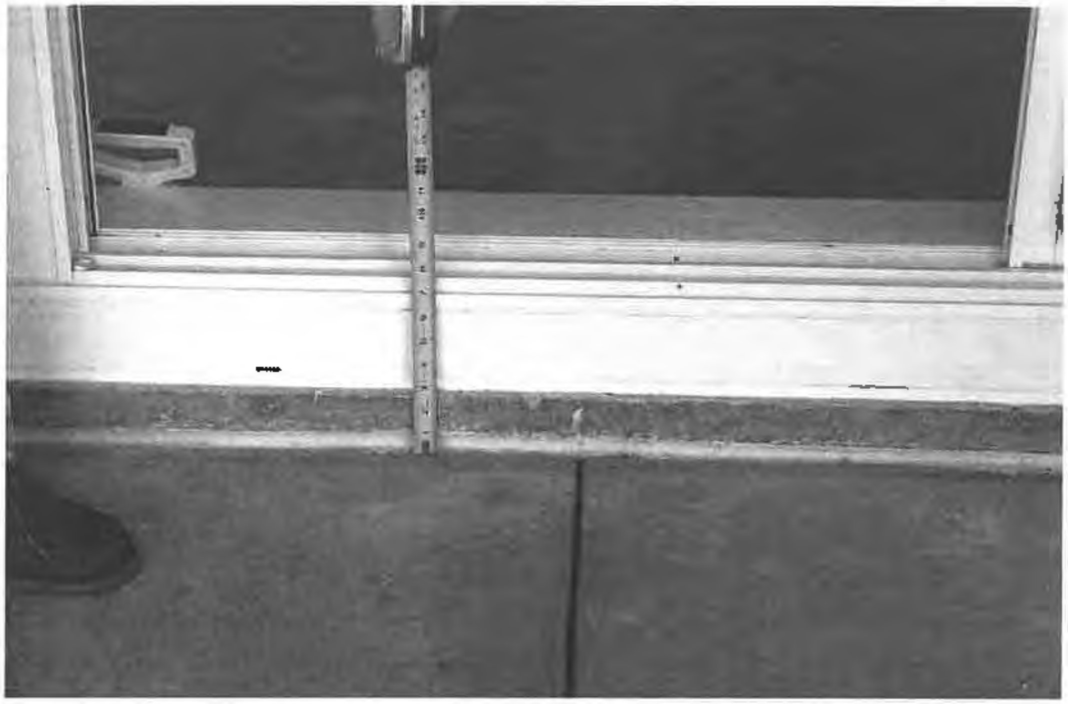


**Figure 1-17. Step-Over Installation of Sliding Glass Door and Strengthened Frame**

**Foundation Failures.** In the case of multi-family dwellings, foundation failures, for practical purposes can be ignored, since few, if any buildings, are built such that they do not have adequate restraints. Foundation failures that have occurred in past hurricanes have almost always been associated with the action of storm surge and waves and not the wind loads. However, the connections for buildings on piers need to be adequately designed and periodically inspected for corrosion.

**Building Envelope.** The building envelope governs the losses for condominium and renter occupancies. These types of buildings, particularly flat roof structures, often exhibit a higher sensitivity to envelope performance than do single-family structures. The multi-unit occupancies of condominium or tenant buildings mean that failure of the roof can affect losses in many units that have experienced no exterior window or door failures.

As with single-family residences, the loads on the building increase dramatically once the envelope fails. The failure of openings on the top floor can lead to significant increases in the loads acting on the underside of the roof and the loads acting on other windows, with the result being that these components are overloaded because of the action of internal pressure, and consequently have a greater chance of failure.

## 1.7 Organization of Report

The organization of this report closely follows the single-family report, "Development of Loss Relativities for Wind Resistive Features of Residential Structures," (Version 2.2, March 2002). The methodology, discussed in Section 2, is similar to that used previously. Sections 3 and 4 discuss the loss relativity results for existing and new construction. A brief summary is presented in Section 5. Appendices are included with additional background information.

# 2.0 METHODOLOGY

## 2.1 Approach

The fundamental approach used herein to develop the loss relativities is to analyze individually-modeled buildings at numerous locations in Florida. Each building is modeled with a specific set of wind-resistive features. The HURLOSS methodology has been used to analyze each modeled building for damage and loss.

The loss costs are estimated for a basic set of insurance parameters: Coverage A (building), C (contents), and D (additional living expenses) limits and deductible. This process is repeated for a large combinatorial set of wind-resistive features for a number of Florida locations (latitude-longitude points).

For each location, the loss relativities are produced by dividing by the loss costs for a selected "typical" building. Therefore, the relativities at each location are simply normalized fractions that provide a measure of the differences in loss based on wind resistive features.

The approach used in this study is to develop loss relativities for existing construction (non-FBC 2001) and new construction (FBC 2001) separately. This separation recognizes the changes brought about by the new code and the fact that the methods used to verify the construction features may be different for existing and new construction. However, for practical reasons, we use a common set of locations in Florida (as described in Section 2.3) to analyze the separate loss relativities for existing and new construction.

As illustrated by the figures in Section 1.4, many key wind features focus on the roof details and openings. Verification of the presence or absence of wind resistive features for existing construction, therefore, cannot be practically accomplished without an "inspection". In the absence of an "inspection", there is no reasonably accurate way to classify an existing building for purposes of providing loss mitigation credits or discounts.

For new construction, the FBC (Section 1606.1.7) requires that the drawings for new construction summarize key design information. This information should be useful for insurance rating purposes. In addition, insurers may wish to or need to perform an inspection of the building or require documentation from the builder.

## 2.2 Florida Building Code Wind Regions, Terrains, and Design Options

Figure 2-1 illustrates the wind speed map for the Florida Building Code (FBC 2001, Figure 1606). The wind speed contours start at 100 mph and go to 150 mph.[1] For buildings located between contours, interpolation is allowable for design. In the absence of interpolation between contours, the building will be designed to the higher of the wind speed contours.

### 2.2.1 Wind-Borne Debris Region

The FBC introduces a Wind-Borne Debris Region where all openings that are not protected with shutters or impact resistant glass are considered to be open. This means a designer has the option of designing the structure as an enclosed building or as a partially enclosed building where the design assumes that wind pressure entering the building adds to the load on the structure.

[1] It is possible that some engineers could interpolate to slightly less than 100 mph in the region inside the 100 mph contour since ASCE 7-98 allows interpolation between basic wind contours.

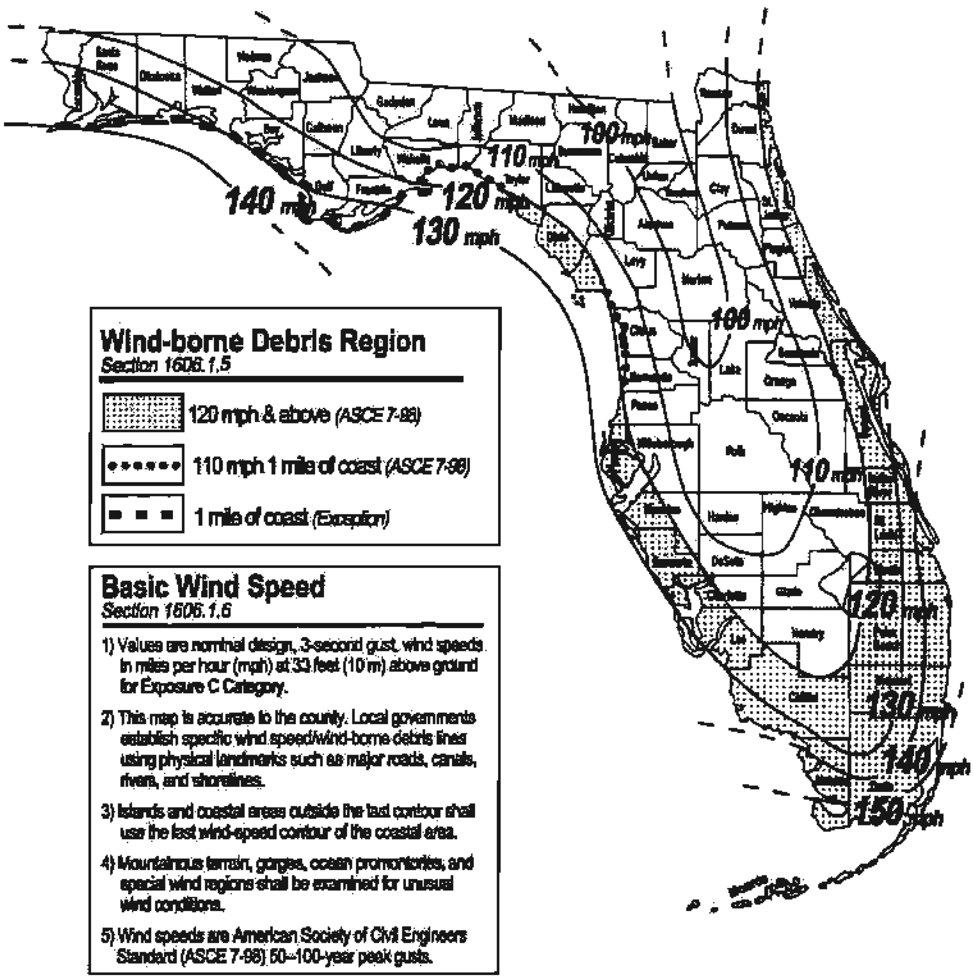



FIGURE 1606
STATE OF FLORIDA
WIND-BORNE DEBRIS REGION & BASIC WIND SPEED

**Figure 2-1. Wind Regions in Florida Building Code**

The Wind-Borne Debris Region (FBC, Section 1606.1.5) includes all areas where the basic wind speed is 120 mph or greater (shaded area of Fig. 2-1) except from the eastern border of Franklin County to the Florida-Alabama line where the region only includes areas within 1 mile of the coast. It also includes areas within 1 mile of the coast where the basic wind speed is 110 mph or greater (see Fig. 2-1).

### 2.2.2 Terrain Exposure Category

The Florida Building Code has adopted the Exposure Category (terrain) definitions of ASCE 7-98 with a few important exceptions (see FBC, Sections 1606.1.8 and 1619.3):

1. Exposure C (open terrain with scattered obstructions) applies to: All locations in HVHZ (Miami-Dade and Broward Counties)
   - Barrier islands as defined per s.161.55(5), Florida Statues, as the land area from the seasonal high water line to a line 5000 ft landward from the Coastal Construction Control line.

- All other areas within 1,500 ft of the coastal construction control line, or within 1,500 ft of the mean high tide line, whichever is less.

2. Exposure B (urban, suburban, and wooded areas) practically applies to all other locations in Florida by virtue of the definitions for Exposures A and D.

Hence, new construction in the state will fall into Exposures B and C. The following paragraphs attempt to provide more background on this important topic as it relates to wind-resistance construction and insurance ratings for buildings.

The effect of terrain (i.e. the reduction in wind speed near the ground produced by the frictional effects of buildings and vegetation) has a significant impact on wind speeds and, hence, wind-induced damage and loss. The magnitude of the reduction of the wind speed at any height is a function of the size and density of the obstructions (buildings, trees, etc) on the ground, as well as the fetch (distance) the wind has blown over a given terrain. The importance of terrain is recognized in most national and international wind loading codes through the use of simplified terrain categories defined, for example, as open terrain, suburban terrain, urban terrain, etc. When designing a building, a design engineer must first determine what terrain a building is going to be built in, and design the building to resist the associated wind loads. In ASCE 7-98, the national wind loading standard, there is a significant increase in the design loads associated with designing a building located in open terrain (Exposure C) compared to the case of a building designed for suburban terrain conditions (Exposure B). For example, the design loads for the cladding (windows, doors, roof sheathing, etc.) of a 15 foot tall building located in Exposure C are 21% more than those for a building located in Exposure B, and for a 25 foot tall building the difference in the design loads is 34%. The true effect of terrain is in most cases greater than that indicated in the building codes which tend to conservatively underestimate the reduction in wind load that is experienced for most buildings located in suburban terrain.

All damage and loss calculations carried out in this study were performed using terrain models representative of typical terrain Exposure "B" and Exposure "C" conditions.

### 2.2.3 High Velocity Hurricane Zone

The FBC identifies a High Velocity Hurricane Zone (HVHZ) for Miami-Dade and Broward Counties (FBC, Sections 202 and 1611ff). This portion of the Florida code comes from the South Florida Building Code (SFBC). The HVHZ has some important differences with the non-HVHZ areas of the FBC, including:

1. More stringent missile impact test criteria.
2. Requirement that all doors and non-glazed openings have missile protection.
3. Does not allow for partially enclosed building design as an alternative to the opening protection requirement.
4. Some restrictions on materials that can be used.
5. Design for Terrain Exposure C conditions.

These requirements make for improved wind resistance for buildings built in the HVHZ.

### 2.2.4 Design Options

There are few prescriptive design methods allowable for residential occupancies in buildings with five or more units. The FBC allows the use of SBCCI SSTD10 for basic windspeed of 130 mph or less in Exposure B and 110 mph or less in Exposure C. Other prescriptive options are not allowable for buildings with five or more units.

The use of SSTD-10 is limited to wood frame buildings of two stories and less, concrete and masonry buildings of three stories and less, for hip and gable roofs only. The use of SSTD-10 is also limited to buildings less than 60 feet wide. Because of these limitations, coupled with the fact that the previous study for single-family houses showed little difference in the relativities between SSTD-10 houses and FBC houses, we did not model SSTD-10 buildings in this study.

All buildings greater than 60 ft high must be designed by ASCE 7-98. ASCE 7-98 designs include enclosed and partially enclosed options. In the wind-borne debris region, enclosed designs will have all glazed openings protected for debris impact.

Table 2-1 summarizes the design cases for new construction in the Florida Building Code. A "3" in a cell indicates a viable FBC design option for that wind speed. The "3" corresponds to the 3 building height groups (see Section 2.4.2). The terrain exposure category was determined by reviewing the FBC definitions for terrain exposure and wind-borne debris regions. As previously discussed, the FBC allows for enclosed building design without impact protection for wind speeds greater than 120 mph in the Panhandle (since the FBC limits the wind-borne debris region in that area to within 1 mile of the coastal mean high water line).

A key objective of this project is to determine how loss costs vary for the design options for new construction shown in Table 2-1. An important point is that these designs are for the code minimum loads. Some condominium and tenant buildings will be designed for higher wind speeds than dictated by the code. Hence, a practical matrix for new construction needs to be expanded beyond the minimal load design. These issues are addressed in Section 4.

**Table 2-1. FBC Minimum Load Design Cases for New Construction (No consideration of topographic speedups)**

| | | FBC: (ASCE 7-98) | | | |
|---|---|---|---|---|---|
| Wind Speed | Terrain Exposure | ASCE 7 Enclosed (non-WBDR)[2] | ASCE 7 Enclosed (WBDR) | ASCE 7 Partially Enclosed (WBDR) | FBC-HVHZ (SFBC) |
| 100 | B[1] | 3 | | | |
| 110 | B[1] | 3 | | | |
| 120 | B | 3 | 3 | 3 | |
| | C | | 3 | 3 | |
| 130 | B | 3 | 3 | 3 | |
| | C | | 3 | 3 | |
| 140 | B | 3 | 3 | 3 | |
| | C | | 3 | 3 | |
| 150 | B | | 3 | 3 | |
| | C | | 3 | 3 | |
| HVHZ-140[3] | C | | | | 3 |
| HVHZ-146[4] | C | | | | 3 |
| Totals[5] | | 15 | 24 | 24 | 6 |

1 Based on the FBC definitions of Exp C, which is limited to barrier islands and within 1500 ft of the coast, there is no design Exp C for these wind zones

2 For 120, 130 and 140 mph wind speeds in the Panhandle, the FBC limits the Wind-borne Debris Region (WBDR) to 1 mile from coast.

3 This corresponds to Broward County.

4 This corresponds to Miami-Dade County.

5 Topographic speedups are not considered in the project because Florida has relatively few locations that qualify per ASCE 7-98.

## 2.3 Locations for Loss Relativity Analysis

Table 2-1 shows that there are 12 combinations of wind speed and terrain exposure that result from the Florida Building Code. For consistency with the previous study, "Development of Loss Relativities for Wind Resistive Features of Residential Structures," we use the same locations for the analysis of losses for new and existing construction. Figure 2-2 shows the selected points. Since we are normalizing the results at each location by the computed loss costs at that location, the consideration of multiple locations serves to test how the relativities may vary by region within the state. The reason for locating multiple points on a contour is to see if the loss relativities vary much for that contour.[1]

Once the locations are specified, the relevant new construction building design options (Table 2-1) are located at each point. In addition, the modeled buildings for existing construction are also analyzed at each point.

For simplicity, we will use these same locations to develop the loss relativities for existing construction. That is, the locations in Fig. 2-2 are used in the analysis in Section 3.

The location of points on each contour are shown in Fig. 2-2a. For each point, the number denotes the wind speed and the letter denotes the terrain. Points with terrain Exposure C are located within 1500 ft of the coastline. Points not within 1500 ft of the coastline are terrain Exposure B, except for those in the HVHZ zone, per the special definitions in the Florida Building Code. Figure 2-2b shows the towns (or geographic feature) where the points are located, or the nearest town. Using the town names to denote point locations is simply a way to label the points and does not necessarily imply that the town is exactly on that contour.

Table 2-2 summarizes the 31 points used to define the locations. Note that 9 of the locations are not on a contour. Two each for HVHZ 140 (Broward) and HVHZ 146 ( Miami Dade). The design wind speed in these counties is constant over the entire county. The other five points are not on contours. These locations are identified in the comment column in Table 2-2. One of the added points is for 120 mph and the other three are all for the 150 mph wind speed. Since the 150 mph wind speed contour only crosses Florida in the Everglades, we felt it was more appropriate to locate the points on buildable land. This is also consistent with our understanding that there will be no required FBC designs to wind speeds greater than 150 mph.

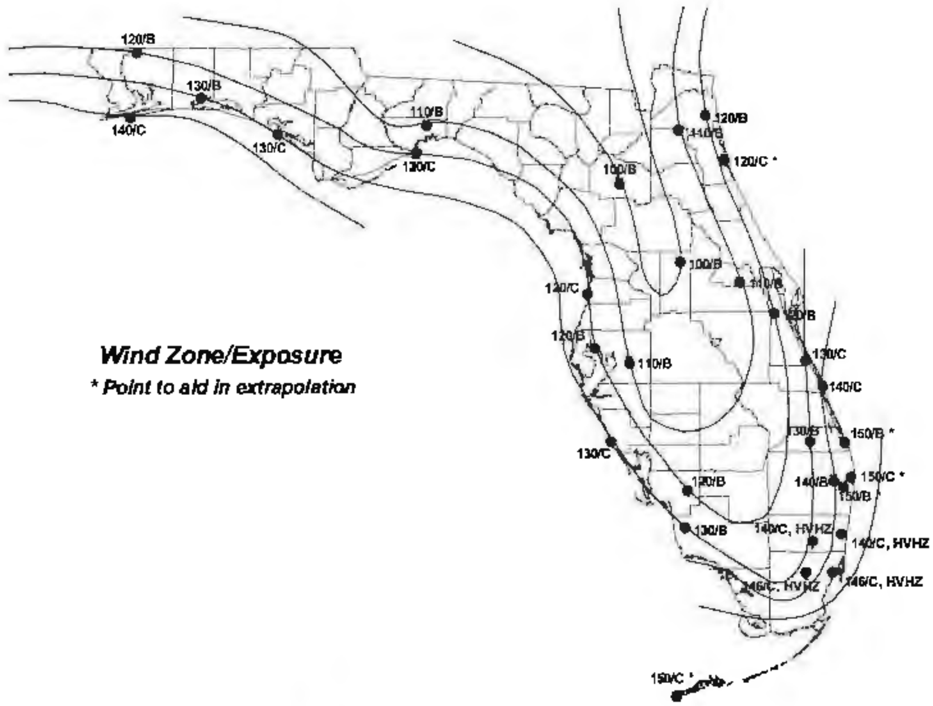
## 2.4 HURLOSS Model

ARA's HURLOSS model is summarized in the public domain submittal to the Florida Commission on Hurricane Loss Projection Methodology (FCHLPM). The model was approved by the Commission for the 1999, 2000, and 2001 standards. The model is used in this study to produce loss costs relativities. Loss costs are not reported in this study since each insurer must perform those calculations for its book of business. The relativities produced herein show how loss costs are expected to vary according to wind resistive features and FBC design options.

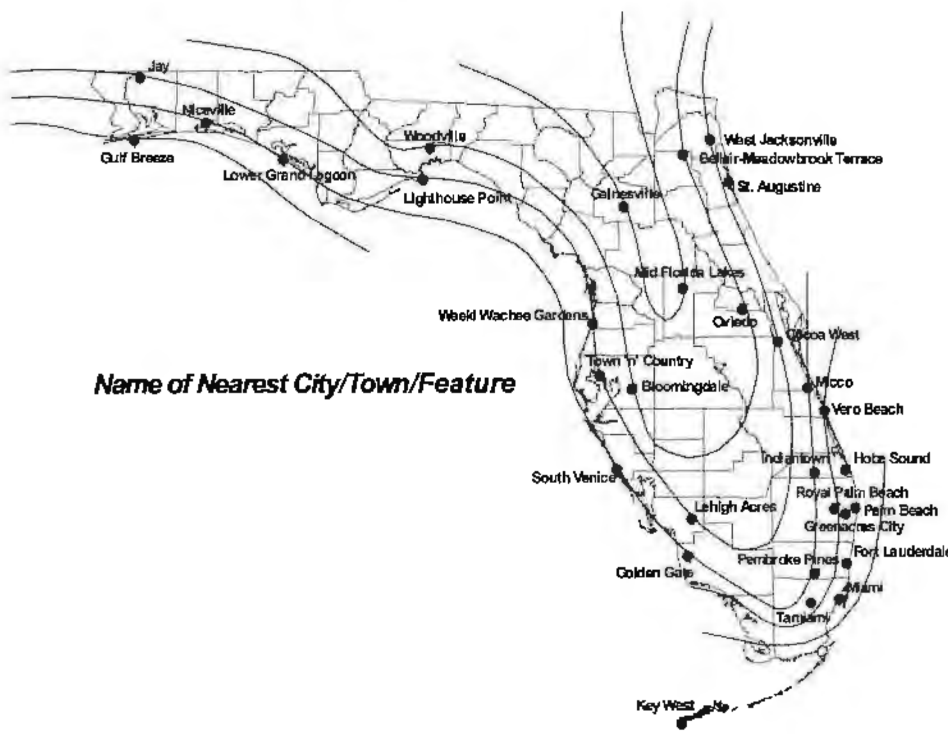
### 2.4.1 Simulated Hurricane Wind Climate

For this study, we simulated 300,000 years of hurricanes in the Atlantic Basin and retained all storms that strike Florida. This large number of years was chosen to ensure statistical convergence of loss costs, recognizing that in some cases the difference in modeled buildings could be a change in a single variable out of many variables. Loss

[1] From ASCE 7-98, the contours represent the hurricane winds corresponding to a 500 year return period divided by the square root of the load factor. The contours essentially represent 50-100 year return period wind speeds, with the actual return period determined by the slope of the hurricane wind speed exceedance probability curve for that location.




a. Points Identified by Contour and Terrain Exposure




b. Nearest Towns or Geographic Features for Point Locations

**Figure 2-2. Map of Location Points for Loss Relativity Analysis**

**Table 2-2. Location Points and Lat-Long Coordinates**

| ID | Wind Contour | Exposure | Place | WBDR | Comment | Label | Latitude (deg) (X_Coord) | Longitude (deg) (Y_Coord) |
|---|---|---|---|---|---|---|---|---|
| 1 | 100 | B | Gainesville | N | | 100/B | -82.35078 | 29.66851 |
| 2 | 100 | B | Mid Florida Lakes | N | | 100/B | -81.75630 | 28.86330 |
| 3 | 110 | B | Woodville | N | | 110/B | -84.26329 | 30.24175 |
| 4 | 110 | B | Bellair-Meadowbrook Terrace | N | | 110/B | -81.75189 | 30.17602 |
| 5 | 110 | B | Oviedo | N | | 110/B | -81.15279 | 28.66395 |
| 6 | 110 | B | Bloomingdale | N | | 110/B | -82.26102 | 27.87761 |
| 7 | 120 | B | Jay | N | | 120/B | -87.14942 | 30.95997 |
| 8 | 120 | B | West Jacksonville | Y | | 120/B | -81.50699 | 30.32542 |
| 9 | 120 | B | Cocoa West | Y | | 120/B | -80.82584 | 28.34633 |
| 10 | 120 | B | Lehigh Acres | Y | | 120/B | -81.66613 | 26.57927 |
| 11 | 120 | B | Town 'n' Country | Y | | 120/B | -82.59261 | 28.00821 |
| 12 | 120 | C | Lighthouse Point | Y | Also analyzed as Terrain B | 120/C | -84.33933 | 29.93707 |
| 13 | 120 | C | Weeki Wachee Gardens | Y | | 120/C | -82.66236 | 28.52765 |
| 14 | 120 | C | St. Augustine | Y | Added point, not on contour | 120/C * | -81.31077 | 29.89192 |
| 15 | 130 | B | Niceville | N | Also analyzed as Terrain C | 130/B | -86.50246 | 30.50508 |
| 16 | 130 | B | Indiantown | Y | | 130/B | -80.46272 | 27.03545 |
| 17 | 130 | B | Golden Gate | Y | | 130/B | -81.68795 | 26.20149 |
| 18 | 130 | C | Lower Grand Lagoon | Y | Also analyzed as Terrain B | 130/C | -85.73581 | 30.12823 |
| 19 | 130 | C | Micco | Y | | 130/C | -80.51389 | 27.87154 |
| 20 | 130 | C | South Venice | Y | | 130/C | -82.40817 | 27.04785 |
| 21 | 140 | B | Royal Palm Beach | Y | | 140/B | -80.23009 | 26.70591 |
| 22 | 140 | C | Gulf Breeze | Y | Also analyzed as Terrain B | 140/C | -87.20833 | 30.32189 |
| 23 | 140 | C | Vero Beach | Y | | 140/C | -80.35962 | 27.64502 |
| 24 | 150 | B | Hobe Sound | Y | Added point, not on contour | 150/B * | -80.13952 | 27.07265 |
| 25 | 150 | B | Greenacres City | Y | Added point, not on contour | 150/B * | -80.13989 | 26.62995 |
| 26 | 150 | C | Palm Beach | Y | Added point, not on contour | 150/C * | -80.03816 | 26.69286 |
| 27 | 150 | C | Key West | Y | Added point, not on contour | 150/C * | -81.77521 | 24.56286 |
| 28 | 140 | C | Fort Lauderdale | Y | HVHZ: Broward | 140/C, HVHZ | -80.13958 | 26.14289 |
| 29 | 140 | C | Inland Broward County | Y | HVHZ: Broward | 140/C, HVHZ | -80.44245 | 26.05956 |
| 30 | 146 | C | Miami | Y | HVHZ: Miami-Dade | 146/C, HVHZ | -80.21093 | 25.77570 |
| 31 | 146 | C | Inland Miami Dade County | Y | HVHZ: Miami-Dade | 146/C, HVHZ | -80.47958 | 25.75599 |

costs are driven by the intense storms and 300,000 years produces a sufficient number of intense hurricanes for loss costs convergence.

Figure 2-3 shows several resulting wind speed plots produced from the simulation. Peak gust open-terrain wind speeds are plotted versus return period for four locations: Jay, Miami, Bloomingdale, and Gainesville.

Note that these are open-terrain peak gust 10 m (above ground) wind speeds and are not sustained wind speeds. Also, for typical suburban terrain, the 10 m wind speeds will be notably less.

The simulated wind speed exceedance probabilities are compared to the ASCE 7-98 wind speeds in Fig. 2-4. The small differences are due to the following:

1. The current simulations are based on a larger historical data set, including hurricanes for 1995-2000.
2. The simulations in this study use 300,000 years versus the 20,000 years used for development of the ASCE 7-98 wind speed map.
3. Enhancements to the model since 1995.

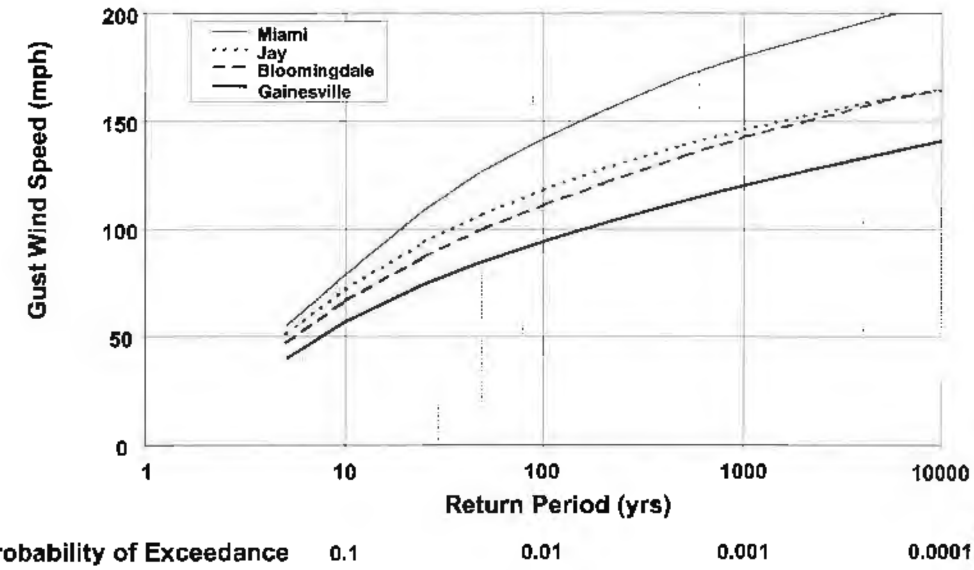



**Figure 2-3. Open-Terrain Peak Gust 10 m Wind Speed Plots**

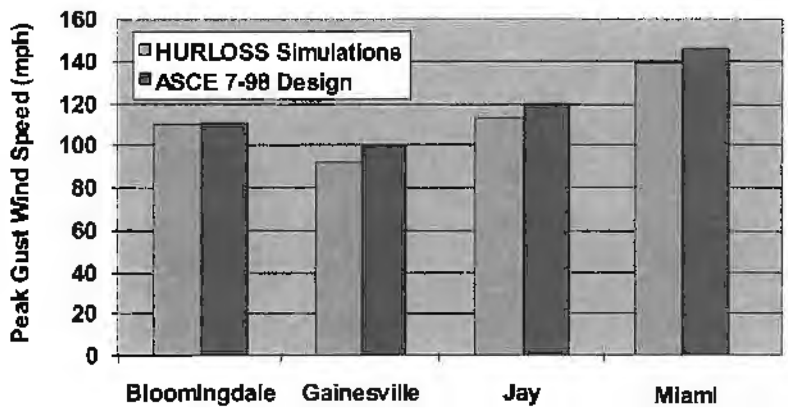



**Figure 2-4. Comparisons of Simulated Wind Speeds and ASCE 7-98 Wind Speeds for Comparable Return Periods**

Nevertheless, the comparisons indicate that the current HURLOSS hurricane model produces similar wind speeds when compared to the national design standards for locations in Florida.

### 2.4.2 Modeled Buildings

The discussion in this subsection is primarily pertinent to existing construction other than those built to the FBC 2001 code or equivalent, requirements of which have been discussed in detail in Section 2.2. Buildings modeled or "built" to the FBC 2001 or equivalent, termed "new construction" in this project, use the same geometry and building height grouping as for existing construction described below, while their wind resistive features are designed according to the more stringent FBC 2001 or equivalent requirements, as detailed in Appendix A.

In this study, three main building groups are considered. The first group of buildings is the lowrise, "marginally engineered" buildings, typically one to three stories in height, constructed with either wood or masonry walls and having a wood truss roof system and plywood sheathing. The construction characteristics of the Group I Buildings are similar to that of single family residential buildings.

The second group of buildings consists of multi-story buildings less than sixty feet tall, with the structure designed by an engineer, usually using the requirements given in the Standard Building Code (SBC). These buildings are usually constructed from steel or concrete, with either steel or concrete roof decks, although in some instances wood decks are used.

The third group of buildings consists of buildings over sixty feet in height. This group exists, since for buildings over sixty feet tall, the design of the building and its components is usually performed using ASCE 7 instead of SBC. The use of ASCE-7 instead of SBC results in a design which must withstand higher wind loads.

Table 2-3 summarizes these three building height groups. This categorization by building height is by predominant construction and design methods that have existed in the building codes. The user should use the appropriate group based on actual construction method. For example, a four story wood frame building should be classified using the parameters in Group I (although Group I typically applies to 1-3 story buildings). All buildings over 60 feet in height should be classified by Group III regardless of the typical construction. The Model Building Height column in Table 2-3 refers to the actual height of the modeled buildings described in the following paragraphs. For each of the three groups of buildings described above, 3-D CAD models have been developed.

**Table 2-3. Building Construction Groups for Condominium and Tenant Buildings**

| Group | Typical Wall Construction | Typical Heights | Model Building Height |
|---|---|---|---|
| I | Masonry or Wood Frame | 1-3 story | 2 stories |
| II | Steel or Concrete Frame or Reinforced Masonry | ≤60 feet | 5 stories |
| III | Steel or Concrete Frame | >60 feet | 8 stories |

***Group I Buildings.*** Models of Group I Buildings have been developed that consist of two geometries, denoted small and large, and 3 roof shapes (hip, gable and flat). The introduction of the flat roof case yields a feature of buildings not considered in the residential loss relativity study, where flat and gable roof buildings were grouped into one class and modeled as a gable. Eliminating the flat roof building in the case of single family residential buildings was reasonable since relatively few single family homes are built with flat roofs. In the case of multi-family units, flat roofs are common, and, hence, are modeled in this study. The model building geometries are shown in Figure 2-5.

The primary characteristics of the Group I lowrise condominium/rental units are described in Table 2-4. As in the single family residential building relativity study, the roof deck attachments used in the past, are largely governed by the prescriptive requirements of the earlier codes rather than by the performance requirements. A total number of Group I building types generated through the combination of parameters presented in Table 2-4 is 1152. The sloped roof buildings are modeled as having shingle roofs, whereas the flat roof buildings are modeled with a built-up roof.

**a. Small Building Flat Roof**

**a. Small Building Gable Roof**

**c. Small Building Hip Roof**

**d. Large Building Flat Roof**

**e. Large Building Gable Roof**

**f. Large Building Hip Roof**

**Figure 2-5. Group I Building Geometries**

**Table 2-4. Existing Construction Classification Variables for Group I Buildings**

| Variable | Categories | General Description |
|---|---|---|
| Building Size | 2 | Small (8 units), Large (16 units) |
| Roof Shape | 3 | Hip, Flat, Gable |
| Roof Construction | 1 | Wood |
| Roof Covering | 2 | FBC Equivalent, Non-FBC Equivalent |
| Secondary Water Protection | 2 | No, Yes |
| Roof-to-Wall Connection | 4 | Toe Nail, Clip, Wrap, Double Wrap |
| Roof Deck Material/Attachment | 3 | Plywood/OSB (3 nail size/spacings), |
| Openings: Protection Level | 2 | None, SFBC/SSTD 12/ASTM E 1996 |
| Secondary Water Resistance | 2 | No, Yes |
| Surrounding Terrain | 2 | FBC Terrain B, FBC Terrain C |

***Group II and III Buildings.*** The characteristics of the group II and III buildings are described in Tables 2-5 and 2-6 with their geometries depicted in Figure 2-6. There are a total of 320 combinations of parameters for the Group II buildings (secondary water resistance is not needed for the concrete roof deck case). The existing Group II buildings have been "designed" for two different code eras, one corresponding to the 1976 SBC and the other corresponding to the 1988 SBC. In the case of the Group III buildings, there are a total of 168 combinations of building parameters (again no SWR is used on the concrete roof deck) and two design codes (1976 SBC and ASCE 7-88).

The ASCE 7-88 designs were performed using the ASCE 7-88 wind loading provisions, which have gone essentially unchanged during the period 1982 through 1995. We have also analyzed the case of a Group III building being designed to meet the provisions of the 1976 SBC, which at the time allowed the use of SBC wind loads for buildings over 60 feet tall. Note that in the case of the ASCE 7-88 designs, we have considered all three of the design terrains which exist in the state of Florida. Terrain is not treated in the 1976 SBC.

### 2.4.3 Modeling Approach to Compute Building Damage and Insured Loss

The HURLOSS model is used to compute ground-up losses and insured losses in this study. The HURLOSS modeling approach is shown in Fig. 2-7, which is taken from ARA's submittal to the FCHLPM. The individual building model approach shown in Fig. 2-7a has been used in this study.

The HURLOSS modeling approach is based on a load and resistance approach which has been validated and verified using both experimental and field data. The model includes the effects of both wind-induced pressures and wind-borne debris on the performance of a structure in a hurricane. The wind loading models replicate the variation of wind loads as a function of wind direction, building geometry and component location, and when coupled with a simulated hurricane wind speed trace, a time history of wind loads acting on the building is produced. The wind loading model has been validated through comparisons with wind tunnel data. The time history of wind loads is used in the damage model to account for the progressive damage that often takes place during a hurricane event. The model also allows the effects of nearby buildings and their impact on the loads acting on the exterior of the structure.

**Table 2-5. Existing Construction Parameters Modeled for Group II Buildings**

| Variable | Categories | Description |
|---|---|---|
| Building Size | 1 | Five Stories with 35 Units |
| Roof Shape | 1 | Flat |
| Roof Construction | 3 | Wood, Concrete, Steel |
| Roof Covering | 2 | FBC Equivalent, Non-FBC Equivalent |
| Design Code and Fastest Mile Design Speed | 8 | SBC 1976 - 90, 100, 110, 120 and 130 mph, SBC 1988 - 90, 100 and 110 mph |
| Opening Protection | 2 | None, SFBC/SSTD 12 |
| Secondary Water Resistance | 2 | No, Yes |
| Surrounding Terrain | 2 | FBC Terrain B, FBC Terrain C |

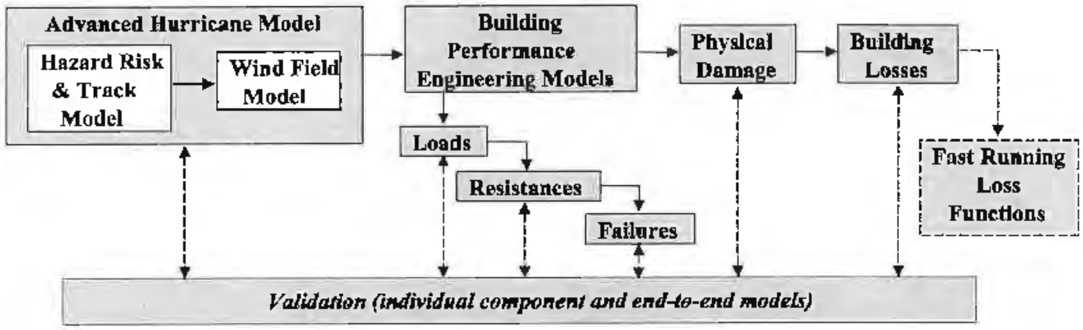
**Table 2-6. Existing Construction Parameters Modeled for Group III Buildings**

| Variable | Categories | Description |
|---|---|---|
| Building Size | 1 | Eight Stories with 56 Units |
| Roof Shape | 1 | Flat |
| Roof Construction | 2 | Concrete, Steel |
| Roof Covering | 2 | FBC Equivalent, Non-FBC Equivalent |
| Design Code and Fastest Mile Design Speed | 8 | SBC 1976 - 90, 100, 110, 120 and 130 mph, ASCE 7-88 - 90, 100 and 110 mph |
| Design Terrain for ASCE 7-88 | 3 | B, C, D |
| Opening Protection | 2 | None, SFBC/SSTD 12 |
| Secondary Water Resistance | 2 | No, Yes |
| Surrounding Terrain | 2 | FBC Terrain B, FBC Terrain C |

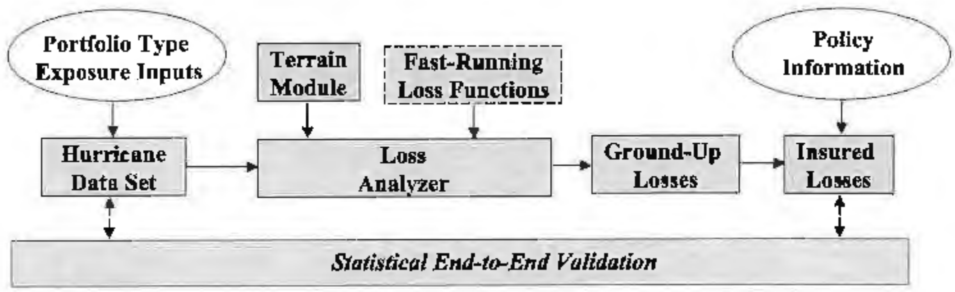
**a. Group II Building Geometry** **b. Group III Building Geometry**

**Figure 2-6. Group II and III Buildings**

Each of the buildings is located at each point in Florida given in Fig. 2-2. In the HURLOSS analysis, the building orientation (with respect to compass direction, N, NE, ...) is modeled as uniformly random. That is, for each simulated storm, an orientation is sampled from 0 to 360 degrees and the house is fixed in that orientation for that simulated storm. This approach is used since actual building orientation varies from house to house. In general, building orientation is important for a particular storm, but when losses are averaged over all hurricanes, a specific building's




**(a) Individual Buildings and Building Class Performance Model**




**(b) Multiple Site – Multiple Building Loss Projections**

**Figure 2-7. HURLOSS Modeling Approach for Hurricane Loss Projections**

orientation generally only affects loss costs by a few percent, particularly in Florida where hurricanes can come from many directions.

The wind resistive features of each house are established for a simulation run of 300,000 years of hurricanes. This is accomplished in the HURLOSS individual risk model by an input file that specifies component and building specifications for each key feature.

At each time step during a simulated storm, the computed wind loads acting on the building and its components are compared to the modeled resistances of the various components. If the computed wind load exceeds the resistance of the component, the component fails. When a component such as a window or a door fails, the wind-induced pressure acting on the exterior of the component is transmitted to the interior of the building. This internal pressure is then added (or subtracted) from the wind loads acting on the exterior of the building to determine if any additional components have been overloaded because of the additional loads produced by the internal pressurization of the building.

The progressive failure damage modeling approach is summarized in Fig. 2-8. Estimates of wind loads as a function of wind direction are produced for building components, including roof cover, roof sheathing, windows and doors, as well as for larger components including the entire roof, walls, and for overturning or sliding of the entire building in cases where a positive attachment to the ground does not exist.

The statistical properties of the resistances of the building components are obtained from laboratory tests and/or

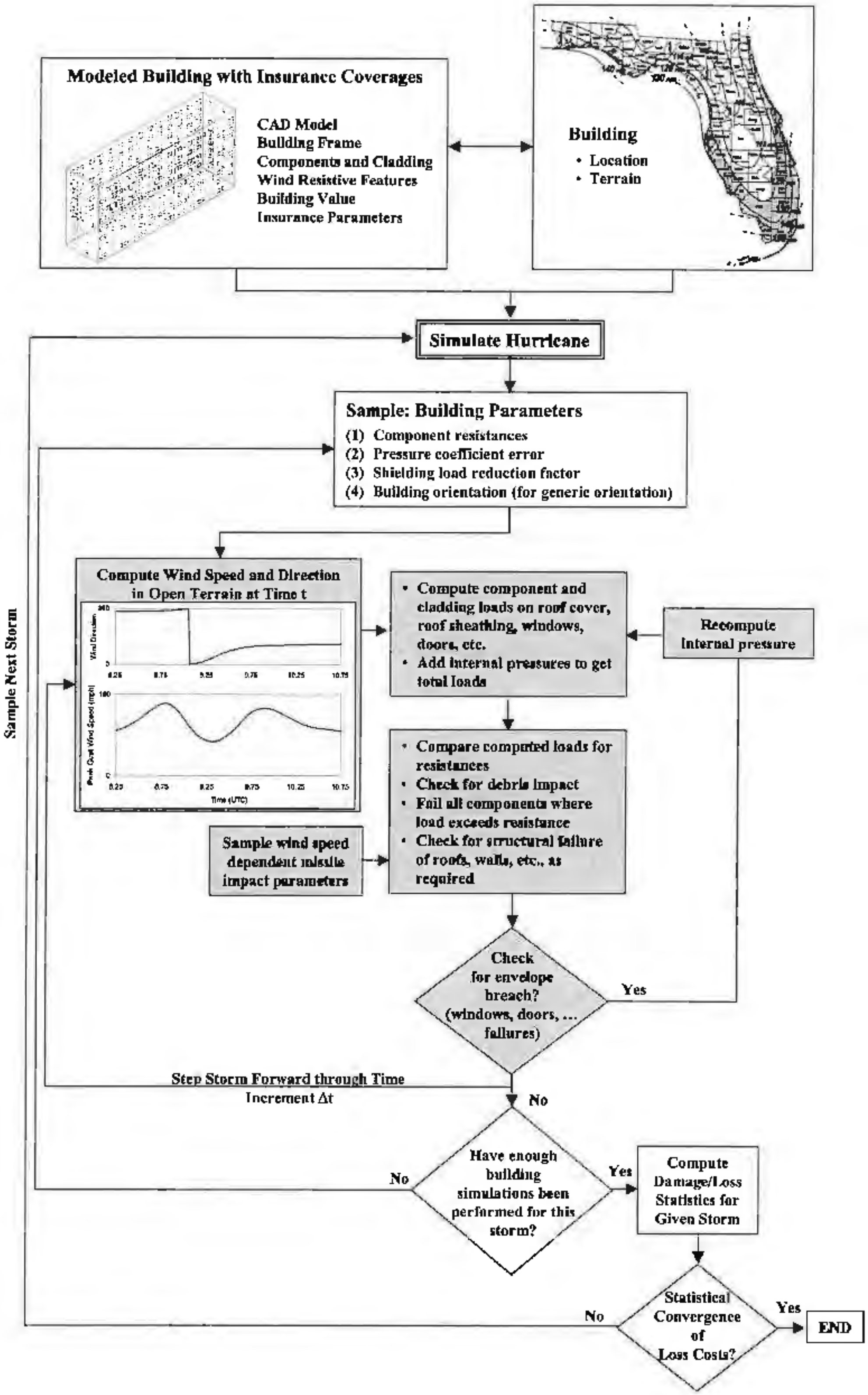



**Figure 2-8. HURLOSS Building Damage Simulation Methodology**

engineering calculations. In the simulation process, the resistances of the individual building components that will be loaded are sampled prior to the simulation of a hurricane, and are held constant throughout the simulation. The model computes a complete history of the failure of the building, which can be used to make a "movie" of the building performance.

Once the building damage has been computed for a given storm and the losses for all coverages computed, the process is repeated for a new set of sampled building component resistances. Once a large number of simulations have been performed, we have derived the data necessary to develop a statistical model for the expected performance of the building given the occurrence of a storm.

With this explicit modeling approach, it is possible to assess the impact of the Florida Building Code on the reduction in physical damage and insured loss. For example, enclosed designs (protected openings) and partially-enclosed designs can be explicitly modeled in the same manner an engineer designs the truss package or the builder selects the windows to comply with the required dynamic pressure rating.

### 2.4.4 Insurance Assumptions

This study covers condominium and tenant occupancies in buildings with five or more units. Hurricane losses to these buildings include damage to the building exterior and roof, interior walls, floor coverings, cabinets, etc., individual unit contents, common area contents, and loss of use. For condominium buildings, the condominium association typically insures the building, including all common areas. In some cases, the condominium unit owner will have responsibility for wall and floor coverings, etc. In tenant occupied buildings, the building owner is responsible for the entire building, including all interior finishing.

Condominium and renter unit coverage generally include loss of contents and loss of use.

The possible allocations of loss to building owner, condominium owner, and tenant have not been evaluated in this study. Instead, loss relativities based on total loss, without separate allocations to building, contents, etc., have been used to provide a simple and practical approach for this basic study. This approach is also consistent with the very simple FWUA class plan for condominium and tenant occupancies.

The increased complexity of extending the analysis into loss allocations would result in significant increase in computational time and increasing the numbers of tables by a factor of 4. This fact also suggests a simpler presentation based on total loss in this initial study. Additional time and effort are required to extend this study and produce more complex loss allocations based on individual interests and policy type.

The value of each modeled building was computed using ARA's construction cost estimation methodology. There are thousands of possible combinations of building features, each one producing a distinct building value. Table 2-7 shows the average square ft costs for each building group. The Coverage C limit was set at 70% of Coverage A[1] and the Coverage D limit at 20% of Coverage A. Loss relativities were computed with 2% deductible (as a percentage of the total coverage) as the base case.

**Table 2-7. Average Square Foot Costs**

| Building Group | Average of Unit Cost ($/sq ft) |
|---|---|
| I | 102.29 |
| II | 87.86 |
| III | 86.71 |

[1] The single-family residential study showed that the loss relativities were not sensitive to Coverage C for the range 50% to 70% of Coverage A.

# 3.0 LOSS RELATIVITIES FOR EXISTING CONSTRUCTION

## 3.1 General

The key construction features for condominium and tenant buildings that influence hurricane losses were introduced in Section 1. This section presents the analysis of key wind mitigation features of existing construction that influence physical damage and loss in a hurricane. Existing construction refers to all multi-family buildings built to any code or standard other than the 2001 Florida Building Code.

As noted in Section 2, the discussion on the construction of the multi-family buildings is separated into three height groups. The first group of buildings are one to three stories high having construction features very similar to single-family homes. The next two groups are the "engineered" structures designed to meet the performance criteria specified in various codes. The second group includes buildings more than 3 stories but less than or equal to sixty feet high. These buildings are normally designed to the wind loads specified in the Standard Building Code. The third group includes all buildings above sixty feet tall, which are normally designed to meet the wind load requirements given in the ASCE 7. Table 2-3 summarizes these groups.

## 3.2 Loss Relativities for Group I Buildings

Group I buildings are masonry and wood frame buildings, one to three stories tall.

Table 2-4 summarizes the wind-resistive features modeled in the analysis of Group I loss relativities. As noted earlier, the construction characteristics of buildings three stories tall or less are very similar to those of one and two family dwellings, and thus the wind-resistive features of these buildings are basically the same as those of one and two family dwellings. Each wind-resistive feature can be analyzed for several distinct "categories", where each category corresponds to a characteristic method of construction. For example, the roof-to-wall connection is assumed to be: (1) toe nail, (2) clip, (3) wrap, or (4) double-wrap connection. These four categories are chosen from a near continuum of possibilities and are categorized into a few distinct cases for practical reasons.

Appendix A discusses the wind-resistive features for the three building groups. As discussed in Appendix A, opening protection can be achieved in several ways, including the use of impact resistant glazing, impact resistant coverings, and also wood structural panels, per the FBC.[1] We note that this study has not analyzed wood structural panels (plywood shutters) because of the limited time and scope of this effort and the need for detailed analysis of test data to properly characterize the impact and pressure cycling resistances of wood panels. We have also not attempted to quantify any added benefits provided by passive in-place protection afforded by impact resistant glazing.[2]

Secondary classification variables include the same factors discussed in the previous study for single-family buildings, namely, dimensional lumber, protection of nonglazed openings, gable end bracing, wall construction, reinforced concrete roof deck and wall-to-foundation restraint. These factors have not been separately analyzed in this study due to time and budget issues. If appropriate, users

[1] For non-HVHZ locations in Florida, wood structural panels can be used for protection of openings in one and two story buildings. See FBC Section 1606.1.4 for wood panel fastening requirements.

[2] Glazing refers to glass or transparent or translucent plastic sheet used in windows, doors, or skylights (ASCE 7-98, Section 6.2).

can apply the secondary factors to Group I buildings exactly as was illustrated in the previous study.

### 3.2.1 Group I Loss Relativity Tables

The main loss relativity tables are given in Tables 3-1 and 3-2 for FBC Terrain B and C, respectively. The rating factors are discussed in Appendix A. These tables are normalized to a "central" building, as discussed in Section 3.2.3.1. These tables are for a 2% deductible (as a percentage of total coverage).

The loss relativities in Table 3-1 for Terrain B are based on averaging the loss relativities for the two model buildings for all 17 Terrain B locations in Table 2-2.

There are 14 Terrain C locations in Table 2-2. These locations are intended to represent:

1. Points located within 1500 feet of coast line.
2. Barrier islands.
3. All of Broward and Dade counties, per the FBC.

The relativities in Table 3-1 for these Terrain C locations are based on averaging the 14 modeled Terrain C locations across the state.

Because Terrain Category C loss costs are higher than Terrain Category B loss costs, the normalizing base class loss costs are different for Tables 3-1 and 3-2. Therefore, although the range in relativities is lower for Terrain C, the base loss costs for these locations are higher, reflecting the open terrain exposure.

Appendix A discusses the analysis and shows how the relativities vary by location. The variation in relativity was not judged to be significant enough to warrant the complexities introduced by separate relativities for each location.

### 3.2.2 Sensitivity Studies on Group I Secondary Variables

Sensitivity analyses were not performed in this study. However, relativity adjustments associated with the following wind-resistive features were analyzed for the previous study of single-family houses:

1. Roof Deck Attachment D (Dimensional Lumber, etc.)
2. Wall Construction
3. Reinforced Concrete Roof Deck
4. Opening Coverage
5. Gable End Bracing
6. Foundation Restraint.

The same secondary adjustments described in the previous report can be applied to the Group I relativities given in Tables 3-1 and 3-2.

### 3.2.3 Discussion of Group I Loss Relativity Results

As expected, there is a wide range of relativities from the weakest to the strongest buildings. The multiplicative ranges are factors of about 9 for Terrain B and 8 for Terrain C. These ranges are not as large as actually exists in a territory because not all variables have been considered separately in the classification.

The following paragraphs discuss the differences in loss relativity for some of the key variables.

#### 3.2.3.1 Normalization

The results in Tables 3-1 and 3-2 have been normalized by the loss costs of a "typical" building, which makes the comparison of the relativities easier. The typical building is

## Table 3-1. Loss Costs Relativities for Group I Buildings – Terrain B Locations with 2% Deductible

| Group I Buildings (Masonry and Wood Frame) Terrain Category B – 2% Deductible | | | | Roof Shape | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Flat | | Gable | | Hip | |
| Roof Cover | Roof Deck Attachment | Roof-Wall Connection | Opening Protection | No Secondary Water Resistance | Secondary Water Resistance | No Secondary Water Resistance | Secondary Water Resistance | No Secondary Water Resistance | Secondary Water Resistance |
| Non-FBC Equivalent | A | Toe Nails | None | 2.55 | 1.98 | 1.61 | 1.51 | 1.41 | 1.30 |
| | | | Hurricane | 1.99 | 1.38 | 0.92 | 0.80 | 0.76 | 0.65 |
| | | Clips | None | 2.32 | 1.74 | 1.26 | 1.15 | 1.03 | 0.92 |
| | | | Hurricane | 1.94 | 1.32 | 0.77 | 0.64 | 0.60 | 0.48 |
| | | Single Wraps | None | 2.27 | 1.68 | 1.15 | 1.04 | 0.93 | 0.83 |
| | | | Hurricane | 1.94 | 1.31 | 0.76 | 0.63 | 0.60 | 0.47 |
| | | Double Wraps | None | 2.26 | 1.67 | 1.10 | 1.00 | 0.92 | 0.82 |
| | | | Hurricane | 1.93 | 1.31 | 0.75 | 0.62 | 0.60 | 0.47 |
| | B | Toe Nails | None | 2.00 | 1.33 | 1.43 | 1.33 | 1.34 | 1.24 |
| | | | Hurricane | 1.31 | 0.59 | 0.72 | 0.61 | 0.69 | 0.57 |
| | | Clips | None | 1.61 | 0.91 | 1.00 | 0.88 | 0.87 | 0.76 |
| | | | Hurricane | 1.14 | 0.39 | 0.49 | 0.35 | 0.46 | 0.33 |
| | | Single Wraps | None | 1.45 | 0.75 | 0.83 | 0.72 | 0.71 | 0.59 |
| | | | Hurricane | 1.12 | 0.37 | 0.46 | 0.33 | 0.45 | 0.31 |
| | | Double Wraps | None | 1.31 | 0.58 | 0.66 | 0.54 | 0.61 | 0.48 |
| | | | Hurricane | 1.10 | 0.35 | 0.44 | 0.31 | 0.43 | 0.29 |
| | C | Toe Nails | None | 2.00 | 1.32 | 1.43 | 1.33 | 1.34 | 1.24 |
| | | | Hurricane | 1.29 | 0.57 | 0.71 | 0.60 | 0.68 | 0.57 |
| | | Clips | None | 1.60 | 0.89 | 0.99 | 0.88 | 0.88 | 0.76 |
| | | | Hurricane | 1.11 | 0.35 | 0.48 | 0.34 | 0.46 | 0.32 |
| | | Single Wraps | None | 1.44 | 0.72 | 0.83 | 0.71 | 0.71 | 0.59 |
| | | | Hurricane | 1.09 | 0.33 | 0.45 | 0.32 | 0.44 | 0.30 |
| | | Double Wraps | None | 1.27 | 0.53 | 0.65 | 0.52 | 0.61 | 0.48 |
| | | | Hurricane | 1.07 | 0.30 | 0.43 | 0.29 | 0.43 | 0.29 |
| FBC Equivalent | A | Toe Nails | None | 1.70 | 1.69 | 1.49 | 1.47 | 1.29 | 1.27 |
| | | | Hurricane | 1.11 | 1.09 | 0.79 | 0.76 | 0.65 | 0.63 |
| | | Clips | None | 1.45 | 1.44 | 1.13 | 1.11 | 0.91 | 0.89 |
| | | | Hurricane | 1.04 | 1.03 | 0.63 | 0.60 | 0.48 | 0.45 |
| | | Single Wraps | None | 1.37 | 1.36 | 1.02 | 1.00 | 0.82 | 0.79 |
| | | | Hurricane | 1.04 | 1.02 | 0.62 | 0.59 | 0.48 | 0.45 |
| | | Double Wraps | None | 1.35 | 1.33 | 0.98 | 0.95 | 0.80 | 0.78 |
| | | | Hurricane | 1.03 | 1.02 | 0.62 | 0.59 | 0.47 | 0.44 |
| | B | Toe Nails | None | 1.26 | 1.24 | 1.31 | 1.29 | 1.23 | 1.21 |
| | | | Hurricane | 0.57 | 0.54 | 0.61 | 0.59 | 0.58 | 0.55 |
| | | Clips | None | 0.87 | 0.84 | 0.88 | 0.86 | 0.77 | 0.74 |
| | | | Hurricane | 0.39 | 0.35 | 0.37 | 0.34 | 0.35 | 0.32 |
| | | Single Wraps | None | 0.72 | 0.69 | 0.73 | 0.70 | 0.60 | 0.58 |
| | | | Hurricane | 0.37 | 0.34 | 0.35 | 0.32 | 0.33 | 0.30 |
| | | Double Wraps | None | 0.56 | 0.53 | 0.55 | 0.52 | 0.50 | 0.47 |
| | | | Hurricane | 0.35 | 0.32 | 0.33 | 0.30 | 0.32 | 0.29 |
| | C | Toe Nails | None | 1.25 | 1.23 | 1.31 | 1.29 | 1.23 | 1.21 |
| | | | Hurricane | 0.56 | 0.53 | 0.61 | 0.59 | 0.58 | 0.55 |
| | | Clips | None | 0.85 | 0.82 | 0.88 | 0.85 | 0.77 | 0.74 |
| | | | Hurricane | 0.36 | 0.32 | 0.37 | 0.33 | 0.35 | 0.31 |
| | | Single Wraps | None | 0.70 | 0.67 | 0.72 | 0.69 | 0.60 | 0.58 |
| | | | Hurricane | 0.34 | 0.30 | 0.34 | 0.31 | 0.33 | 0.30 |
| | | Double Wraps | None | 0.52 | 0.49 | 0.54 | 0.51 | 0.49 | 0.46 |
| | | | Hurricane | 0.32 | 0.28 | 0.32 | 0.29 | 0.32 | 0.28 |

Notes:
1. This table is based on averaging the relativities for each modeled building for all 17 Terrain B locations.
2. This table applies to masonry and wood frame buildings one to three stories in height in Terrain B except those with a reinforced concrete roof deck.
3. Secondary factors are not considered in this table, including: (i) board roof decks (dimensional lumber and tongue and groove); (ii) masonry walls and reinforced masonry walls; (iii) all openings protected versus just glazed opening protected; (iv) unbraced gable end for gable roofs (other roof shape); and (v) unrestrained foundation.

## Table 3-2. Loss Costs Relativities for Group I Buildings – Terrain C Locations with 2% Deductible

| Group I Buildings (Masonry and Wood Frame) Terrain Category C – 2% Deductible | | | | Roof Shape | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Flat | | Gable | | Hip | |
| Roof Cover | Roof Deck Attachment | Roof-Wall Connection | Opening Protection | No Secondary Water Resistance | Secondary Water Resistance | No Secondary Water Resistance | Secondary Water Resistance | No Secondary Water Resistance | Secondary Water Resistance |
| Non-FBC Equivalent | A | Toe Nails | None | 1.58 | 1.39 | 1.28 | 1.25 | 1.20 | 1.16 |
| | | | Hurricane | 1.12 | 0.89 | 0.68 | 0.62 | 0.61 | 0.54 |
| | | Clips | None | 1.48 | 1.29 | 1.13 | 1.09 | 1.04 | 0.99 |
| | | | Hurricane | 1.08 | 0.85 | 0.56 | 0.49 | 0.47 | 0.39 |
| | | Single Wraps | None | 1.46 | 1.26 | 1.09 | 1.05 | 0.98 | 0.93 |
| | | | Hurricane | 1.08 | 0.84 | 0.56 | 0.48 | 0.47 | 0.38 |
| | | Double Wraps | None | 1.44 | 1.25 | 1.04 | 1.00 | 0.96 | 0.91 |
| | | | Hurricane | 1.08 | 0.84 | 0.55 | 0.48 | 0.46 | 0.38 |
| | B | Toe Nails | None | 1.38 | 1.15 | 1.20 | 1.16 | 1.17 | 1.12 |
| | | | Hurricane | 0.77 | 0.47 | 0.57 | 0.50 | 0.56 | 0.48 |
| | | Clips | None | 1.21 | 0.96 | 1.00 | 0.95 | 0.95 | 0.90 |
| | | | Hurricane | 0.64 | 0.30 | 0.37 | 0.28 | 0.36 | 0.25 |
| | | Single Wraps | None | 1.15 | 0.90 | 0.94 | 0.88 | 0.86 | 0.80 |
| | | | Hurricane | 0.62 | 0.29 | 0.35 | 0.25 | 0.34 | 0.23 |
| | | Double Wraps | None | 1.00 | 0.74 | 0.78 | 0.72 | 0.73 | 0.66 |
| | | | Hurricane | 0.61 | 0.28 | 0.34 | 0.24 | 0.33 | 0.22 |
| | C | Toe Nails | None | 1.38 | 1.15 | 1.21 | 1.17 | 1.17 | 1.12 |
| | | | Hurricane | 0.76 | 0.46 | 0.57 | 0.50 | 0.56 | 0.48 |
| | | Clips | None | 1.20 | 0.95 | 1.00 | 0.95 | 0.95 | 0.90 |
| | | | Hurricane | 0.61 | 0.26 | 0.36 | 0.27 | 0.35 | 0.25 |
| | | Single Wraps | None | 1.14 | 0.88 | 0.94 | 0.88 | 0.86 | 0.80 |
| | | | Hurricane | 0.59 | 0.24 | 0.34 | 0.24 | 0.34 | 0.23 |
| | | Double Wraps | None | 0.98 | 0.71 | 0.78 | 0.72 | 0.72 | 0.66 |
| | | | Hurricane | 0.58 | 0.23 | 0.32 | 0.22 | 0.32 | 0.22 |
| FBC Equivalent | A | Toe Nails | None | 1.30 | 1.29 | 1.24 | 1.23 | 1.16 | 1.15 |
| | | | Hurricane | 0.78 | 0.77 | 0.62 | 0.60 | 0.54 | 0.52 |
| | | Clips | None | 1.19 | 1.18 | 1.08 | 1.07 | 0.99 | 0.97 |
| | | | Hurricane | 0.73 | 0.72 | 0.49 | 0.47 | 0.40 | 0.37 |
| | | Single Wraps | None | 1.16 | 1.15 | 1.04 | 1.03 | 0.93 | 0.91 |
| | | | Hurricane | 0.73 | 0.72 | 0.48 | 0.46 | 0.39 | 0.37 |
| | | Double Wraps | None | 1.13 | 1.12 | 0.99 | 0.98 | 0.90 | 0.89 |
| | | | Hurricane | 0.73 | 0.72 | 0.48 | 0.46 | 0.39 | 0.36 |
| | B | Toe Nails | None | 1.13 | 1.12 | 1.16 | 1.15 | 1.12 | 1.11 |
| | | | Hurricane | 0.47 | 0.45 | 0.51 | 0.49 | 0.50 | 0.48 |
| | | Clips | None | 0.94 | 0.92 | 0.95 | 0.94 | 0.90 | 0.89 |
| | | | Hurricane | 0.31 | 0.28 | 0.30 | 0.27 | 0.28 | 0.25 |
| | | Single Wraps | None | 0.87 | 0.86 | 0.88 | 0.87 | 0.80 | 0.79 |
| | | | Hurricane | 0.30 | 0.27 | 0.28 | 0.25 | 0.26 | 0.23 |
| | | Double Wraps | None | 0.72 | 0.71 | 0.72 | 0.71 | 0.66 | 0.64 |
| | | | Hurricane | 0.29 | 0.26 | 0.27 | 0.23 | 0.25 | 0.22 |
| | C | Toe Nails | None | 1.12 | 1.12 | 1.16 | 1.15 | 1.12 | 1.11 |
| | | | Hurricane | 0.45 | 0.43 | 0.51 | 0.49 | 0.49 | 0.47 |
| | | Clips | None | 0.93 | 0.91 | 0.95 | 0.93 | 0.90 | 0.88 |
| | | | Hurricane | 0.28 | 0.25 | 0.29 | 0.26 | 0.28 | 0.25 |
| | | Single Wraps | None | 0.86 | 0.85 | 0.88 | 0.87 | 0.80 | 0.78 |
| | | | Hurricane | 0.26 | 0.23 | 0.27 | 0.24 | 0.26 | 0.23 |
| | | Double Wraps | None | 0.70 | 0.68 | 0.72 | 0.70 | 0.66 | 0.64 |
| | | | Hurricane | 0.25 | 0.22 | 0.25 | 0.22 | 0.25 | 0.21 |

Notes: 1. This table is based on averaging the relativities for each modeled building for all 14 Terrain C locations.
2. This table applies to masonry and wood frame buildings one to three stories in height in Terrain C except those with a reinforced concrete roof deck.
3. Secondary factors are not considered in this table, including: (i) board roof decks (dimensional lumber and tongue and groove); (ii) masonry walls and reinforced masonry walls; (iii) all openings protected versus just glazed opening protected; (iv) unbraced gable end for gable roofs (other roof shape); and (v) unrestrained foundation.

selected as a gable roof shape, clip roof-to-wall connection, deck attachment B, and non-FBC roof cover with no secondary water resistance. Note that the "typical" building is not necessarily the "most likely" or "average" building for a territory or construction era.

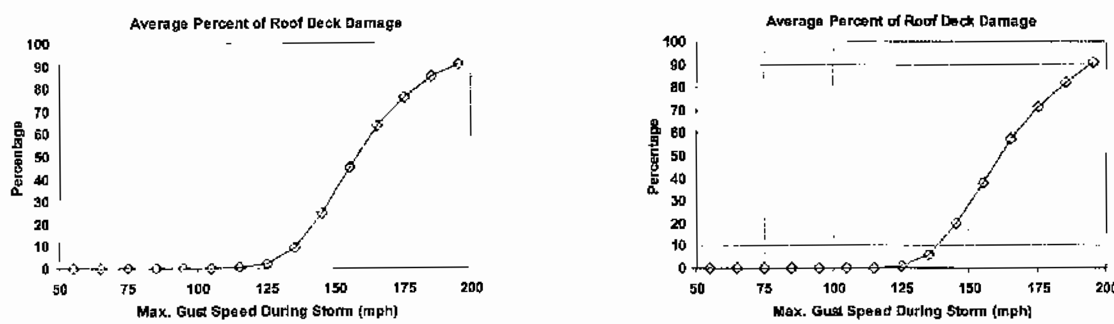
We see that the weakest building in Terrain B has loss costs 2.55 times that of a "typical" building. The strongest building has loss costs of only 0.28 of the "typical" building, reflecting the stronger roof, opening protection, hip roof shape, and SWR. These differences are readily explained by differences in component and connection strength and impact resistance. Some insurers may choose to renormalize the results to the weakest building for purposes of implementation. Renormalization, of course, has no mathematical influence on the computation of rates.

### 3.2.3.2 Roof Deck and Roof-to-Wall Connections

The effect of improved roof deck attachment can be seen in Fig. 3-1, which compares HURLOSS predicted deck attachment failure rates for a gable roof example for a Terrain B location. This plot shows the average percent of roof deck that has failed from the negative pressures and resulting pressure (suction) loads on the plywood roof deck. The sheathing on both buildings are nailed with a 6″/12″ nailing pattern. We see that if these buildings experience winds associated with a maximum reference peak gust speed (10 m above ground) of 125 mph, the building with 6d nails on average loses 2% of its roof deck while the building with 8d nails loses on average 1% of its deck. At 155 mph, the building with 6d nails loses 45% of its roof deck on average and the building with 8d nails loses about 38% of its roof deck on average.

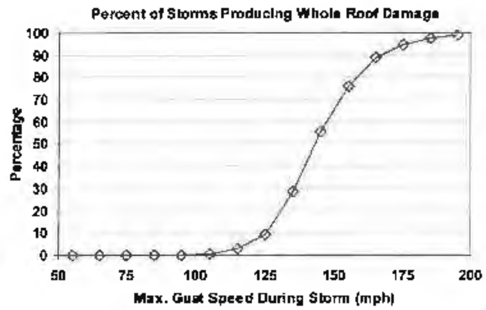
Figure 3-2 plots the percent of storms that produce whole roof failures for these two buildings. Whole roof failure occurs when the loads on the roof exceed the uplift resistance of the roof-to-wall connections. The roof, or major portions of it, fail and lift off the building. The difference in strength between toe nails and straps (single-wrap) results in a much reduced frequency of whole roof failures for straps. For 125 mph reference peak gust winds, the toe-nail case experiences whole roof type failures in about 10% of the hurricanes whereas the strapped case experiences whole roof failures in only 2% of the storms.

The combination of strengthening these two connections significantly reduces the failure rates of roof deck and whole roof failures.

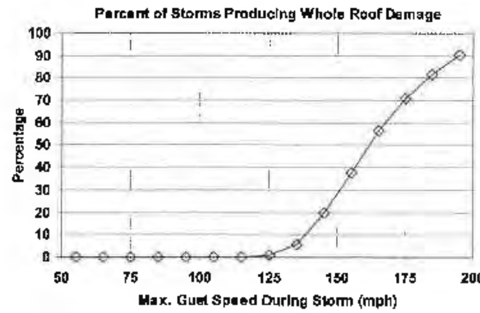



**(a) Gable Example 6d Roof Deck Nails** **(b) Gable Example 8d Roof Deck Nails**

**Figure 3-1. Comparison of HURLOSS Estimated Roof Deck Damage for 6d versus 8d Nails for Terrain B Location – Group I Buildings**




(a) Toe-Nail



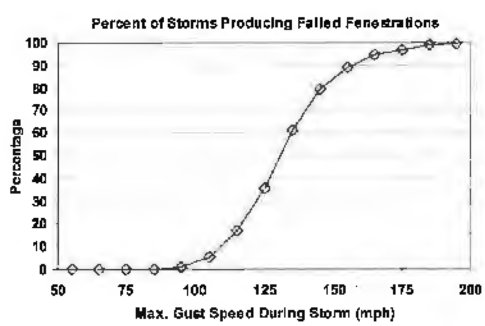

(b) Strap

**Figure 3-2. Comparison of HURLOSS Estimated Whole Roof Failures for Toe-Nail versus Strap for Terrain B Location – Group I Buildings**

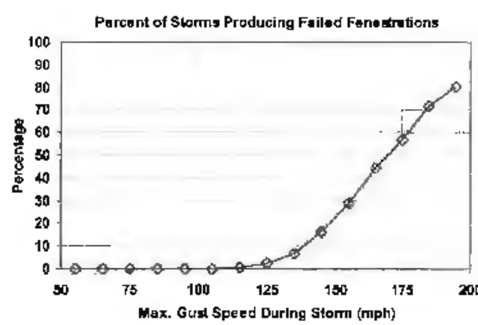
### 3.2.3.3 Protection of Openings

Hurricane opening protection refers to impact resistant glass or shutters for all glazed openings. The significant effect of hurricane opening protection can be seen in several ways. First, consider the percentage of storms producing failed openings. Figure 3-3 compares the percentage of storms producing failed fenestrations for the gable roof example without opening protection to the same building with opening protection. For the unprotected building, about 35 percent of the storms with 125 mph peak gust winds result in one or more failed fenestrations, whereas 2-3% of these storms produce one or more failed openings for the protected building. At 150 mph peak gust winds, the difference is just as dramatic: about 84% of the storms result in failed openings for the unprotected building whereas only 23% of such storms produce failures for the protected building.

A second result from the protection of openings is a reduction in the number of whole roof failures. To see this effect, we need to compare two identical buildings with the only difference being the protection of openings. Two examples are examined below. Figure 3-4 shows the difference in whole roof failures experienced by a gable building with toe-nail roof-wall connections. At 150 mph peak gust winds, the building with hurricane protection of openings experiences about ½ the whole roof failure rate (30%) versus the building with no opening protection (60% failure rate). The

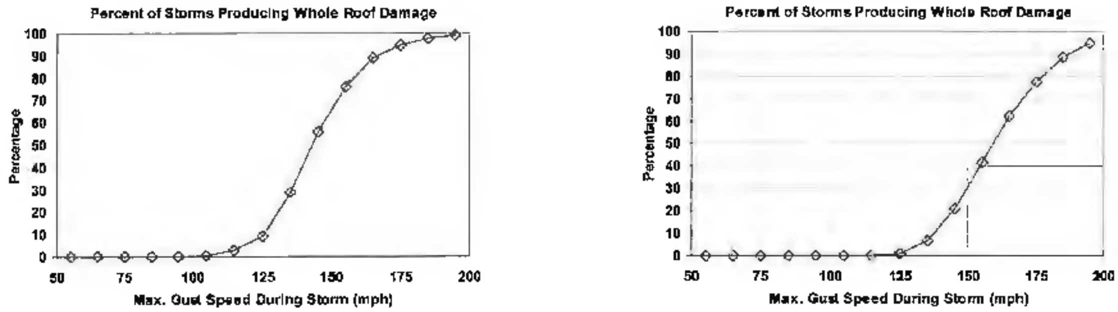



(a) Gable Roof (No Opening Protection)




(b) Same Building with Opening Protection

**Figure 3-3. Comparison of HURLOSS Estimated Fenestration Failures for No Opening Protection versus Opening Protection for Terrain B Location – Group I Buildings**




(a) **No Opening Protection – Toe Nail** (b) **Opening Protection – Toe Nail**

**Figure 3-4. Comparison of HURLOSS Estimated Whole Roof Failures(Gable Roof) – Group I Buildings in Terrain B**

same comparison for a stronger building (single-wrap roof-wall connections) is shown in Fig. 3-5. Wc see the same effect except the relative difference in whole roof failures is shifted towards higher wind speeds for the stronger building.

### 3.2.3.4 Roof Shape

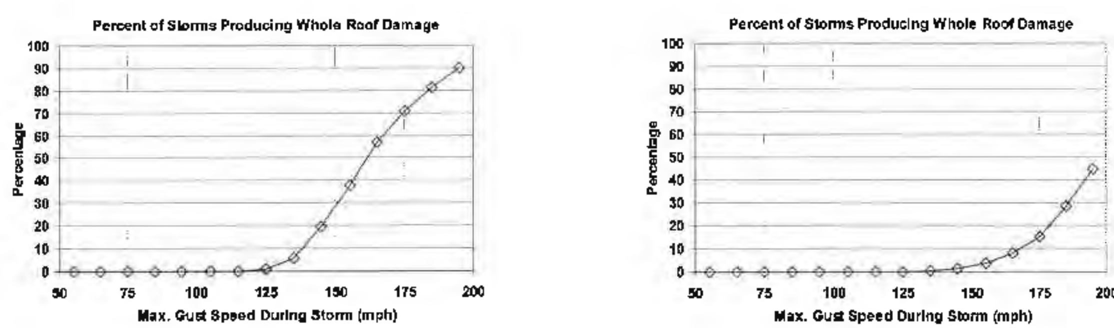
The effect of roof shape can be illustrated by comparing roof deck failures and whole roof failures for flat, gable and hip roof buildings. Figures 3-6 shows these comparisons for the case of toe-nail roof-wall connections, and no opening protection.

The failure rates for each of these components are reduced as the roof shape changes from flat to gable to hip, reflecting the improved aerodynamics and the fact that the hip has roof-to-wall connections on 4 sides versus 2 sides for the flat and gable roofs. Hence, there is a sizable relativity difference for the effect of roof shape. This difference is also highly nonlinear, being much more for a weaker building than for a stronger building.

## 3.3 Loss Relativities for Group II Buildings

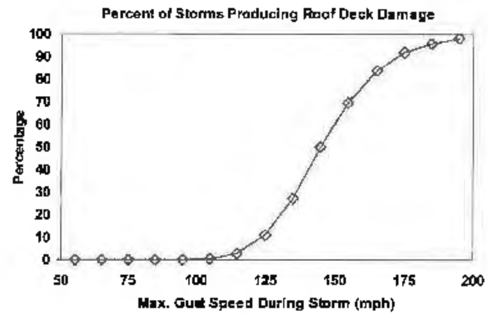
Group II buildings are steel, concrete, or reinforced masonry frame structures that are 60 ft high or less.

Table 2-5 summarizes the wind-resistive features modeled in the analysis for Group II Buildings. Referring to Appendix A, the two construction eras considered are 1982

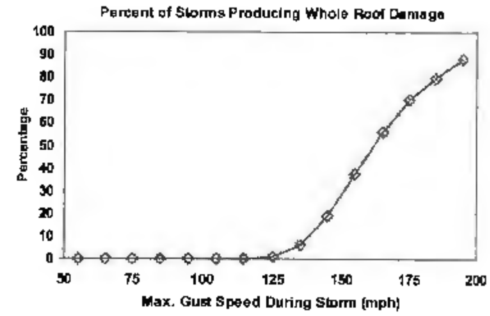



(a) **No Opening Protection – Strap** (b) **Opening Protection – Strap**

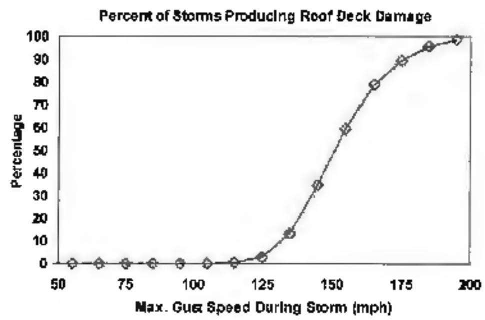
**Figure 3-5. Comparison of HURLOSS Estimated Whole Roof Failures for a Gable Roof – Group I Buildings in Terrain B**



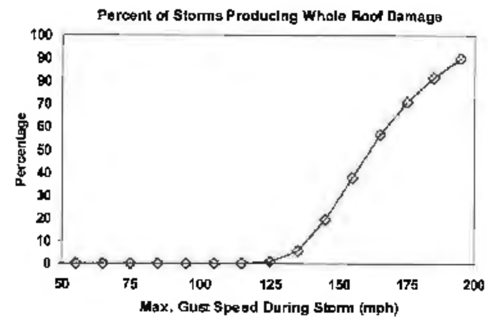

**(a) Flat**



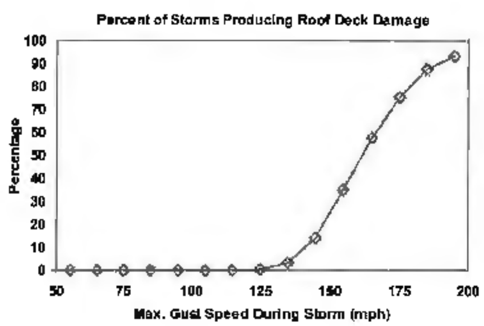

**(b) Flat**



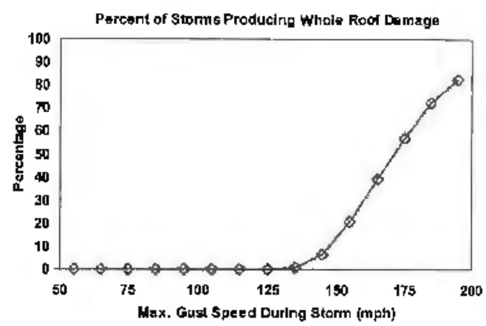

**(c) Gable**




**(d) Gable**




**(e) Hip**




**(f) Hip**

**Figure 3-6. Comparison of HURLOSS Estimated Roof Damage for Flat, Gable and Hip Roofs – Group I Buildings in Terrain B**

or earlier, for which SBC 1976 is assumed to be applicable, and between 1983 and 2001, for which SBC 1988 is assumed to be applicable. Buildings constructed in 2002 or later are designed according to FBC (ASCE 7-98), as discussed in Section 4.

Design wind speed maps for SBC 1976, SBC 1988, and ASCE 7-88 (see Section 3.4) are given in Appendix A. These maps may be useful to determine the applicable design wind speed for existing Groups II and III buildings.

### 3.3.1 Group II Loss Relativity Tables

The Group II loss relativity data for a 2% deductible (as a percentage of total coverage) are given in Tables 3-3 and 3-4 for FBC Terrains B and C respectively. Within each of these tables, the loss costs have been normalized within each design windspeed location by a building with a metal deck, no SWR, and no opening protection. This building

**Table 3-3. Loss Cost Relativities for Group II Buildings – Terrain B Locations with 2% Deductible**

| Design Code | Design Speed | Roof Cover | Opening Protection | Roof Deck: Wood Deck, No SWR | Roof Deck: Wood Deck, With SWR | Roof Deck: Metal Deck, No SWR | Roof Deck: Metal Deck, With SWR | Roof Deck: Reinforced Concrete Deck |
|---|---|---|---|---|---|---|---|---|
| 1982 or Earlier (SBC 1976) | 90 | Non-FBC Equivalent | No | 1.540 | 1.181 | 1.000 | 0.630 | 0.515 |
| | | | Yes | 1.386 | 1.023 | 0.705 | 0.333 | 0.234 |
| | | FBC Equivalent | No | 1.044 | 1.031 | 0.646 | 0.618 | 0.508 |
| | | | Yes | 0.886 | 0.871 | 0.361 | 0.330 | 0.234 |
| | 100 | Non-FBC Equivalent | No | 1.658 | 1.271 | 1.000 | 0.596 | 0.529 |
| | | | Yes | 1.499 | 1.117 | 0.722 | 0.317 | 0.255 |
| | | FBC Equivalent | No | 1.126 | 1.111 | 0.621 | 0.588 | 0.524 |
| | | | Yes | 0.972 | 0.956 | 0.351 | 0.315 | 0.255 |
| | 110 | Non-FBC Equivalent | No | 1.614 | 1.337 | 1.000 | 0.671 | 0.614 |
| | | | Yes | 1.439 | 1.159 | 0.682 | 0.338 | 0.269 |
| | | FBC Equivalent | No | 1.219 | 1.206 | 0.699 | 0.661 | 0.608 |
| | | | Yes | 1.045 | 1.031 | 0.383 | 0.335 | 0.268 |
| | ≥120 | Non-FBC Equivalent | No | 1.640 | 1.411 | 1.000 | 0.700 | 0.646 |
| | | | Yes | 1.470 | 1.238 | 0.693 | 0.377 | 0.296 |
| | | FBC Equivalent | No | 1.314 | 1.301 | 0.738 | 0.691 | 0.639 |
| | | | Yes | 1.140 | 1.126 | 0.432 | 0.374 | 0.295 |
| 1983 to 2001 (SBC 1988) | 90 | Non-FBC Equivalent | No | 1.749 | 1.327 | 1.000 | 0.562 | 0.435 |
| | | | Yes | 1.639 | 1.223 | 0.792 | 0.353 | 0.263 |
| | | FBC Equivalent | No | 1.176 | 1.159 | 0.590 | 0.552 | 0.433 |
| | | | Yes | 1.064 | 1.047 | 0.393 | 0.352 | 0.263 |
| | 100 | Non-FBC Equivalent | No | 1.270 | 0.592 | 1.000 | 0.491 | 0.333 |
| | | | Yes | 1.064 | 0.390 | 0.790 | 0.277 | 0.161 |
| | | FBC Equivalent | No | 0.659 | 0.577 | 0.542 | 0.485 | 0.330 |
| | | | Yes | 0.468 | 0.380 | 0.336 | 0.272 | 0.161 |
| | 110 | Non-FBC Equivalent | No | 1.328 | 0.681 | 1.000 | 0.505 | 0.327 |
| | | | Yes | 1.129 | 0.475 | 0.794 | 0.293 | 0.167 |
| | | FBC Equivalent | No | 0.764 | 0.665 | 0.573 | 0.499 | 0.325 |
| | | | Yes | 0.570 | 0.461 | 0.372 | 0.288 | 0.167 |

has been selected arbitrarily and the tables can be easily renormalized by any other building type. A key point in the use of these tables is the fact that they have been normalized within each design windspeed group. The user determines the relevant design windspeed that a building was based on by knowing its year built and location. Location (street address or zip code) is easily converted to design windspeed by using the windspeed map of the code (see Appendix A, section A.3.1.2) according to year built.

The data listed in Table 3-3 are the averages over the seventeen Terrain B locations listed in Table 2-2. The data listed in Table 3-4 are the averages over the fourteen Terrain C locations listed in Table 2-2.

As discussed in the single-family loss relativity report, the loss costs used to normalize these tables are different for Terrain B and Terrain C.

As an example of how to use these tables, consider the case of a 4 story condominium building built in 1990 in Tampa. The building is reinforced concrete with a reinforced concrete roof deck and non-FBC equivalent roof cover. The building has no opening protection. Since the building was built in 1990 we use the results for the SBC 1988. From Fig. A-8, we see that the design windspeed for Tampa was 100 mph fastest mile windspeeds. Therefore, we find the loss relativity to be 0.333. This means that for 2% deductible (as a percentage of total loss) that the loss costs are 0.333 that of the reference metal deck building at that location.

**Table 3-4. Loss Cost Relativities for Group II Buildings – Terrain C Locations with 2% Deductible**

| | | | | Roof Deck | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | Wood Deck | | Metal Deck | | Reinforced Concrete Deck |
| Design Code | Design Speed | Roof Cover | Opening Protection | No SWR | With SWR | No SWR | With SWR | |
| 1982 or Earlier (SBC 1976) | ≤100 | Non-FBC Equivalent | No | 1.489 | 1.211 | 1.000 | 0.692 | 0.603 |
| | | | Yes | 1.309 | 1.031 | 0.650 | 0.328 | 0.235 |
| | | FBC Equivalent | No | 1.097 | 1.085 | 0.710 | 0.680 | 0.596 |
| | | | Yes | 0.924 | 0.910 | 0.363 | 0.325 | 0.235 |
| | 110 | Non-FBC Equivalent | No | 1.458 | 1.250 | 1.000 | 0.746 | 0.677 |
| | | | Yes | 1.272 | 1.063 | 0.615 | 0.339 | 0.247 |
| | | FBC Equivalent | No | 1.165 | 1.154 | 0.769 | 0.736 | 0.671 |
| | | | Yes | 0.976 | 0.964 | 0.384 | 0.336 | 0.247 |
| | 120 | Non-FBC Equivalent | No | 1.502 | 1.321 | 1.000 | 0.760 | 0.686 |
| | | | Yes | 1.334 | 1.151 | 0.630 | 0.366 | 0.264 |
| | | FBC Equivalent | No | 1.246 | 1.235 | 0.792 | 0.752 | 0.681 |
| | | | Yes | 1.071 | 1.059 | 0.422 | 0.364 | 0.264 |
| | 130 | Non-FBC Equivalent | No | 1.312 | 0.979 | 1.000 | 0.742 | 0.652 |
| | | | Yes | 0.907 | 0.552 | 0.620 | 0.337 | 0.205 |
| | | FBC Equivalent | No | 1.032 | 0.969 | 0.786 | 0.736 | 0.649 |
| | | | Yes | 0.626 | 0.546 | 0.404 | 0.333 | 0.204 |
| 1983 to 2001 (SBC 1988) | 90 | Non-FBC Equivalent | No | 1.449 | 1.208 | 1.000 | 0.730 | 0.573 |
| | | | Yes | 1.275 | 1.030 | 0.651 | 0.359 | 0.224 |
| | | FBC Equivalent | No | 1.109 | 1.098 | 0.753 | 0.724 | 0.572 |
| | | | Yes | 0.933 | 0.920 | 0.394 | 0.354 | 0.224 |
| | 100 | Non-FBC Equivalent | No | 1.244 | 0.721 | 1.000 | 0.606 | 0.438 |
| | | | Yes | 0.953 | 0.422 | 0.706 | 0.299 | 0.154 |
| | | FBC Equivalent | No | 0.787 | 0.712 | 0.652 | 0.599 | 0.435 |
| | | | Yes | 0.498 | 0.412 | 0.361 | 0.296 | 0.154 |
| | 110 | Non-FBC Equivalent | No | 1.280 | 0.824 | 1.000 | 0.654 | 0.445 |
| | | | Yes | 0.998 | 0.533 | 0.700 | 0.334 | 0.163 |
| | | FBC Equivalent | No | 0.896 | 0.810 | 0.712 | 0.649 | 0.444 |
| | | | Yes | 0.622 | 0.524 | 0.412 | 0.331 | 0.162 |

These tables are based on minimal load design for each building code era. Over design is not considered in these loss relativities for existing buildings.

### 3.3.2 Discussion of Group II Loss Relativity Results

As expected, there is a wide range of relativities from the weakest to the strongest buildings. The multiplicative range are factors between 9 and 10 for either Terrain B or Terrain C. The following paragraphs discuss the differences in loss relativity for some of the key variables.

#### 3.3.2.1 Roof Deck

In the case of the wood roof system, the effect of improved roof deck attachment is shown in Fig. 3-7, which compares HURLOSS predicted damage rates for a five-story building example with Terrain B surroundings. Plots (a) and (b) show the estimated roof deck damage rates for the two limiting designs (90 mph and 130 mph) of the SBC 1976 building code. Plots (c) and (d) show the equivalent curves for the two limiting design cases (90 mph and 110 mph) for the SBC 1988 building code. The weakest roof deck design for both the SBC 1976 and SBC 1988 building codes yields a plywood fastening schedule of 6 penny nails with a 6"/12" nailing pattern, thus resulting in similar damage rate curves (see left-hand plots in Fig. 3-7).

Similarly, the strongest roof deck design for both the SBC 1976 and SBC 1988 building codes yields a plywood fastening schedule of 8 penny nails with a 6"/12" nailing pattern, again resulting in similar damage rate curves (see right-hand plots in Fig. 3-7). It can be seen from Fig. 3-7 that the performance of the plywood roof deck can vary significantly with the design wind speed for either of the SBC building codes considered. For example, if a Group II Building designed according to SBC 1976 were situated in a Terrain B environment and subject to a hurricane producing peak gust speeds equal to 100 mph it will have about a 90% chance of experiencing roof deck damage if it were designed using a 90 mph design speed and only about a 30% chance if it were designed using the upper limiting design speed.

The roof deck performance of a Group II Building constructed with a metal roof system is shown in Fig. 3-8 for the same limiting designs as discussed above for the Group II Building with a wood roof system. The four metal roof deck damage rate curves shown in Fig. 3-8 are very similar since the minimum metal deck attachment schedule for roof zone 1 remains in effect for all design speeds for both of the code eras.

#### 3.3.2.2 Roof Framing and Roof-Wall Connection

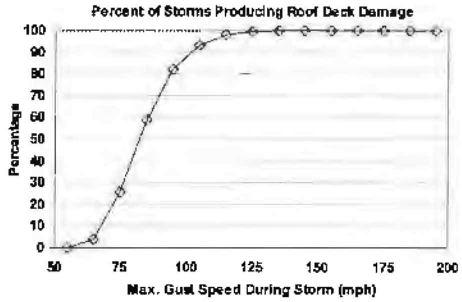
For Group II type buildings with a wood roof system, whole roof failure rates are shown in Fig. 3-9 for the two limiting SBC 1976 designs and the two limiting SBC 1988 designs. As is clearly shown in Fig. 3-9, there are large variations in whole roof failure rates for different design speeds within each of the two code eras. For example, considering the Group II Building designed according to SBC 1988, hurricanes producing a peak gust speed of 150 mph are estimated to produce whole roof failures about 35% of the time when designed using a 90 mph design speed versus only about 3% of the time when designed using a 110 mph design speed.

The failure rate of the steel joist-to-wall connection is shown in Fig. 3-10. The plots show the percent of storms producing at least one joist-to-wall failure versus the peak gust speed produced by the storm. Due to minimum weld requirements, the uplift capacity of the joist at the wall connection is modeled the same for both code eras and all design speeds. However, the moment capacities of the joists vary with both code era and design speed. Since moment failures cause adjacent joists to carry increased loads, they will influence uplift failures. The uplift failures thus vary with design era and design speed, as can be seen in Fig. 3-10.

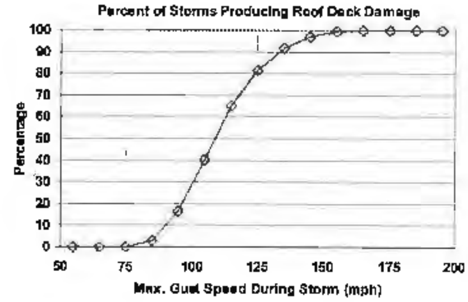
#### 3.3.2.3 Protection of Openings

The protection of glazed openings (windows and sliding glass doors) has the direct effect of reducing the frequency of glazing failures due to impact by windborne debris. In addition, shutters will relieve some of the wind pressure load taken by the glazing due to load sharing, thus reducing the number of pressure failures at the same time. Figure 3-11 shows the reduction in fenestration damage resulting from glazing protection for Group II Buildings.

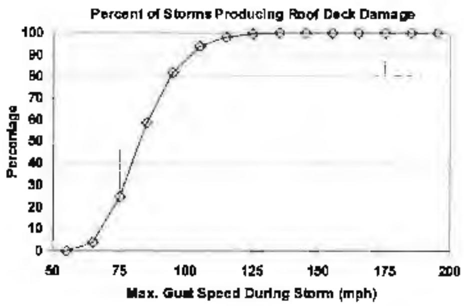
Fewer instances of fenestration damage afforded by the glazing protection results in fewer instances that the interior of the building on the top floor becomes pressurized, thus leading to a reduction in damage to the roof deck (see Fig. 3-12), and to open web steel joists for metal roof systems (Fig. 3-13).



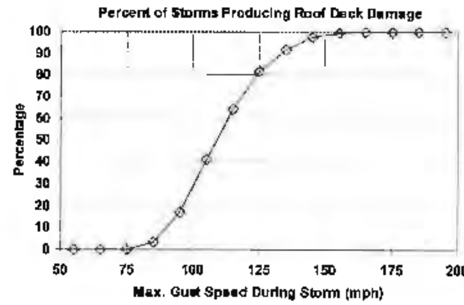

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure

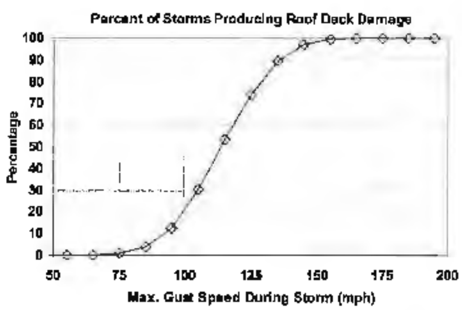



(c) SBC 1988 - 90 mph Zone, Standard Exposure

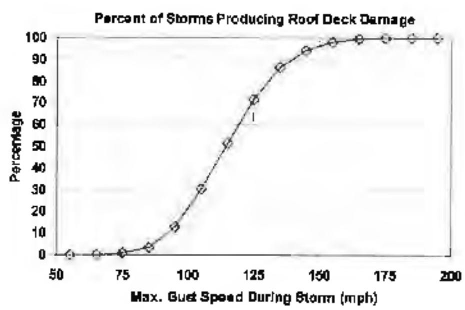



(d) SBC 1988 - 110 mph Zone, Standard Exposure

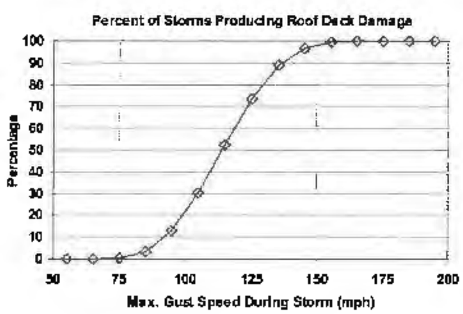
**Figure 3-7. Comparison of HURLOSS Estimated Roof Deck Damage Rates for a Group II Building with a Wood Roof System in Terrain B**



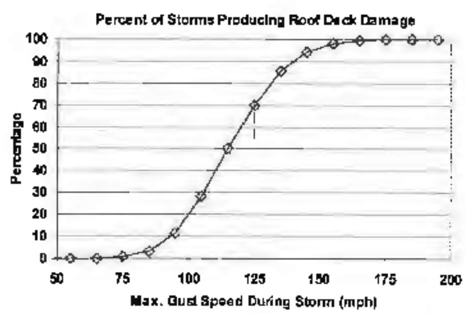

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure

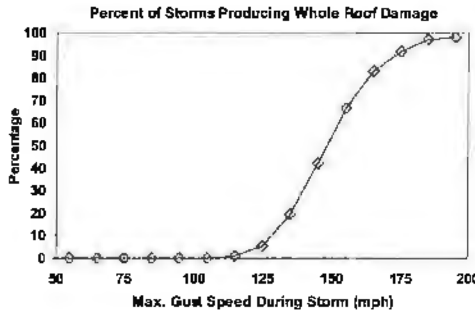



(c) SBC 1988 - 90 mph Zone, Standard Exposure

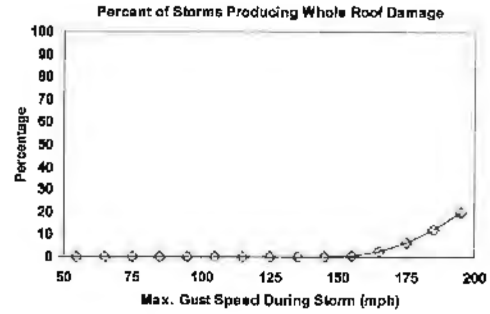



(d) SBC 1988 - 110 mph Zone, Standard Exposure

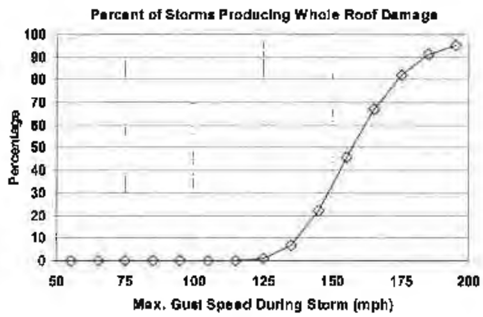
**Figure 3-8. Comparison of HURLOSS Estimated Roof Deck Damage Rates for a Group II Building with a Metal Roof System in Terrain B**



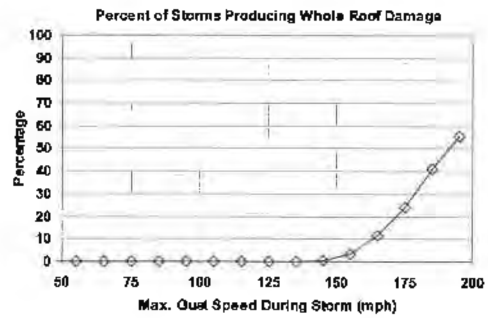

**(a) SBC 1976 - 90 mph Zone, Standard Exposure**




**(b) SBC 1976 - 130 mph Zone, Standard Exposure**

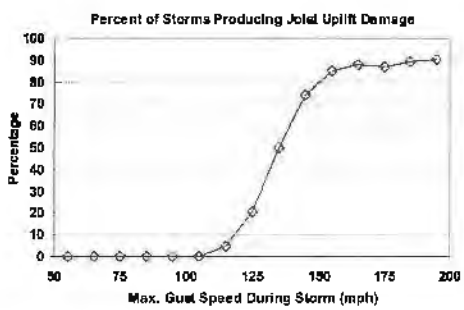



**(c) SBC 1988 - 90 mph Zone, Standard Exposure**

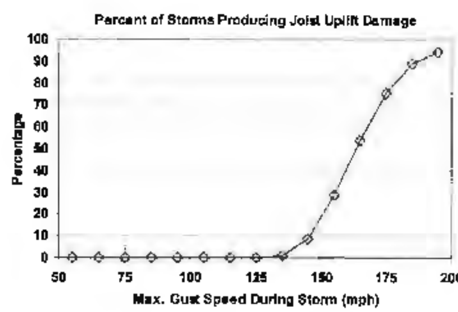



**(d) SBC 1988 - 110 mph Zone, Standard Exposure**

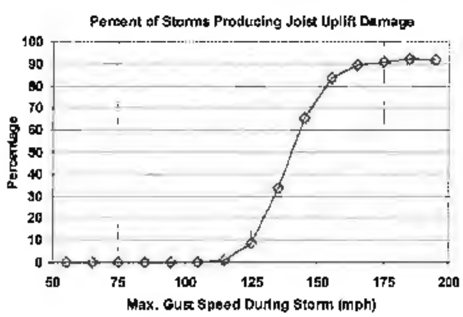
**Figure 3-9. Comparison of HURLOSS Estimated Whole Roof Failures for a Group II Building with a Wood Roof System in Terrain B**



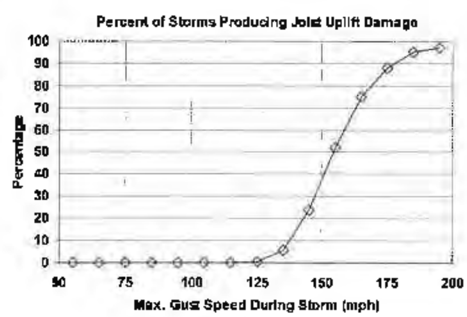

**(a) SBC 1976 - 90 mph Zone, Standard Exposure**




**(b) SBC 1976 - 130 mph Zone, Standard Exposure**

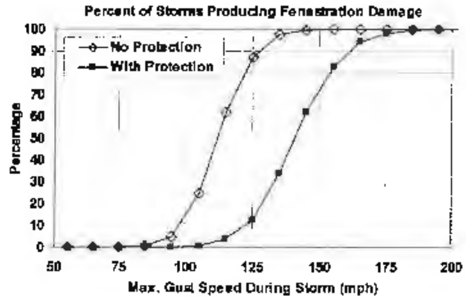



**(c) SBC 1988 - 90 mph Zone, Standard Exposure**

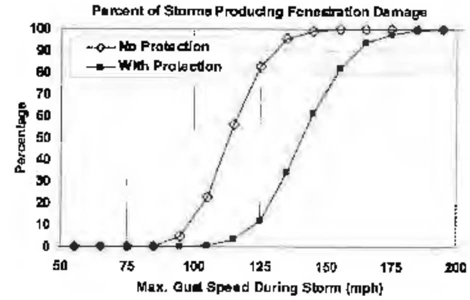



**(d) SBC 1988 - 110 mph Zone, Standard Exposure**

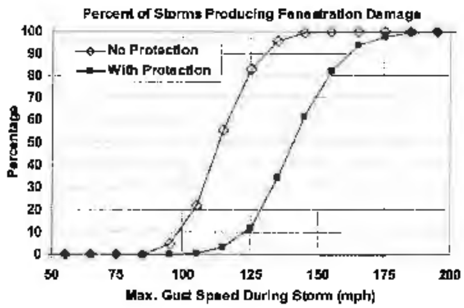
**Figure 3-10. Comparison of HURLOSS Estimated Joist Uplift Damage Rates for a Group II Building with a Metal Roof System in Terrain B**



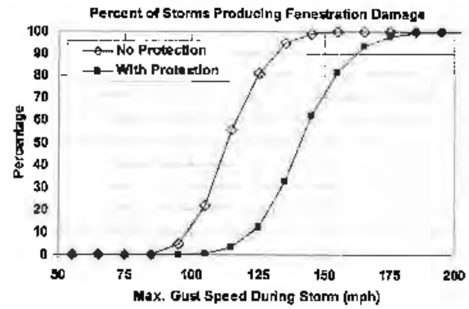

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure

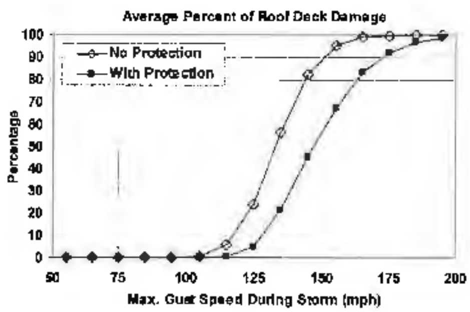



(c) SBC 1988 - 90 mph Zone, Standard Exposure

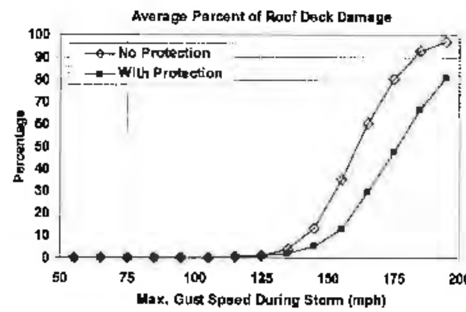



(d) SBC 1988 - 110 mph Zone, Standard Exposure

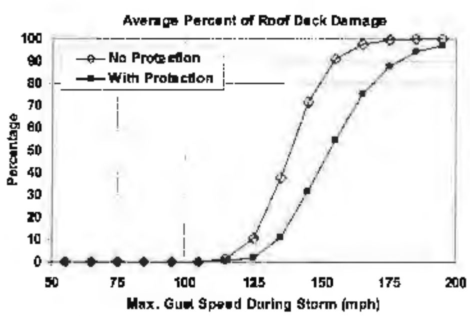
**Figure 3-11. Comparison of HURLOSS Estimated Fenestration Damage Rates for a Group II Building in Terrain B**



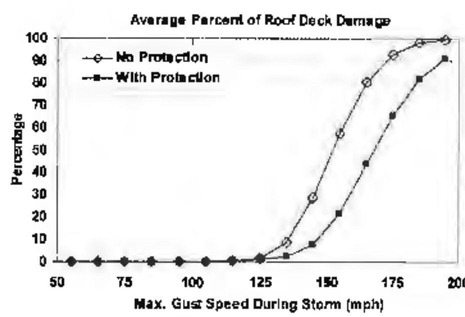

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure

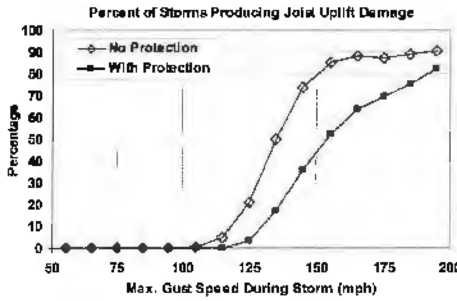



(c) SBC 1988 - 90 mph Zone, Standard Exposure

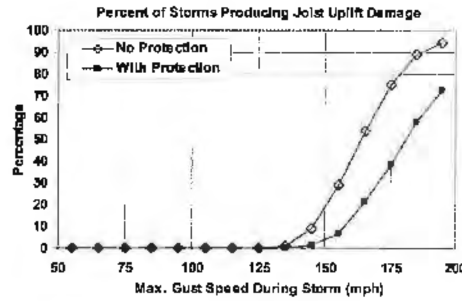



(d) SBC 1988 - 110 mph Zone, Standard Exposure

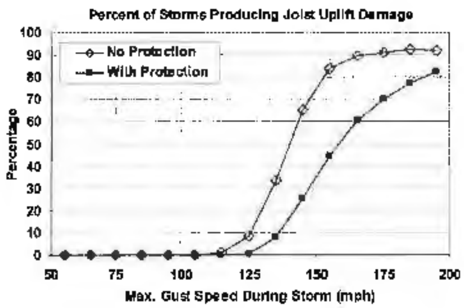
**Figure 3-12. Comparison of HURLOSS Estimated Average Roof Deck Damage for a Group II Building with a Metal Roof System in Terrain B**



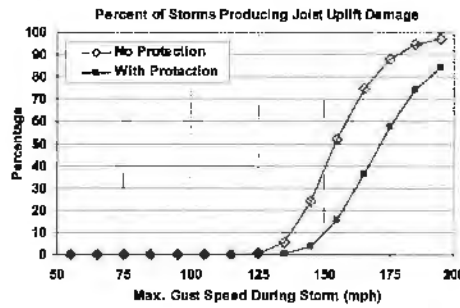

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure




(c) SBC 1988 - 90 mph Zone, Standard Exposure




(d) SBC 1988 - 110 mph Zone, Standard Exposure

**Figure 3-13. Comparison of HURLOSS Estimated Joist Uplift Damage Rates for a Group II Building with a Metal Roof System in Terrain B**

## 3.4 Loss Relativities for Group III Buildings

Table 2-6 summarizes the wind-resistive features modeled in the analysis Group III Buildings. Similar to Group II Buildings,the two construction eras considered are 1982 or earlier, for which the SBC 1976 building code is assumed to be applicable, and between 1982 and 2001, for which the ASCE 7-88 building code is assumed to be applicable. Buildings constructed in 2002 or later are designed according to FBC (ASCE 7-98) as discussed in Section 4.

### 3.4.1 Group III Loss Relativity Tables

The loss relativity data for a 2% deductible are given in Tables 3-5 and 3-6 for FBC Terrains B and C respectively. The shaded cell in each of the two tables highlights the building by which the losses have been normalized. The data listed in Table 3-5 are the averages over the seventeen Terrain B locations listed in Table 2-2. The data listed in Table 3-6 are the averages over the fourteen Terrain C locations listed in Table 2-2.

Terrain exposure is not treated for the SBC code era in Tables 3-5 and 3-6. The 1976 SBC does not have different terrains in the design procedure. As was done in Tables 3-3 and 3-4, each design windspeed location is normalized to an arbitrary building, which is a metal roof deck with no secondary water resistance and no opening protection. The user must determine roof deck type, SWR, and roof cover to classify the building given knowledge of year built and location.

For the 1988 code era, design according to ASCE 7 was assumed. Terrains B, C, and D were treated since the terrain is a key parameter

## Table 3-5. Loss Costs Relativities for Group III Buildings – Terrain B Locations with 2% Deductible

| | | | | | Roof Deck | | |
|---|---|---|---|---|---|---|---|
| | | | | | Metal Deck | | Reinforced Concrete Deck |
| Design Code | Design Speed (Fastest Mile) | Design Exposure | Roof Cover | Opening Protection | No SWR | With SWR | |
| 1982 or Earlier (SBC 1976) | 90 | Standard | Non-FBC Equivalent | No | 1.000 | 0.688 | 0.535 |
| | | | | Yes | 0.646 | 0.337 | 0.155 |
| | | | FBC Equivalent | No | 0.718 | 0.678 | 0.529 |
| | | | | Yes | 0.377 | 0.335 | 0.155 |
| | 100 | | Non-FBC Equivalent | No | 1.000 | 0.646 | 0.531 |
| | | | | Yes | 0.630 | 0.284 | 0.158 |
| | | | FBC Equivalent | No | 0.687 | 0.638 | 0.526 |
| | | | | Yes | 0.332 | 0.281 | 0.158 |
| | 110 | | Non-FBC Equivalent | No | 1.000 | 0.713 | 0.605 |
| | | | | Yes | 0.605 | 0.312 | 0.175 |
| | | | FBC Equivalent | No | 0.763 | 0.709 | 0.604 |
| | | | | Yes | 0.373 | 0.311 | 0.175 |
| | ≥120 | | Non-FBC Equivalent | No | 1.000 | 0.700 | 0.564 |
| | | | | Yes | 0.661 | 0.356 | 0.175 |
| | | | FBC Equivalent | No | 0.768 | 0.696 | 0.563 |
| | | | | Yes | 0.433 | 0.353 | 0.175 |
| 1983 to 2001 (SBC 1988) | 90 | B | Non-FBC Equivalent | No | 1.000 | 0.736 | 0.633 |
| | | | | Yes | 0.516 | 0.251 | 0.157 |
| | | | FBC Equivalent | No | 0.765 | 0.732 | 0.630 |
| | | | | Yes | 0.287 | 0.249 | 0.156 |
| | | C | Non-FBC Equivalent | No | 0.496 | 0.224 | 0.203 |
| | | | | Yes | 0.421 | 0.159 | 0.146 |
| | | | FBC Equivalent | No | 0.262 | 0.224 | 0.202 |
| | | | | Yes | 0.196 | 0.159 | 0.146 |
| | | D | Non-FBC Equivalent | No | 0.447 | 0.177 | 0.166 |
| | | | | Yes | 0.405 | 0.149 | 0.145 |
| | | | FBC Equivalent | No | 0.214 | 0.178 | 0.166 |
| | | | | Yes | 0.183 | 0.149 | 0.145 |
| | 100 | B | Non-FBC Equivalent | No | 1.000 | 0.669 | 0.573 |
| | | | | Yes | 0.601 | 0.269 | 0.160 |
| | | | FBC Equivalent | No | 0.724 | 0.668 | 0.573 |
| | | | | Yes | 0.330 | 0.268 | 0.160 |
| | | C | Non-FBC Equivalent | No | 0.593 | 0.252 | 0.221 |
| | | | | Yes | 0.493 | 0.164 | 0.147 |
| | | | FBC Equivalent | No | 0.314 | 0.252 | 0.221 |
| | | | | Yes | 0.223 | 0.164 | 0.147 |
| | | D | Non-FBC Equivalent | No | 0.550 | 0.210 | 0.195 |
| | | | | Yes | 0.476 | 0.152 | 0.147 |
| | | | FBC Equivalent | No | 0.272 | 0.210 | 0.195 |
| | | | | Yes | 0.209 | 0.152 | 0.147 |
| | 110 | B | Non-FBC Equivalent | No | 1.000 | 0.635 | 0.540 |
| | | | | Yes | 0.628 | 0.264 | 0.158 |
| | | | FBC Equivalent | No | 0.717 | 0.631 | 0.537 |
| | | | | Yes | 0.352 | 0.263 | 0.158 |
| | | C | Non-FBC Equivalent | No | 0.659 | 0.286 | 0.252 |
| | | | | Yes | 0.519 | 0.162 | 0.148 |
| | | | FBC Equivalent | No | 0.375 | 0.285 | 0.251 |
| | | | | Yes | 0.247 | 0.162 | 0.148 |
| | | D | Non-FBC Equivalent | No | 0.614 | 0.240 | 0.231 |
| | | | | Yes | 0.507 | 0.151 | 0.148 |
| | | | FBC Equivalent | No | 0.334 | 0.240 | 0.231 |
| | | | | Yes | 0.234 | 0.151 | 0.148 |

## Table 3-6. Loss Costs Relativities for Group III Buildings – Terrain C Locations with 2% Deductible

| Design Code | Design Speed (Fastest Mile) | Design Exposure | Roof Cover | Opening Protection | Roof Deck: Metal Deck: No SWR | Roof Deck: Metal Deck: With SWR | Roof Deck: Reinforced Concrete Deck |
|---|---|---|---|---|---|---|---|
| 1982 or Earlier (SBC 1976) | ≤100 | Standard | Non-FBC Equivalent | No | 1.000 | 0.746 | 0.632 |
| | | | | Yes | 0.567 | 0.312 | 0.169 |
| | | | FBC Equivalent | No | 0.777 | 0.738 | 0.627 |
| | | | | Yes | 0.354 | 0.310 | 0.168 |
| | 110 | | Non-FBC Equivalent | No | 1.000 | 0.793 | 0.696 |
| | | | | Yes | 0.546 | 0.329 | 0.187 |
| | | | FBC Equivalent | No | 0.830 | 0.790 | 0.694 |
| | | | | Yes | 0.378 | 0.327 | 0.187 |
| | 120 | | Non-FBC Equivalent | No | 1.000 | 0.786 | 0.656 |
| | | | | Yes | 0.583 | 0.357 | 0.179 |
| | | | FBC Equivalent | No | 0.834 | 0.783 | 0.654 |
| | | | | Yes | 0.420 | 0.355 | 0.179 |
| | 130 | | Non-FBC Equivalent | No | 1.000 | 0.797 | 0.663 |
| | | | | Yes | 0.589 | 0.374 | 0.186 |
| | | | FBC Equivalent | No | 0.851 | 0.799 | 0.664 |
| | | | | Yes | 0.439 | 0.371 | 0.185 |
| 1983 to 2001 (SBC 1988) | 90 | B | Non-FBC Equivalent | No | 1.000 | 0.878 | 0.783 |
| | | | | Yes | 0.421 | 0.280 | 0.190 |
| | | | FBC Equivalent | No | 0.887 | 0.870 | 0.777 |
| | | | | Yes | 0.304 | 0.279 | 0.190 |
| | | C | Non-FBC Equivalent | No | 0.470 | 0.325 | 0.292 |
| | | | | Yes | 0.315 | 0.171 | 0.149 |
| | | | FBC Equivalent | No | 0.352 | 0.325 | 0.292 |
| | | | | Yes | 0.200 | 0.170 | 0.149 |
| | | D | Non-FBC Equivalent | No | 0.378 | 0.232 | 0.209 |
| | | | | Yes | 0.296 | 0.153 | 0.146 |
| | | | FBC Equivalent | No | 0.259 | 0.232 | 0.209 |
| | | | | Yes | 0.181 | 0.153 | 0.146 |
| | 100 | B | Non-FBC Equivalent | No | 1.000 | 0.819 | 0.731 |
| | | | | Yes | 0.474 | 0.278 | 0.184 |
| | | | FBC Equivalent | No | 0.850 | 0.818 | 0.729 |
| | | | | Yes | 0.320 | 0.277 | 0.184 |
| | | C | Non-FBC Equivalent | No | 0.526 | 0.327 | 0.287 |
| | | | | Yes | 0.366 | 0.169 | 0.150 |
| | | | FBC Equivalent | No | 0.369 | 0.328 | 0.288 |
| | | | | Yes | 0.214 | 0.170 | 0.150 |
| | | D | Non-FBC Equivalent | No | 0.461 | 0.261 | 0.240 |
| | | | | Yes | 0.351 | 0.157 | 0.149 |
| | | | FBC Equivalent | No | 0.304 | 0.260 | 0.240 |
| | | | | Yes | 0.199 | 0.157 | 0.148 |
| | 110 | B | Non-FBC Equivalent | No | 1.000 | 0.817 | 0.731 |
| | | | | Yes | 0.497 | 0.301 | 0.196 |
| | | | FBC Equivalent | No | 0.862 | 0.817 | 0.731 |
| | | | | Yes | 0.358 | 0.300 | 0.196 |
| | | C | Non-FBC Equivalent | No | 0.600 | 0.403 | 0.360 |
| | | | | Yes | 0.376 | 0.178 | 0.155 |
| | | | FBC Equivalent | No | 0.456 | 0.401 | 0.358 |
| | | | | Yes | 0.238 | 0.178 | 0.155 |
| | | D | Non-FBC Equivalent | No | 0.539 | 0.336 | 0.321 |
| | | | | Yes | 0.362 | 0.164 | 0.154 |
| | | | FBC Equivalent | No | 0.395 | 0.336 | 0.321 |
| | | | | Yes | 0.223 | 0.164 | 0.154 |

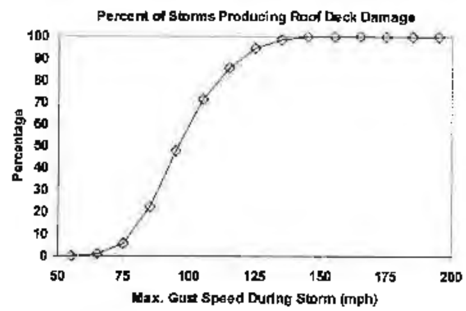
of the design.[3] Normalization of this part of the table is also by design windspeed. In the absence of documentation on the Terrain exposure used in the design, it seems reasonable to assume Terrain B. Coastal exposures were likely designed by the engineer to Terrain C loads.

[3] Terrain exposure A was not treated in this study. Terrain exposure A is a New York City type of high rise development and hence was likely not used in Florida designs. Normalization of this part of the table is also by design windspeed

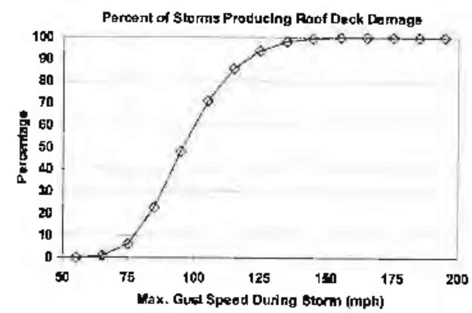
### 3.4.2 Discussion of Group III Loss Relativity Results

As expected, there is a wide range of relativities from the weakest to the strongest buildings. The multiplicative range is approximately 10 for both Terrains B and C.

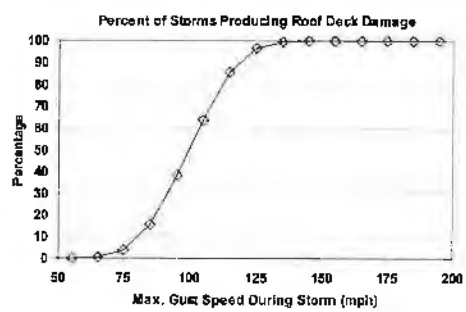
Plots showing damage rates associated with the Group III Buildings are given in Figs. 3-14 through 3-16. In a qualitative sense, the discussion of the damage results given in Section 3.3 for Group II Buildings is applicable to the damage results given in this section for the Group III Buildings.



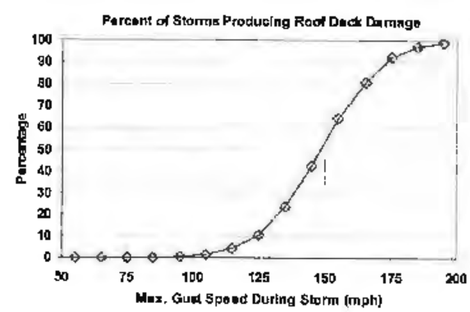

**(a) SBC 1976 - 90 mph Zone, Standard Exposure**




**(b) SBC 1976 - 130 mph Zone, Standard Exposure**

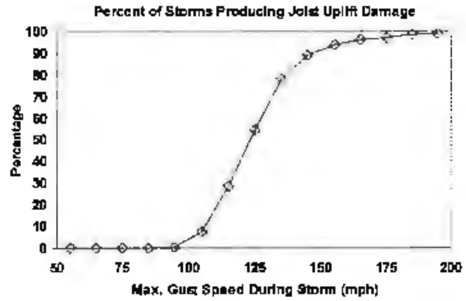



**(c) ASCE 7-88 - 90 mph Zone, Exposure B**

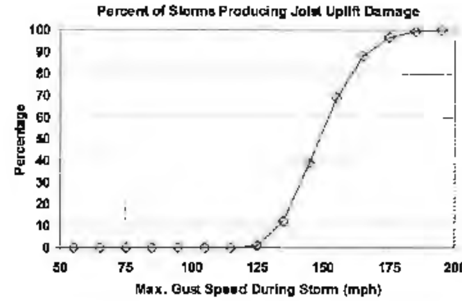



**(d) ASCE 7-88 - 110 mph Zone, Exposure D**

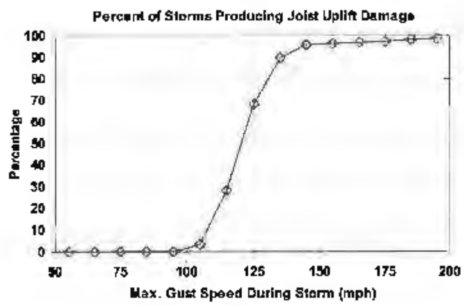
**Figure 3-14. Comparison of HURLOSS Estimated Roof Deck Damage Rates for a Group III Building with a Metal Roof System in Terrain B**



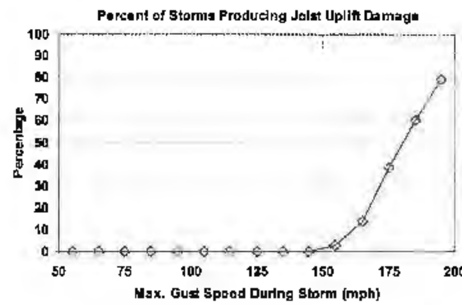

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure

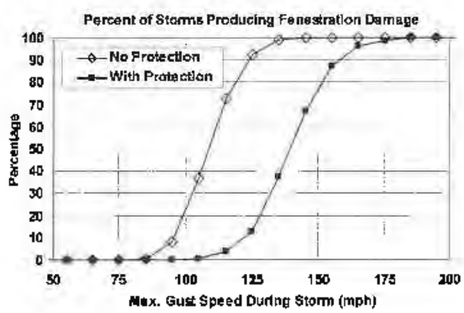



(c) ASCE 7-88 - 90 mph Zone, Exposure B

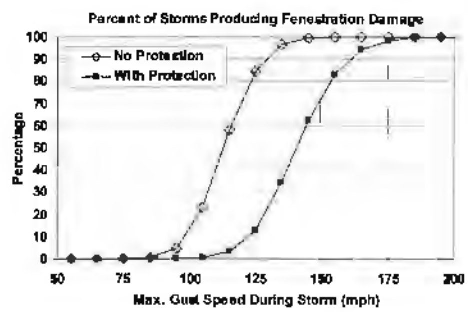



(d) ASCE 7-88 - 110 mph Zone, Exposure D

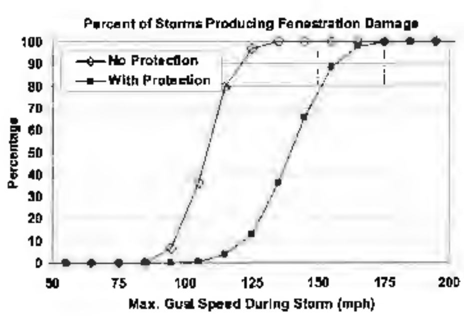
**Figure 3-15. Comparison of HURLOSS Estimated Joist Uplift Damage Rates for a Group III Building with a Metal Roof System in Terrain B**



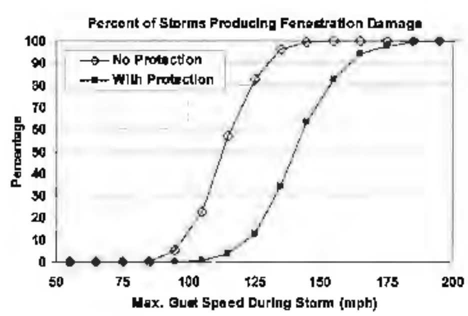

(a) SBC 1976 - 90 mph Zone, Standard Exposure




(b) SBC 1976 - 130 mph Zone, Standard Exposure




(c) ASCE 7-88 - 90 mph Zone, Exposure B




(d) ASCE 7-88 - 110 mph Zone, Exposure D

**Figure 3-16. Comparison of HURLOSS Estimated Fenestration Damage Rates for a Group III Building in Terrain B**

# 4.0 LOSS RELATIVITIES FOR NEW CONSTRUCTION

## 4.1 General

The FBC will have a beneficial impact on new construction in the state of Florida. The code will improve the design and construction of new buildings with regard to wind loads, particularly in the windborne debris regions. Prior to the FBC, only a few counties in the state required consideration of windborne debris.

The development of the loss relativities for new construction requires consideration of two design options in the wind-borne dehris zone: design as an enclosed building or design as a partially-enclosed building. Section 4.2 presents a summary of the major design issues of the FBC. Appendix A provides a more in-depth discussion and also presents the analysis of the loss relativities for new construction to the FBC. Appendix B includes an example of the design calculations that were performed by ARA in order to model the critical wind resistive features of multi-unit buildings built to the new code. Section 4.3 presents the loss relativity tables for new construction, Section 4.4 presents loss relativities for over-design cases, and Section 4.5 presents a brief discussion of rating verification issues for new construction.

## 4.2 Effect of the Florida Building Code on New Construction

With respect to the rating of buildings for insurance purposes, the FBC makes the following wind-related changes to construction requirements in the state.

- The introduction of a Wind-Borne Debris Region (WBDR) means that new buildings in this region must now either have impact resistance protection on all glazed openings *or* be designed for higher wind pressures than previously. This changc means that a designer must now choose between designing the structure as either an enclosed or partially enclosed building. For buildings taller than 30 ft, a different impact test standard is required for small missiles.
- A new wind speed map and new terrain exposure categories mean that buildings in some parts of the state will be designed for higher wind pressures than they were previously under the SBC. This change will affect the design of several parts of the structurc including the strength of the windows, the strength of the roof deck and its connections, the wall design, and the foundations.
- A designer will now consider only 60% of the dead load in resisting uplift loads in the FBC, which means that roof-wall straps will be stronger than they were using the SBC.
- More wind resistant roof coverings will now become the standard roof covering in most of the state. For design wind speeds of 110 mph and greater, the asphalt shingles must be tested according to ASTM D 3161 (modified to 110-mph) or Miami-Dade PA 107.

### 4.2.1 Design Scenarios in Wind-Borne Debris Region (WBDR)

An "Enclosed" structure is designed assuming that all the openings are closed and therefore the wind loads are determined using a small internal pressure inside the building. To be designed as an "Enclosed" structure, a building must have all its glazed openings being impact resistant, achieved either by having qualified protection devices or using qualified impact resistant glazing.

Alternatively, a "Partially Enclosed" building is designed assuming that one or more areas on the building are open to allow the wind to enter the building and pressurize the interior. This pressurization means that individual parts of the building, such as the windows, doors, trusses, and roof decking must be designed to be stronger than the same features in an "Enclosed" building.

For insurance rating purposes, the distinction between the enclosed and partially-enclosed designs in the WBDR with respect to loss costs is largely determined by the presence or absence of opening protection on all glazed openings[1]. Enclosed designs in the WBDR will perform better than partially-enclosed designs and will have lower losses because of the effect of the opening protection. Section 3 discusses the significance of opening protection in reducing damage and loss.

For tall buildings, the impact protection standard changes compared to residential structures. For openings between 0 and 30 ft, the large missile impact test referred to in the residential study is required, where a wood 2x4 stud is fired at the protection device or impact glazing, followed by pressure cycling tests. For openings between 30 and 60 ft above ground, the protection device must meet the small missile impact test, where 30 steel balls are fired at the device. Openings above 60 ft do not need impact protection, except in the HVHZ where all openings greater than 30 ft above ground must meet the small missile test.

Examination of the results in Appendix A indicates that the partially-enclosed designs are only marginally better than an equivalent enclosed design without protected openings. The small increase in performance is due to the stronger roof-to-wall connection, tighter roof deck nailing pattern, and stronger window and door assemblies in terms of pressure resistance.

[1] In the HVHZ, all openings must be protected (see Section 1626 of FBC 2001).

### 4.2.2 Definition of Terrain "Exposure Category"[2]

The FBC has adopted a different definition of Exposure C than the one that appears in the text of ASCE 7-98. Exposure C (known as the open country exposure) is defined in the FBC as Broward and Miami-Dade counties (HVHZ), barrier islands within 5000 ft of the high water line, and 1500 ft from the coastline in the rest of the state. All other buildings will be designed for Exposure B. Loss relativities are computed for buildings designed for terrain Exposures B and C separately.

## 4.3 Loss Cost Relativity Tables

For each of the 31 locations, the roof deck attachment, the roof-to-wall connection, and the window design pressures on the model buildings were designed to the minimum requirements of the Florida Building Code as described above. These "designed" buildings were analyzed with HURLOSS to estimate the loss cost of each of the buildings at each location. Over one hundred FBC 2001 building designs were produced, reflecting the different design wind speeds, treatment of internal pressure, building height, and roof shape. Over 1,000 HURLOSS computations were performed for these FBC buildings at different locations in Florida.

The average loss costs for the base class (typical) buildings in the existing building study were calculated for each location, and used to determine the relativity of each FBC building. That is, we normalized the new construction relativities by the same values in the existing building study so that the relativity tables would be consistent with each other.

[2] ASCE-7 uses the term "Exposure" to define the earth's surface roughness for purposes of grouping this roughness into several distant categories for wind load estimation. Insurers need to be aware of this use of the term "Exposure" when reading building code and wind engineering literature.

The analysis summarized in Appendix A shows that for classification purposes for hurricanes, the key variables for new construction are:

- Terrain Exposure Category
- Roof Shape
- Roof Deck Type
- Opening Protection
- Design Wind Speed
- Internal Pressure Design

Appendix A contains a more detailed explanation of how these factors affect the strength of various features of the building. It also discusses several other definitions from the FBC that affect the overall strength of the building.

Tables 4-1, 4-2, and 4-3 present the relativity results for new construction for 2% deductible for Group I, II, and III buildings respectively. The top part of Table 4-1 covers all new construction that does not have a reinforced concrete roof deck. The bottom part of Table 4-1 is for Group I buildings with a reinforced concrete roof deck. The results for Group II buildings include wood, metal and concrete roof decks. The results for Group III buildings are for metal and concrete roof decks. The lower portion of each table applies only to those buildings with a reinforced concrete roof deck built to ACI 318 and tied integrally to the walls.

The wind speeds in Tables 4-1 through 4-3 are peak gust wind speeds that correspond to the FBC wind speed map (Fig. 2-1). The user should be aware that the wind speeds in Tables 3-3 through 3-6 in Section 3 correspond to fastest mile wind speeds. The reason for this difference is that the earlier design standards for Groups II and II buildings used fastest mile wind speeds instead of peak gust wind speeds.

Our analysis of the results indicates that the variation in relativities between wind speeds is notable for the lower wind speed levels (100 and 110 mph) and that the higher wind speeds can be grouped into ≥ 120 mph. Therefore, Tables 4-1 through 4-3 show only three wind speeds: 100 mph, 110 mph, and ≥ 120 mph. Buildings with reinforced concrete deck have not been observed to fail in hurricanes. Therefore, in this study, design wind speed is not considered as a rating factor for buildings with reinforced concrete roof decks.

We note that Opening Protection in Terrain Exposure B and Exposure C means that all glazed openings (i.e., those with glass or plastic) are protected with impact rated glazing or shutters. The requirements for the High Velocity Hurricane Zone (HVHZ) are slightly different in that all openings including doors and garage doors must he protected with shutters or impact resistant products. The results of our simulations of buildings in the HVHZ include this additional protection requirement for the HVHZ in Tables 4-1 through 4-3.

The analysis for opening protection for new construction was performed only for devices that meet the impact and pressure cycling test standards. Although wood structural panels (plywood) are allowed by the FBC (except in the HVHZ), modeling and analysis of that option was not performed in this study.

## 4.4 Mitigation and Over-Design to FBC Minimum-Design Relativities

Each of the designs prepared for the study buildings (summarized in Appendix A) meet the *minimum* requirements of the FBC. There are many opportunities in most parts of the state to exceed these requirements, and build to a higher design wind speed, or protect

## Table 4-1. Loss Relativities for Minimum-Design Construction to FBC 2001, Building Group I (2% Deductible)[1]

| FBC 2001 Construction | | | | | Flat | | Gable | | Hip | |
|---|---|---|---|---|---|---|---|---|---|---|
| Roof Deck | Terrain Exp.[2] | Gust Wind Speed[10] (mph) | Internal Pressure Design[3] | WBDR[4] | No Opening Protection | Opening Protection | No Opening Protection | Opening Protection | No Opening Protection | Opening Protection |
| Other Roof Deck[9] | B | 100 | Enclosed | No | 0.61 | -[5] | 0.54 | -[5] | 0.44 | -[5] |
| | | 110 | Enclosed | No | 0.60 | -[5] | 0.52 | -[5] | 0.43 | -[5] |
| | | ≥ 120 | Enclosed | No | 0.50[6] | - | 0.41[6] | - | 0.38[6] | - |
| | | | | Yes | - | 0.38 | - | 0.34 | - | 0.33 |
| | | | Part. Enclosed | Yes | 0.44 | - | 0.37 | - | 0.36 | - |
| | C | ≥ 120 | Enclosed | Yes | - | 0.23 | - | 0.21 | - | 0.21 |
| | | | Part. Enclosed | Yes | 0.25 | | 0.23 | | 0.23 | |
| | HVHZ | ≥ 120 | Enclosed | Yes | -[8] | 0.23 | -[8] | 0.21 | -[8] | 0.21 |
| Reinforced Concrete Roof Deck[9] | B | Any | Enclosed | No | 0.34 | - | 0.34 | -[5] | 0.32 | -[5] |
| | | | | Yes | - | 0.27 | - | 0.27 | - | 0.27 |
| | | | Part. Enclosed | Yes | 0.30 | -[7] | 0.30 | -[7] | 0.30 | -[7] |
| | C | Any | Enclosed | Yes | - | 0.17 | - | 0.17 | - | 0.17 |
| | | | Part. Enclosed | Yes | 0.19 | -[7] | 0.19 | -[7] | 0.18 | -[7] |
| | HVHZ | Any | Enclosed | Yes | -[8] | 0.16 | -[8] | 0.16 | -[8] | 0.16 |

[1] Table is for buildings built to Minimum Wind Loads of FBC 2001. Buildings built to higher loads should use this table and the adjustments in Table 4-4.
[2] See Figure 2-1 and FBC 1606.1.8.
[3] FBC 1606.1.4.
[4] WBDR = Wind-Borne Debris Region (FBC 1606.1.5 and Section 2.2.1 of this report).
[5] Not applicable to Minimum Load Design in non-WBDR.
[6] This relativity applies to non-WBDR locations.
[7] Not applicable to Minimum Load Design for Partially Enclosed Buildings in WBDR.
[8] HVHZ requires WBD Opening Protection.
[9] No secondary rating factor adjustments to these relativities.
[10] FBC peak gust wind speed corresponding to building location.

the building with opening protection. A builder may consider this when his geographic area of business extends across several wind speed regions, or the builder is attempting to differentiate his product from others in the area. It is also possible to add features that are not required by the building code, such as Secondary Water Resistance (SWR). For these conditions, the relativities shown in Tables 4-1 through 4-3 should be adjusted with factors from Tables 4-4 through 4-6.

To determine the changes in loss relativity for the over-design and/or mitigation cases, six locations with FBC design wind speeds ranging from 100 mph to 150 mph were selected and buildings at each location were re-designed using higher design wind speeds (at an increment of 10 mph) than the minimum requirements. Additional mitigation features, such as opening protection and SWR, were assumed for those buildings as well (if none existed previously). More than 700 cases were determined in this manner and simulated using HURLOSS. The simulation results were then normalized by the results from the minimum design tables (Tables 4-1 through 4-3) to produce modification factors for over-design cases (see Tables 4-4 through 4-6). The column labeled as "Location Wind Speed" in Tables 4-4 through 4-6 lists the minimum design wind speed required by new FBC for each location selected.

These tables show that the biggest factor is the addition of opening protection, which offers up to 70% reduction in loss costs from the minimum design case. Also, buildings in the 100 mph region with no opening protection could benefit by approximately 20% for Groups I and II buildings and 40% for Group III buildings when built to 110 mph wind speed. The benefit could be even higher when higher wind speeds are used in the design.

## Table 4-2. Loss Relativities for Minimum-Design Construction to FBC 2001, Building Group II (2% Deductible)[1]

| Roof Deck | Terrain Exposure[2] | Gust Wind Speed (mph)[9] | WBDR[4] | Enclosed[3] | | Partially Enclosed[3] |
|---|---|---|---|---|---|---|
| | | | | No Opening Protection | With Opening Protection | No Opening Protection |
| Wood Deck[8] | B | 100 | No | 0.56 | -[5] | -[5] |
| | | 110 | No | 0.67 | -[5] | -[5] |
| | | ≥ 120 | No | 0.53[6] | - | - |
| | | | Yes | - | 0.35 | 0.41 |
| | C | ≥ 120 | Yes | - | 0.28 | 0.35 |
| | HVHZ | ≥ 120 | Yes | -[8] | 0.27 | -[8] |
| Metal Deck[8] | B | 100 | No | 0.61 | -[5] | -[5] |
| | | 110 | No | 0.72 | -[5] | -[5] |
| | | ≥ 120 | No | 0.54[6] | - | - |
| | | | Yes | - | 0.32 | 0.39 |
| | C | ≥ 120 | Yes | - | 0.27 | 0.36 |
| | HVHZ | ≥ 120 | Yes | -[7] | 0.29 | -[7] |
| Reinforced Concrete Roof Deck[8] | B | Any | No | 0.43 | -[5] | -[5] |
| | | | Yes | - | 0.19 | 0.27 |
| | C | Any | Yes | - | 0.16 | 0.25 |
| | HVHZ | HVHZ | Yes | -[7] | 0.16 | -[7] |

[1] Table is for buildings built to Minimum Wind Loads of FBC 2001. Buildings built to higher loads should use this table and the adjustments in Table 4-5.
[2] See Figure 2-1 and FBC 1606.1.8.
[3] FBC 1606.1.4.
[4] WBDR = Wind-Borne Debris Region (FBC 1606.1.5 and Section 2.2.1 of this report).
[5] Not applicable to Minimum Load Design in non-WBDR.
[6] This relativity applies to non-WBDR locations.
[7] HVHZ requires WBD Opening Protection.
[8] No secondary rating factor adjustments to these relativities.
[9] FBC peak gust wind speed corresponding to building location.

## Table 4-3. Loss Relativities for Minimum-Design Construction to FBC 2001, Building Group III (2% Deductible)[1]

| Roof Deck | Terrain Exposure[2] | Gust Wind Speed (mph)[9] | WBDR[4] | Enclosed[3] | | Partially Enclosed[3] |
|---|---|---|---|---|---|---|
| | | | | No Opening Protection | With Opening Protection | No Opening Protection |
| Metal Deck[8] | B | 100 | No | 0.88 | -[5] | -[5] |
| | | 110 | No | 0.74 | -[5] | -[5] |
| | | ≥ 120 | No | 0.48[6] | - | - |
| | | | Yes | - | 0.27 | 0.34 |
| | C | ≥ 120 | Yes | - | 0.26 | 0.36 |
| | HVHZ | ≥ 120 | Yes | -[7] | 0.25 | -[7] |
| Reinforced Concrete Roof Deck[8] | B | Any | No | 0.46 | - | - |
| | | | Yes | - | 0.16 | 0.23 |
| | C | Any | Yes | - | 0.16 | 0.28 |
| | HVHZ | HVHZ | Yes | -[7] | 0.16 | -[7] |

[1] Table is for buildings built to Minimum Wind Loads of FBC 2001. Buildings built to higher loads should use this table and the adjustments in Table 4-6.
[2] See Figure 2-1 and FBC 1606.1.8.
[3] FBC 1606.1.4.
[4] WBDR = Wind-Borne Debris Region (FBC 1606.1.5 and Section 2.2.1 of this report).
[5] Not applicable to Minimum Load Design in non-WBDR.
[6] This relativity applies to non-WBDR locations.
[7] HVHZ requires WBD Opening Protection.
[8] No secondary rating factor adjustments to these relativities.
[9] FBC peak gust wind speed corresponding to building location.

**Table 4-4. Modification Factors ($N_i$) for Over-Design and/or Mitigation of New Construction FBC Buildings (Group I)**

| Gust Wind Speed of Location[1] | Gust Wind Speed (mph) of Design[2] | No Opening Protection | | Opening Protection | |
|---|---|---|---|---|---|
| | | No SWR | SWR | No SWR | SWR |
| 100 mph | 100 | 1.00 | 0.92 | 0.53 | 0.48 |
| | 110 | 0.80 | 0.74 | 0.52 | 0.47 |
| | 120 | 0.61 | 0.55 | 0.51 | 0.46 |
| | 130 | 0.57 | 0.51 | 0.51 | 0.46 |
| | 140 | 0.52 | 0.47 | 0.51 | 0.46 |
| | 150 | 0.49 | 0.45 | 0.47 | 0.43 |
| 110 mph | 110 | 1.00 | 0.93 | 0.61 | 0.56 |
| | 120 | 0.73 | 0.67 | 0.59 | 0.54 |
| | 130 | 0.69 | 0.63 | 0.59 | 0.53 |
| | 140 | 0.64 | 0.57 | 0.58 | 0.53 |
| | 150 | 0.59 | 0.53 | 0.54 | 0.49 |
| 120 mph | 120 | 1.00 | 0.92 | 0.78 | 0.70 |
| | 130 | 0.92 | 0.84 | 0.77 | 0.69 |
| | 140 | 0.85 | 0.76 | 0.77 | 0.69 |
| | 150 | 0.78 | 0.70 | 0.70 | 0.63 |
| 130 mph | 130 | 1.00 | 0.92 | 0.81 | 0.72 |
| | 140 | 0.90 | 0.81 | 0.80 | 0.72 |
| | 150 | 0.81 | 0.72 | 0.72 | 0.62 |
| 140 mph | 140 | 1.00 | 0.91 | 0.89 | 0.80 |
| | 150 | 0.91 | 0.81 | 0.79 | 0.69 |
| 150 mph | 150 | 1.00 | 0.90 | 0.86 | 0.75 |

[1] Wind speed for where building is located.

[2] Wind speed that building is designed or mitigated to withstand

**Table 4-5. Modification Factors ($N_i$) for Over-Design and/or Mitigation of New Construction FBC Buildings (Group II)**

| Gust Wind Speed of Location[1] | Gust Wind Speed (mph) of Design[2] | No Opening Protection | | Opening Protection | |
|---|---|---|---|---|---|
| | | No SWR | SWR | No SWR | SWR |
| 100 mph | 100 | 1.00 | 0.93 | 0.46 | 0.40 |
| | 110 | 0.79 | 0.72 | 0.43 | 0.37 |
| | 120 | 0.50 | 0.42 | 0.42 | 0.34 |
| | 130 | 0.44 | 0.37 | 0.42 | 0.33 |
| | 140 | 0.41 | 0.34 | 0.38 | 0.31 |
| | 150 | 0.39 | 0.32 | 0.38 | 0.31 |
| 110 mph | 110 | 1.00 | 0.90 | 0.46 | 0.38 |
| | 120 | 0.58 | 0.48 | 0.44 | 0.34 |
| | 130 | 0.49 | 0.40 | 0.43 | 0.33 |
| | 140 | 0.44 | 0.35 | 0.39 | 0.29 |
| | 150 | 0.42 | 0.32 | 0.38 | 0.28 |
| 120 mph | 120 | 1.00 | 0.75 | 0.70 | 0.50 |
| | 130 | 0.83 | 0.69 | 0.68 | 0.52 |
| | 140 | 0.74 | 0.59 | 0.61 | 0.45 |
| | 150 | 0.69 | 0.53 | 0.59 | 0.44 |
| 130 mph | 130 | 1.00 | 0.81 | 0.77 | 0.56 |
| | 140 | 0.88 | 0.74 | 0.68 | 0.51 |
| | 150 | 0.81 | 0.64 | 0.65 | 0.48 |
| 140 mph | 140 | 1.00 | 0.81 | 0.75 | 0.54 |
| | 150 | 0.91 | 0.74 | 0.71 | 0.54 |
| 150 mph | 150 | 1.00 | 0.85 | 0.77 | 0.54 |

[1] Wind speed for where building is located.

[2] Wind speed that building is designed or mitigated to withstand

**Table 4-6. Modification Factors ($N_i$) for Over-Design and/or Mitigation of New Construction FBC Buildings (Group III)**

| Gust Wind Speed of Location[1] | Gust Wind Speed (mph) of Design[2] | No Opening Protection | | Opening Protection | |
|---|---|---|---|---|---|
| | | No SWR | SWR | No SWR | SWR |
| 100 mph | 100 | 1.00 | 0.90 | 0.30 | 0.23 |
| | 110 | 0.58 | 0.51 | 0.27 | 0.21 |
| | 120 | 0.32 | 0.25 | 0.28 | 0.20 |
| | 130 | 0.28 | 0.21 | 0.26 | 0.19 |
| | 140 | 0.26 | 0.20 | 0.25 | 0.19 |
| | 150 | 0.26 | 0.19 | 0.24 | 0.18 |
| 110 mph | 110 | 1.00 | 0.83 | 0.39 | 0.29 |
| | 120 | 0.50 | 0.40 | 0.39 | 0.27 |
| | 130 | 0.42 | 0.31 | 0.36 | 0.25 |
| | 140 | 0.38 | 0.28 | 0.34 | 0.24 |
| | 150 | 0.37 | 0.27 | 0.32 | 0.22 |
| 120 mph | 120 | 1.00 | 0.79 | 0.69 | 0.53 |
| | 130 | 0.82 | 0.64 | 0.64 | 0.45 |
| | 140 | 0.73 | 0.55 | 0.61 | 0.42 |
| | 150 | 0.70 | 0.52 | 0.55 | 0.39 |
| 130 mph | 130 | 1.00 | 0.89 | 0.69 | 0.52 |
| | 140 | 0.89 | 0.72 | 0.66 | 0.48 |
| | 150 | 0.83 | 0.65 | 0.59 | 0.42 |
| 140 mph | 140 | 1.00 | 0.93 | 0.72 | 0.53 |
| | 150 | 0.94 | 0.75 | 0.65 | 0.46 |
| 150 mph | 150 | 1.00 | 0.96 | 0.67 | 0.51 |

[1] Wind speed for where building is located.
[2] Wind speed that building is designed or mitigated to withstand

Due to restraints in time and available resources, the effects of roof shape on modification factor for over-design and mitigation of new FBC buildings were not investigated in this study. In a companion study, "Development of Loss Relativities for Wind Resistive Features of Residential Structures," roof shape has shown limited effects on the modification factor. Therefore, to simplify the rating process, the modification factors listed in Tables 4-4 through 4-6 can be assumed to be applicable to any roof shape.

To use these tables, one must know the minimum wind speed zone for where the building is located, and also the design wind speed for which the structure was actually designed. For example, if the building is located in Mid Florida Lakes, the minimum wind speed zone for that location is 100 mph, Exposure B. Now, lets say the building was actually designed for 120 mph, Exposure C wind loads, and also has hurricane opening protection and no SWR. For a Group I building with flat roof and other roof deck the adjusted relativity would be a simple multiplication

$$R' = R_{min} \cdot N_i \tag{4-1}$$

where $R_{min} = 0.61$ (relativity for FBC minimum design in Table 4-1) and $N_i = 0.51$ from Table 4-4. This multiplication produces $R' = 0.31$.

## 4.5 Verification Issues for New Construction

FBC Section 1606.17 summarizes the required wind load information that must be shown on construction drawings:

1. Basic Wind Speed
2. Wind Importance Factor and Building Category
3. Terrain Exposure
4. Applicable Internal Pressure Coefficient
5. Design Wind Pressure of Components and Cladding.

With this information and the following additional data (from the drawings or certified by the design professional) one can properly rate the building.

1. Location of Building
2. Wall Construction
3. Roof Deck Type
4. Roof Shape
5. Additional Mitigation Factors (all openings protected, SWR),

All of these items may be summarized on a form to be completed by the design professional and/or verified by a trained inspector.

# 5.0 SUMMARY

## 5.1 General

A research project has been conducted to estimate the effects of wind-resistive building features in reducing hurricane damage and loss to residential occupancies in condominium and tenant buildings located in the state of Florida. The scope of this project has included both new construction to the Florida Building Code 2001 (FBC) and existing construction.

The results of this study are based on the analysis of individually modeled buildings at numerous locations in Florida. Each building has been modeled with a specific set of wind resistive features. The features considered in this project include: roof shape, roof covering, secondary water resistance, roof-to-wall connection, roof deck material/attachment, opening protection, and wall construction. For new construction, the buildings have been designed to the FBC 2001 according to the design wind speed, wind-borne debris region design options, and FBC definitions of Terrain Category. In the wind-borne debris region, designs for both enclosed and partially enclosed structures have been evaluated, per the FBC and ASCE 7-98.

## 5.2 Florida Building Code

The FBC is the central piece of a new statewide building code system. The single statewide code is developed and maintained by the Florida Building Commission. The FBC supersedes all local codes and is automatically effective on the date established by state law. The new building code system requires building code education for all licensees and uniform procedures and quality control in a product approval system.

The FBC 2001 will have a notable impact on new construction in the state of Florida. The code is expected to improve the design and construction of new buildings with regard to wind loads, particularly in the windborne debris regions. The key impacts of the FBC on construction include:

1. A Wind Borne Debris Region (WBDR) that encompasses a significant part of the state.
2. Adoption of ASCE 7-98 Terrain Exposure Categories, with some exceptions.
3. Options for Partially Enclosed and Enclosed Design in WBDR.
4. HVHZ in Miami-Dade and Broward Counties; enclosed design required in HVHZ.
5. Opening protection in WBDR applies to glazed openings in lower 60 ft of building, except that all openings must be protected in HVHZ.
6. Wood structural panels are not allowable as opening protection for multi-family buildings with more than two stories.
7. For buildings taller than 30 feet in the WBDR, the small missile can be used for opening protection for glazing above 30 feet. No opening protection is required by the code for glazing above 60 feet in the WBDR. The exception to this is in the HVHZ, where small missile protection is required above 60 feet.
8. Load combinations for ASCE 7-98 for Allowable Stress Design will result in larger connection sizes for roof-to-wall connections.
9. Chapter 34 requires buildings that are damaged beyond 25% to be repaired according to the FBC. For buildings

damaged beyond 50%, the entire building must be repaired to conform to the FBC.

The wind speed map for the FBC is repeated in Fig. 5-1. The Wind-Borne Debris Region includes all areas where the basic wind speed is 120 mph or greater except for Panhandle area where the region includes areas only within 1 mile of the coast. The FBC adopted the Terrain Exposure Categories of the ASCE 7-98 with a few exceptions. Terrain Exposure C (open terrain) applies to all locations in Miami-Dade and Broward Counties (the High Velocity Hurricane Zone, HVHZ), barrier islands, and all locations within 1500 ft of the coastline. Terrain Exposure B (urban, suburban, and wooded areas) virtually applies to all other locations in Florida.

Discussion of the FBC is contained in Sections 2, 4, and Appendix A. Appendix B contains example sets of FBC design.

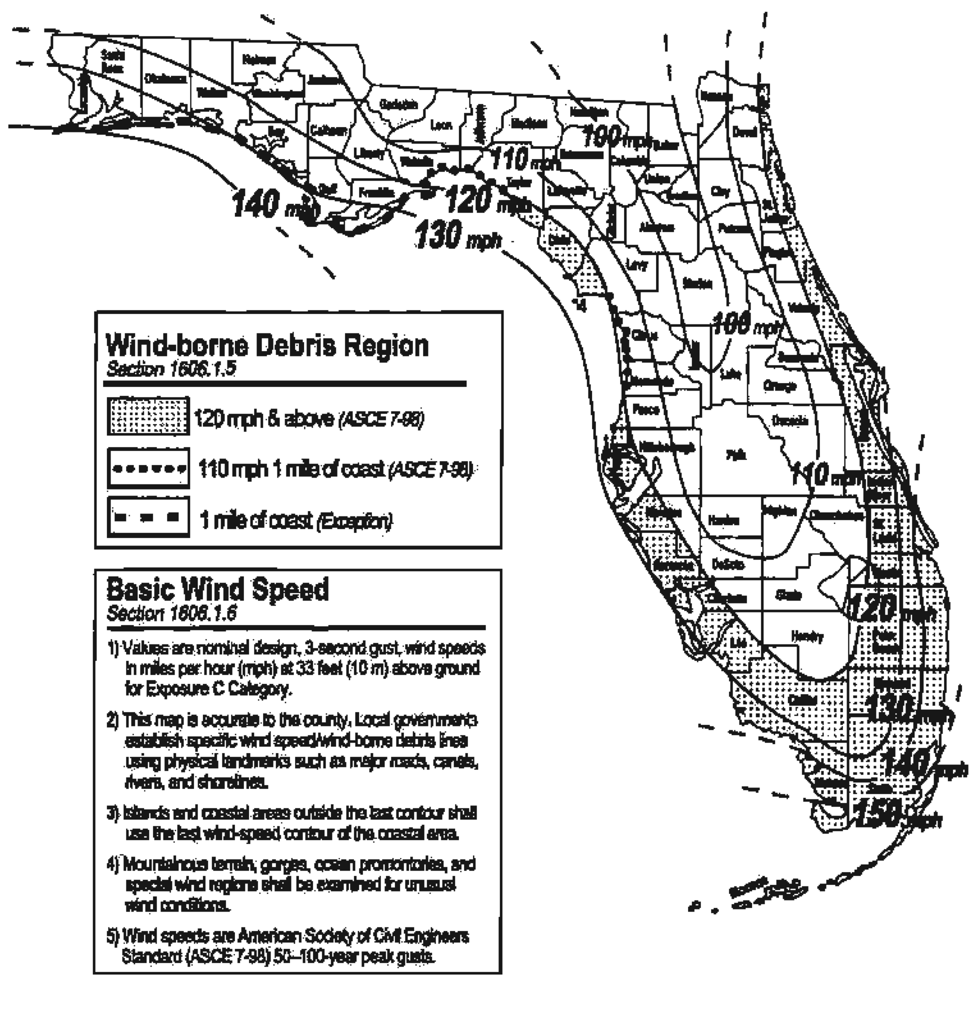



FIGURE 1606
STATE OF FLORIDA
WIND-BORNE DEBRIS REGION & BASIC WIND SPEED

**Figure 5-1. Wind Regions in Florida Building Code**

## 5.3 Methodology

As discussed in Section 2, loss cost relativities were produced at 31 locations in Florida. Each modeled building with each combination of wind resistive features was analyzed at these locations. Building designs for the possible building code eras and controlling windspeeds were produced and analyzed at the relevant locations. The results were then examined to see if location dependent relativities were justified. The variation in loss cost relativity by location was judged to be small. Hence, a single set of relativities is provided statewide for existing construction and by winspeed interval for new construction.

The loss costs were computed using the HURLOSS computer model. Three hundred thousand years of hurricanes were simulated for each building case analyzed in this study. The hurricane windspeed risk used in this study was compared to the national design standard (ASCE 7-98) and the results were essentially identical.

The relativities were compressed by an engineering judgment factor to reflect modeling uncertainties and limitations due to the scope and schedule.

## 5.4 Loss Relativities

The loss costs relativities for existing construction are developed in the form of a set of tables. Three groups of building types were used to classify condominium and tenant buildings. These include:

- *Group I Buildings* – Masonry or wood frame structures, typically 1-3 stories.
- *Group II Buildings* – Steel, concrete, or reinforced masonry frame buildings 60 feet tall or less.
- *Group III Buildings* – Steel or concrete frame buildings that are greater than 60 feet tall.

Three sets of loss relativity tables arc provided to cover these construction groups. Group II and III Buildings are further classified based on the year built for existing construction.

For new construction to the FBC 2001, a single table covers the minimum load designs for each Group. The analysis indicates that there is a small difference in relativity between an enclosed design without opening protection and a partially enclosed design (also without opening protection). Hence, in the tables in Section 4, there is only a small difference in enclosed and partially enclosed designs without opening protection.

Not all of the FBC new construction will be designed and built to just the minimal loads required by the code. Engineers may design buildings to higher loads than the minimum. Alternately, the owner may mitigate the building at a later date with SWR or opening protection. A separate table of modification factors has been developed to handle these cases.

The tables in Sections 3 and 4 have been normalized to a "typical" building, which is a representative building as opposed to the weakest building. The relativity for the central building is one. The Terrain B results are primarily for inland locations and the Terrain C results are primarily for barrier islands and locations within 1500 feet of the coastline.

Opening Protection in these tables mean that all glazed openings (i.e., those with glass or plastic) are protected with impact-rated glazing or shutters. The requirements for the High Velocity Hurricane Zone (HVHZ) are slightly different in that all openings, including doors and garage doors, must be protected with shutters or impact resistant products. The results in Section 4 include protection for all openings that is required in the HVHZ.

The loss relativities are based on total loss costs corresponding to 2% deductibles. The possible allocations of loss to building owner, condominium owner, and tenant, and the associated relativities based on these allocations have not been evaluated in this study. Instead, loss relativities based on total loss, without separate allocations to building, contents, etc., have been used to provide a simple and practical approach for this basic study.

Refer to Sections 3, 4, and Appendix A to fully appreciate the issues associated with implementation of these loss relativities for existing and new construction.

## 5.5 Limitations and Discussion

The following discussion represents the independent opinions of the ARA authors of this report and should not be interpreted as representing views of the State of Florida.

***Building Features Not Considered.*** As described and discussed in Appendix A, there are some key variables not explicitly considered in this study. These include:

1. Complex building plan geometries
2. Multiple level roof geometry or multiple types of roof cover on the same building
3. Parapets
4. Roof top equipment
5. Tile roof coverings (not considered in the modeled buildings)
6. Skylights
7. Porches, balconies, and carports
8. Location of building next to a source of high elevation missiles, such as gravel roof ballast on an adjacent building
9. Variation of opening protection with building height. In this study, an all or nothing approach was used for opening protection. That is, the building openings were assumed to be either protected uniformly or all openings were treated as unprotected. No cases of partial protection were considered nor of changes in level of protection at certain building heights (e.g., 30 ft or 60 ft).
10. Over design of existing Group II and Group III buildings.

While some of these limitations may be of secondary importance, others may have a more noticeable effect on the relativities. These limitations can be considered in future studies with adequate resources and schedule. In the absence of such an effort to quantify these sensitivities, individual users may want to perform additional analyses to expand the set of features and parameters considered in this study.

Other features, such as variations in percent glazing, were treated somewhat in the modeled buildings but were not analyzed as separate classification variables.

***Actuarial Judgments.*** The relativities computed herein do not include any "actuarial" types of adjustments. For example, the quality of the rating data obtained from either the building owner or through an inspection has not been considered.

***Individual Building Rating.*** The scope of this study has focused on specific wind loss mitigation features and relativities on a building-by-building basis. Such relativities, when applied, attempt to capture differences in loss costs for buildings with/without specific wind mitigation features. These relativities will obviously affect insurance rates on a building-by-building basis. However, these relativities are separate from an overall rate increase/ decrease across a book of business.

***Separate Loss Relativities for Building and Units.*** The loss relativity tables presented in this study are based on the total loss costs (building, contents and ALE), without considering possible loss allocation to different owners. This simplified method is practical for this initial study.

As discussed in Section 2, a multifamily residential building and the contents have multiple interest and distinct insurance policies. Each party has its own responsibilities. For example, the condominium association is responsible for a building's structural, exterior, electrical and mechanical components and/or common areas in the building (such as corridors and lobby). Unit owners are responsible for contents, loss of use, and may be responsible for the interior finishes of their units. Since different owners are typically insured under different policies, the approach should be to produce separate loss relativities for condominium association, condominium owner, building owner and tenant. Depending on a building's configuration, the difference in loss relativity for different owners can be significant. For example, the loss costs for condominium owner or tenant are generally more sensitive to the wind resistive features on a building than those of condominium association or building owner. Moreover, location of the unit within the building is also an important quantifier, particularly for Group II and III Buildings. Therefore, additional work is needed to develop relativities for the different interests in multifamily buildings. The results herein provide a basic first step for these types of occupancies within the available resources and schedule.

***Need for Building Stock Distribution.*** This study has not developed data on the building stock distribution of wind resistive features for condominium and tenant buildings. Such a data development effort would have some useful benefits to the state. A public domain source of the frequency of building types and wind resistive features would provide a benchmark to gauge average rating factors that may be estimated by individual insurers. In addition, we would be able to evaluate additional types of construction practices (other than those considered herein) for Group II and III Buildings. The vulnerability of Florida's multifamily building stock could also be estimated based on the developed building stock distribution. Building code issues would also be identified.

***Vulnerability of Roof Edge Systems to Wind Damage -- Existing and New.*** Roof edge systems generally include parapet wall copings, gravel stops, edge fascias and other roof edge termination assemblies, as well as flashings. They are used normally to provide wind/water tightness and aesthetic appearance along the roof perimeter of buildings, particularly buildings with flat or low-slope roofs, often in connection with roof membranes. Post-hurricane surveys have shown repeatedly that failure of roof edge systems is one of the most common phenomena among building envelope components or assemblies. This has been the case even for weak hurricanes. The failure modes are generally buckling and/or peeling off from fasteners under wind uplift loads. The failure of a roof edge system often causes roof membranes being lifted off the roof structure that the roof edge system is designed to protect. This in turn produces damages to other building assemblies and contents due to rainwater infiltration.

Designs of roof edge systems have used the same edge-zone wind uplift design loads as those specified in building codes or standards for other roof components or assemblies that normally have larger tributary areas (or "effective wind areas" as termed in ASCE 7-98). The effective wind areas associated with roof edge systems are inherently very small due to their narrow strip configurations, and their uplift loads are very high owing to their proximity to the roof edge and corner.

Development of an improved roof edge loads standard for the FBC would pay enormous benefits at low costs.

***Failure of Rooftop Equipment.*** The failure of heating, ventilating, air conditioning and other roof-top equipment often occurs in strong wind storms. Not only is the equipment damaged, but occasionally holes in the building envelope are created which allow water entry to interior spaces. This study did not examine this failure mechanism. The FBC now requires all equipment exposed to wind loads to meet the same wind pressure and missile impact standards referenced in Chapter 16 of the FBC. However, this requirement does not appear in early versions of the SBC. The scope of this study should be extended to develop a relativity modification for roof top equipment anchorage.

***Modeling of Missile Environment.*** This study made the assumption that the large missile protection standard was used uniformly on all glazed openings regardless of height. We also used a missile model based on large missiles typical of residential or light commercial areas. However the FBC only requires missile protection in the lower 60 feet with large missile protection devices. Follow-up studies that examine the risk due to gravel missiles, and the associated small missile impact standard are a needed sensitivity analysis.

***Additional Hurricane Damage Data.*** It is recommended that a public domain study be performed on analyzing damage and loss of a sample of Group I, II, and III Buildings after each Category 3 and higher storm that makes landfall in Florida. Data needs to be collected for each storm on several hundred randomly selected buildings that document the construction features and physical damage of each building. When available, the loss claims would be obtained for both building owners and unit owner/renter to individually document the losses for all interests (with insurance company name deleted). With proper analysis of building orientation (important for individual storms) and actual surrounding terrain, validation of loss relativities could be developed.

By repeating this process for several Category 3 or higher hurricanes, improved measures of loss relativity for new and existing construction can be developed and demonstrated. Improvements to the building code and code enforcement may be identified. Because of the nonlinear nature of loss, the many building specific variables involved, and real terrain variations, simplistic efforts that look at a single storm are doomed to give incomplete if not misleading results without an associated analysis effort of building loads, resistances, and physical damage.

***Cost-Benefit Analysis of Possible Improvements to the Florida Building Code.*** The Florida Building Code generally provides for much improved building design and construction in the State. It has certain wind mitigation features at a very modest cost increase. These improvements will reduce future losses in hurricanes. There are several additional areas where code improvements may have large benefits at modest cost impacts. These include: secondary water resistance; wind-borne debris protection for Group II and III buildings, reviewing the partially enclosed option; further improvements to roof coverings and roof edge attachments; improved wind load characterization in tall tree environments; and quantifying tree fall risk, damage, and loss to buildings.

# 6. REFERENCES


ACI 530/ASCE 5/TMS 402-99, *Building Code Requirements for Masonry Structures*, American Concrete Institute, Farmington Hills, MI, 1999.

ANSI/AF&PA, *National Design Specification for Wood Construction* and *Design Values for Wood Construction*, a Supplement to the National Design Specification, American Forest and Paper Association/American Wood Council, Washington, DC, 1997.

Applied Research Associates, Inc., "Florida Commission on Hurricane Loss Projection Methodology," Florida Hurricane Commission, Tallahassee, FL, February 2001.

Applied Research Associates, Inc., "Development of Loss Relativities for Wind Resistive Features of Residential Structures," Prepared for Florida Department of Community Affairs, Tallahassee, FL, Version 2.2, March 2002.

Arya, S.P.S., *Introduction to Micrometeorology*, Academic Press, 1988

ASCE 7-98, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, New York, NY, 2000.

ASTM E 1886-97, *Standard Test Method for Performance of Exterior Windows, Glazed Curtain Walls, Doors and Storm Shutters Impacted by Missile(s) and Exposed to Cyclic Pressure Differentials*, American Society for Testing and Materials, Philadelphia, PA, 1997.

ASTM E 1996-99, *Standard Specification for Performance of Exterior Windows, Glazed Curtain Walls, Doors and Storm Shutters Impacted by Wind-borne Debris in Hurricanes*, American Society for Testing and Materials, Philadelphia, PA, 1999.

Case, P.C., "Wind Loads on Low Buildings with 4:12 Gable Roofs," M.S. Thesis, University of Western Ontario, London, Ontario, Canada, May 1996.

Chouinard, L.E, Liu Chang, and C.K. Cooper, "Model for Severity of Hurricanes in Gulf of Mexico", *Journal of Waterway, Port, Coastal and Ocean Engineering*, Vol. 123, No. 3, pp. 120-129, 1997.

Darling, R.W.R., "Estimating Probabilities of Hurricane Wind Speeds Using a Large-Scale Empirical Model," *Journal of Climate*, Vol. 4, No. 10, pp. 1035-1046, 1991.

Emanuel, K.A., "The Maximum Intensity of Hurricanes", *Journal of the Atmospheric Sciences*, Vol. 45, pp. 1143-1155, 1988.

FEMA, "Building Performance: Hurricane Andrew in Florida," Federal Emergency Management Agency, Federal Insurance Administration, FIA-22 (2/93), December 21, 1993.

Florida Building Code 2001, State of Florida, Tallahassee, FL, 2001.

Florida Windstorm Underwriting Associates, *Manual of Rates, Rules, and Procedures*, Jacksonville, FL, July 2001.

Georgiou, P.N., "Design Windspeeds in Tropical Cyclone-Prone Regions," Ph.D. Thesis, Faculty of Engineering Science, University of Western Ontario, London, Ontario, Canada, 1985.

Hartigan, J.A., "Statistical Theory in Clustering," *Journal of Classification*, Vol. 2, pp. 63-76, 1985.

Ho, T.C.E., "Variability of Low Building Wind Loads," Ph.D. Thesis, Faculty of Engineering Science, University of Western Ontario, London, Ontario, Canada, 1992.

Jarvinen, B.R., C.J. Neumann, and M.A.S. Davis, "A Tropical Cyclone Data Tape for the North Atlantic Basin 1886-1983: Contents, Limitations and Uses," NOAA Tech. Memo. NWS NHC 22, U.S. Department of Commerce, Washington, DC, 1984.

Mehta, K.C. and R.D. Marshall, *Guide to the Use of the Wind Load Provisions of ASCE 7-95*, American Society of Civil Engineers, Reston, VA, 1998.

Powell, M.D., and S.H. Houston, "Hurricane Andrew's Landfall in South Florida – Part II: Surface Wind Fields and Potential Real-Time Applications," *Weather and Forecasting*, Vol. 11, No. 3, pp. 329-349, 1996.

Rosowksy, D.V., and S.D. Schiff, "Probabilistic Modeling of Roof Sheathing Uplift Capability," *Proceedings ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability*, Worcester, MA, pp. 334-337,.

Rosowsky, D.V, and N. Cheng, "Reliability of Light-Frame Roof Systems Subject to Wind Uplift, "Final Report to NAHB Research Center, Coastal Hazards Assessment and Mitigation Program, Clemson University, Clemson, SC, 1996.

Shapiro, L.J., "The Asymmetric Boundary Layer Flow Under a Translating Hurricane," *Journal of Atmospheric Science,* Vol. 40, No. 8, pp. 1984-1998, 1983.

South Florida Building Code, 1984 Edition, Miami, Metropolitan Dade County, FL, 1984.

South Florida Building Code, 1994 Edition, Miami, Metropolitan Dade County, FL, 1994.

South Florida Building Code, 1999 Revisions, Miami, Metropolitan Dade County, FL, 1999.

SSTD 10-99, *SBCCI Standard for Hurricane Resistant Residential Construction*, Southern Building Code Congress International, Inc., Birmingham, AL, 1999.

SSTD 12-99, *SBCCI Test Standard for Determining Impact Resistance from Wind-borne Debris*, Southern Building Code Congress International, Inc., Birmingham, AL, 1999.

Standard Building Code 1997 Edition, Southern Building Code Congress International, Inc., Birmingham, AL, 1997.

Twisdale, L.A., "A Risk-Based Design Against Tornado Missiles," *Proceedings Third ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities*, Boston, MA, April 1979.

Twisdale, L.A., and P.J. Vickery, "Extreme Wind Risk Assessment," Chapter 20, *Probabilistic Structural Mechanics Handbook*, Van Nostrund Reinhold, New York, 1995.

Twisdale, L.A., and P.J. Vickery, "Uncertainties in the Prediction of Hurricane Windspeeds," *Proceedings ASCE Conference on Hurricanes 1992*, Miami, FL, December 1993.

Twisdale, L.A., et al., "Tornado Missile Risk Analysis," Volumes I and II, EPRI NP-769, Electric Power Research Institute, Palo Alto, CA, May 1978.

Twisdale, L.A., P.J. Vickery and J.-X. Lin, "Analysis of Hurricane Windborne Debris Impact Risk for Residential Structures: Part I," to be submitted to the *ASCE Journal of Structural Engineering*, 2000a.

Twisdale, L.A., P.J. Vickery and J.-X. Lin, "Analysis of Hurricane Windborne Debris Impact Risk for Residential Structures: Part II," to be submitted to the *ASCE Journal of Structural Engineering*, 2000b.

Twisdale, L.A., W.L. Dunn, et al., "Tornado Missile Simulation and Design Methodology," Volumes I and II, EPRI NP-2005, Research Triangle Institute, Research Triangle Park, North Carolina, for Electric Power Research Institute, Palo Alto, CA, August 1981.

Vickery, P.J. and L.A. Twisdale, "Wind-Field and Filling Models for Hurricane Wind-Speed Predictions," *Journal of Structural Engineering*, Vol. 121, No. 11, pp. 1700-1709, 1995a.

Vickery, P.J. and L.A. Twisdale, "Prediction of Hurricane Wind Speeds in The United States," *Journal of Structural Engineering*, Vol. 121, No. 11, pp. 1691-1699, 1995b.

Vickery, P.J., J.-X. Lin, and L.A. Twisdale, "Analysis of Hurricane Windborne Debris Impact Risk for Residential Structures: Part II," Final Rep. 5609, Rev. 1, Applied Research Associates, Inc., Raleigh, NC, April 1998.

Vickery, P.J., P.F. Skerlj, A.C. Steckley, and L.A. Twisdale, "Hurricane Wind Field Model for Use in Hurricane Simulations," *Journal of Structural Engineering*, Vol. 126, No. 10, pp. 1203-1221, 2000.

Vickery, P.J., P.F. Skerlj, and L.A. Twisdale, "Simulation of Hurricane Risk in the United States Using an Empirical Storm Track Modeling Technique," *Journal of Structural Engineering*, Vol. 126, No. 10, pp. 1222-1237, 2000.

# APPENDIX A:

# WIND RESISTIVE FEATURES AND LOSS ANALYSIS

# APPENDIX A: WIND RESISTIVE FEATURES AND LOSS ANALYSIS

## A.1 Introduction

This appendix includes three main sections. Section A.2 presents general definitions of the wind resistive features used in the development of the loss relativities for construction in Sections 3 and 4. Section A.3 describes the design work that has been completed on the sample buildings in this study under various building codes, as they relate to the wind resistance of the building. Section A.4 discusses how the computer runs were performed and the results integrated to produce the final relativity tables in Sections 3 and 4. It also presents the basic relativity results from our damage/loss simulations and the methods that have been used to simplify the final tables to those that appear in Sections 3 and 4.

## A.2 Modeled Wind-Resistive Rating Variables

This section generally defines the wind resistive features used in the modeled buildings. This information is intended to provide only general guidelines that can be used by insurers to develop more detailed definitions and procedures for their individual filings.

These variables apply to all three Groups of building types as defined in Section 2.4.2. For Group I buildings which are only "marginally engineered", a selection of typical values for the key physical parameters have been selected for modeling. For Group II and III buildings, which have traditionally received design attention, some of the key variables of the simulation have been determined via design calculations, and thus the relativity tables are presented in terms of wind load design parameters instead of unique physical parameters.

The following sections are the key variables used in the model building simulations. Refer to Section A.3 for a discussion of rating by design parameters

### A.2.1 Building Height

Because of the differences described in Section 2.4.2, the building height has been used as separate rating variable in these studies. Results are presented by building Groups I through III.

### A.2.2 Roof Covering

The most common roof covering for sloped roofs in Florida are composition shingles and tiles. Other roof covering materials used for residential construction include built-up, metal, slate, wood shakes, and single ply membranes. Built-up and single ply membranes are the most common roof covers on flat roofed Groups II and III residential buildings. A key factor in roof covering performance is the method of attachment of the roof covering to the roof deck.

The Florida Building Code 2001 (Section 1504) has material requirements and attachment specifications that are superior to common roof covering building practices in the past. For composition shingles, these requirements include improved self-seal strips and compliance with ASTM D-3161 (Modified for 110 mph). This requirement is commonly referred to as the "110 mph" rated shingle.

The roof covering specifications of the 1994 SFBC also require improved attachment methods and testing to a similar protocol. Therefore, these roof coverings are considered to be sufficiently similar to FBC roof coverings to be classified in the "FBC Equivalent" category in Table 3-1.

The rating of roof covering for existing construction can be achieved by requiring the roofing contractor to certify that a prior installation met the 1994 SFBC or the FBC 2001 requirements. Otherwise, the current roof covering should be rated as non-FBC equivalent. Insurers should remind owners of existing buildings that when they recover their roofs they need to have the contractor certify that the installation meets the FBC 2001, Chapter 15 requirements in order to receive the new roof covering credit.

### A.2.3 Secondary Water Resistance

Secondary water resistance (SWR) is a layer of protection that protects the building if the roof covering fails. SWR was included in the FWUA class plan because of its cost-effectiveness as a mitigation technique. SWR can be applied to wood and metal roof decks for new construction and re-roofing of existing construction.

This mitigation technique is aimed at keeping rain water out of the building once the roof covering fails. Roof coverings often begin to peel off in peak wind gusts ranging from about 70 to 100 mph. Water enters through the space between sections of the wood decks and through the joints of the metal decks. SWR covers these seams and provides for redundant water proofing of the building.

***Wood Roof Decks.*** The most economical way to achieve SWR is to apply Self-Adhering Modified Bitumen Tape to the plywood joints. This self-adhering tape is generically known as Ice & Water Shield or Peel N Seal and is a rubber-like product applied directly to a roof deck to prevent damage from ice dams in northern climates. Here, the product is applied to the outside of a clean plywood/OSB deck prior to application of regular underlayments and roof covering. The most economical use of this product is to use 6" widths as shown in Fig. A-1. This is done when a new roof covering is being put on the building.

Another way to achieve SWR for wood decks is through a foamed polyurethane structural adhesive applied from inside the attic to cover the joints between all plywood sheets. Figure A-2 shows this product installed in an attic. Note that this product is also used to reinforce the connection between trusses and roof sheathing, qualifying for improved roof deck attachment. Structural adhesives that meet AFG-01 should not be confused with foamed insulating products.

The verification of SWR must be done at the time of application since, once covered, it is difficult to verify. The foamed structural adhesive applied from inside the attic, however, is readily verified with an attic inspection. Roofing contractors should complete a form to provide certification for the owner in order to receive this credit. Education of contractors is needed since the sealing of the plywood joints is a relatively new concept. If not carefully communicated, roofing contractors may incorrectly assume that the underlayment or hot-mopped felts are SWR. These standard roofing applications do not qualify for SWR because they may be blown off the roof deck at high wind speeds. In contrast, off-the-shelf self-adhering bitumen tape has been tested to negative pressures of over 150 psf without failure of the SWR strips.

***Metal Roof Decks.*** The concept of SWR can be applied to metal roof decks as well, provided that tar is used to cover any perforations in the deck associated with mechanical attachments of the deck to the underlying joist structure.

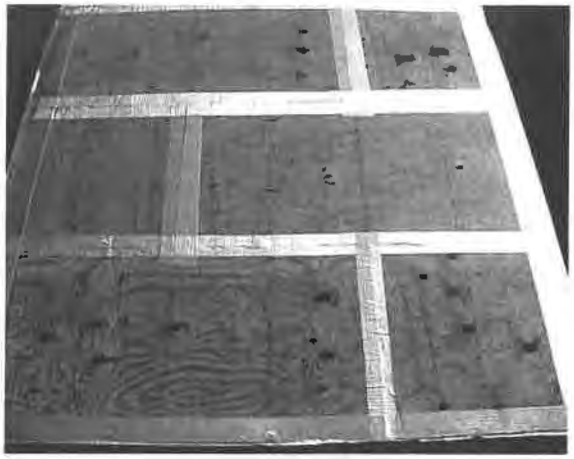


**Figure A-1. Self-Adhering Modified Bitumen Strips Applied to Plywood Joints of Roof Deck**

**Figure A-2. Sprayed on Structural Adhesives to Seal Plywood Joints (SWR) and Strengthen Roof Deck Attachment**

### A.2.4 Roof-to-Wall Connection

The roof-to-wall connection is another critical connection that keeps the roof on the building and acts to transfer the uplift loads into the vertical walls. This connection is key to the performance of the building due to the large negative pressures acting on the roof.

***Wood Roof Frame.*** A common connection detail in non-hurricane prone areas is the toenail, where approximately 3 nails are driven at an oblique angle through the rafter and into the top plate. An example of a toenail connection is shown in Fig. A-3.

There are several manufacturers of metal connectors for hurricane uplift connectors and each company has a fairly wide line of products. For practical purposes, a classification is used herein to distinguish the uplift capacity of these connections based on connector type. The most important feature of any of these connectors, other than toe nails, is that the fasteners used to transfer the loads from rafter/truss to strap to top plate or side wall are always loaded in shear (perpendicular to the nail direction), or the strap is embedded into the bond beam of the masonry wall. Proper installation is critical to connector performance.

Some of the older straps in Florida are simply strips of galvanized metal that were pounded into shape on site to perform the same functions as the straps shown here. These galvanized straps were often 1″ by 1/8″ thick pieces of galvanized steel. If these straps are installed correctly and are not compromised by corrosion, they will perform adequately.

Our analysis for loss relativities has evaluated how four levels of roof-to-wall connections affect loss costs for Group I buildings (Table A-1). The uplift resistance capacities are mean ultimate values based on tests results. By providing the ultimate capacities used in this study, we are indicating what actual values were used in the loss relativity calculations. The ultimate values are distinctly different from the design value of the connection. For example, a 386 lb rated clip has an ultimate capacity of about 866 lbs.

We offer the following general descriptions of these connections (see Fig. A-4):

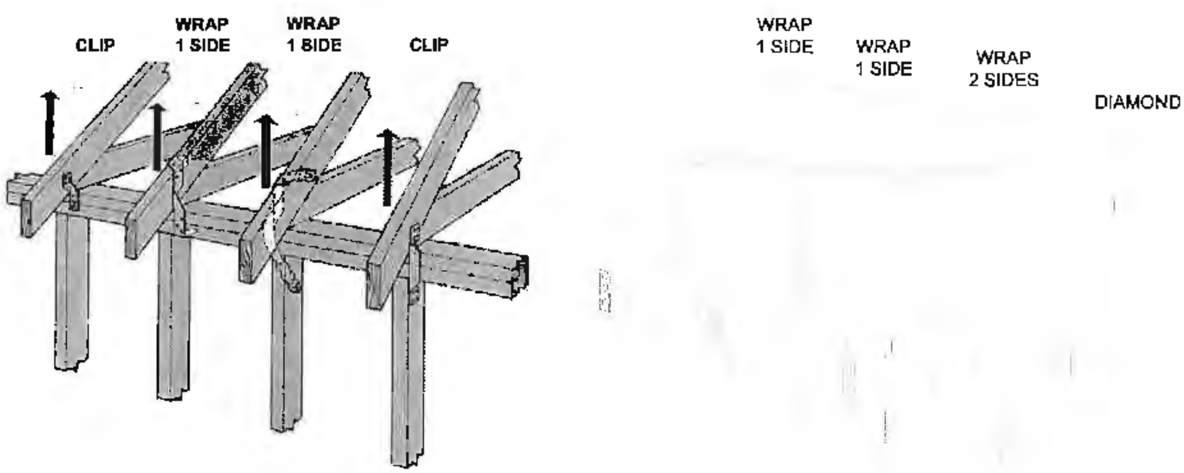
- **Clips and Diamond Connectors:** Clips are defined as pieces of metal that are nailed into the side of the rafter/truss and into the side of the top plate or wall

**Figure A-3. Example of a Toenail Connection Used for Rafter-to-Top Plate Connection**

**Table A-1. Roof-to-Wall Connections Analyzed for Loss Relativities**

| Description | Typical Design Strength* (lbs) | Mean Ultimate Strength Used in Calculations (lbs) |
|---|---|---|
| Toe Nail (3-16d) | 185 | 415 |
| Clip | 386 | 866 |
| Wrap | 535 | 1200 |
| Double Wrap | 891 | 2000 |

* Includes 60% increase for wind loading




**(a) Wood Frame** **(b) Masonry**

**Figure A-4. Typical Hurricane Roof-to-Wall Metal Connector**

stud. The metal does not wrap around the top of the rafter/truss, and the clip is only located on one side of the connection. The approximate design capacity of this type of strap is in the order of 400-500 lbs uplift. The approximate design uplift capacity for two clips is 800 lbs. A diamond is a piece of metal that has a slot in the middle to accept the rafter, and nails to the outside edge of the top plate. It has a design uplift capacity of approximately 500 lbs.

- **Straps: Wrap 1 Side and Wrap 2 Side:** The wrap style straps are attached to the side and/or bottom of the top plate and are nailed to the rafter/truss. Straps that are wrapped on both sides have double the capacity of a single strap.

Verification of the type of roof-wall connector requires an inspection for accurate building ratings.

***Metal Frame.*** For buildings with steel roofs, the roof is usually constructed using open web steel joists, with a welded connection to the wall frame. These connections are designed according to the applicable building code for year built. Hence, the classification for metal frame buildings is based on year built.

***Concrete Frame.*** The roof-to-wall connections for concrete deck and frame are designed according to ACI 318. These buildings generally do not ever fail under wind loading. The differences in relativities for reinforced concrete buildings is due to the design loads for the openings and the type of roof cover.

### A.2.5 Roof Deck Material and Attachment

The performance of the roof deck is of critical importance in keeping hurricane losses to a minimum. It usually only takes the loss of a small portion of the roof deck before the losses for the building become substantial. Rain enters the building and produces water damage to the interior and contents.

#### A.2.5.1 Wood Decks

Roof decks for residential occupancies in single family buildings and buildings with 1-4 units are typically constructed with plywood, OSB, dimensional lumber, tongue and groove boards, or batten.

The most common roof deck types are plywood and Oriented Strand Board (OSB) decks. Prior to the availability of plywood, the most common roof decking material was dimensional lumber or tongue and groove (T&G) board. Dimensional lumber or T&G are usually 4″ to 8″ wide boards that are nominally 1″ thick (¾″ actual thickness) and are laid in a fashion that is parallel to the ridge or diagonal to the ridge. These roof decks are fastened by at least two nails per truss/rafter connection. Because of the inherently large number of nails in dimensional lumber or T&G, the uplift capacity is generally far greater than typical plywood/OSB decks.

By far the most important feature of roof decks is the attachment to the framing, which is usually achieved by nail fasteners. Nail size, type, spacing, and penetration depth into the truss or rafters determines the uplift resistance of the deck. The difference in uplift capacity of 8d (2½″) nails at a typical nail spacing and 6d (2″) nails at the same spacing is a factor of about two times stronger, which makes a significant difference in deck performance in hurricanes.

The thickness of the deck material is important primarily in the determination of the penetration depth of the nail into the truss/rafter. Prescriptive building codes specify longer nails for thicker decks (see Table A-2). Thicker decks have an added advantage of

**Table A-2. Nailing Patterns from Standard Building Code**

| Typical Roof Sheathing Nailing Pattern – Non-High Wind Zones (SBC 1997) | | | |
|---|---|---|---|
| Thickness of Sheathing | Attachment Size | Edge Spacing | Field Spacing |
| ½″ or less | 6d nails | 6″ | 12″ |
| 19/32″ and up | 8d nails | 6″ | 12″ |
| Typical Roof Sheathing Nailing Pattern – High Wind Zones (SSTD 10-93) | | | |
| Thickness of Sheathing | Attachment Size | Edge Spacing* | Field Spacing |
| 15/32″ and up | 8d common nails | 6″ | 6″ |

* At gable ends, sheathing nails should be installed at 4″ on center.

adding additional weight to the roof, which helps to resist whole roof failures. However, thicker decks by themselves do not make a notable difference for deck attachment failures as local pressures govern these. The effect of deck thickness is therefore relatively minor and has not been analyzed in this study.

For existing construction, the only practical way to determine deck type and fastener type and spacing is by a trained inspector going into the attic.

We have analyzed roof deck attachments for the following cases:

Level A. Plywood/OSB nailed with 6 penny common nails at 6″ spacing on the edge and 12″ in the field on 24″ truss spacing. This provides for a mean uplift resistance of 55 lbs per square foot.

Level B. Plywood/OSB nailed with 8 penny common nails at 6″ spacing on the edge and 12″ in the field on 24″ truss spacing. This provides for a mean uplift resistance of 103 lbs per square foot.

Level C. Plywood/OSB nailed with 8 penny common nails at 6″ spacing on the edge and 6″ in the field on 24″ truss spacing. Within 4′ of a gable end the nail spacing is 4″. This provides for a mean uplift resistance of 182 lbs per square foot for non-gable end locations and 219 lbs per sq foot for gable end locations.

The panel uplift resistances given above are based on a combination of experimental data obtained from individual nail withdrawal tests and laboratory uplift tests performed using full sizes (4′ by 8′) sheets of plywood and OSB. Note that the uplift resistance of a panel is dependent upon the species of wood of the underlying truss or rafters and the moisture content of the wood. Decks attached with screws and or adhesives should be rated according to the equivalent uplift resistance of these attachments using the categories above.

Based on the RCMP and FWUA inspections in Florida, more than about 60% of the existing Group I roof deck/attachments will be superior to Level A (6d nails at 6″/12″ spacing).

There are many technical issues that affect the proper rating of the roof deck, including a great variety of available nail sizes, nail penetration depths, the consideration of missed nails, etc. Proper inspection guidelines and training are essential to determining the deck attachment of existing buildings (see Fig. A-5). Without proper training/retraining, roof deck attachment ratings will likely have significant classification errors, possibly greater than 30%.

Batten deck is a system where boards are laid perpendicular to the rafters and spaced apart from each other. This deck forms the basis for which to install wood shakes or wood shingles. There is no continuous deck in this roofing system. Batten decks with wood shakes have not been analyzed separately in this study. An interim recommendation is to use Roof Deck Attachment Level A.

### A.2.5.2 Concrete Roof Deck

There are many mid-rise and high-rise multi-family residential buildings constructed with concrete roof decks. When these building are equipped with wind-borne debris impact resistant opening protection, they are extremely resistant to building failures. Damage to the building will largely consist of damage to the wall finish and roof covering (if any). The hurricane loss costs are therefore reduced dramatically. Although many concrete roof decks on engineered buildings may have perforations for roof drains, or HVAC equipment that may produce a significant opening if they fail, the effect of roof deck penetrations has not been considered in this study.

A reasonable requirement for this type of construction is that the roof deck be designed and constructed in accordance with the provisions of ACI (American Concrete Institute) 318, with engineered connections to the wall frame.

### A.2.5.3 Metal Roof Deck

A significant number of multifamily residential buildings have flat roofs that are constructed using metal deck on open-web steel joists. The variety of metal decking systems is substantial. Different deck materials, dimensions, fastener types, and installation procedures are used to fit the particular needs of a project. Therefore, it is almost impossible to investigate all possible combinations. Rather, it is intended herein to model a selection of design details considered to be the most commonly used metal decking systems in the

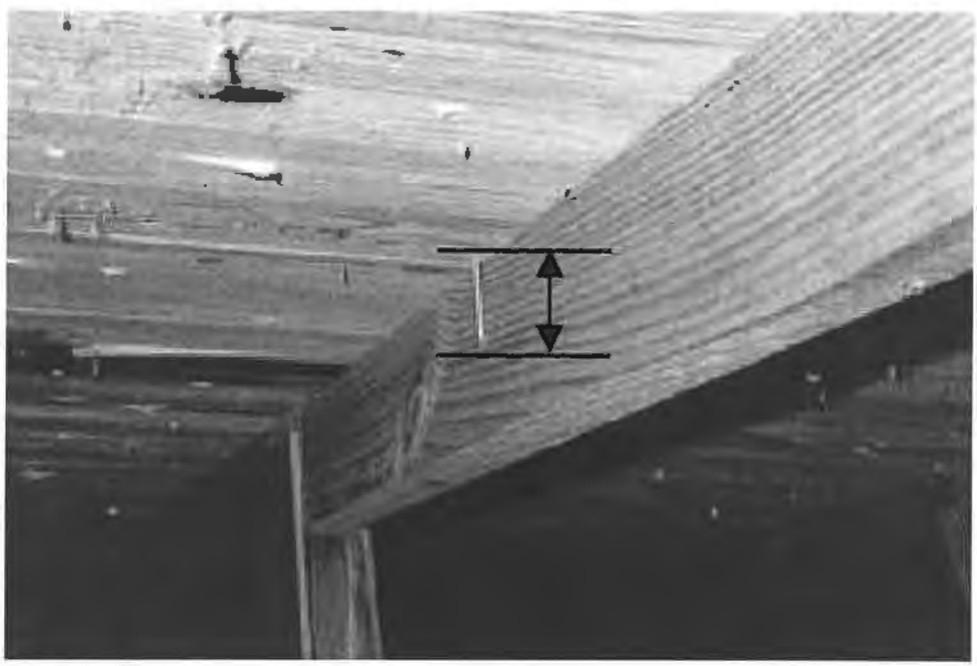


**Figure A-5. Roof Deck Attachment Rating Requires an Attic Inspection**

field. These cases were designed according to various wind speeds that are valid for the design era considered.

Metal deck is typically made from cold forming structural grade sheet steel that shall conform to ASTM Designations A611 Grade C, D, or E (for painted deck) or A653 Grade 33, 40, 50 or 80 (for galvanized deck) (USD, 1997). The minimum yield strength of the steel is 33 ksi. The metal deck can be categorized into 4 types (i.e., A, B, F, and N) according to different profiles of the ribs. Standard deck width varies from 12″ to 36″ with an incremental of 6″ (the length may vary depending on the spacing of bar joists or purlins). Typical thickness of the metal deck is 16, 18, 20, and 22 gage.

The metal decks are typically attached to the building frame with arc puddle welds, self-drilling screws, and powder-actuated or pneumatically driven pins. Sheet to sheet fastening is done with screws, button punching (crimping), or welds. The deck is typically end-lapped a minimum of 2″ and shall occur over supports. The minimum end bearing is 1-1/2″.

In this study, screw type of fastener has been assumed. Screws are typically #12s or ¼-in diameter when fastening the roof deck to structural members. Sheet to sheet connections (also known as stitch connections) typically use self drilling #8 to ¼-in diameter. Screws are assumed to be valley-fixed.

### A.2.6 Roof Shape

Roof shape refers to the geometry of the roof and not the type of roof covering. There are many common roof shapes in residential construction. Gable, hip, and flat are the most common for Group I buildings. Dutch hip, gambrel, mono slope, and many shape combinations are possible for these structures. Gable roofs have vertical walls that extend all the way to the top of the inverted V, and are very common throughout Florida. A hip roof has sloping ends and sloping sides down to the roof eaves line. Predominant roof shapes vary by region within the state.

The Groups II and III buildings have been modeled with flat roofs. Flat roof shapes dominate the construction for these types of buildings. Parapets and other geometrical variations on flat roof buildings have not been considered.

This study has not attempted to quantify the effects of complex roof shapes, including architectural gables, combination shapes, etc.

Insurance classification procedures for roof shapes are best developed with many example photos and supporting discussion/rules to ensure accurate ratings. Because the relative difference in hurricane losses for roof shape is significant, roof shape ratings should be done as accurately as possible.

### A.2.7 Openings

Openings in the wall and roof include windows, doors, sliding glass doors, skylights, and garage doors. Gable end vents and other roof vents are not considered openings for purposes of this study. Openings are vulnerable to wind-borne debris impacts in hurricanes and other windstorms. Typical single and double strength glazing are easily broken by impact from lightweight debris that is generated from roof covering failures during high winds. In addition, heavier debris, such as roof tiles, 2″ by 4″ wood members, and plywood will easily penetrate openings that are not protected by impact resistant products.

The protection of openings is perhaps the greatest single loss mitigation strategy for a building. The reason for this is that once a window or door fails, the pressure inside of the structure increases due to the breach in the building envelope. The positive pressure inside of the building produces an additive load on the building envelope. The increase in load can be up to twice the loads the building experiences without a breach of the envelope. This approximate doubling of the load can easily put the roof, other windows, doors, in an overload situation. The result is often additional failures that occur after the original opening fails. This type of failure sequence has become a well-documented phenomenon in the wind engineering literature since the 1970s. Unfortunately, the protection of openings for debris impact has only recently made it into certain design standards and building codes. Hence, many buildings remain vulnerable to debris impact failures of unprotected openings.

The first building code to adopt protection requirements in the United States was the South Florida Building Code in 1994. The testing protocol in this code requires the protection device to withstand impacts by 2 by 4 studs followed by pressure cycle loading. The Standard Building Code's SSTD-12 has similar reqnirements. In 1999, the ASTM also came out with a debris impact standard (E 1996) and test (E 1886). These standards include requirements for both wind pressure and debris impact. Opening protection products manufactured before 1994 would not have been tested to these standards. Figure A-6 shows an example of opening protection with the Miami-Dade County sticker showing product compliance with test standards.

There are many untested opening protection products that bavc been installed in Florida both prior to and after the development of the impact/pressure cycling standards. In general, these products provide some protection for pressure and missile impact, but there is no practical way to quantify all the possible variations in debris impact and pressure cycling resistance. The FWUA class plan has an "Ordinary" protection level based on ASCE 7-88 wind pressure design that provides an intermediate level of protection between the Miami-Dade standard and no opening protection. This protection level was not simulated in this study.

In the case of the Group II and III buildings, all windows were protected to the same level (all with the FBC large missile protection) with the assumption being that uniformity in the appearance of a shuttered building would require that the windows above 30 ft be protected to the same level as windows below 30 ft.

### A.2.8 Wall Construction

The most common two types of wall construction used for Group I buildings are wood frame, masonry, and combinations of the two. The different construction materials are important for fire resistance considerations, but are less important for wind resistance. Masonry walls are further distinguished by whether or not there is steel reinforcing to carry vertical and horizontal loads.

*Frame* construction is composed of a stick frame made from wood or metal studs and is often sheathed with plywood or Oriented Strand Board (OSB) upon which an exterior finish is installed.

*Masonry* construction is built from Poured Concrete, Insulated Concrete Forms (ICF) or Concrete Block Masonry Units (CMU's) which may be left unfinished, stuccoed, or have a veneer system hung from the masonry units.

*Reinforced Masonry* construction has exterior walls constructed of masonry materials that are reinforced with both vertical and horizontal steel reinforcement and are relied upon for structural stability. It is important that the vertical reinforcement is fully grouted in the hollow cells of CMU, and that horizontal reinforcement be fully grouted in specially formed units. Tilt-up or poured concrete wall units will be reinforced with reinforcing steel both vertically and horizontally.

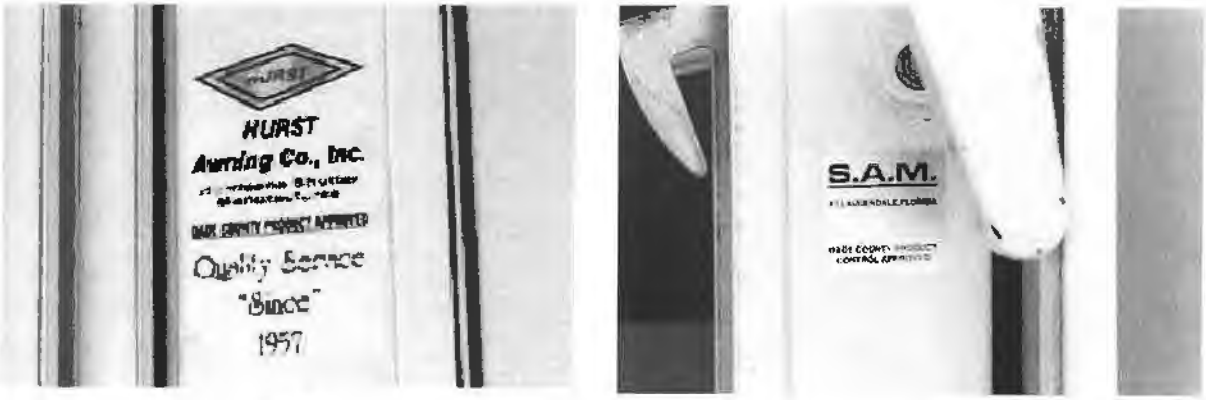



**Figure A-6. Two product approval sticker on accordion shutters indicating that they meet Miami-Dade County impact resistance and wind pressure load standards. These labels contain the words "Dade County Product Approved" or equivalent.**

*Steel or Concrete Frame* consists of engineering frame construction. Groups II and III buildings fall into this category.

### A.2.9 Wall-to-Foundation

Foundation failures from wind forces alone are very rare. Typically, foundation failures associated with hurricanes occur when the surge from the water damages the foundation and structure.

Groups II and III buildings are engineered structures and have adequate wall-to-foundation strength. The following discussion applies to Group I buildings.

Typical foundations include the following, as shown in Fig. A-7:

- Crawl space (Stem Wall)
- Basement
- Slab on Grade with Stem Wall
- Monolithic slab
- Piles
- Piers/Posts

A crawl space is a perimeter foundation that creates an enclosed under-floor space that is not habitable. The perimeter foundation is typically a continuous footing with a stem wall that is attached to the wall/flooring structure of the building. The interior area in a crawl space may or may not extend below grade. Alternatively, a basement foundation is a wall foundation that extends below grade and encloses an area that may be used for living space or storage.

A slab on grade foundation with a stem wall is a concrete floor that is supported directly by the soil, and an independent stem wall that supports the weight of the building. A monolithic slab is a concrete floor that has an integrated footing that supports the weight of the building.

Pile foundations are necessary when the weight of the building must be transmitted to a deeper soil layer that is more stable, or when the structure must be elevated above required flood elevations. Pier or Post foundations are sometimes an economical alternative to stem wall perimeter foundations. These foundations may or may not have bracing between

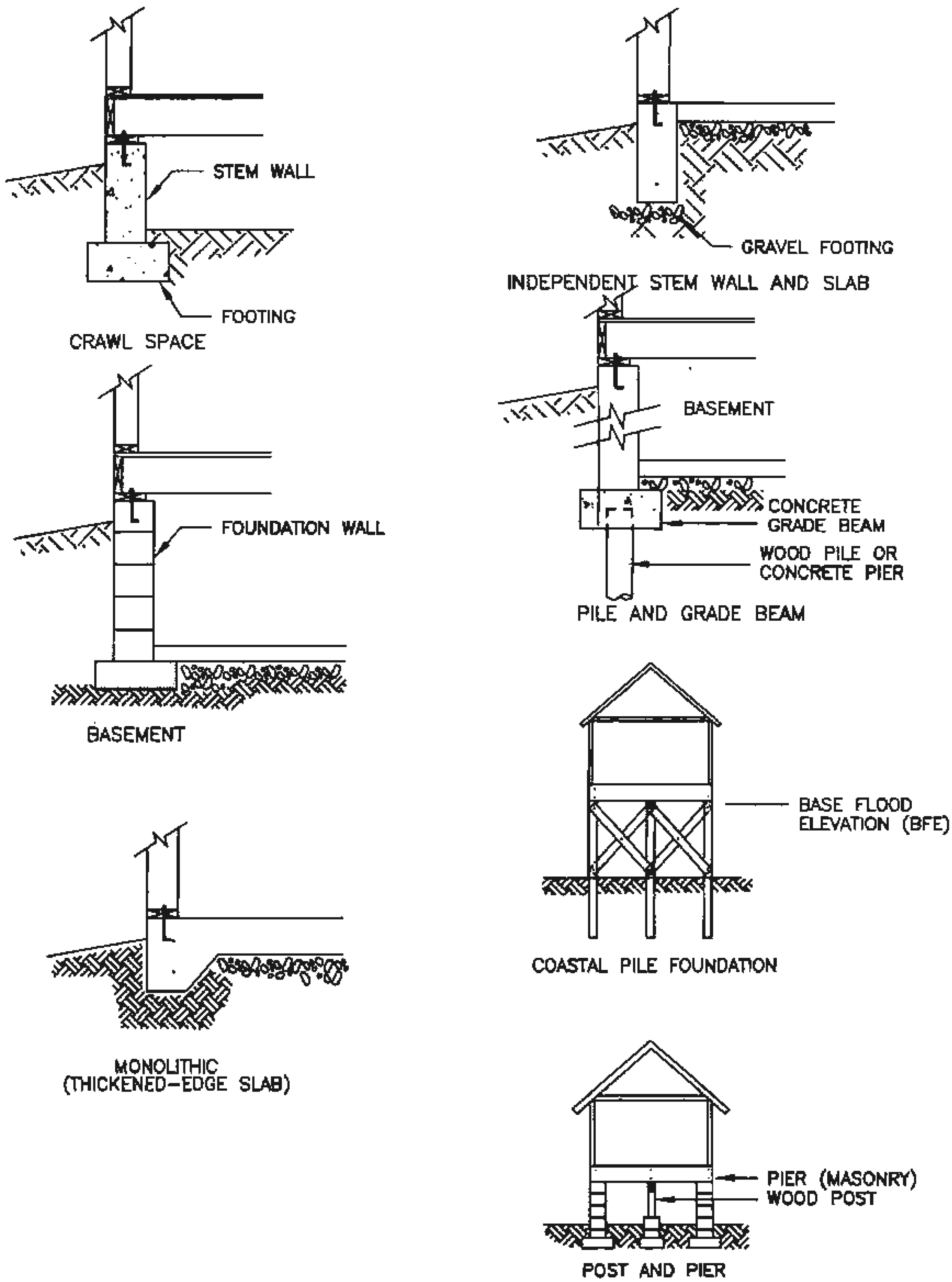



**Figure A-7. Typical Foundation Types in Group I Residential Construction (adapted from Residential Structural Design Guide, 2000 Edition, US. Dept of Housing and Urban Development, March 2000)**

posts/piers depending on the height of the post/pier compared to its width. There may also be bracing or in-filled masonry walls between the posts and piers to resist lateral loads. Note that pile foundations are typically much deeper than post/pier foundations.

Inspections of foundation attachments are not practical for common slab-on-grade construction. Inspections of stem wall foundations require access through a crawl space. Because of these issues and the fact that foundation failures are very rare for hurricane winds (and, if they do occur, the building is usually significantly damaged from other failures), we have classified foundations into:

1. Restrained: Foundations are assumed to have sufficient horizontal and vertical restraining forces unless classified as unrestrained.
2. Unrestrained: Buildings on posts, piles, or concrete blocks that rely solely on gravity and friction forces for resistance to uplift and lateral loads.

The previous study of single-family houses evaluated these two general classes of foundations for two failure modes – sliding of the building off the foundation and overturning of the entire building (i.e., the wind lifts the building up off the foundation).

Almost all site-built buildings will qualify as restrained. Building codes and inspections of buildings confirm that there is almost always an attachment mechanism that provides suitable uplift and lateral resistance, especially when the building weight is also considered.

### A.2.10 Terrain

Terrain and the built environment significantly influence the pressure loads and debris impact loads on a building. The correct modeling of terrain (as defined by the aerodynamic roughness length, $z_o$) is one of critical importance in the prediction of wind loads, wind damage and, hence, wind loss. The surface roughness length, $z_o$, is a function of the density and height of the objects on the ground, including the buildings themselves and vegetation (i.e., trees). In areas of moderate to heavy tree density, the effect of the trees on the wind speeds near the ground can be as important as the surrounding building characteristics. An awareness of the importance of trees in the estimation of the surface roughness has prompted a change in the new wind loading provisions in the United States (ASCE 7-98), which now provides a methodology for the building designer to estimate the surface roughness taking into account the effect of trees.

The wind-borne debris environment depends on the location and type of adjacent buildings. Most residences are in suburban terrain with other low-rise structures. Buildings facing open fields and water are exposed to higher wind speeds and have higher pressures. In South Florida, the trees are shorter than those in North Florida and the surface roughness is correspondingly different.

Terrain is treated as a rating variable in this study for existing construction in the following manner:

1. Terrain Category B (Inland): All existing buildings not on a barrier island nor within 1500 ft of the mean coastal high water line.
2. Terrain Category C (Coastal): All existing buildings on a barrier island or within 1500 ft of the mean coastal high water line.

This classification basically follows the terrain exposure categories specified in the Florida Building Code (Section 1606.1.8) for new construction. While this is a simplified representation, it serves to capture the significant difference in loss costs and loss costs' relativities for buildings situated in highly vulnerable coastal locations.

## A.3 Wind-Resistive Rating Variables for Designed Buildings

When compared to the Standard Building Code, the number of unique wind design cases in FBC increases as designers consider terrain, WBDR, and internal pressure. The following sections describe these new features and how they affect the wind load calculations in FBC. The design techniques described here apply to each of the three building groups in this study.

ARA has performed design calculations for wind loads on various components of each of the three building groups in this study. Design calculations were done for three eras of building codes. The following sections show how certain key components vary with each design combination of wind speed, exposure, and internal pressure assumptions.

For Group I buildings, the following items that affect relativities were examined:

- Wood Roof Deck Nailing Pattern on Flat Roof
- Wood Roof Deck Nailing Pattern on Gable Roof
- Wood Truss Roof Wall Tie Down on Flat Roof
- Wood Truss Roof Wall Tie Down on Gable Roof
- Window and Door Design Pressure

For Group II buildings:

- Roof Deck Nailing Pattern on Flat Wood Deck
- Roof Wall Tie Down of Wood Truss on Flat Roof
- Window and Door Design Pressure
- Metal Deck design
- Metal Joist Design

For Group III buildings:

- Metal Deck design
- Metal Joist Design
- Window and Door Design Pressure

### A.3.1 Evolution of Wind Loading in Previous Building Codes

ARA has researched various items related to wind resistance from the Standard Building Code series from 1946 to the present version of the code. The research has indicated that there are four basic eras of building codes to consider for wind resistance in the case of Florida. Those eras are from:

A. 1946 to 1976,

B. 1976 to 1982

C. 1982 to 2001

D. 2001 onward (FBC)

We have prepared design calculations for the last three major wind design eras, represented by the versions of the SBC in 1976 for era B, SBC 1988 for era C, and the FBC for era D.

From 1946 until 1976, wind loads were given very little attention in the building codes. The wall wind load was specified as a function of height alone. It was increased if the building was near the coast, and the roof load was generally a fixed ratio of 1.25 times the wall wind load at the same height. There was no wind speed map, so the wind loads were the same regardless of where the building was located. Buildings from this era are treated in this study in two different ways depending on the building height class. The Group I building runs are based on a range of physical parameters that will cover the range of the construction details for this era. The Group II and III designs are considered to be the same as the designs produced by the 1976 building code because the wind load calculations from the

1976 era essentially result in a set of minimal fastening details.

In 1976, the SBC introduced a wind speed map, recognized that wind pressure is a function of wind speed and height, and adopted the use of shape factors to apply different loads to different shapes of structures and different parts of structures. There was some acknowledgement of the difference in loading primary members versus the structural frame. This concept is the precursor to today's concepts of Components & Cladding and Main Wind Force Resisting loading patterns.

In 1982, a key change in the wind load standards occurred in the recognition of the difference in wind loads on low-rise versus high-rise buildings was manifested in the adoption of separate loading patterns and coefficients for low rise buildings. This was the time when the SBC adopted the Main Wind Force Resisting Systems (MWFRS) and Components & Cladding (C&C) techniques of today's codes. In addition the decreasing correlation of wind loads with increasing area was adopted through the Components and Claddings graphs. Edge zones on roofs and walls recognized that high local pressures and suctions occurred at windward edges.

Later versions of the SBC have included additional features to further refine the loading criteria for different shapes of buildings such as hip roofs, a more complete range of roof slopes for C&C and MWFRS loads, and wind loading coefficients for specific roofing products.

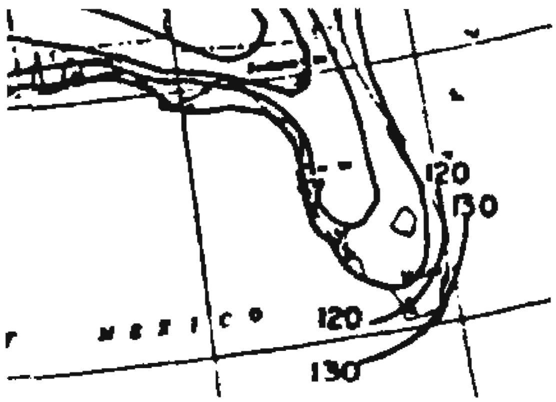
All of these codes used the Fastest Mile wind speed until the adoption of the FBC, which has converted to the 3-second gust wind speed.

In 1994, the Standard Building Code officially adopted the ASCE 7-88 standard for high-rise buildings (greater than 60 feet in height) and dropped the previous provisions for that class of building height. The ASCE 7-88 wind loads are slightly higher than those found in SBC up until that point. Even though the official adoption of the ASCE 7-88 is relatively late in this era (1982-2001), in this study, we have assumed that the majority of Group III buildings were designed according to ASCE 7-88. Schedule constraints did not allow an examination of the differences to be made between ASCE 7-88 and the high-rise version of SBC 88. This examination may reveal the need for an additional era for high-rise buildings. In the absence of this information, those buildings that were designed according to the SBC high-rise provisions during the 1982-01 era should use the relativities for the 1976-82 era.

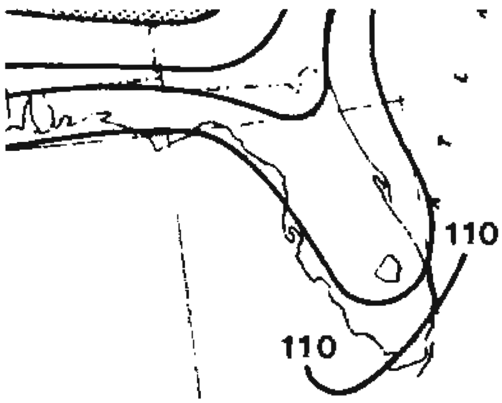
In 2001, Florida has adapted a building code based on the International Code series (which is a evolution of the Standard Building Code, the Uniform Building Code and the BOCA code) and the South Florida Building Code.

#### A.3.1.1 Comparison of Wind Speed Maps in SBC

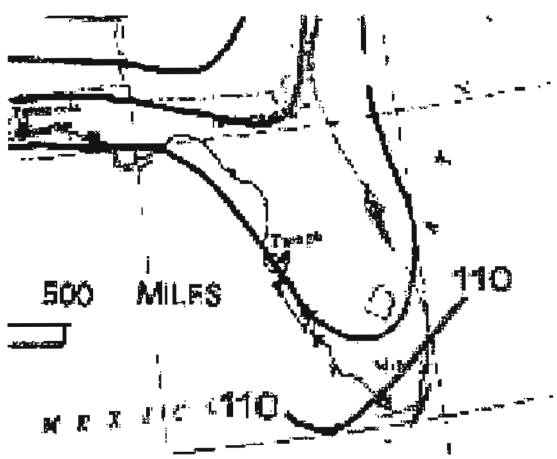
Close examination of the wind speed maps in the SBC76 code versus the SBC88 code (Fig A-8) indicates that there is a wider range of design wind speeds in the older version of the building code than the SBC88 version. The SBC88 code has contours for 90, 100 and 110 mph (fastest mile) in the state of Florida, whereas the SBC 76 code has wind speeds that can range from 80 – 130 mph (fastest mile). Thus the design tables presented in this section show only valid design cases according to each of the code eras. We have also chosen to present the design results in Section A.4 by aligning equivalent wind speeds together - that is, comparing a design for 110 mph fastest mile to a 130 mph 3-second gust design.




**(a) SBC 1976**




**(b) SBC 1988**




**(c) ASCE 7-88**

**Figure A-8. Fastest-Mile Design Wind Speeds for SBC 1976, SBC 1988, and ASCE 7-88**

### A.3.1.2 Using Wind Speed and Design Exposure as Rating Variables for Group II and III Existing Construction

The loss relativity tables in Section 3 (Tables 3-3 through 3-6) for Group II and Group III buildings include a classification variable called "Fastest Mile Design Wind Speed". These buildings represent structures that have been designed by a Professional Engineer. The member sizes and connections have been determined based on engineering analysis. These buildings have been constructed to performance-based criteria as opposed to prescriptive criteria, which have generally been used for existing Group I buildings. Because of this fundamental difference and the difficulty of inspecting for connection strength using visible means (without an engineering analysis), a practical way to classify these types of buildings is by the design code and era of year built. This approach was used in this study.

A practical way to determine the "design wind speed" of existing Group II and Group III buildings is by using the reference design wind speed maps of the design standard in use at the time the building was designed. Figure A-8 shows the fastest-mile wind speed contours for Florida for the three existing construction standards for Groups II and III buildings considered in this study. With knowledge of building location and year built, the user can (by reference to these maps) select the appropriate cell in Tables 3-3 through 3-6.

Note that for Tables 3-5 and 3-6, there is also a classification column for "Design Exposure" for the ASCE 7-88 era. This column refers to the terrain roughness, which is discussed in Section 2.2 for the new Florida Building Code. In the ASCE 7-88 document, Exposure D applies to buildings within 1500 feet of the coast, Exposure C to open terrain, and Exposure B to urban and suburban areas. In the absence of obtaining design information for these older buildings, it seems reasonable to assume that Exposure D applies to buildings built within 1500 feet of the coast and Exposure B applies elsewhere.

A preferred alternative to classifying by year built (and Design Exposure for Group III Buildings built after 1983) would be to obtain the design documents and classify the building according to the actual design parameters.

#### A.3.1.3 South Florida Building Code

A final point is that, due to schedule and resource limitations, we did not analyze Group II and III buildings built to the 1994 SFBC as a separate class. In the absence of such an analysis, it seems reasonable to classify Group II buildings built to the 1994 SFBC using the bottom row in Tables 3-4. That is, the bottom row corresponds to FBC equivalent roof covers, hurricane opening protection, and the highest level of design wind speed considered in Chapter 3. For Group III buildings built to the 1994 SFBC, one could use the bottom row (Exposure D) in Table 3-6 for locations within 1500 feet of the coastline and the third row from the bottom (Exposure C, Hurricane Opening Protection) for all other locations. Again, if design documents are available, that is the preferential approach to classify these buildings.

### A.3.2 Design Options Under FBC

The FBC allows a broader range of design conditions to be used now compared to previous versions of various building codes. From an insurance rating perspective, the three most important changes are the creation of a Wind Borne Debris Region, a new definition of "openings" within that region, and the uniform adoption of terrain types. There are other changes to the code that affect the designs to a lesser degree, which will also be discussed. In order to fully understand the impact of the first two items, one must be familiar with the various internal pressure scenarios under the building code – i.e. the difference between enclosed and partially enclosed designs.

#### A.3.2.1 Internal Pressure: Enclosed vs. Partially Enclosed

In designing a building, an engineer must consider the effect of whether the wind is able to enter the building and change the loading pattern on the building components. Building codes define three conditions. The first is an "Enclosed" building where the envelope is completely closed, and only minimal wind "leaking" around doors, windows, framing, etc. is allowed to affect the interior of the building. The second condition is called an "Open" building such as a stadium grand stand where wind can freely enter the inside of the structure.

In between these two conditions is the third case, which is a "Partially Enclosed" building, where openings are assumed to exist in one or more faces of the building. These openings allow the wind to create pressures or suctions inside the building. These "internal" pressures for partially enclosed designs are typically larger in magnitude than the internal pressures in an enclosed building. Hence, partially enclosed designs that are based on larger internal pressures typically result in individual parts of the structure being stronger than if designed to an "enclosed" condition. In the past, to qualify as a partially enclosed, a structure had to have a certain ratio of permanent openings compared to the wall area. However in the FBC, this criterion has changed and will be discussed in the following sections.

#### A.3.2.2 Wind Borne Debris Region

The introduction of a Wind-Borne Debris Region (WBDR) means that new buildings in this region must now either have impact resistant protection on all glazed openings *or* be designed for higher wind pressures as partially enclosed structures. This change means that a designer must now choose between designing the structure as either an enclosed or partially enclosed building. This region is any area where the wind speeds are greater than 120 mph (gust) and the areas within 1 mile of coast where wind speeds are greater than 110 mph (gust) except for the Florida panhandle where the region includes only areas within 1 mile of the coast regardless of the wind speeds.

### A.3.2.3 The Definition of "Openings" in WBDR

In the SBC97, an opening was defined as: *"windows, doors and skylights that are not designed as components and cladding."* The implication of this definition is that if a designer specified the wind load for a window that must meet components and cladding loads, then the window is not considered to be an opening. Based on this definition, the building does not have to be designed as a partially enclosed structure even when the windows have no impact protection.

In contrast, ASCE 7-98 and the FBC have adopted a different definition of opening as: *"in wind borne debris regions, exterior glazing shall be assumed open unless impact resistant or shuttered."* This change in opening definition means that for those buildings in the wind borne debris region – the structure must have some form of impact protection for all glazed openings, or alternatively be designed as a partially-enclosed structure (to withstand higher wind pressures that occur when an "opening" occurs in the exterior of the building). Designing for the partially enclosed condition means that all design pressures are increased as a result of potentially higher internal pressure loads that the structure may experience. This includes loads on the roof deck, roof trusses, windows and doors, as well as all other parts of the structure.

However, the openings (windows, doors, etc.) in partially enclosed designs are vulnerable to wind-borne debris impact failures and the resulting wind and rainwater damage to the building interior and contents. Determining which condition is appropriate for a given building depends on the number and size of the openings in a building.

For taller buildings, the FBC requires that all glazed openings with height less than 30 ft meet the Large Missile Impact test of the FBC, which consists of firing a 2x4 at the product. For openings within the 30 to 60 ft height range, the openings shall meet the provisions of the Small Missile Test, which consists of firing 30 steel balls at the product. No impact protection is required at heights greater than 60 ft. Note that in the HVHZ, small missile impact protection is required everywhere above 30 ft, including heights greater than 60 ft.

For insurance rating purposes, clearly the design option chosen for a building in the Wind-Borne Debris Region of the FBC (see Section 2.2) is a key factor in hurricane loss mitigation. Enclosed designs in the Wind-Borne Debris Region will have all glazed openings protected[1] for debris impact. These buildings will perform better than partially enclosed designs and will have lower losses.

In the FBC opening definition, strictly speaking, doors without glazing escape the impact rating requirements because the definition of openings is phrased in terms of "glazed" openings. The FBC definition of glazed openings is assumed to mean any door or window containing glass. Thus, garage doors and entrance doors without windows only have to meet wind pressure requirements in the wind borne debris region; they do not have to meet any of the referenced impact standards. The current rules for opening protection credits used by many insurance companies, such as FWUA, require all windows *and* doors to be protected. Thus, buildings designed strictly to the FBC enclosed scenario will require a new class that corresponds to protection of only glazed openings.

[1] In the HVHZ, all openings must be protected (see Section 1626 of FBC 2001).

### A.3.2.4 The Definition of Terrain Exposure[2]

The FBC has adopted a different definition of Exposure C than the one that appears in the text of ASCE 7-98. Exposure C (known as the open country exposure) is defined in the FBC as Broward and Miami-Dade counties (HVHZ), barrier islands, and 1500 ft from the coastline in the rest of the state. All other buildings will be designed for Exposure B. Hence; the loss relativities for new construction are computed separately for terrain exposures B and C since the design loads are dependent on terrain.

### A.3.2.5 Other Changes to Wind Loading in FBC

Other changes that affect the strength of building components under wind loads include a change of how wind speeds are reported, a change in the load combinations considered with wind loads, and a change in the load assumptions for truss tie-downs.

**Wind Speed Change.** When discussing wind speeds, it is always important to consider the time interval over which the measurement occurs and the average is taken. Shorter averaging times yield wind speeds that are higher than longer averaging times. The Standard Building Code and ASCE 7-88 measured wind speed according to Fastest Mile. This was based on old anemometers that timed how long it took for the anemometer to spin the equivalent of 1 mile. This method of collection meant that the time interval for averaging the wind speed was a function of the wind speed itself, ranging from a few tens of seconds at high wind speeds to several minutes at very low wind speeds. The ASCE 7-95 and 7-98 standards (and by adoption of ASCE 7-98, the FBC and International code series) have been converted to a 3-second gust which is the maximum wind speed with an averaging interval of three seconds that occurs during the storm.

In terms of comparing the designs from the Standard Building Code and the FBC, one can consider the change to be analogous to a change of units. The difference in Florida between the fastest mile wind speeds and the 3-second gust is nominally 20 mph.

**Load Combinations.** There has been a change in the design load combinations for the Allowable Stress Design method specified in ASCE 7-98 and thus in FBC. Previously, a designer calculates the wind loads on the assembly and calculates the forces considering both the full dead load of the assembly and the wind loads. In ASCE 7-98, the designer is now required to consider a design scenario where the full wind loads and only 60% of the dead load simultaneously act upon the assembly. The net result of this change is that connection sizes may be significantly larger than those calculated strictly by earlier codes, such as the Standard Building Code provisions.

**Effect of Loading Assumptions in Truss Strap Design.** When designing the roof straps, a designer is presented two methods of calculating the loads on the roof straps under the SBC and the FBC. One set of loads in the code is called Components and Cladding (C&C) loads and these are to be applied to any cladding or member that receives wind loads directly from the wind. These loading pressures take into account the lack of correlation of the wind gusts over larger and larger areas gradually. The other set of loads in the code are called Main Wind Force Resisting System (MWFRS) loads and are intended to calculate the effect of loads acting on several surfaces at once. Much discussion and debate among design professionals over which loading set is appropriate for roof trusses has ensued over the years.

[2] ASCE-7 and wind engineers use the term "Exposure" to define the earth's surface roughness for purposes of grouping this roughness into several distant categories for wind load estimation. Insurers need to be aware of this use of the term "Exposure" when reading building code and wind engineering literature.

The ASCE 7-98 document says that trusses are to be considered as both C&C loading and MWFRS loading (see page 243 of ASCE 7-98 commentary). The commentary describes the situation where long span trusses should be designed for MWFRS loads and individual members of the truss designed for C&C loads. Unfortunately, the commentary does not discuss what is appropriate for the straps holding the truss to the wall, nor does it define what constitutes a long span truss. Section 6.5.12.1.3 of the ASCE 7-98 does indicate a threshold of 700 square ft of tributary area for considering a component to be designed with MWFRS loads. From this threshold, a logical argument could be made that most trusses are not large enough to qualify for the MWFRS loads, and therefore should be designed for C&C loads, and subsequently, the strap size chosen to be consistent with C&C loads. Both the MWFRS and the C&C loads should be checked, and the larger of the two chosen. Typically, the C&C loads are significantly higher than the MWFRS loads.

The language in Section 1606 of the SBC is quite vague on which loading set is appropriate for strap uplift calculations. However, the prescriptive codes referenced by the FBC are founded on the SBC97 (or SBC95) building code, and clearly state that the truss strap design has been completed with MWFRS loads. Conversations with designers and truss manufacturers indicate that much of the industry is conforming to the MWFRS loads. Therefore, we have evaluated the design options under the Standard Building Codes using the MWFRS loads and under the FBC using C&C loads.

### A.3.3 Model Parameters Determined via Design Methods

Each of the buildings built to the new FBC was considered in the HURLOSS simulations to be a masonry structure (Group I) or steel frame (Group II & III) structure with FBC Equivalent shingle or built up roof covers and no Secondary Water Resistance.

The following tables show comparisons of the key model parameters designed according to the three code eras. An example of the design calculations for one of the buildings at 130 mph design wind speed is shown in Appendix B. These calculations were repeated for the wind speed/exposure combinations at each of the 31 points in this study (see Table 2-2).

#### A.3.3.1 Wood Deck Nailing Pattern

A wood roof deck has been used on the Group I flat, gable, and hip roofs and the Group II flat roofs. Note that only the Group II relativity tables (Table 3-3 and 3-4) are organized by design parameters, whereas the Group I tables (Table 3-1 and 3-2) are organized by physical parameters. The nailing pattern has been determined based on Zone 2/3 pressures and is applied uniformly across the entire roof. The withdrawal resistance of a single fastener is compared to the component and cladding wind pressure applied over standard tributary areas. The appropriate nailing pattern is chosen such that the single fastener in the field of the roof can withstand the required design pressure.

In Table A-3, the design calculations indicate that a minimum nail size of 8d should be used throughout the state under the FBC. The nailing pattern for the roof varies from the standard 6″/12″ pattern in the lower wind speed zones in the state, to the 6″/6″ spacing in the high wind zone areas. In all of these designs, the nailing pattern at the edge of the roof is assumed to drop to a 4-inch spacing next to the gable end (if appropriate).

**Table A-3. Comparison of Wood Decking Nailing Patterns Across Different Building Code Eras (Flat Roof)**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group I | | | | | | Group II | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC 76 | | SBC 88 | | FBC | | SBC 76 | | SBC 88 | | FBC | |
| | | | | Nail Size | Spacing (in.) | Nail Size | Spacing (in.) | Nail Size | Spacing (in.) | Nail Size | Spacing (in.) | Nail Size | Spacing (in.) | Nail Size | Spacing (in.) |
| 100 | 80 | B | Enclosed | 6d | 12 | | | 8d | 12 | 6d | 12 | | | 8d | 12 |
| 100 | 80 | B | Part Encl. | | | | | 8d | 12 | | | | | 8d | 12 |
| 110 | 90 | B | Enclosed | 6d | 12 | 6d | 12 | 8d | 12 | 6d | 12 | 6d | 12 | 8d | 12 |
| 110 | 90 | B | Part Encl. | | | | | 8d | 12 | | | | | 8d | 12 |
| 120 | 100 | B | Enclosed | 6d | 12 | 6d | 12 | 8d | 12 | 6d | 12 | 8d | 12 | 8d | 12 |
| 120 | 100 | B | Part Encl. | | | | | 8d | 12 | | | | | 8d | 12 |
| 120 | 100 | C | Enclosed | 6d | 12 | 6d | 12 | 8d | 12 | 6d | 12 | 8d | 12 | 8d | 9.6 |
| 120 | 100 | C | Part Encl. | | | | | 8d | 9.6 | | | | | 8d | 9.6 |
| 130 | 110 | B | Enclosed | 6d | 12 | 6d | 12 | 8d | 12 | 6d | 12 | 8d | 12 | 8d | 12 |
| 130 | 110 | B | Part Encl. | | | | | 8d | 12 | | | | | 8d | 9.6 |
| 130 | 110 | C | Enclosed | 6d | 12 | 6d | 12 | 8d | 9.6 | 6d | 12 | 8d | 12 | 8d | 9.6 |
| 130 | 110 | C | Part Encl. | | | | | 8d | 9.6 | | | | | 8d | 8 |
| 140 | 120 | B | Enclosed | 6d | 12 | | | 8d | 12 | 6d | 12 | | | 8d | 9.6 |
| 140 | 120 | B | Part Encl. | | | | | 8d | 9.6 | | | | | 8d | 9.6 |
| 140 | 120 | C | Enclosed | 6d | 12 | | | 8d | 9.6 | 6d | 12 | | | 8d | 8 |
| 140 | 120 | C | Part Encl. | | | | | 8d | 8 | | | | | 8d | 6.9 |
| 146 | | HVHZ | Enclosed | | | | | 8d | 8 | | | | | 8d | 6.9 |
| 150 | 130 | B | Enclosed | 6d | 12 | | | 8d | 9.6 | 8d | 12 | | | 8d | 9.6 |
| 150 | 130 | B | Part Encl. | | | | | 8d | 9.6 | | | | | 8d | 8 |
| 150 | 130 | C | Enclosed | 6d | 12 | | | 8d | 8 | 8d | 12 | | | 8d | 6.9 |
| 150 | 130 | C | Part Encl. | | | | | 8d | 6.9 | | | | | 8d | 6 |

Note:
- Spacing number indicated spacing of nails along truss supporting interior of the plywood. Spacing on the edge of plywood is always 6 inches.
- Wood roof deck does not apply to Group III building.
- Truss spacing = 24″
- Design results in shaded area were prepared for comparison purpose only. Existing construction runs of Group I buildings were based on range of physical parameters instead of unique design combinations

Note that the minimum nailing patterns under the previous versions of the codes typically require only 6d nails except for the highest of wind speeds. There is a significant difference in the uplift resistance between 6d and 8d nails, and this affects the relativities reported in Table 3-3 and 3-4. The dramatic change in the loss relativity from one design wind speed to another is largely due to the change in roof deck nailing pattern required by the code. Hence, if the 90 mph fastest mile design to SBC 1988 was actually nailed with 8d nails (as opposed to the minimum required 6d nails), the relativity in Table 3-3 would change significantly. This difference is seen by the drop in relativity from the 90 mph to the 100 mph fastest mile design (SBC88), which requires 8d nails. Thus, it is preferable to inspect building with wood roof decks to determine the actual connections and then use the appropriate relativity based on physical parameters rather than design parameters.

### A.3.3.2 Wood Truss Tie Downs

The hurricane strap size has been calculated for a truss spanning from one side of the building to the other, spaced at 24 inches apart. In FBC, the designs were prepared with C&C loads as discussed above. Tables A-4 and A-5 present the reaction of an interior truss that

**Table A-4. Comparison of Modeled Resistances (Ultimate Load) of Roof-Wall Connections for Wood Decks Across Different Code Eras (Flat Roofs)**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group I | | | Group II | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC 76 | SBC 88 | FBC | SBC 76 | SBC 88 | FBC |
| 100 | 80 | B | Enclosed | -730 | | -995 | -958 | | -1318 |
| 100 | 80 | B | Part Encl. | | | -1475 | | | |
| 110 | 90 | B | Enclosed | -1059 | -1174 | -1300 | -1355 | -1813 | -1696 |
| 110 | 90 | B | Part Encl. | | | -1882 | | | |
| 120 | 100 | B | Enclosed | -1428 | -1629 | -1635 | -1799 | -2428 | -2110 |
| 120 | 100 | B | Part Encl. | | | -2327 | | | -2968 |
| 120 | 100 | C | Enclosed | -1428 | -1629 | -2296 | -1799 | -2428 | -3001 |
| 120 | 100 | C | Part Encl. | | | -3207 | | | -4153 |
| 130 | 110 | B | Enclosed | -1835 | -2133 | -1999 | -2290 | -3107 | -2561 |
| 130 | 110 | B | Part Encl. | | | -2811 | | | -3567 |
| 130 | 110 | C | Enclosed | -1835 | -2133 | -2775 | -2290 | -3107 | -3606 |
| 130 | 110 | C | Part Encl. | | | -3844 | | | -4958 |
| 140 | 120 | B | Enclosed | -2281 | | -2392 | -2827 | | -3047 |
| 140 | 120 | B | Part Encl. | | | -3334 | | | -4214 |
| 140 | 120 | C | Enclosed | -2281 | | -3292 | -2827 | | -4260 |
| 140 | 120 | C | Part Encl. | | | -4531 | | | -5827 |
| 146 | | HVHZ | Enclosed | | | -3620 | | | -4675 |
| 150 | 130 | B | Enclosed | -2766 | | -2814 | -3411 | | -3570 |
| 150 | 130 | B | Part Encl. | | | -3895 | | | -4909 |
| 150 | 130 | C | Enclosed | -2766 | | -3847 | -3411 | | -4962 |
| 150 | 130 | C | Part Encl. | | | -5270 | | | -6761 |

Note:
- Wood roof deck not used on Group III building
- Design results in shaded area were prepared for comparison purpose only. Existing construction runs of Group I buildings were based on range of physical parameters instead of unique design combinations
- Truss spacing assumed to be 24″ on center.

is typical for 75% of the roof-wall connections in a given building.[3] Table A-4 is for the flat roof version of the building, and Table A-5 is for the gable/hip version of the same building. The required uplift resistances for the flat roof are slightly lower than the sloped roofs in this study. Because the FBC now uses a load combination of 60% of the dead load of the roof to resist uplift, the design values of the straps are larger than they were for the immediately previous version of the SBC (1997). Note that the roof-wall connections shown in Table A-4 for FBC partially enclosed design are often factors of two or more greater than those designed to the SBC 1976 code.

### A.3.3.3 Metal Deck

A finite element analysis was carried out to investigate the failure load (pressure) of these selected case studies. Particular attention was paid to the uplift capacities of metal roof decks since they control the failure modes in high wind events (such as hurricanes and tornadoes). Serviceability was also used as a criterion to determine whether or not a metal deck fails. Based on the finite element simulation of the metal deck, it was found that after the first fastener failed, the remaining fasteners would disengage (pull-out or pull-over for screw connections) from the structural members very rapidly and it would take less than 10-15% of extra pressure for the whole metal deck to reach failure status. Therefore, the metal deck system can be modeled as a

[3] Assuming uniform spacing of similar size trusses throughout roof plan.

**Table A-5. Comparison of Modeled Resistances (Ultimate Load) of Roof-Wall Connections for Wood Decks across Different Code Eras (Gable/Hip Roofs)**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group I | | |
|---|---|---|---|---|---|---|
| | | | | SBC 76 | SBC 88 | FBC |
| 100 | 80 | B | Enclosed | -725 | | -1125 |
| 100 | 80 | B | Part Encl. | | | -1605 |
| 110 | 90 | B | Enclosed | -1054 | -1079 | -1458 |
| 110 | 90 | B | Part Encl. | | | -2039 |
| 120 | 100 | B | Enclosed | -1421 | -1512 | -1822 |
| 120 | 100 | B | Part Encl. | | | -2514 |
| 120 | 100 | C | Enclosed | -1421 | -1512 | -2543 |
| 120 | 100 | C | Part Encl. | | | -3453 |
| 130 | 110 | B | Enclosed | -1827 | -1991 | -2218 |
| 130 | 110 | B | Part Encl. | | | -3031 |
| 130 | 110 | C | Enclosed | -1827 | -1991 | -3064 |
| 130 | 110 | C | Part Encl. | | | -4133 |
| 140 | 120 | B | Enclosed | -2271 | | -2646 |
| 140 | 120 | B | Part Encl. | | | -3588 |
| 140 | 120 | C | Enclosed | -2271 | | -3627 |
| 140 | 120 | C | Part Encl. | | | -4866 |
| 146 | | HVHZ | Enclosed | | | -3985 |
| 150 | 130 | B | Enclosed | -2754 | | -3106 |
| 150 | 130 | B | Part Encl. | | | -4187 |
| 150 | 130 | C | Enclosed | -2754 | | -4232 |
| 150 | 130 | C | Part Encl. | | | -5654 |

Note:
- Wood roof deck not used on Group III building
- Flat roof configuration does not apply to Group II building
- Design results in shaded area were prepared for comparison purpose only. Existing construction runs of Group I buildings were based on range of physical parameters instead of unique design combinations

series system and the pressure resistance capability is dependent on the probability that its "weakest" link survives. In this report, the series system analysis was taken to predict the failure pressure statistics (mean and standard deviation) of a metal deck on open web steel joist.

Design calculations for the metal roof decks have been based on a series system analysis of the fastener spacing and size. Table A-6 shows the resistances of various combinations of fastener size, deck material and fastener spacing. Wind loads for three areas (zones) on the roof were then calculated according to the three eras of building codes. Table A-7 shows the fastener pattern called for by each design wind speed for the three code eras.

Note that the wind load pattern specified in the SBC 76 code is a uniform pressure across the entire roof surface. Resulting from research in the 1970's the SBC 88 code includes a more complicated loading pattern for Components and Cladding loads that features an edge and corner zone. Thus Table A-7 does not show any design for Zone 2 and 3 for the SBC76 code. In these cases, the design shown in Zone 1 has been assumed to apply uniformly to the entire roof area.

One may also notice that the design for the SBC76 code is the same regardless of wind speed. This is because the calculated wind loads from this code were so low that they did not dictate that anything other than the minimum fastening requirements was necessary.

## Table A-6. Failure Capacity for Selected Metal Deck Design Cases

| Case ID | Screw Size | Steel Deck | | | Base Metal | | | Screw Spacing (in) | Ultimate Pressure (psf) |
|---|---|---|---|---|---|---|---|---|---|
| | | Thickness | Fy (ksi) | Fu (ksi) | Thickness (in.) | Fy (ksi) | Fu (ksi) | | |
| s1/16″ | #10 | 0.030 (gauge 22) | 40 | 55 | 0.125 | 33 | 45 | 16 | 149 |
| s1/12″ | | | | | | | | 12 | 184 |
| s1/6″ | | | | | | | | 6 | 318 |
| s2/16″ | #10 | 0.030 (gauge 22) | 50 | 60 | 0.125 | 33 | 45 | 16 | 168 |
| s2/12″ | | | | | | | | 12 | 208 |
| s2/6″ | | | | | | | | 6 | 361 |
| s3/12″ | #10 | 0.036 (gauge 20) | 50 | 60 | 0.125 | 40 | 55 | 12 | 227 |
| s3/6″ | | | | | | | | 6 | 396 |
| s4/12″ | #12 | 0.036 (gauge 20) | 50 | 60 | 0.125 | 40 | 55 | 12 | 245 |
| s4/6″ | | | | | | | | 6 | 425 |

## Table A-7. Metal Deck Designs for Various Code Eras (Flat Roof)

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group II | | | | | | | | | Group III | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC76 | | | SBC88 | | | FBC | | | SBC76 | | | ASCE 7-88 | | | FBC | | |
| | | | | Zone 1 | Zone 2 | Zone 3 | Zone 1 | Zone 2 | Zone 3 | Zone 1 | Zone 2 | Zone 3 | Zone 1 | Zone 2 | Zone 3 | Zone 1 | Zone 2 | Zone 3 | Zone 1 | Zone 2 | Zone 3 |
| 100 | 80 | B | Enclosed | s1/16" | N/A | N/A | | | | s1/16" | s1/16" | s1/16" | s1/16" | N/A | N/A | | | | s1/16" | s1/16" | s2/16" |
| 100 | 80 | B | Part Encl. | | | | | | | | | | | | | | | | | | |
| 110 | 90 | B | Enclosed | s1/16" | N/A | N/A | s1/16" | s1/16" | s1/12" | s1/16" | s1/16" | s1/16" | s1/16" | N/A | N/A | s1/16" | s2/16" | s3/12" | s1/16" | s1/16" | s1/12" |
| 110 | 90 | B | Part Encl. | | | | | | | | | | | | | | | | | | |
| 120 | 100 | B | Enclosed | s1/16" | N/A | N/A | s1/16" | s1/16" | s3/12" | s1/16" | s1/16" | s1/12" | s1/16" | N/A | N/A | s2/16" | s1/12" | s4/12" | s1/16" | s1/12" | s3/12" |
| 120 | 100 | B | Part Encl. | | | | | | | s1/16" | s1/16" | s1/12" | | | | | | | s1/16" | s1/12" | s3/12" |
| 120 | 100 | C | Enclosed | s1/16" | N/A | N/A | s1/16" | s1/16" | s3/12" | s1/16" | s2/16" | s3/12" | s1/16" | N/A | N/A | s2/16" | s1/12" | s4/12" | s1/16" | s2/12" | s4/12" |
| 120 | 100 | C | Part Encl. | | | | | | | s1/16" | s1/12" | s4/12" | | | | | | | s1/12" | s3/12" | s1/6" |
| 130 | 110 | B | Enclosed | s1/16" | N/A | N/A | s1/16" | s1/12" | s4/12" | s1/16" | s1/16" | s2/12" | s1/16" | N/A | N/A | s1/12" | s3/12" | s1/6" | S1/16" | s1/12" | s4/12" |
| 130 | 110 | B | Part Encl. | | | | | | | s1/16" | s2/16" | s3/12" | | | | | | | s2/16" | s3/12" | s4/12" |
| 130 | 110 | C | Enclosed | s1/16" | N/A | N/A | s1/16" | s1/12" | s4/12" | s1/16" | s1/12" | s4/12" | s1/16" | N/A | N/A | s1/12" | s3/12" | s1/6" | s2/16" | s3/12" | s1/6" |
| 130 | 110 | C | Part Encl. | | | | | | | s1/16" | s3/12" | s1/6" | | | | | | | s2/12" | s4/12" | s1/6" |
| 140 | 120 | B | Enclosed | s1/16" | N/A | N/A | | | | s1/16" | s2/16" | s3/12" | s1/16" | N/A | N/A | | | | s1/16" | s3/12" | s1/6" |
| 140 | 120 | B | Part Encl. | | | | | | | s1/16" | s1/12" | s4/12" | | | | | | | s1/12" | s4/12" | s1/6" |
| 140 | 120 | C | Enclosed | s1/16" | N/A | N/A | | | | s1/16" | s2/12" | s1/6" | s1/16" | N/A | N/A | | | | s1/12" | s4/12" | s2/6" |
| 140 | 120 | C | Part Encl. | | | | | | | s1/12" | s3/12" | s1/6" | | | | | | | s3/12" | s1/6" | s3/6" |
| 146 | | HVHZ | Enclosed | | | | | | | s1/16" | s3/12" | s1/6" | | | | | | | s2/12" | s1/6" | s3/6" |
| 150 | 130 | B | Enclosed | s1/16" | N/A | N/A | | | | s1/16" | s1/12" | s4/12" | s1/16" | N/A | N/A | | | | s1/12" | s4/12" | s1/6" |
| 150 | 130 | B | Part Encl. | | | | | | | s1/16" | s3/12" | s1/6" | | | | | | | s2/12" | s4/12" | s2/6" |
| 150 | 130 | C | Enclosed | s1/16" | N/A | N/A | | | | s1/16" | s3/12" | s1/6" | s1/16" | N/A | N/A | | | | s2/12" | s1/6" | s3/6" |
| 150 | 130 | C | Part Encl. | | | | | | | s1/12" | s4/12" | s2/6" | | | | | | | s4/12" | s1/6" | s4/6" |

Notes: 1. Zone 1 = interior, Zone 2 = edge of roof, Zone 3 = corner of roof
2. In SBC 76, Zone 2 and 3 did not exist.– pattern for Zone 1 applied to entire roof.

### A.3.3.4 Metal Bar Joists

Most metal bar joist failures occur due to inadequate uplift resistance capacity at the roof-wall connection or result from buckling of the lower chord or lower members of the metal bar joist. The relatively common uplift failure modes involve the failure of the weld connecting the joist to either a steel bearing plate or a larger steel beam, or the failure of the un-reinforced masonry wall at the mortar interface between blocks. Common bending failure modes include bottom flange buckling, bottom chord yielding, or web members yielding or buckling.

The likely resistance capacities associated with these failure mechanisms are estimated primarily based on design loads specified in building codes that govern or prevail within the jurisdiction at the time of construction, combined with reasonable randomness expected to result from design, manufacture, erection and aging processes. The estimated resistance capacities are presented in Table A-8 for uplift reaction at a joist connection to the wall and Table A-9 for moment at the mid-span of a joist, which is presumably the most vulnerable location for a beam with uniform section.

**Table A-8. Resistance Capacity Associated with Uplift Reaction at Metal Joist-to-Wall Connection Estimated from Design Loads Specified in Various Building Code Eras (lbf.)**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group II | | | Group III | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC-76 | SBC-88 | FBC | SBC-76 | ASCE 7-88 | FBC |
| 100 | 80 | B | Enclosed | -1538 | -2174 | -2154 | -1760 | -2818 | -3428 |
| 100 | 80 | B | Part Encl. | | | -2778 | | | -4140 |
| 110 | 90 | B | Enclosed | -1946 | -2752 | -2608 | -2226 | -3566 | -4148 |
| 110 | 90 | B | Part Encl. | | | -3362 | | | -5010 |
| 120 | 100 | B | Enclosed | -2404 | -3398 | -3104 | -2748 | -4404 | -4936 |
| 120 | 100 | B | Part Encl. | | | -4000 | | | -5962 |
| 120 | 100 | C | Enclosed | -2404 | -3398 | -4172 | -2748 | -6598 | -6404 |
| 120 | 100 | C | Part Encl. | | | -5378 | | | -7736 |
| 130 | 110 | B | Enclosed | -2908 | -4110 | -3642 | -3326 | -5328 | -5792 |
| 130 | 110 | B | Part Encl. | | | -4694 | | | -6996 |
| 130 | 110 | C | Enclosed | -2908 | -4110 | -4896 | -3326 | -7984 | -7516 |
| 130 | 110 | C | Part Encl. | | | -6310 | | | -9078 |
| 140 | 120 | B | Enclosed | -3460 | -4892 | -4224 | -3958 | -6342 | -6718 |
| 140 | 120 | B | Part Encl. | | | -5444 | | | -8114 |
| 140 | 120 | C | Enclosed | -3460 | -4892 | -5678 | -3958 | -9500 | -8718 |
| 140 | 120 | C | Part Encl. | | | -7320 | | | -10528 |
| 146 | | HVHZ | Enclosed | | | -6174 | | | -9480 |
| 150 | 130 | B | Enclosed | -4062 | -5740 | -4848 | -4644 | -7442 | -7712 |
| 150 | 130 | B | Part Encl. | | | -6250 | | | -9314 |
| 150 | 130 | C | Enclosed | -4062 | -5740 | -6518 | -4644 | -11150 | -10006 |
| 150 | 130 | C | Part Encl. | | | -8402 | | | -12086 |

Note:
- Joist spacing equal to 4 ft
- Metal joists not used on Group I buildings

**Table A-9. Moment Resistance Capacity Associated with Joist Mid-Span Estimated from Design Loads Specified in Various Building Codes Eras (lbf.-ft.)**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group II | | | Group III | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC-76 | SBC-88 | FBC | SBC-76 | ASCE 7-88 | FBC |
| 100 | 80 | B | Enclosed | -15379.8 | -21740.9 | -21549.8 | -17590.1 | -28182.2 | -34274.7 |
| 100 | 80 | B | Part Encl. | | | -27779.1 | | | -41399.3 |
| 110 | 90 | B | Enclosed | -19465.0 | -27515.8 | -26075.3 | -22262.5 | -35668.2 | -41472.4 |
| 110 | 90 | B | Part Encl. | | | -33612.7 | | | -50093.1 |
| 120 | 100 | B | Enclosed | -24030.9 | -33970.2 | -31031.8 | -27484.6 | -44034.8 | -49355.6 |
| 120 | 100 | B | Part Encl. | | | -40001.9 | | | -59614.9 |
| 120 | 100 | C | Enclosed | -24030.9 | -33970.2 | -41714.9 | -27484.6 | -65977.6 | -64043.4 |
| 120 | 100 | C | Part Encl. | | | -53773.1 | | | -77355.7 |
| 130 | 110 | B | Enclosed | -29077.3 | -41103.9 | -36419.2 | -33256.4 | -53282.1 | -57924.3 |
| 130 | 110 | B | Part Encl. | | | -46946.7 | | | -69964.7 |
| 130 | 110 | C | Enclosed | -29077.3 | -41103.9 | -48957.1 | -33256.4 | -79832.9 | -75162.0 |
| 130 | 110 | C | Part Encl. | | | -63108.8 | | | -90785.6 |
| 140 | 120 | B | Enclosed | -34604.4 | -48917.0 | -42237.7 | -39577.8 | -63410.1 | -67178.5 |
| 140 | 120 | B | Part Encl. | | | -54447.0 | | | -81142.5 |
| 140 | 120 | C | Enclosed | -34604.4 | -48917.0 | -56778.6 | -39577.8 | -95007.7 | -87170.1 |
| 140 | 120 | C | Part Encl. | | | -73191.2 | | | -105289.8 |
| 146 | | HVHZ | Enclosed | | | -61749.7 | | | -94802.0 |
| 150 | 130 | B | Enclosed | -40612.2 | -57409.6 | -48487.1 | -46449.0 | -74418.7 | -77118.1 |
| 150 | 130 | B | Part Encl. | | | -62502.9 | | | -93148.3 |
| 150 | 130 | C | Enclosed | -40612.2 | -57409.6 | -65179.6 | -46449.0 | -111502.1 | -100067.8 |
| 150 | 130 | C | Part Encl. | | | -84020.5 | | | -120868.4 |

Note: • Joist spacing equal to 4 ft
• Metal joists not used on Group I buildings

### A.3.3.5 Window Design Pressure

Window designs are based on the calculation of the positive and negative wind loads on each of the openings on the building. For simplicity, we have assumed that windows were chosen to exactly match the minimum design pressures as calculated. In reality, the window strengths were likely chosen to exceed these minimum requirements in a way that minimized the number of different window types that were ordered. Area reductions in the design pressure were used where allowed in the appropriate version of the building code. A summary of the window design pressures is shown in Table A-10.

## A.4 Analysis of Loss Costs Relativities

The HURLOSS model was run in its individual risk analysis mode to produce loss costs for each modeled building. The buildings were modeled with the wind-resistive features described previously. Two sets of runs were made for the two different terrain categories.

For Group I existing buildings, a full combinatorial set of the variables from Table 2-4 were run. This includes roof covering, secondary water resistance, roof-to- wall connection, roof deck attachment, opening protection level, and roof shape yielding 1152 separate simulations. For Group I new construction, one of the 21 unique combinations of wind speed, terrain exposure, and internal pressure were simulated for each building at each of the 31 locations around the state.

Group II and Group II existing and new buildings were all designed and therefore the model parameters were selected based on a unique combination of wind speed, terrain exposure (if applicable) and internal pressure (if applicable). For each of the 31 locations, the roof deck, the roof-wall connection, and the

**Table A-10. Comparison of Window Design Pressures on Condominiums Across Different Building Codes**

| Wind Speed (Gust), mph | Wind Speed (Fast Mile), mph | Terrain Exposure | Internal Pressure | Group I | | | | | | Group II | | | | | | Group III | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | SBC 76 | | SBC 88 | | FBC | | SBC 76 | | SBC 88 | | FBC | | SBC 76 | | ASCE 88 | | FBC | |
| | | | | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP | Max DP | Min DP |
| 100 | 80 | B | Enclosed | 18 | -9 | | | 16 | -18 | 21 | -10 | | | 19 | -21 | 24 | -12 | | | 22 | -22 |
| 100 | 80 | B | Part Encl. | | | | | 22 | -23 | | | | | 26 | -28 | | | | | 29 | -29 |
| 110 | 90 | B | Enclosed | 23 | -11 | 22 | -22 | 20 | -21 | 27 | -13 | 28 | -28 | 23 | -25 | 31 | -15 | 23 | -24 | 26 | -27 |
| 110 | 90 | B | Part Encl. | | | | | 26 | -28 | | | | | 31 | -33 | | | | | 35 | -35 |
| 120 | 100 | B | Enclosed | 28 | -14 | 27 | -27 | 23 | -25 | 33 | -16 | 35 | -35 | 28 | -30 | 38 | -19 | 28 | -29 | 31 | -32 |
| 120 | 100 | B | Part Encl. | | | | | 31 | -33 | | | | | 37 | -40 | | | | | 42 | -42 |
| 120 | 100 | C | Enclosed | 28 | -14 | 27 | -27 | 31 | -33 | 33 | -16 | 35 | -35 | 37 | -40 | 38 | -19 | 47 | -48 | 40 | -41 |
| 120 | 100 | C | Part Encl. | | | | | 41 | -44 | | | | | 50 | -53 | | | | | 55 | -55 |
| 130 | 110 | B | Enclosed | 34 | -17 | 33 | -33 | 27 | -30 | 40 | -20 | 43 | -43 | 33 | -35 | 46 | -23 | 34 | -35 | 36 | -37 |
| 130 | 110 | B | Part Encl. | | | | | 37 | -39 | | | | | 44 | -47 | | | | | 49 | -50 |
| 130 | 110 | C | Enclosed | 34 | -17 | 33 | -33 | 36 | -39 | 40 | -20 | 43 | -43 | 44 | -47 | 46 | -23 | 57 | -58 | 48 | -48 |
| 130 | 110 | C | Part Encl. | | | | | 48 | -51 | | | | | 59 | -62 | | | | | 64 | -64 |
| 140 | 120 | B | Enclosed | 41 | -20 | | | 32 | -35 | 49 | -24 | | | 38 | -41 | 56 | -28 | | | 42 | -43 |
| 140 | 120 | B | Part Encl. | | | | | 43 | -46 | | | | | 51 | -54 | | | | | 57 | -57 |
| 140 | 120 | C | Enclosed | 41 | -20 | | | 42 | -45 | 49 | -24 | | | 51 | -55 | 56 | -28 | | | 55 | -56 |
| 140 | 120 | C | Part Encl. | | | | | 56 | -60 | | | | | 68 | -72 | | | | | 74 | -75 |
| 146 | | HVHZ | Enclosed | | | | | 45 | -49 | | | | | 55 | -60 | | | | | 60 | -61 |
| 150 | 130 | B | Enclosed | 48 | -23 | | | 37 | -40 | 55 | -28 | | | 43 | -47 | 64 | -32 | | | 48 | -50 |
| 150 | 130 | B | Part Encl. | | | | | 49 | -52 | | | | | 58 | -62 | | | | | 65 | -66 |
| 150 | 130 | C | Enclosed | 48 | -23 | | | 48 | -52 | 55 | -28 | | | 58 | -63 | 64 | -32 | | | 63 | -64 |
| 150 | 130 | C | Part Encl. | | | | | 64 | -68 | | | | | 78 | -83 | | | | | 85 | -86 |

window design pressures on each model building were designed to the minimum requirements of the SBC or FBC as appropriate. The buildings were also modeled with an appropriate roof cover, and foundation characteristics consistent with the FBC or SBC. These "designed" buildings were analyzed with HURLOSS to estimate the loss cost of each of the buildings at each location.

As described in Section 2, 300,000 years of hurricanes were simulated in HURLOSS. For each storm that produced winds greater than 50 mph peak gust winds at the building location, the loads on the building were computed and the response of the building modeled as the storm was stepped along it is simulated track. Damage and loss were computed and this process repeated for all storms. Loss costs were then computed for each combination of coverage and deductible.

### A.4.1 Choice of Base Class in Relativity Tables

Dividing the loss costs for each modeled building by the loss costs of a "central" building produces the relativities reported in this study. The "central" building is the one considered to be the most typical building. For the Group I existing construction buildings, that was a building with clips (866 lbf. resistance), 8d nails at 6″/12″ pattern, 60 mph roof cover, gable roof, with no opening protection, and no SWR.

The Group I new construction relativities are normalized by the same as the

Group I existing building study so that the relativity tables would be consistent with each other.

A comparison of the SBC 76 and SBC 88 designs for Group I buildings allowed a rational selection of a base case for the Group II and Group III buildings to be made. The Group I base case was found to roughly correspond to a SBC 76 design at 110 mpb (fastest mile). Therefore the reference case for the Groups II and III buildings is based on the SBC 76, 110 mph design.

### A.4.2 Use of Engineering Judgment Factor

The relativities produced by this process directly reflect the differences in loss costs for different construction features on a set of modeled buildings. Since the loss costs at each location are normalized by the loss costs of a "central" building at that same location, the relativities become multipliers to the insurer's estimated base loss costs for each territory. This normalization on a location-by-location basis clearly eliminates some of the modeling differences that depend on the specific approach. However, since the modeling process is not perfect and not all variables have been considered, a logical judgment factor was applied to compress the relativity range produced from these basic calculations.

### A.4.3 Simplification of Relativity Tables for Design Cases

Tables A-11 through A-14 present the relativities for FBC construction by building group, and roof deck material. These results present relevant design options for each of the locations. For example, no partially enclosed condition is shown for points in the High Velocity Hurricane Zone because all buildings in this zone must be designed as enclosed structures with opening protection.

In order to make these results useful, we have considered ways to reduce the relativity table for new construction to a smaller, easier to use table. We have examined the results in search of those variables that have the least effect on the relativities and aggregated across the range of those variables. The first is the reduction of the number of wind speed zones, and the second is the combination of the Enclosed/Partially Enclosed design options with the opening protection variable. This leaves the following key variables to consider: the terrain exposure, the wind speed zones, the roof shape, and the opening protection. The following paragraphs examine the data from Table A-11 to determine which variables must be retained in the simplified version of the new construction tables and which can be averaged into the final results.

Extra runs for Table A-11 were completed that showed the effect of applying WBDR design options to areas that did not require these options according to the FBC. This sensitivity study was completed to more accurately gauge the effect of partially enclosed options relative to enclosed options without opening protection.

#### A.4.3.1 Hip Roof vs. Gable Roof vs. Flat Roof

Notice that the difference in loss relativity (Table A-11) between gable and hip roofs for the Group I buildings is much smaller than that for single-family residential buildings. The longer length of these condo buildings means that the additional framing and straps on the hip roof are a smaller percentage of the overall roof structure in these larger buildings. Thus the beneficial effect of the hip roof is less pronounced for these types of structures compared to the single-family residential buildings.

Flat roof buildings however generally have slightly higher wind loads than gable roofs and thus the relativities for flat roofs are larger than for the equivalent hip or gable roof.

## Table A-11. Average of Relativity for Minimal Designed FBC Group I Buildings at All Simulated Points with Wood Roof Deck (2% Deductible)

| Relativity – 2% Deductible | | | Non-WBDR (Enclosed)[1] No Opening Protection | | | WBDR (Enclosed)[2] Opening Protection | | | WBDR (Part. Enclosed)[3] No Opening Protection | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Terrain Exposure | Wind Speed (Gust), mph | ID | Flat Roof | Gable Roof | Hip Roof | Flat Roof | Gable Roof | Hip Roof | Flat Roof | Gable Roof | Hip Roof |
| B | 100 | 1 | 0.596 | 0.536 | 0.446 | 0.317 | 0.296 | 0.292 | 0.433 | 0.367 | 0.321 |
| | | 2 | 0.605 | 0.539 | 0.432 | 0.318 | 0.302 | 0.294 | 0.457 | 0.384 | 0.329 |
| | 110 | 3 | 0.611 | 0.528 | 0.440 | 0.359 | 0.326 | 0.314 | 0.432 | 0.365 | 0.343 |
| | | 4 | 0.579 | 0.496 | 0.418 | 0.341 | 0.311 | 0.301 | 0.402 | 0.346 | 0.325 |
| | | 5 | 0.542 | 0.473 | 0.391 | 0.333 | 0.308 | 0.297 | 0.387 | 0.335 | 0.317 |
| | | 6 | 0.631 | 0.555 | 0.465 | 0.369 | 0.330 | 0.321 | 0.445 | 0.374 | 0.347 |
| | 120 | 7 | 0.469 | 0.390 | 0.350 | 0.338 | 0.308 | 0.302 | 0.396 | 0.334 | 0.327 |
| | | 8 | | | | 0.372 | 0.336 | 0.324 | 0.446 | 0.373 | 0.361 |
| | | 9 | | | | 0.373 | 0.334 | 0.324 | 0.439 | 0.367 | 0.353 |
| | | 10 | | | | 0.368 | 0.325 | 0.316 | 0.430 | 0.356 | 0.343 |
| | | 11 | | | | 0.380 | 0.336 | 0.326 | 0.447 | 0.375 | 0.359 |
| | 130 | 15 | 0.498 | 0.409 | 0.385 | 0.388 | 0.339 | 0.332 | 0.467 | 0.361 | 0.354 |
| | | 16 | | | | 0.360 | 0.320 | 0.315 | 0.428 | 0.341 | 0.336 |
| | | 17 | | | | 0.384 | 0.334 | 0.327 | 0.459 | 0.355 | 0.348 |
| | 140 | 21 | | | | 0.390 | 0.330 | 0.326 | 0.399 | 0.365 | 0.359 |
| | 150 | 24 | | | | 0.361 | 0.338 | 0.335 | 0.417 | 0.372 | 0.367 |
| | | 25 | | | | 0.358 | 0.336 | 0.333 | 0.415 | 0.368 | 0.363 |
| C | 120 | 12 | | | | 0.237 | 0.202 | 0.201 | 0.236 | 0.217 | 0.213 |
| | | 13 | | | | 0.227 | 0.196 | 0.197 | 0.224 | 0.209 | 0.206 |
| | | 14 | | | | 0.239 | 0.204 | 0.203 | 0.240 | 0.220 | 0.216 |
| | 130 | 18 | | | | 0.214 | 0.201 | 0.200 | 0.241 | 0.214 | 0.211 |
| | | 19 | | | | 0.220 | 0.206 | 0.206 | 0.249 | 0.221 | 0.218 |
| | | 20 | | | | 0.225 | 0.208 | 0.208 | 0.256 | 0.225 | 0.221 |
| | 140 | 22 | | | | 0.229 | 0.206 | 0.206 | 0.243 | 0.227 | 0.225 |
| | | 23 | | | | 0.226 | 0.205 | 0.205 | 0.237 | 0.223 | 0.221 |
| | 150 | 26 | | | | 0.223 | 0.212 | 0.212 | 0.247 | 0.240 | 0.237 |
| | | 27 | | | | 0.241 | 0.225 | 0.224 | 0.273 | 0.261 | 0.257 |
| HVHZ | 140 | 28 | | | | 0.240 | 0.213 | 0.213 | | | |
| | | 29 | | | | 0.217 | 0.199 | 0.200 | | | |
| | 146 | 30 | | | | 0.228 | 0.217 | 0.217 | | | |
| | | 31 | | | | 0.213 | 0.206 | 0.206 | | | |

Notes [1] Relativities for non-Wind Borne Debris Regions
[2] Relativities for Wind Borne Debris Regions with opening protection (shutters or impact resistant glazing)
[3] Relativities for Wind Borne Debris Regions where design based on partially enclosed assumption with no opening protection.
Shaded area represents cases that exceed minimum requirements of FBC.

The difference in roof geometry is only significant on wood deck Group I buildings. Table A-12 indicates the difference in relativities for concrete roofs is insignificant. For Groups II and III buildings, only flat roofs were considered in this study, and, thus, roof shape does not appear in Tables A-13 and Table A-14.

### A.4.3.2 Variation of Results with Design Wind Speed and Exposure

There is a significant difference in relativity for buildings in Terrain Exposure C verses Terrain Exposure B. Therefore, the final tables have been grouped by design exposure. The relativities from Table A-11 have been plotted on graphs in Fig. A-9 to show the variation of the relativities with location/wind speed. These graphs indicate that the variation along wind speed contours is quite small and therefore a simplified version of the minimally designed new construction relativity tables may be independent of actual location. One may also note that the variation between wind speed regions is really only significant at 100, 110 and ≥ 120 mph levels. Therefore the simplified tables (presented in Section 4) are reduced to three wind speed regions.

## Table A-12. Average of Relativity for Minimal Designed FBC Group I Buildings at All Simulated Points with Concrete Roof Deck (2% Deductible)

| Relativity – 2% Deductible | | | Non-WBDR (Enclosed)[1] | | | WBDR (Enclosed)[2] | | | WBDR (Part. Enclosed)[3] | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | No Opening Protection | | | Opening Protection | | | No Opening Protection | | |
| Terrain Exposure | Wind Speed (Gust), mph | ID | Flat Roof | Gable Roof | Hip Roof | Flat Roof | Gable Roof | Hip Roof | Flat Roof | Gable Roof | Hip Roof |
| B | 100 | 1 | 0.282 | 0.289 | 0.283 | | | | | | |
| | | 2 | 0.303 | 0.319 | 0.300 | | | | | | |
| | 110 | 3 | 0.382 | 0.379 | 0.359 | | | | | | |
| | | 4 | 0.362 | 0.361 | 0.343 | | | | | | |
| | | 5 | 0.341 | 0.342 | 0.322 | | | | | | |
| | | 6 | 0.389 | 0.387 | 0.366 | | | | | | |
| | 120 | 7 | 0.321 | 0.305 | 0.296 | | | | | | |
| | | 8 | | | | 0.271 | 0.273 | 0.271 | 0.297 | 0.299 | 0.295 |
| | | 9 | | | | 0.270 | 0.271 | 0.270 | 0.291 | 0.292 | 0.287 |
| | | 10 | | | | 0.267 | 0.267 | 0.266 | 0.284 | 0.287 | 0.283 |
| | | 11 | | | | 0.271 | 0.272 | 0.270 | 0.295 | 0.297 | 0.292 |
| | 130 | 15 | 0.336 | 0.323 | 0.315 | 0.271 | 0.273 | 0.271 | 0.296 | 0.301 | 0.297 |
| | | 16 | | | | 0.264 | 0.266 | 0.265 | 0.281 | 0.286 | 0.284 |
| | | 17 | | | | 0.269 | 0.270 | 0.269 | 0.290 | 0.294 | 0.291 |
| | 140 | 21 | | | | 0.270 | 0.272 | 0.271 | 0.296 | 0.302 | 0.299 |
| | 150 | 24 | | | | 0.273 | 0.276 | 0.274 | 0.304 | 0.311 | 0.307 |
| | | 25 | | | | 0.271 | 0.274 | 0.273 | 0.302 | 0.308 | 0.305 |
| C | 120 | 12 | | | | 0.164 | 0.164 | 0.164 | 0.177 | 0.177 | 0.174 |
| | | 13 | | | | 0.162 | 0.162 | 0.162 | 0.171 | 0.170 | 0.169 |
| | | 14 | | | | 0.163 | 0.163 | 0.163 | 0.177 | 0.177 | 0.174 |
| | 130 | 18 | | | | 0.163 | 0.163 | 0.162 | 0.172 | 0.174 | 0.172 |
| | | 19 | | | | 0.164 | 0.164 | 0.163 | 0.176 | 0.178 | 0.175 |
| | | 20 | | | | 0.165 | 0.165 | 0.165 | 0.178 | 0.180 | 0.177 |
| | 140 | 22 | | | | 0.165 | 0.164 | 0.163 | 0.182 | 0.185 | 0.182 |
| | | 23 | | | | 0.163 | 0.162 | 0.162 | 0.177 | 0.180 | 0.177 |
| | 150 | 26 | | | | 0.163 | 0.163 | 0.163 | 0.188 | 0.192 | 0.188 |
| | | 27 | | | | 0.168 | 0.167 | 0.166 | 0.203 | 0.208 | 0.203 |
| HVHZ | 140 | 28 | | | | 0.162 | 0.162 | 0.162 | | | |
| | | 29 | | | | 0.159 | 0.160 | 0.160 | | | |
| | 146 | 30 | | | | 0.162 | 0.163 | 0.162 | | | |
| | | 31 | | | | 0.160 | 0.161 | 0.160 | | | |

[1] Relativities for non-Wind Borne Debris Regions
[2] Relativities for Wind Borne Debris Regions with opening protection (shutters or impact resistant glazing)
[3] Relativities for Wind Borne Debris Regions where design based on partially enclosed assumption with no opening protection.

### A.4.3.3 Partially Enclosed vs. Enclosed (No Shutters)

The results in Table A-11 indicate that the partially enclosed design case is not as effective at reducing losses as the enclosed design case. Although the partially enclosed case has stronger components, it still does not address the issue of protecting the openings on the building. Figure A-10 shows the damage curves for the Partially Enclosed, the Enclosed with no opening protection, and the Enclosed with opening protection versions of one of the Group I buildings in Niceville, Exposure B (Point 15). The difference between the simulations is in the roof-wall connection, and

**Table A-13. Average of Relativity for Minimal Designed FBC Group II Buildings at All Simulated Points (2% Deductible)**

| Relativity – 2% Deductible | | | Wood Deck | | | Metal Deck | | | Concrete Deck | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Non-WBDR | WBDR | | Non-WBDR | WBDR | | Non-WBDR | WBDR | |
| Terrain Exposure | Wind Speed (Gust) | ID | No Opening Protect.[1] | Opening Protect.[2] | No Opening Protect.[3] | No Opening Protect.[1] | Opening Protect.[2] | No Opening Protect.[3] | No Opening Protect.[1] | Opening Protect.[2] | No Opening Protect.[3] |
| B | 100 | 1 | 0.532 | | | 0.602 | | | 0.406 | | |
| | | 2 | 0.556 | | | 0.632 | | | 0.419 | | |
| | 110 | 3 | 0.693 | | | 0.742 | | | 0.571 | | |
| | | 4 | 0.637 | | | 0.691 | | | 0.521 | | |
| | | 5 | 0.618 | | | 0.672 | | | 0.487 | | |
| | | 6 | 0.705 | | | 0.766 | | | 0.570 | | |
| | 120 | 7 | 0.521 | | | 0.534 | | | 0.385 | | |
| | | 8 | | 0.354 | 0.441 | | 0.329 | 0.413 | | 0.196 | 0.286 |
| | | 9 | | 0.349 | 0.423 | | 0.317 | 0.389 | | 0.191 | 0.264 |
| | | 10 | | 0.362 | 0.443 | | 0.325 | 0.406 | | 0.190 | 0.269 |
| | | 11 | | 0.345 | 0.417 | | 0.314 | 0.381 | | 0.192 | 0.262 |
| | 130 | 15 | 0.518 | 0.374 | 0.390 | 0.503 | 0.327 | 0.390 | 0.367 | 0.195 | 0.271 |
| | | 16 | | 0.363 | 0.366 | | 0.311 | 0.365 | | 0.186 | 0.245 |
| | | 17 | | 0.385 | 0.399 | | 0.335 | 0.401 | | 0.193 | 0.270 |
| | 140 | 21 | | 0.324 | 0.408 | | 0.321 | 0.390 | | 0.192 | 0.271 |
| | 150 | 24 | | 0.335 | 0.387 | | 0.318 | 0.390 | | 0.195 | 0.280 |
| | | 25 | | 0.332 | 0.384 | | 0.316 | 0.386 | | 0.193 | 0.277 |
| C | 120 | 12 | | 0.269 | 0.337 | | 0.259 | 0.333 | | 0.162 | 0.225 |
| | | 13 | | 0.256 | 0.311 | | 0.246 | 0.307 | | 0.160 | 0.206 |
| | | 14 | | 0.275 | 0.356 | | 0.268 | 0.355 | | 0.162 | 0.238 |
| | 130 | 18 | | 0.280 | 0.328 | | 0.259 | 0.336 | | 0.161 | 0.232 |
| | | 19 | | 0.289 | 0.341 | | 0.268 | 0.351 | | 0.162 | 0.239 |
| | | 20 | | 0.295 | 0.350 | | 0.273 | 0.363 | | 0.163 | 0.246 |
| | 140 | 22 | | 0.265 | 0.346 | | 0.266 | 0.350 | | 0.164 | 0.262 |
| | | 23 | | 0.264 | 0.335 | | 0.265 | 0.340 | | 0.162 | 0.246 |
| | 150 | 26 | | 0.261 | 0.365 | | 0.278 | 0.372 | | 0.167 | 0.283 |
| | | 27 | | 0.279 | 0.406 | | 0.301 | 0.417 | | 0.174 | 0.323 |
| HVHZ | 140 | 28 | | 0.286 | | | 0.290 | | | 0.168 | |
| | | 29 | | 0.263 | | | 0.264 | | | 0.161 | |
| | 146 | 30 | | 0.270 | | | 0.312 | | | 0.170 | |
| | | 31 | | 0.254 | | | 0.285 | | | 0.163 | |

[1] Relativities for non-Wind Borne Debris Regions
[2] Relativities for Wind Borne Debris Regions with opening protection (shutters or impact resistant glazing)
[3] Relativities for Wind Borne Debris Regions where design based on partially enclosed assumption with no opening protection.

the opening protection as shown in Table A-15. The partially enclosed case has roof straps that are 37% stronger than the enclosed case, which means that the whole roof fails about one thirdas often during the severe wind events. However, the window damage for the partially enclosed case is essentially the same as the enclosed case without opening protection. There is some savings from the higher DP rating of the partially enclosed case, but otherwise the damage level of the windows is the same. The higher levels of fenestration damage cause more damage internally which drives up the loss costs to higher levels. Note

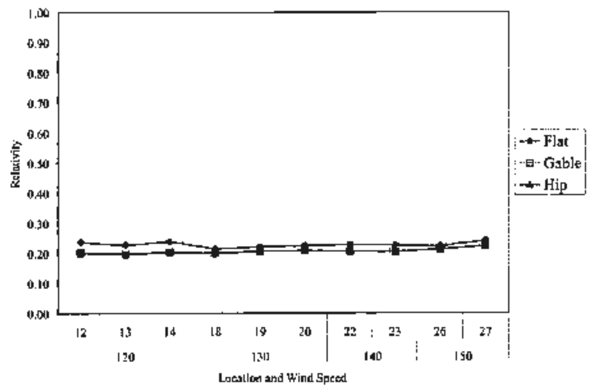
**Table A-14. Average of Relativity for Minimal Designed FBC Group III Buildings at All Simulated Points (2% Deductible)**

| Relativity – 2% Deductible | | | Metal Deck | | | Concrete Deck | | |
|---|---|---|---|---|---|---|---|---|
| | | | Non-WBDR | WBDR | | Non-WBDR | WBDR | |
| Terrain Exposure | Wind Speed (Gust) | ID | No Opening Protect.[1] | Opening Protect.[2] | No Opening Protect.[3] | No Opening Protect.[1] | Opening Protect.[2] | No Opening Protect.[3] |
| B | 100 | 1 | 0.837 | | | 0.633 | | |
| | | 2 | 0.912 | | | 0.686 | | |
| | 110 | 3 | 0.747 | | | 0.606 | | |
| | | 4 | 0.708 | | | 0.563 | | |
| | | 5 | 0.698 | | | 0.541 | | |
| | | 6 | 0.770 | | | 0.615 | | |
| | 120 | 7 | 0.511 | | | 0.382 | | |
| | | 8 | | 0.272 | 0.356 | | 0.148 | 0.234 |
| | | 9 | | 0.273 | 0.343 | | 0.148 | 0.217 |
| | | 10 | | 0.278 | 0.357 | | 0.147 | 0.224 |
| | | 11 | | 0.271 | 0.338 | | 0.149 | 0.216 |
| | 130 | 15 | 0.444 | 0.276 | 0.332 | 0.323 | 0.148 | 0.225 |
| | | 16 | | 0.271 | 0.312 | | 0.145 | 0.202 |
| | | 17 | | 0.282 | 0.342 | | 0.146 | 0.227 |
| | 140 | 21 | | 0.273 | 0.326 | | 0.146 | 0.225 |
| | 150 | 24 | | 0.244 | 0.319 | | 0.148 | 0.228 |
| | | 25 | | 0.242 | 0.318 | | 0.147 | 0.228 |
| C | 120 | 12 | | 0.265 | 0.339 | | 0.162 | 0.252 |
| | | 13 | | 0.258 | 0.314 | | 0.161 | 0.228 |
| | | 14 | | 0.269 | 0.359 | | 0.162 | 0.267 |
| | 130 | 18 | | 0.248 | 0.338 | | 0.161 | 0.259 |
| | | 19 | | 0.255 | 0.352 | | 0.162 | 0.269 |
| | | 20 | | 0.258 | 0.361 | | 0.164 | 0.278 |
| | 140 | 22 | | 0.248 | 0.357 | | 0.164 | 0.289 |
| | | 23 | | 0.248 | 0.345 | | 0.163 | 0.273 |
| | 150 | 26 | | 0.245 | 0.382 | | 0.168 | 0.316 |
| | | 27 | | 0.260 | 0.423 | | 0.176 | 0.358 |
| HVHZ | 140 | 28 | | 0.263 | | | 0.168 | |
| | | 29 | | 0.246 | | | 0.161 | |
| | 146 | 30 | | 0.252 | | | 0.170 | |
| | | 31 | | 0.237 | | | 0.163 | |

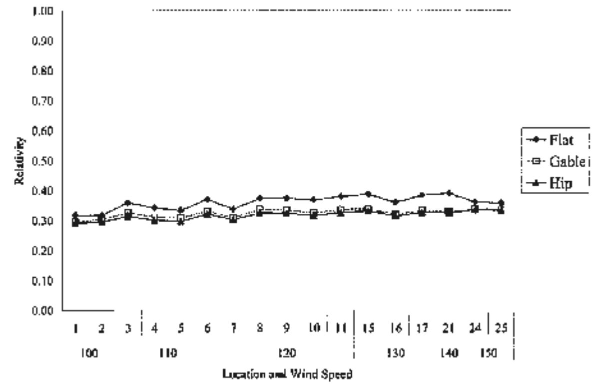
[1] Relativities for non-Wind Borne Debris Regions
[2] Relativities for Wind Borne Debris Regions with opening protection (shutters or impact resistant glazing)
[3] Relativities for Wind Borne Debris Regions where design based on partially enclosed assumption with no opening protection.

how the addition of opening protection has an effect on the whole roof failures that is similar to the 37% larger roof straps. The opening protection prevents breaches of the envelope and prevents large internal pressures from developing inside the structure.

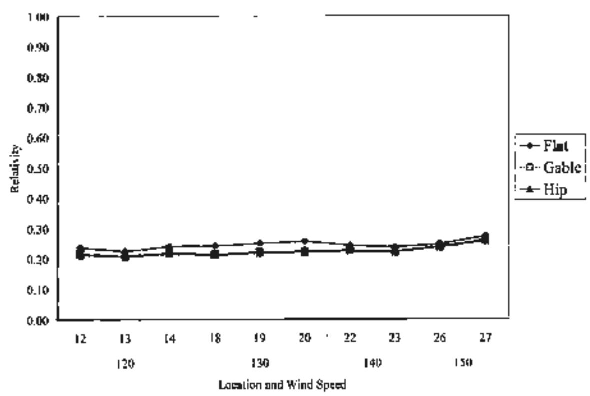
Closer examination of the results in Table A-11 indicates that the difference between the two cases without opening protection decrease with increasing wind speed. That is the effectiveness of simply strengthening the roofing connections and window DPs decreases as the number of wind borne missiles and the energy of those missiles



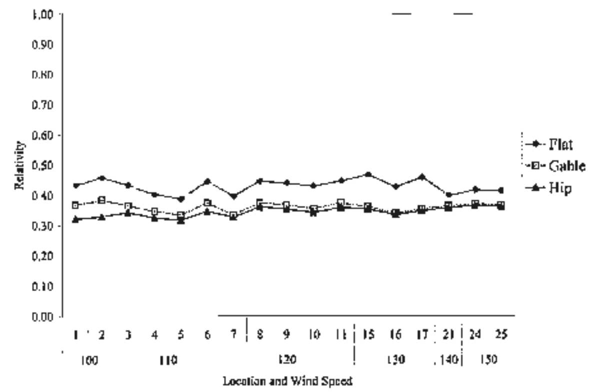

**(a) Exposure C, Opening Protection, WBDR**




**(b) Exposure B, Opening Protection, WBDR**



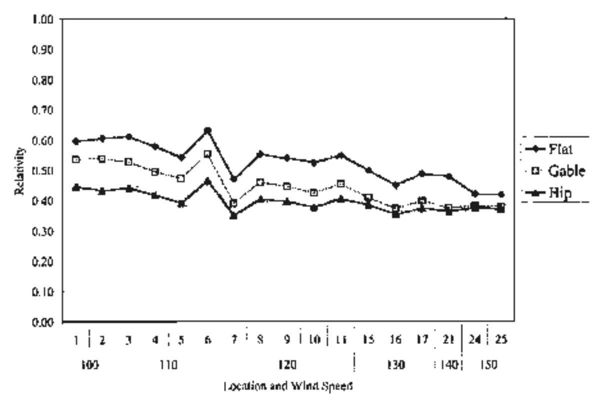

**(c) Exposure C, No Opening Protection, WBDR**




**(d) Exposure B, No Opening Protection, WBDR**

**No figure shown because combination of Exposure C and non-WBDR does not exist in FBC**

**(e) Exposure C, No Opening Protection, Non-WBDR**




**(e) Exposure B, No Opening Protection, Non-WBDR**

**Figure A-9. Comparison of Loss Relativity (2% Deductible) across Location and Wind Speed for Minimum Designed FBC Group I Buildings with Wood Roof Deck (Table A-11)**

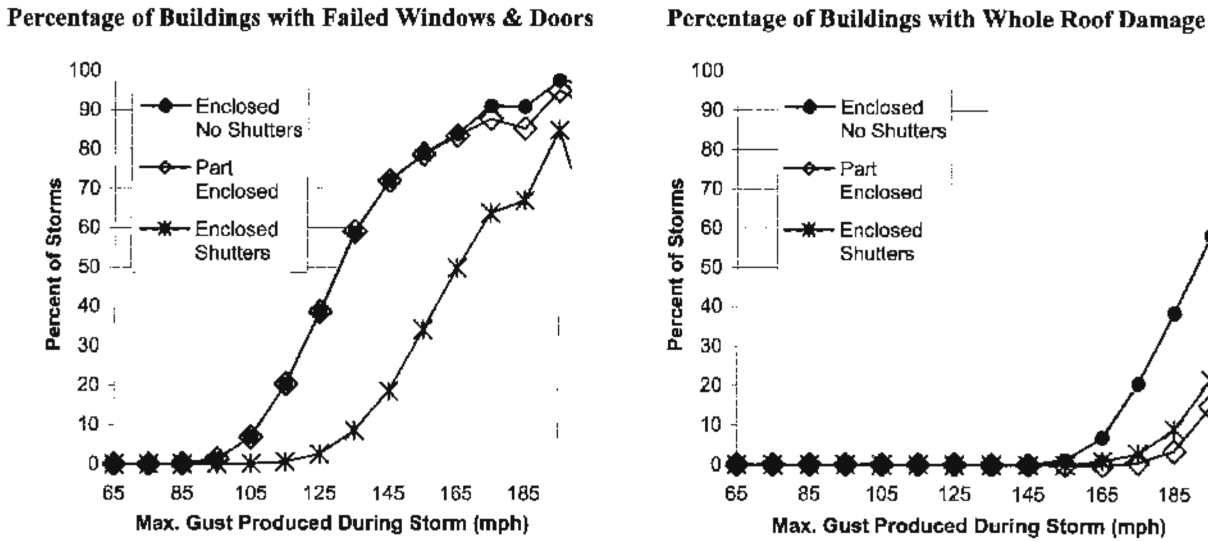



**Figure A-10. Comparison of Design Options for Group I Buildings in Exposure B, 120 mph at Location 15**

**Table A-15. Difference in Modeled Parameters for Cases in Figure A-10**

| Parameter | Internal Pressure | | |
|---|---|---|---|
| | **Enclosed** | **Partially Enclosed** | **Enclosed** |
| Roof-Wall Strap | 1822 lbf. | 2514 lbf. | 1822 lbf. |
| Roof Deck Nailing Pattern | 8d @ 6″/12″ | 8d @ 6″/12″ | 8d @ 6″/12″ |
| Max Fen DP ratings | -28 psf | -36 psf | -28 psf |
| Opening Protection | No | No | Yes |

increases with wind speed. The difference becomes significant at the 100-120 mph wind speed regions. Thus, the final relativities table will retain the internal pressure variable as a separate rating variable.

### A.4.4 Comparison of New Construction Relativities to Existing Constrnction

The relativity of the new construction designs has been referenced to the existing construction matrix to ensure consistent application of relativities. This section compares the relativity from Section 3 with an equivalent relativity from Section 4 and explains the reason that there are slight differences between the tables.

To determine where the new construction parameters map onto the existing building matrix, one must know the design capacity of the straps labeled as Clip, Single Wrap and Double Wrap, as given in Table A-1.

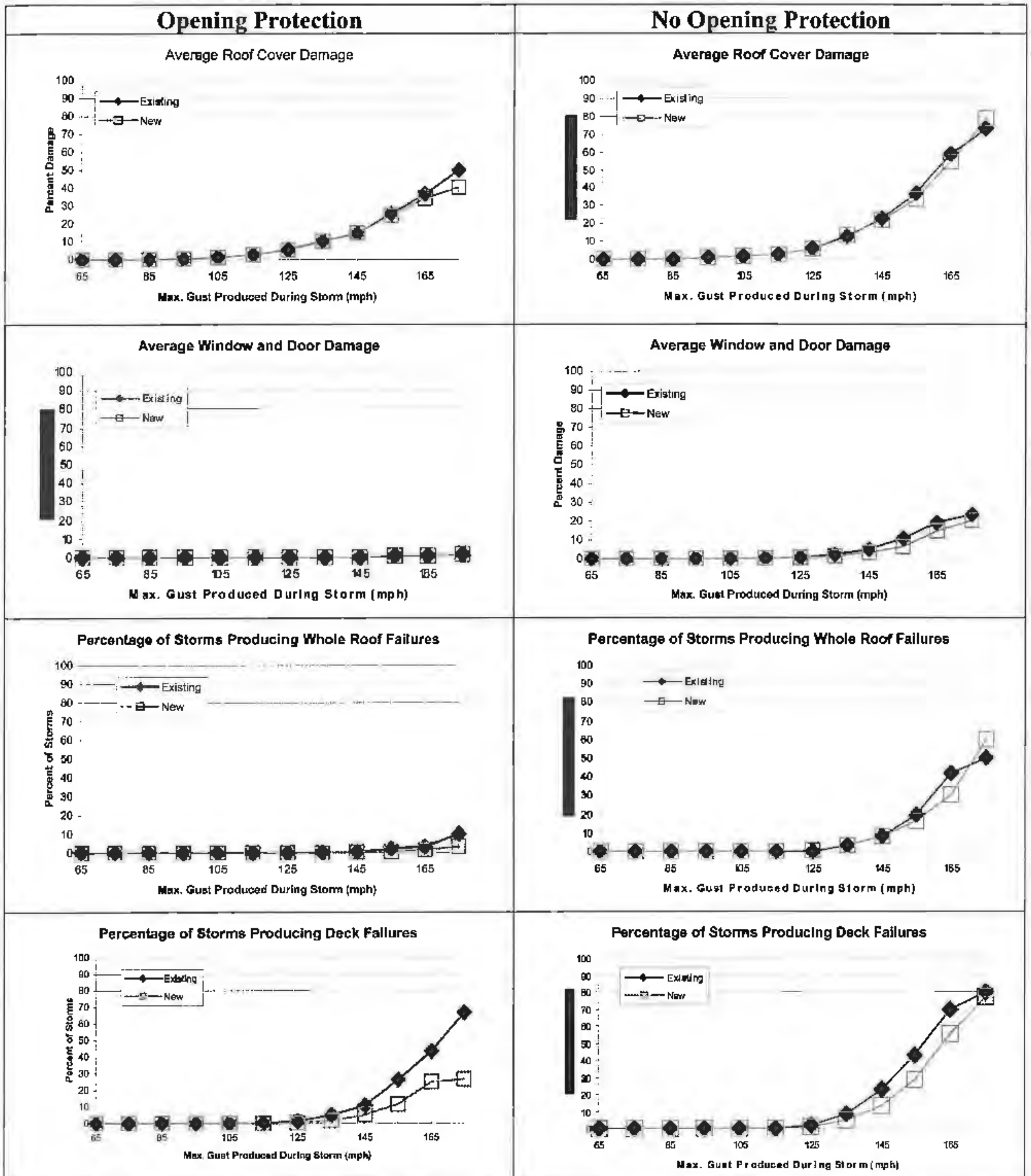
*Compare Design for 100 mph Exposure B.* We will compare one of the existing construction cases to the enclosed design for 100 mph gust wind speed in Exposure B from Table A-11. Location 1 is in Gainesville. We assume both the existing and new construction cases have FBC Equivalent roof covers on a gable roof. These design conditions indicate that for a wood roof, the roof strap capacity by the FBC is 1125 lbf. (see Table A-5), and the deck nailing pattern by the

FBC is 8d at 6″/12″ pattern (Table A-3). This combination of roof strap and nailing patterns corresponds to the Single Wrap, Deck Attachment B with FBC Equivalent roof cover from Table 3-1. The relativity for these existing cases are 0.73 without opening protection and 0.35 with opening protection. We compare these relativities to the equivalent new construction relativity for the 100 mph Exp B case of 0.54 without opening protection, and 0.30 with opening protection in Table A-11.

The mean strengths of various components are listed at the bottom of Fig. A-11. Although the mean strength of the roof straps is lower for new construction, the modeled strength of the straps on existing construction actually has a wider range of simulated values, and therefore allow the whole roof to fail more often than in the new construction case. The wider range is used in the existing construction because the single wrap strap is representative of a wide range of strap classes and capacities. The same concept applies to the window DP ratings as well. Although the mean strength of the window DP for new construction is slightly lower than the existing construction, the range of simulated resistances is wider for the existing construction, and will allow more window failures than the new construction simulation. Also note the difference in nailing patterns between the new and existing construction cases. The patterns are essentially the same except that a 4" spacing is used at the gable end as required in the FBC, instead of the 6" spacing which was maintained on the existing construction cases. This change means that the roof deck sections at the gable end that are the most susceptible to failure, are stronger in the new construction case than the existing construction case.

Figure A-11 presents damage curves for roof cover, roof deck, windows, and whole roof failures for these four cases. Note that the damage curves for the existing and new cases in this figure are very similar to each other. For the cases with no opening protection, one can observe slightly smaller damage curves for windows, whole roof (roof straps), and roof deck failures. The relativity of the new construction for this case (0.54) is less than the existing construction table (0.73) primarily because of the improved roof-deck nailing pattern. There are also some benefits seen from the more focused distribution of window and roof strap strengths in the new construction simulation.

Figure A-11 also offers another example of the effectiveness of opening protection on the relativity. When opening protection is added, the window and door damage drops to nearly zero, and the whole roof failures of the new construction are dropped by a factor of 4 as well, which translates into reduced roof covering losses as well.




| | Existing | New | | Existing | New |
|---|---|---|---|---|---|
| Strap (lbs): | 1200 | 1125 | Fen DP: | 20 psf. | 18 psf. |
| Deck Nailing: | 8d @ 6"/12"/6" | 8d @ 6"/12"/4" | Roof Cover | FBC Equiv | FBC Equiv |

**Figure A-11. Comparison of Existing Construction and New Construction Simulations for Group I Building at Gainesville in Exposure B**

# APPENDIX B:

# EXAMPLE DESIGN CALCULATIONS BY FBC, SBC 88, AND SBC 76

# APPENDIX B: EXAMPLE DESIGN CALCULATIONS BY ASCE 7-98, SBC 88, AND SBC 76

This appendix contains one sample set of design calculations of the analysis completed in Section 4 of this report. This sample is for a Group I building designed according to Florida Building Code Section 1606 (based on ASCE 7-98), Standard Building Code 1988, and Standard Building Code 1976.

The dimensions of the building, and other key parameters such as truss spacing are defined on page B-3 under the section called "Geometry of Building". A sample of the sizes of the windows, and doors are defined on page B-9. Once the configuration of the building is established, these calculations compute the design parameters for the following:

- Roof deck nailing,
- Fenestration design pressures,
- Roof-wall connection design,

The input parameters are the design wind speed and terrain exposure, and the internal pressure condition (Enclosed vs. Partially Enclosed), as appropriate. Note that SBC 76 and SBC 88 do not have an exposure variable.

This particular sample design has been prepared for 130 mph gust design wind speed in Terrain Exposure C for an Enclosed Building condition under FBC and 110 mph fastest mile wind speed under SBC. Recall that 110 mph fastest mile wind speed is equivalent to about 130 mph gust wind speed.

This set of calculations was repeated for each of the FBC/SBC combinations of wind speed, terrain exposure, and internal pressure condition listed in Table 2-1 for each of the modeled buildings. The results of these calculations are summarized in Appendix A of this report.

One may note that the nailing patterns for wood decks on Group I buildings appear to be slightly weaker than those reported in the single family report, even though the wind loads are higher for two story than single story structures. This design calculation is based on a higher wood density, which is more common on commercial and large-scale residential projects. This higher wood density yields a higher nail pullout strength and thus the Group I designs reported here will use slightly fewer nails.

# ASCE7-98 (FBC)

Loads on single story building with flat roof slope (less than 10 degrees)

**Input Parameters**

$in0 := (130\ \ C\ \ Enclosed)$

Define design parameters

**Variables for Exposure**

$$\begin{pmatrix} A \\ B \\ C \\ D \end{pmatrix} \equiv \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \end{pmatrix}$$

**Variables for Enclosed/Part Encl.**

$Enclosed \equiv 0$

$PartEnclosed \equiv 1$

**Design Parameters**

$V := \left|in0^{\langle 0\rangle}\right| \cdot mph$ $\qquad V = 130\,mph$

$I := 1.0$ Importance for Class II Building

$Exp := \left|in0^{\langle 1\rangle}\right|$ $\qquad Exp = 2$

$IntPressure := \left|in0^{\langle 2\rangle}\right|$ $\qquad IntPressure = 0$

$Case := 1$ Case 1 = C&C and MWFRS for low rise bldgs

**Geometry of Building:** Building Name: 0023 - Condo project

$h := \left(\frac{22.18 + 22.18}{2}\right) \cdot ft$ ht of building $\qquad h = 22.18\,ft$

$\theta := atan\left(\frac{0}{12}\right)$ $\qquad \theta = 0\,deg$ $\qquad$ roof slope

$o := 0.0 \cdot ft$ overhang width

$o_g := 0 \cdot ft$

$W := 38ft + 2 \cdot o$ dimensions of building

$L := 192ft + 2 \cdot o$

$\Delta := 24 \cdot in$ Truss spacing

Roof cover: Shingle

$h_{wall} := 9 \cdot ft$ Height of Wall, single story

**Dead load of roof**

$DL_{roof} := 9 \cdot psf$ Hip roof, shingle, trusses, underlayment (from SBC Appendix A)

$DL_{sheath} := (0.5 \cdot in) \cdot \left(\frac{0.4psf}{.125 \cdot in}\right)$ $\qquad DL_{sheath} = 1.6\,psf$

Dead load of roof is composed of following: Truss/Sheathing (7 psf), Tile (10psf). If shingles are used, use 2 psf instead of 10 psf.

$L_{attic} := 30 \cdot psf$ SBC Table 1604.1

$\phi := 0.6$ Fraction of DeadLoad used in combination with Wind Load

$L_{floor} := 40 \cdot psf$

$L_{roof} := 16 \cdot psf$

$DL_{wall} := \begin{pmatrix} 10 \\ 55 \end{pmatrix} \cdot psf$ Wood Frame wall weight / Masonry Wall Weight

$DL_{misc} := 15 \cdot psf$ Miscellaneous: Contents, carpet, cabinets, fixtures)

## Dynamic Wind Pressure

### Terrain Exposure Constants

$$z_g := \begin{pmatrix} 1500 \cdot ft \\ 1200 \cdot ft \\ 900 \cdot ft \\ 700 \cdot ft \end{pmatrix} \quad \alpha := \begin{pmatrix} 5.0 \\ 7.0 \\ 9.5 \\ 11.5 \end{pmatrix} \quad h_{min} := \begin{pmatrix} 60 \\ 30 \\ 15 \\ 7 \end{pmatrix} \cdot ft$$

Exposures = A,B,C,D

$$h_{min} := \left| \begin{array}{ll} \begin{pmatrix} 100 \\ 30 \\ 15 \\ 15 \end{pmatrix} \cdot ft & \text{if } Case = 1 \\ \begin{pmatrix} 15 \\ 15 \\ 15 \\ 15 \end{pmatrix} \cdot ft & \text{otherwise} \end{array} \right.$$

$$K_z(h) := \left| \begin{array}{ll} 2.01 \cdot \left( \dfrac{15 \cdot ft}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{if } (h < 15 \cdot ft) \\ 2.01 \cdot \left( \dfrac{h_{min_{Exp}}}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{if } \left( h \le h_{min_{Exp}} \right) \\ 2.01 \cdot \left( \dfrac{h}{z_{g_{Exp}}} \right)^{\frac{2}{\alpha_{Exp}}} & \text{otherwise} \end{array} \right.$$

$h_{min_{Exp}} = 15\,ft$

$K_z(h) = 0.92$

$K_{zt} := 1.0$ No topographic speedup

$K_d := 0.85$ Directionality factor (0.85 used when doing combination loads - with dead load)

$$q_h := .00256 \frac{slug}{2.15111 ft^3} \cdot K_z(h) \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I \qquad q_h = 33.9\,psf$$

Dynamic Wind Pressure

### Internal Pressure coefficient

$$GC_{pi} := \left| \begin{array}{ll} \begin{pmatrix} -0.18 \\ 0.18 \end{pmatrix} & \text{if } IntPressure = Enclosed \\ \begin{pmatrix} -0.55 \\ 0.55 \end{pmatrix} & \text{if } IntPressure = PartEnclosed \\ \begin{pmatrix} -20 \\ 20 \end{pmatrix} & \text{otherwise} \end{array} \right.$$

Dummy value in Case Int Pressure is invalid

$$GC_{pi} = \begin{pmatrix} -0.18 \\ 0.18 \end{pmatrix}$$

internal pressure range variable

$posneg := 0..1$

**Gust Factor:**

Terrain Exposure Constants from Table 6-4

$$l := \begin{pmatrix} 180 \\ 320 \\ 500 \\ 650 \end{pmatrix} \cdot \text{ft} \qquad \varepsilon := \begin{pmatrix} \frac{1}{2} \\ \frac{1}{3} \\ \frac{1}{5} \\ \frac{1}{8} \end{pmatrix} \qquad c := \begin{pmatrix} 0.45 \\ 0.3 \\ 0.2 \\ 0.15 \end{pmatrix} \quad z_{min} := \begin{pmatrix} 60 \\ 30 \\ 15 \\ 7 \end{pmatrix} \cdot \text{ft}$$

$$z_e := \begin{pmatrix} 0.6 \cdot h \\ z_{min_{Exp}} \end{pmatrix} \qquad z_c := \max(z_c) \qquad z_e = 15\,\text{ft}$$ Equivalent height of structure

$$I_z := c_{Exp} \cdot \left(\frac{33 \cdot \text{ft}}{z_e}\right)^{\frac{1}{6}} \qquad I_z = 0.23$$ Turbulence Intensity (eqn 6-3)

$$L_z := l_{Exp} \cdot \left(\frac{z_c}{33 \cdot \text{ft}}\right)^{\varepsilon_{Exp}} \qquad L_z = 427.06\,\text{ft}$$ Integral Length Scale of Turbulence (Eqn 6-5)

$$Q := \sqrt{\frac{1}{1 + 0.63 \cdot \left(\frac{W + h}{L_z}\right)^{0.63}}} \qquad Q = 0.92$$ Background Response (Eqn 6-4)

$$g_Q := 3.4 \qquad g_v := 3.4$$

$$G := 0.925 \cdot \left(\frac{1 + 1.7 \cdot g_Q \cdot I_z \cdot Q}{1 + 1.7 \cdot g_v \cdot I_z}\right) \qquad G = 0.88$$ Gust Factor (Eqn 6-2)

**External Pressure Coefficients: Figure 6-5B**

Limits of External Pressure Coefficients for each Zone in C&C loads
( first row neg coefficients, second row positive coefficents)

10SF neg 100SF neg, 1000SF neg
10SF pos 100SF pos, 1000 SF pos

If slope is less than 10 degrees:

$$GCp\_10deg := \begin{bmatrix} 0 \\ \begin{pmatrix} -1 & -0.9 & -0.9 \\ 0.3 & 0.2 & 0.2 \end{pmatrix} \\ \begin{pmatrix} -1.8 & -1.1 & -1.1 \\ 0.3 & 0.2 & 0.2 \end{pmatrix} \\ \begin{pmatrix} -2.8 & -1.1 & -1.1 \\ 0.3 & 0.2 & 0.2 \end{pmatrix} \\ 0.9 \cdot \begin{pmatrix} -1.1 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ 0.9 \cdot \begin{pmatrix} -1.4 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ \begin{pmatrix} -1.7 & -1.6 & -1.1 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -1.7 & -1.6 & -1.1 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -2.8 & -0.8 & -0.8 \\ 0 & 0 & 0 \end{pmatrix} \end{bmatrix}$$

Zone 1 roof coefficients ASCE7-98: Figure 6-5B
Zone 2
Zone 3
Zone 4 wall coefficients, ASCE7-98 Fig 6-5A
Zone 5
Zone 1 , overhang
Zone 2, overhang
Zone 3, overhang

Alim is the x axis values of the change in slope of the GCP graphs in Fig 6-5 and 6-4

$$Alim\_10deg := \begin{bmatrix} 0 \\ (10 \ \ 100 \ \ 1000) \\ (10 \ \ 100 \ \ 1000) \\ (10 \ \ 100 \ \ 1000) \\ (10 \ \ 500 \ \ 1000) \\ (10 \ \ 500 \ \ 1000) \\ (10 \ \ 100 \ \ 500) \\ (10 \ \ 100 \ \ 500) \\ (10 \ \ 100 \ \ 1000) \end{bmatrix} \cdot ft^2$$

Zone 1
Zone 2
Zone 3
Zone 4
Zone 5
Zone 1 , ohang
Zone 2, ohang
Zone 3, ohang

If slope is 10 to 30 degrees:

$$GCp\_30deg := \begin{bmatrix} 0 \\ \begin{pmatrix} -0.9 & -0.8 & -0.8 \\ 0.5 & 0.3 & 0.3 \end{pmatrix} \\ \begin{pmatrix} -2.1 & -1.4 & -1.4 \\ 0.5 & 0.3 & 0.3 \end{pmatrix} \\ \begin{pmatrix} -2.1 & -1.4 & -1.4 \\ 0.5 & 0.3 & 0.3 \end{pmatrix} \\ \begin{pmatrix} -1.1 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ \begin{pmatrix} -1.4 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ \begin{pmatrix} -0.9 & -0.8 & -0.8 \\ 0.5 & 0.3 & 0.3 \end{pmatrix} \\ \begin{pmatrix} -2.2 & -2.2 & -2.2 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -3.7 & -2.5 & -2.5 \\ 0 & 0 & 0 \end{pmatrix} \end{bmatrix}$$

Zone 1 roof coefficients ASCE7-98: Figure 6-5B

Zone 2

Zone 3

Zone 4 wall coefficients, ASCE7-98 Fig 6-5A

Zone 5

Zone 1 , overhang - assumed same as Zone 1 no overhang

Zone 2, overhang

Zone 3, overhang

$$Alim\_30deg := \begin{bmatrix} 0 \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 500 \;\; 1000) \\ (10 \;\; 500 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \end{bmatrix} \cdot ft^2$$

If slope is 30 to 45 degrees:

$$GCp\_45deg := \begin{bmatrix} 0 \\ \begin{pmatrix} -1.0 & -0.8 & -0.8 \\ 0.9 & 0.8 & 0.8 \end{pmatrix} \\ \begin{pmatrix} -1.2 & -1.0 & -1.0 \\ 0.9 & 0.8 & 0.8 \end{pmatrix} \\ \begin{pmatrix} -1.2 & -1.0 & -1.0 \\ 0.9 & 0.8 & 0.8 \end{pmatrix} \\ \begin{pmatrix} -1.1 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ \begin{pmatrix} -1.4 & -0.8 & -0.8 \\ 1 & 0.7 & 0.7 \end{pmatrix} \\ \begin{pmatrix} -1.0 & -0.8 & -0.8 \\ 0.9 & 0.8 & 0.8 \end{pmatrix} \\ \begin{pmatrix} -2.0 & -1.8 & -1.8 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -2.0 & -1.8 & -1.8 \\ 0 & 0 & 0 \end{pmatrix} \end{bmatrix}$$

Zone 1 roof coefficients ASCE7-98: Figure 6-5B

Zone 2

Zone 3

Zone 4 wall coefficients, ASCE7-98 Fig 6-5A

Zone 5

Zone 1 , overhang - assumed same as Zone 1 no overhang

Zone 2, overhang

Zone 3, overhang

$$Alim\_45deg := \begin{bmatrix} 0 \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 500 \;\; 1000) \\ (10 \;\; 500 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \\ (10 \;\; 100 \;\; 1000) \end{bmatrix} \cdot ft^2$$

Select appropriate set of parameters according to slope of roof:

$\theta = 0\,\mathrm{deg}$

$$GCp := \begin{cases} GCp\_10deg & \text{if } (\theta \le 10\cdot deg) \\ GCp\_30deg & \text{if } 10\cdot deg < \theta \le 30\cdot deg \\ GCp\_45deg & \text{if } 30\cdot deg < \theta \le 45\cdot deg \\ 0 & \text{otherwise} \end{cases} \qquad Alim := \begin{cases} Alim\_10deg & \text{if } (\theta \le 10\cdot deg) \\ Alim\_30deg & \text{if } 10\cdot deg < \theta \le 30\cdot deg \\ Alim\_45deg & \text{if } 30\cdot deg < \theta \le 45\cdot deg \\ 0 & \text{otherwise} \end{cases}$$

Calculate slopes of parts of pressure coefficient graphs for interpolation:

$$slope_{GCp}(Zone) := \frac{\left(GCp_{Zone}\right)^{\langle 1\rangle} - \left(GCp_{Zone}\right)^{\langle 0\rangle}}{\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right] - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]}$$

Slope of first section of line

$$slope_{GCp2}(Zone) := \frac{\left(GCp_{Zone}\right)^{\langle 2\rangle} - \left(GCp_{Zone}\right)^{\langle 1\rangle}}{\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 2\rangle}\right|}{ft^2}\right] - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right]}$$

slope of secondary section of line
(usually flat)

$$GC_p(Area, Zone) := \begin{cases} \left(GCp_{Zone}\right)^{\langle 0\rangle} & \text{if } Area < \left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right| \\ \left[\left(slope_{GCp}(Zone)\right)\cdot\left[\log\left(\frac{Area}{ft^2}\right) + -\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]\right] + \left(GCp_{Zone}\right)^{\langle 0\rangle}\right] & \text{if } \left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right| \le Area < \left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right| \\ \left[\left(slope_{GCp2}(Zone)\right)\cdot\left[\log\left(\frac{Area}{ft^2}\right) + -\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right]\right] + \left(GCp_{Zone}\right)^{\langle 1\rangle}\right] & \text{if } \left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right| \le Area < \left|\left(Alim_{Zone}\right)^{\langle 2\rangle}\right| \\ \left(GCp_{Zone}\right)^{\langle 2\rangle} & \text{otherwise} \end{cases}$$

For Example:

$$GC_p\left(11\cdot ft^2, 4\right) = \begin{pmatrix} -0.98 \\ 0.89 \end{pmatrix} \qquad GC_p\left(10\cdot ft^2, 5\right) = \begin{pmatrix} -1.26 \\ 0.9 \end{pmatrix} \qquad GC_p\left(50\cdot ft^2, 1\right) = \begin{pmatrix} -0.93 \\ 0.23 \end{pmatrix}$$

# Window Design Pressure

The following input table was imported from an excel sheet that had a list of fens for this building. Each column represents the width, height, area, and zone of each fen respectively.

Fen :=

D:\..\fen dp.xls

Dummy Width and Height

Size := 3

Zone := 4

Fen =

| | 0 | 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 |
| 1 | 0 | 1 | 1 | 14 | 4 |
| 2 | 0 | 2 | 2 | 14 | 4 |
| 3 | 0 | 3 | 3 | 20 | 4 |
| 4 | 0 | 4 | 4 | 20 | 4 |
| 5 | 0 | 5 | 5 | 14 | 4 |
| 6 | 0 | 6 | 6 | 20 | 4 |
| 7 | 0 | 7 | 7 | 14 | 4 |
| 8 | 0 | 8 | 8 | 20 | 4 |
| 9 | 0 | 9 | 9 | 14 | 4 |
| 10 | 0 | 10 | 10 | 14 | 4 |
| 11 | 0 | 11 | 11 | 20 | 4 |
| 12 | 0 | 12 | 12 | 20 | 4 |
| 13 | 0 | 13 | 13 | 14 | 4 |
| 14 | 0 | 14 | 14 | 20 | 4 |
| 15 | 0 | 15 | 15 | 14 | 4 |

Number of windows/doors

rows(Fen) = 196

j := 0 .. rows(Fen) − 2

$$DP^{\langle j\rangle} := \begin{cases} q_h \cdot \left( GC_p\left( \overrightarrow{\left[\left(Fen^{\langle Size\rangle}\right)_{j+1}\cdot ft^2\right]}, \overrightarrow{\left(Fen^{\langle Zone\rangle}\right)_{j+1}} \right) + GC_{pi} \right) & \text{if } \left(Fen^{\langle Zone\rangle}\right)_{j+1} \neq 45 \\ \left[ \begin{array}{l} q_h \cdot \left( GC_p\left( \overrightarrow{\left[\left(Fen^{\langle Size\rangle}\right)_{j+1}\cdot ft^2\right]}, 5 \right) + GC_{pi} \right) \cdot \left(Fen^{\langle Fraction\rangle}\right)_{j+1} \cdots \\ + q_h \cdot \left( GC_p\left( \overrightarrow{\left[\left(Fen^{\langle Size\rangle}\right)_{j+1}\cdot ft^2\right]}, 4 \right) + GC_{pi} \right) \cdot \left[ 1 - \left(Fen^{\langle Fraction\rangle}\right)_{j+1} \right] \end{array} \right] & \text{otherwise} \end{cases}$$

Effective Area of fenestrations are set according to the area of the element resisting the load, as opposed to the area of the entire fenestration. For example, a sliding glass door is made of 3 doors spanning vertically, each door is 4x8. The doors do not transfer wind load horizontally, therefore the wind loads are correlated only over the single door, and thus instead of an effective area of 96 square feet, the effective area is 32 square feet.

DP =

| | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | -38.87 | -38.87 | -38.04 | -38.04 | -38.87 | -38.04 | -38.87 | -38.04 | -38.87 | -38.87 |
| 1 | 35.82 | 35.82 | 34.99 | 34.99 | 35.82 | 34.99 | 35.82 | 34.99 | 35.82 | 35.82 |

psf

for example window : Design pressures are: $DP^{\langle 4\rangle} = \begin{pmatrix} -38.87 \\ 35.82 \end{pmatrix}$ psf

# Design of Nailing Pattern for Roof Deck

Tributary area for single fastener: $Area := 10 \cdot ft^2$

| Zone1 | Zone 2 | Zone 3 |
|---|---|---|
| $GC_p(Area,1) = \begin{pmatrix} -1 \\ 0.3 \end{pmatrix}$ | $GC_p(Area,2) = \begin{pmatrix} -1.8 \\ 0.3 \end{pmatrix}$ | $GC_p(Area,3) = \begin{pmatrix} -2.8 \\ 0.3 \end{pmatrix}$ |

Design load: Zone2

$p_{single} := q_h \cdot (GC_p(Area,2) + GC_{pi})$ $\qquad p_{single} = \begin{pmatrix} -67.12 \\ 16.27 \end{pmatrix} psf$

Tributary area for single sheet of plywood fastener: $Area := 32 \cdot ft^2$

One 4x8ft sheet of plywood/OSB = 32 FT tributary area

| Zone1 | Zone 2 | Zone 3 |
|---|---|---|
| $GC_p(Area,1) = \begin{pmatrix} -0.95 \\ 0.25 \end{pmatrix}$ | $GC_p(Area,2) = \begin{pmatrix} -1.45 \\ 0.25 \end{pmatrix}$ | $GC_p(Area,3) = \begin{pmatrix} -1.94 \\ 0.25 \end{pmatrix}$ |

$p_{panel} := q_h \cdot (GC_p(Area,2) + GC_{pi})$ $\qquad p_{panel} = \begin{pmatrix} -55.13 \\ 14.56 \end{pmatrix} psf$

**Resistance of single 8d Nail**

Load Case : Wind + 60% of dead load

$q_r := 41 \cdot \frac{lbf}{in}$ 8d common nail, NDS 1997, page 30, diameter 0.131", specific Gravity 0.55 (Southern Pine)

$l_{nail} := 2.5in$ length of nail, 8d

$t := .5 \cdot in$ Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 2\,in$ penetration length

Southern Pine SG - 0.55 on page 29, Table 12A of NDS-S97

$C_D := 1.6$ Duration factor for short term loads - wind = 10 minutes

$C_m := 1.0$ Condition Factor = assume that wood moisture content at time of construction is same as long term value

$R_{nail} := q_r \cdot l_p \cdot C_D \cdot C_m$ $\qquad R_{nail} = 131.2\,lbf$

**Maximum Spacing for 8d nail:**

$$A_t := \frac{R_{nail}}{\left(\left|p_{single_0} + 0.6 \cdot DL_{sheath}\right| \cdot 2 \cdot ft\right)} \qquad A_t = 11.9\,in$$

maximum required spacing of fasteners

Select nailing pattern that meets max spacing criteria

practical number of nails that meets nailing spacing criteria listed above (Zone 2/3)

$$ceil(linterp(s_{possible}, N_{possible}, A_t)) = 6$$

lookup nailing pattern to meet Zone2/3

$$II_s := floor(linterp(s_{possible}, II, A_t))$$

$$s_i := s_{possible_{II_s}}$$

spacing, nails

NailSched =

| | |
|---|---|
| 4.36 | 12 |
| 4.8 | 11 |
| 5.33 | 10 |
| 6 | 9 |
| 6.86 | 8 |
| 8 | 7 |
| 9.6 | 6 |
| 12 | 5 |
| 16 | 4 |
| 24 | 3 |
| 48 | 2 |

**USE the following spacing:**

$s_e := 6in$ edge spacing $\qquad s_i = 9.6\,in$ interior spa

$$N_{nails} := 2 \cdot \left(\frac{48in}{s_e} + 1\right) + 3 \cdot \left(\frac{48in}{s_i} + 1\right) \qquad N_{nails} = 36$$

Check whole panel resistance

$$L_{panel} := \left(\left|p_{panel_0} + 0.6 \cdot DL_{sheath}\right|\right) \cdot 32ft^2 \qquad L_{panel} = 1733.42\,lbf$$

uplift

$$R_{total} := R_{nail} \cdot N_{nails} \qquad R_{total} = 4723.2\,lbf$$

$$Status_{RoofNail} := R_{total} > L_{panel} \qquad Status_{RoofNail} = 1$$

PASS = 1, FAIL = 0

# ROOF STRAPS DESIGN (Uplift): Design of Typical Truss at Center of Building

The ARA roof-strap model simulates failure of the entire roof assembly as a whole, and not any one specific truss connection.. Therefore, strap size in model should be based on strap representative of the majority of the connections, and therefore is based on section at middle of structure.

We have considered the C&C loads that are acting on a single truss in the middle of the roof.

Edge zone

$\mathrm{LeastHorDim} := \min(W, L) \qquad \mathrm{LeastHorDim} = 38\,\mathrm{ft}$

$a := \left(\begin{pmatrix} 0.1\cdot\mathrm{LeastHorDim} \\ 0.4\cdot h \end{pmatrix}\right) \qquad a = \begin{pmatrix} 3.8 \\ 8.87 \end{pmatrix}\mathrm{ft} \quad a := \min(a) \qquad a := \max\left(\begin{pmatrix} a \\ 0.04\cdot\mathrm{LeastHorDim} \\ 3\cdot\mathrm{ft} \end{pmatrix}\right) \qquad a = 3.8\,\mathrm{ft}$

$l_r := \dfrac{W}{2\cdot\cos(\theta)} \qquad l_r = 19\,\mathrm{ft}$ length of top chord of truss

$a_\theta := \dfrac{a}{\cos(\theta)}$ length of edge zones along roof slope - assume that "a" in ASCE7 figures are widths in plan.

Method 1: Center Roof Truss Design based on Components and Cladding loads from ASCE 7-98

Effective wind area of a truss equals maximum of actual area and span times 1/3 span length

External Gust Factors

$A_{eff} := \left(\begin{pmatrix} W\cdot\Delta \\ W\cdot\dfrac{W}{3} \end{pmatrix}\right) \qquad A_{eff} = \begin{pmatrix} 76 \\ 481.33 \end{pmatrix}\mathrm{ft}^2 \qquad A_{eff} := \max(A_{eff}) \qquad A_{eff} = 481.33\,\mathrm{ft}^2$

$GC_p(A_{eff}, 1) = \begin{pmatrix} -0.9 \\ 0.2 \end{pmatrix} \qquad GC_p(A_{eff}, 2) = \begin{pmatrix} -1.1 \\ 0.2 \end{pmatrix} \qquad GC_p(A_{eff}, 3) = \begin{pmatrix} -1.1 \\ 0.2 \end{pmatrix}$

$k := 1..3 \qquad p_k := \left(GC_p(A_{eff}, k)_0 + GC_{pi_0}\right)\cdot q_h$

$p = \begin{pmatrix} 0 \\ -36.61 \\ -43.39 \\ -43.39 \end{pmatrix}\mathrm{psf}$ Design Pressures for Zones 1, 2, and 3

$V = 130\,\mathrm{mph}$
$\mathrm{Exp} = 2$

Overhang pressures, Zone 2 $\qquad p_o := \left(GC_p(A_{eff}, 7)_0\right)\cdot q_h \qquad GC_p(A_{eff}, 7) = \begin{pmatrix} -1.11 \\ 0 \end{pmatrix} \qquad p_o = -37.69\,\mathrm{psf}$

WIND Perpendicular to Ridge: Loading pattern according to ASCE 7-98 guide by K. Metha

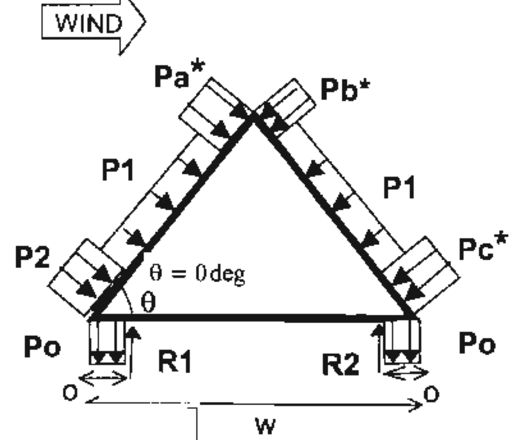
Set pa and Pc equal to p1, because ASCE7-98 guidebook indicates that truss loads should follow patterns where Zone2 is not applied simultaneously to all locations according to wind tunnel tests.

$$p_c := p_1$$

Pattern is slightly different for low slope roofs

$$p_a := \begin{cases} p_1 & \text{if } \theta < 10\cdot\text{deg} \\ p_1 & \text{otherwise} \end{cases}$$

$$p_b := \begin{cases} p_1 & \text{if } \theta < 10\cdot\text{deg} \\ p_2 & \text{otherwise} \end{cases}$$




Sum Moments: about R2 reaction point

$$\begin{aligned}
R_1 := \frac{1}{W-2\cdot o}\cdot\Bigg[ & p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(W-o-\frac{o}{2}\right)\ldots \\
& + p_2\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left(W-o-\frac{a-o}{2}-o\right)\ldots \\
& + p_b\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{W}{2}-o-\frac{a_\theta}{2}\cdot\cos(\theta)\right)\ldots \\
& + p_a\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left[W-o-\left(l_r-\frac{a_\theta}{2}\right)\cdot\cos(\theta)\right]\ldots \\
& + \left[p_c\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left[\frac{1}{2}\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\right]\right]\ldots \\
& + \left[p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\cos(\theta)\cdot\left[\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)+\left(\frac{W}{2}-o-\frac{l_r}{2}\cdot\cos(\theta)\right)\right]\right]\ldots \\
& + -p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{o}{2}\right)\ldots \\
& + -\left[p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\right]\ldots \\
& + -\left[p_2\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta-\frac{1}{2}\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\right]\ldots \\
& + -\left[p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ldots \\
& + -\left[p_a\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r-\frac{a_\theta}{2}\right)\sin(\theta)\right]\ldots \\
& + p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\ldots \\
& + p_1\cdot(l_r-2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\ldots \\
& + p_b\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r-\frac{a_\theta}{2}\right)\cdot\sin(\theta)\ldots \\
& + p_c\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta-\frac{1}{2}\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\ldots \\
& + \phi\cdot DL_{roof}\cdot\Delta\cdot W\cdot\left(\frac{W}{2}-o\right)\Bigg]
\end{aligned}$$

Dead load factor, ASD

$\phi = 0.6$

$R_1 = -1234.86\,\text{lbf}$

Sum Forces in Vertical

$$R_2 := \left[\left[\begin{aligned}&2\cdot\left(p_o\cdot\frac{o}{\cos(\theta)}\cdot\cos(\theta)\cdot\Delta\right)\;\ldots\\&+\left[p_2\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\right]\;\ldots\\&+\left(p_b\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\;\ldots\\&+2\cdot p_1\cdot\left(l_r-2\cdot a_\theta\right)\cdot\cos(\theta)\cdot\Delta\;\ldots\\&+p_a\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\;\ldots\\&+p_c\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\end{aligned}\right]+\phi\cdot DL_{roof}\cdot(\Delta\cdot W)\right]-R_1$$

$$R_2 = -1188.49\,\mathrm{lbf}$$

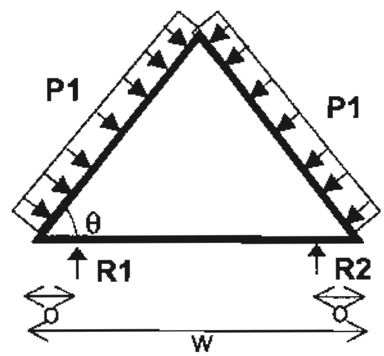
<u>WIND Parallel to Ridge</u>

$$R_3 := \frac{\Delta}{W-2\cdot o}\cdot\left[\begin{aligned}&p_1\cdot l_r\cdot\cos(\theta)\cdot\left[\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)+\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\;\ldots\\&+\phi\cdot DL_{roof}\cdot W\cdot\left(\frac{W}{2}-o\right)\end{aligned}\right]$$

$$R_4 := 2\cdot p_1\cdot l_r\cdot\Delta\cdot\cos(\theta)-R_3+\phi\cdot DL_{roof}\cdot\Delta\cdot W$$

$$R_3 = -1185.92\,\mathrm{lbf}$$

$$R_4 = -1185.92\,\mathrm{lbf}$$



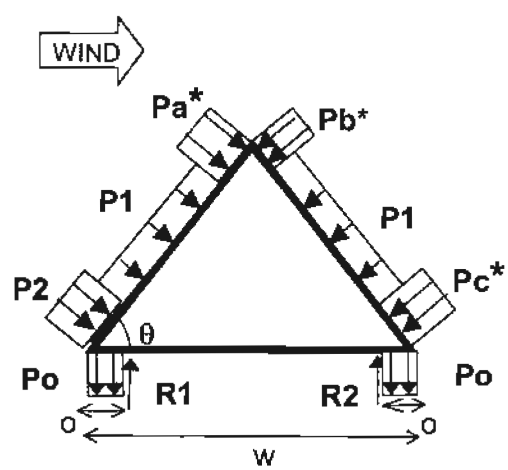

<u>Wind perpendicular to ridge</u>, applied at all edge zones simultaneously (note that this is an unrealistic condition, but is one that may be checked by a designer).

If slope less than 10 degrees, pa and pb do not exist

$$p_a := \begin{cases} p_1 & \text{if } \theta < 10\cdot deg \\ p_2 & \text{otherwise}\end{cases}$$

$$p_b := \begin{cases} p_1 & \text{if } \theta < 10\cdot deg \\ p_2 & \text{otherwise}\end{cases}$$

$$p_c := \begin{cases} p_2 & \text{if } \theta < 10\cdot deg \\ p_2 & \text{otherwise}\end{cases}$$



$$
\begin{aligned}
R_1 := \frac{1}{W - 2\cdot o}\cdot\Bigg[ & p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{o}{2}\right)\ldots \\
& + p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left(W - o - \frac{a-o}{2} - o\right)\ldots \\
& + p_b\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{W}{2} - o - \frac{a_\theta}{2}\cdot\cos(\theta)\right)\ldots \\
& + p_a\cdot a_\theta\cdot\Delta\cdot\cos(\theta)\cdot\left[W - o - \left(l_r - \frac{a_\theta}{2}\right)\cdot\cos(\theta)\right]\ldots \\
& + \left[p_c\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\cos(\theta)\cdot\left[\frac{1}{2}\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\right]\right]\ldots \\
& + \left[p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\cos(\theta)\cdot\left[\left(W - o - \frac{l_r}{2}\cdot\cos(\theta)\right) + \left(\frac{W}{2} - o - \frac{l_r}{2}\cdot\cos(\theta)\right)\right]\right]\ldots \\
& + -p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\cos(\theta)\cdot\left(\frac{o}{2}\right)\ldots \\
& + -\left[p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\right]\ldots \\
& + -\left[p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta - \frac{1}{2}\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\right]\ldots \\
& + -\left[p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\right]\ldots \\
& + -\left[p_a\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a_\theta}{2}\right)\sin(\theta)\right]\ldots \\
& + p_o\cdot\frac{o}{\cos(\theta)}\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{o}{2\cdot\cos(\theta)}\cdot\sin(\theta)\right)\ldots \\
& + p_1\cdot(l_r - 2\cdot a_\theta)\cdot\Delta\cdot\sin(\theta)\cdot\left(\frac{l_r}{2}\cdot\sin(\theta)\right)\ldots \\
& + p_b\cdot a_\theta\cdot\Delta\cdot\sin(\theta)\cdot\left(l_r - \frac{a_\theta}{2}\right)\cdot\sin(\theta)\ldots \\
& + p_c\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\Delta\cdot\sin(\theta)\cdot\left[\left[a_\theta - \frac{1}{2}\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\right]\cdot\sin(\theta)\right]\ldots \\
& + \phi\cdot DL_{roof}\cdot\Delta\cdot W\cdot\left(\frac{W}{2} - o\right)\Bigg]
\end{aligned}
$$

$p_2 = -43.39\,psf$

$p_1 = -36.61\,psf$

$p_o = -37.69\,psf$

$p_a = -36.61\,psf$

$p_b = -36.61\,psf$

$p_c = -43.39\,psf$

$a_\theta = 3.8\,ft$

$W = 38\,ft$

$l_r = 19\,ft$

$\Delta = 2\,ft$

$R_1 = -1237.44\,lbf$

$$
\begin{aligned}
R_2 := \Bigg[\Bigg[ & 2\cdot\left(p_o\cdot\frac{o}{\cos(\theta)}\cdot\cos(\theta)\cdot\Delta\right)\ldots \\
& + \left[p_2\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\right]\ldots \\
& + (p_b\cdot a_\theta\cdot\cos(\theta)\cdot\Delta)\ldots \\
& + 2\cdot p_1\cdot(l_r - 2\cdot a_\theta)\cdot\cos(\theta)\cdot\Delta\ldots \\
& + (p_a\cdot a_\theta\cdot\cos(\theta)\cdot\Delta)\ldots \\
& + p_c\cdot\left(a_\theta - \frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\Bigg] + \phi\cdot DL_{roof}\cdot(\Delta\cdot W)\Bigg] - R_1
\end{aligned}
$$

$R_2 = -1237.44\,lbf$

Compared to theoretically correct loading pattern:

$$\frac{\text{Pattern\_Load}}{\text{Full\_Zone\_Load}} \qquad \frac{R_1}{R_1} = 1 \qquad \frac{R_2}{R_2} = 0.96$$

$$R_1 := R_1 \qquad R_2 := R_2$$ Use full pattern loading

The theorectically correct loading pattern produces maximum uplifts that are only ~6-7% lower than the full pattern loading. Therefore, since ASCE7 doesn't clearly indicate the which pattern loading is appropriate, and since the difference is relatively minor, then default to full pattern loading.

**Summary of Strap Design**

Strap Design of interior zone truss:

Components and Cladding:
Interior Truss

$$R = \begin{pmatrix} 0 \\ -1237.44 \\ -1237.44 \\ -1185.92 \\ -1185.92 \end{pmatrix} \text{lbf} \qquad \min(R) = -1237.44\,\text{lbf} \qquad R_{design} := \min(R)$$

Convert from 5%ile of Ultimate Distribution to Mean and SD of Ultimate

$$ratio_{5\%UltMean} := 1.196$$

$$ratio_{5\%UltSD} := 0.1196$$

Ultimate Failure Capacity

$$R_U := \frac{R_{design}}{1.6} \cdot \left[3 \cdot \begin{pmatrix} ratio_{5\%UltMean} \\ ratio_{5\%UltSD} \end{pmatrix}\right] \qquad R_U = \begin{pmatrix} -2774.96 \\ -277.5 \end{pmatrix} \text{lbf}$$

**Shear on Roof-Wall Connectors**

Lateral shear loads on connectors are assumed to be adequate.

# SUMMARY:

Design Parameters: $V = 130\,\text{mph}$ $\quad IntPressure = 0$ $\quad Exp = 2$

Nail Spacing: $s_e = 6\,\text{in}$ edge of plywood $\quad s_i = 9.6\,\text{in}$ interior of plywood

Straps: C&C loads $\quad R_{design} = -1237.44\,\text{lbf}$ $\quad R_U = \begin{pmatrix} -2774.96 \\ -277.5 \end{pmatrix} \text{lbf}$

Window Design Pressure: $\max(DP) = 36.61\,\text{psf}$ $\quad \min(DP) = -39.66\,\text{psf}$

# SBC88

Wind Loads by SBC 1988 version

**Design Parameters**

$in0 := (110 \;\; 1 \;\; Enclosed)$

$V := \left|in0^{\langle 0 \rangle}\right| \cdot mph \qquad V = 110\,mph$

$IntPressure := \left|in0^{\langle 2 \rangle}\right|$

**Variables for Enclosed/Part Encl.**

$Enclosed \equiv 0$

$PartEnclosed \equiv 1$

**Geometry of Building:** Building Name: 0023 - condo project

$h := \left(\frac{22.18 + 22.18}{2}\right) \cdot ft$ ht of building

$\theta := atan\left(\frac{0}{12}\right) \qquad \theta = 0\,deg$

$o := 0.0 \cdot ft$ overhang width

$o_g := 0 \cdot ft$

$W := 38ft + 2 \cdot o$ dimensions of building

$L := 192ft + 2 \cdot o$

$\Delta := 24 \cdot in$ Truss spacing

Roof cover: Shingle

$h_{wall} := 9 \cdot ft$ Height of Wall, single story

**Dead load of roof**

$DL_{roof} := 9 \cdot psf$ Hip roof, Tile, trusses, underlayment (from SBC Appendix A)

$DL_{sheath} := (0.5 \cdot in) \cdot \left(\frac{0.4psf}{.125 \cdot in}\right) \qquad DL_{sheath} = 1.6\,psf$

Dead load of 17 psf is composed of following: Truss/Sheathing (7 psf), Tile (10psf). If shingles are used, use 2 psf instead of 10 psf.

$L_{attic} := 30 \cdot psf$ SBC Table 1604.1

$L_{floor} := 40 \cdot psf$

$L_{roof} := 16 \cdot psf$

$DL_{wall} := \begin{pmatrix} 10 \\ 55 \end{pmatrix} \cdot psf$ Wood Frame wall weight / Masonry Wall Weight

$DL_{misc} := 15 \cdot psf$ Miscellaneous: Contents, carpet, cabinets, fixtures)

Dynamic Wind Pressure

$h_{min} := 15\cdot ft$

$$q_h := \begin{cases} \left[\left[ .00256\cdot V^2\cdot\left(\frac{h}{30\cdot ft}\right)^{\frac{2}{7}}\cdot\frac{slug}{2.15111\cdot ft^3}\right]\right] & \text{if } (h > h_{min}) \\ \left[.00256\cdot V^2\cdot\left(\frac{15\cdot ft}{30ft}\right)^{\frac{2}{7}}\right]\cdot\frac{slug}{2.15111\cdot ft^3} & \text{otherwise} \end{cases}$$

Dynamic Wind Pressure( Table 1606.2A)

$q_h = 28.415\,psf$

Edge zone $\quad LeastHorDim := \min\left(\begin{pmatrix} W \\ L \end{pmatrix}\right)$ $\quad LeastHorDim = 38\,ft$ $\quad h = 22.18\,ft$

$a := \min\left(\begin{pmatrix} 0.1\cdot LeastHorDim \\ 0.4\cdot h \end{pmatrix}\right)$ $\quad a = 3.8\,ft$ $\quad a := \max\left(\begin{pmatrix} a \\ 0.04\cdot LeastHorDim \\ 3\cdot ft \end{pmatrix}\right)$ $\quad a = 3.8\,ft$

$a_\theta := \frac{a}{\cos(\theta)}$ length of edge zones along roof slope - assume that "z" in Figures 1205 are widths in plan not along roof

$a_\theta = 3.8\,ft$

$l_r := \frac{W}{2\cdot\cos(\theta)}$ $\quad l_r = 19\,ft$ $\quad$ length of top chord of truss

**Internal Pressure coefficient**

$$GC_{pi} := \begin{cases} \begin{pmatrix} 0 \\ 0 \end{pmatrix} & \text{if } IntPressure = Enclosed \\ \begin{pmatrix} -0.4 \\ 0.1 \end{pmatrix} & \text{if } IntPressure = PartEnclosed \\ \begin{pmatrix} -20 \\ 20 \end{pmatrix} & \text{otherwise} \end{cases}$$

dummy value

$GC_{pi} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$

**External Pressure Coefficients: Components and Cladding**

Limits of External Pressure Coefficients for each Zone in C&C loads
( first row neg coefficients, second row positive coefficents)

10SF neg 100SF neg, 1000SF neg
10SF pos 100SF pos, 1000 SF pos

If slope is less than 10 degrees:

$$
\mathrm{GCp\_10deg} := \left[\begin{array}{l}
\begin{pmatrix} -1.3 & -1.15 & -1.15 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -1.3 & -1.15 & -1.15 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -1.7 & -1.4 & -1.4 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -1.7 & -1.4 & -1.4 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -2.9 & -1.4 & -1.4 \\ 0 & 0 & 0 \end{pmatrix} \\
0.9\cdot\begin{pmatrix} -1.3 & -1.1 & -1.1 \\ 1.3 & 1.0 & 1.0 \end{pmatrix} \\
0.9\cdot\begin{pmatrix} -1.5 & -1.1 & -1.1 \\ 1.3 & 1.0 & 1.0 \end{pmatrix} \\
\begin{pmatrix} -1.3 & -0.95 & -0.95 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -1.5 & -1.2 & -1.2 \\ 0 & 0 & 0 \end{pmatrix} \\
\begin{pmatrix} -2.7 & -0.95 & -0.95 \\ 0 & 0 & 0 \end{pmatrix}
\end{array}\right]
\begin{array}{l}
\text{Zone r} \\
\text{Zone re} \\
\text{Zone si} \\
\text{Zone se} \\
\text{Zone c} \\
\text{Zone w} \\
\text{Zone e} \\
\text{Zone r, overhang} \\
\text{Zone s, overhang} \\
\text{Zone c, overhang}
\end{array}
$$

Zone r, Zone re, Zone si, Zone se: roof coefficients Table 1205.2D

Zone c, Zone w, Zone e: wall coefficients, Figure 1205.2C (reduced by 10% when roof slope less than 10deg)

Alim is the x axis values of the change in slope of the GCP graphs

$$
\mathrm{Alim\_10deg} := \begin{bmatrix}
(10 & 100 & 1000) \\
(10 & 100 & 1000) \\
(63 & 100 & 1000) \\
(63 & 100 & 1000) \\
(10 & 100 & 1000) \\
(10 & 500 & 1000) \\
(10 & 500 & 1000) \\
(10 & 100 & 1000) \\
(48 & 72 & 1000) \\
(10 & 100 & 1000)
\end{bmatrix}\cdot\mathrm{ft}^2
\begin{array}{l}
\text{Zone r} \\
\text{Zone re} \\
\text{Zone si} \\
\text{Zone se} \\
\text{Zone c} \\
\text{Zone w} \\
\text{Zone e} \\
\text{Zone r, overhang} \\
\text{Zone s, overhang} \\
\text{Zone c, overhang}
\end{array}
$$

If slope is 10 to 30 degrees:

$$GCp\_30deg := \begin{bmatrix} \begin{pmatrix} -1.2 & -1.1 & -1.1 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -1.2 & -1.1 & -1.1 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -1.4 & -1.2 & -1.2 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -2.1 & -1.8 & -1.8 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -2.7 & -1.8 & -1.8 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -1.3 & -1.1 & -1.1 \\ 1.3 & 1.0 & 1.0 \end{pmatrix} \\ \begin{pmatrix} -1.5 & -1.1 & -1.1 \\ 1.3 & 1.0 & 1.0 \end{pmatrix} \\ \begin{pmatrix} -1.0 & -0.9 & -0.9 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -1.2 & -1.0 & -1.0 \\ 0 & 0 & 0 \end{pmatrix} \\ \begin{pmatrix} -2.5 & -1.6 & -1.6 \\ 0 & 0 & 0 \end{pmatrix} \end{bmatrix}$$

Zone r

Zone re — roof coefficients Table 1205.2D

Zone si

Zone se

Zone c — wall coefficients, Figure 1205.2C (reduced by 10% when roof slope less than 10deg)

Zone w

Zone e

Zone r, overhang

Zone s, overhang

Zone c, overhang

$$Alim\_30deg := \begin{bmatrix} (10 & 100 & 1000) \\ (10 & 100 & 1000) \\ (10 & 100 & 1000) \\ (48 & 100 & 1000) \\ (10 & 100 & 1000) \\ (10 & 500 & 1000) \\ (10 & 500 & 1000) \\ (10 & 100 & 1000) \\ (10 & 100 & 1000) \\ (10 & 100 & 1000) \end{bmatrix} \cdot ft^2$$

Zone r
Zone re
Zone si
Zone se
Zone c
Zone w
Zone e
Zone r, overhang
Zone s, overhang
Zone c, overhang

$GCp\_45deg := 0$

Dummy Values for high slope roofs: Not part of this study

$Alim\_45deg := 0$

Select appropriate set of parameters according to slope of roof:

$\theta = 0\,deg$

$$GCp := \begin{array}{|l} GCp\_10deg \ \text{ if } (\theta \le 10 \cdot deg) \\ GCp\_30deg \ \text{ if } 10 \cdot deg < \theta \le 30 \cdot deg \\ GCp\_45deg \ \text{ if } 30 \cdot deg < \theta \le 45 \cdot deg \\ 0 \ \text{ otherwise} \end{array} \qquad Alim := \begin{array}{|l} Alim\_10deg \ \text{ if } (\theta \le 10 \cdot deg) \\ Alim\_30deg \ \text{ if } 10 \cdot deg < \theta \le 30 \cdot deg \\ Alim\_45deg \ \text{ if } 30 \cdot deg < \theta \le 45 \cdot deg \\ 0 \ \text{ otherwise} \end{array}$$

Calculate slopes of parts of pressure coefficient graphs for interpolation:

$$\mathrm{slope}_{GCp}(\mathrm{Zone}) := \frac{\left(GCp_{Zone}\right)^{\langle 1\rangle} - \left(GCp_{Zone}\right)^{\langle 0\rangle}}{\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right] - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]}$$

Slope of first section of line

$$\mathrm{slope}_{GCp2}(\mathrm{Zone}) := \frac{\left(GCp_{Zone}\right)^{\langle 2\rangle} - \left(GCp_{Zone}\right)^{\langle 1\rangle}}{\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 2\rangle}\right|}{ft^2}\right] - \log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right]}$$

slope of secondary section of line
(usually flat)

$$GC_p(\mathrm{Area},\mathrm{Zone}) := \begin{cases} \left(GCp_{Zone}\right)^{\langle 0\rangle} & \text{if } \mathrm{Area} < \left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right| \\ \left[\left(\mathrm{slope}_{GCp}(\mathrm{Zone})\right)\cdot\left[\log\left(\frac{\mathrm{Area}}{ft^2}\right) \ldots \\ + -\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right|}{ft^2}\right]\right] \ldots \\ + \left(GCp_{Zone}\right)^{\langle 0\rangle}\right] & \text{if } \left|\left(Alim_{Zone}\right)^{\langle 0\rangle}\right| \le \mathrm{Area} < \left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right| \\ \left[\left(\mathrm{slope}_{GCp2}(\mathrm{Zone})\right)\cdot\left[\log\left(\frac{\mathrm{Area}}{ft^2}\right) \ldots \\ + -\log\left[\frac{\left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right|}{ft^2}\right]\right] \ldots \\ + \left(GCp_{Zone}\right)^{\langle 1\rangle}\right] & \text{if } \left|\left(Alim_{Zone}\right)^{\langle 1\rangle}\right| \le \mathrm{Area} < \left|\left(Alim_{Zone}\right)^{\langle 2\rangle}\right| \\ \left(GCp_{Zone}\right)^{\langle 2\rangle} & \text{otherwise} \end{cases}$$

For Example:

$$GC_p\left(72\cdot ft^2, co\right) = \begin{pmatrix} -1.2 \\ 0 \end{pmatrix} \qquad GC_p\left(10\cdot ft^2, r\right) = \begin{pmatrix} -1.3 \\ 0 \end{pmatrix} \qquad GC_p\left(50\cdot ft^2, e\right) = \begin{pmatrix} -1.202 \\ 1.059 \end{pmatrix}$$

$$GC_p\left(510\cdot ft^2, si\right) = \begin{pmatrix} -1.4 \\ 0 \end{pmatrix} \qquad GC_p\left(200\cdot ft^2, e\right) = \begin{pmatrix} -1.074 \\ 0.963 \end{pmatrix}$$

# Window Design Pressure

The following input table was imported from an excel sheet that had a list of fens for this building. Each column represents the widht, height, area, and zone of each fen respectively.

Fen :=

D:\..\fen dp SBC88.xls

Fen =

| | 0 | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 1 | 0 | 1 | 1 | 14 | 5 | 0 |
| 2 | 0 | 2 | 2 | 14 | 5 | 0 |
| 3 | 0 | 3 | 3 | 20 | 5 | 0 |
| 4 | 0 | 4 | 4 | 20 | 5 | 0 |
| 5 | 0 | 5 | 5 | 14 | 5 | 0 |
| 6 | 0 | 6 | 6 | 20 | 5 | 0 |
| 7 | 0 | 7 | 7 | 14 | 5 | 0 |
| 8 | 0 | 8 | 8 | 20 | 5 | 0 |
| 9 | 0 | 9 | 9 | 14 | 5 | 0 |
| 10 | 0 | 10 | 10 | 14 | 5 | 0 |
| 11 | 0 | 11 | 11 | 20 | 5 | 0 |
| 12 | 0 | 12 | 12 | 20 | 5 | 0 |

Dummy Width and Height

$\mathrm{Size} := 3$

$\mathrm{Zone} := 4$

$\mathrm{Fraction} := 5$

Number of Fens: $\mathrm{rows(Fen)} = 196$

$j := 1 .. \mathrm{rows(Fen)} - 2$

$$p_{wall} := q_h \cdot \left(GC_p\left(10 \cdot ft^2, w\right) + GC_{pi}\right) \qquad GC_p\left(10 \cdot ft^2, e\right) = \begin{pmatrix} -1.35 \\ 1.17 \end{pmatrix} \qquad p_{wall} = \begin{pmatrix} -33.246 \\ 33.246 \end{pmatrix} psf$$

$$DP^{\langle j \rangle} := \left[ \begin{array}{ll} q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, \overrightarrow{\left(Fen^{\langle Zone \rangle}\right)_j}\right) + GC_{pi}\right) & \text{if } \left(Fen^{\langle Zone \rangle}\right)_j \neq 45 \\ \left[ \begin{array}{l} q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, e\right) + GC_{pi}\right) \cdot \left(Fen^{\langle Fraction \rangle}\right)_j \ldots \\ + q_h \cdot \left(GC_p\left(\overrightarrow{\left[\left(Fen^{\langle Size \rangle}\right)_j \cdot ft^2\right]}, w\right) + GC_{pi}\right) \cdot \left[1 - \left(Fen^{\langle Fraction \rangle}\right)_j\right] \end{array} \right] & \text{otherwise} \end{array} \right]$$

DP =

| | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | -32.806 | -32.806 | -32.34 | -32.34 | -32.806 | -32.34 | -32.806 | -32.34 |
| 1 | 0 | 32.586 | 32.586 | 31.886 | 31.886 | 32.586 | 31.886 | 32.586 | 31.886 |

psf

$\max(DP) = 33.246\,psf \qquad \min(DP) = -33.246\,psf$

# Design of Nailing Pattern for Roof Deck

Load on one nail: use 10 SF as effective area

$\mathrm{Area} := 10\cdot ft^2$ $\quad GC_p(\mathrm{Area}, r) = \begin{pmatrix} -1.3 \\ 0 \end{pmatrix}$ $\quad GC_p(\mathrm{Area}, si) = \begin{pmatrix} -1.7 \\ 0 \end{pmatrix}$ $\quad GC_p(\mathrm{Area}, c) = \begin{pmatrix} -2.9 \\ 0 \end{pmatrix}$

Design Load: Zone si

$p_{single} := q_h \cdot (GC_p(\mathrm{Area}, si) + GC_{pi})$ $\qquad p_{single} = \begin{pmatrix} -48.306 \\ 0 \end{pmatrix} psf$

Tributary Area of single sheet of plwood: (4ftx8ft)

$\mathrm{Area} := 32\cdot ft^2$ $\quad GC_p(\mathrm{Area}, r) = \begin{pmatrix} -1.224 \\ 0 \end{pmatrix}$ $\quad GC_p(\mathrm{Area}, si) = \begin{pmatrix} -1.7 \\ 0 \end{pmatrix}$ $\quad GC_p(\mathrm{Area}, c) = \begin{pmatrix} -2.142 \\ 0 \end{pmatrix}$

$p_{panel} := q_h \cdot (GC_p(\mathrm{Area}, si) + GC_{pi})$ $\qquad p_{panel} = \begin{pmatrix} -48.306 \\ 0 \end{pmatrix} psf$

## Resistance of Single Nail

6d common nail

$q_r := 35\cdot\frac{lbf}{in}$ — 6d common nail, Southern Pine (specifig gravtiy =0.55) — NDS 1997-S Table 12.2A

$l_{nail} := 2.0in$ — length of nail, 6d

$t := .5\cdot in$ — Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 1.5\,in$ — penetration length

$C_D := 1.6$ — Duration factor for short term loads - wind = 10 minutes

$C_m := 1.0$ — Condition Factor = assume that wood moisture content at time of construction is same as long term value

$R_{nail_0} := q_r \cdot l_p \cdot C_D \cdot C_m$

8d common nail

$q_r := 41\cdot\frac{lbf}{in}$

$l_{nail} := 2.5in$ — length of nail, 8d, Southern Pine (SG=0.55), NDS 97-S Table 12.2A

$t := .5\cdot in$ — Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 2\,in$ — penetration length

$R_{nail_1} := q_r \cdot l_p \cdot C_D \cdot C_m$ $\qquad R_{nail} = \begin{pmatrix} 84 \\ 131.2 \end{pmatrix} lbf$ — Resistance of single Nail, 6d and 8d respectively

**Maximum Spacing for nails:**

$$A_t := \frac{R_{nail}}{\left(\left|P_{single_0} + DL_{sheath}\right| \cdot 2 \cdot ft\right)} \qquad A_t = \begin{pmatrix} 10.791 \\ 16.854 \end{pmatrix} in$$

maximum allowable spacing of fasteners

Select nailing pattern that meets max spacing criteria

*Check 6d nail first*

number of nails that meets nailing pattern criteria for Zone si

$$\text{ceil}\left(\text{linterp}\left(s_{possible}, N_{possible}, A_{t_0}\right)\right) = 6$$

lookup nailing pattern to meet Zone2/3

$$II_s := \text{floor}\left(\text{linterp}\left(s_{possible}, II, A_{t_0}\right)\right) \qquad II_s = 6$$

$$s_{i6} := s_{possible_{II_s}} \qquad s_{i6} = 9.6\,in$$

*check 8d nail*

$$\text{ceil}\left(\text{linterp}\left(s_{possible}, N_{possible}, A_{t_1}\right)\right) = 4$$

$$II_s := \text{floor}\left(\text{linterp}\left(s_{possible}, II, A_{t_1}\right)\right) \qquad II_s = 8$$

$$s_{i8} := s_{possible_{II_s}} \qquad s_{i8} = 16\,in$$

spacing, nails

NailSched =

| | |
|---|---|
| 4.364 | 12 |
| 4.8 | 11 |
| 5.333 | 10 |
| 6 | 9 |
| 6.857 | 8 |
| 8 | 7 |
| 9.6 | 6 |
| 12 | 5 |
| 16 | 4 |
| 24 | 3 |
| 48 | 2 |

**USE the following spacing:**

edge spacing $\quad s_e := 6in$

interior spacing

$$s_i := \begin{vmatrix} s_{i8} & \text{if } s_{i6} < 12 \cdot in \\ s_{i6} & \text{otherwise} \end{vmatrix} \qquad nailsize := \begin{vmatrix} 8 & \text{if } s_{i6} < 12 \cdot in \\ 6 & \text{otherwise} \end{vmatrix}$$

$$nailsize = 8 \qquad s_i = 16\,in$$

Check whole panel resistance

$$N_{nails} := 2 \cdot \left(\frac{48in}{s_e} + 1\right) + 3 \cdot \left(\frac{48in}{s_i} + 1\right) \qquad N_{nails} = 30$$

$$L_{panel} := \left(\left|p_{panel_0} + DL_{sheath}\right|\right) \cdot 32ft^2 \qquad L_{panel} = 1494.589\,ft^2\,psf$$ uplift

$$R_{total} := R_{nail_{\left(\frac{nailsize-6}{2}\right)}} \cdot N_r \; R_{total} = 3936\,lbf$$

$$Status_{RoofNail} := R_{total} > L_{panel} \qquad Status_{RoofNail} = 1$$ PASS = 1, FAIL = 0

# ROOF STRAPS DESIGN

Roof Truss Design should be based on Components and Cladding loads

Effective wind area of a truss equals maximum of actual area and span times 1/3 span length

$$A_{eff} := \begin{pmatrix} W \cdot \Delta \\ W \cdot \frac{W}{3} \end{pmatrix} \qquad A_{eff} = \begin{pmatrix} 76 \\ 481.333 \end{pmatrix} ft^2 \qquad A_{eff} := \max(A_{eff})$$

Since Aeff is greater than 100SF, Use 100SF for GCp values

External Gust Factors

$$GC_p(A_{eff}, r) = \begin{pmatrix} -1.15 \\ 0 \end{pmatrix} \qquad k := 0..9$$

Note there is no combo load case that has a reduction in dead load in SBC (section 1609)

$$GC_p(A_{eff}, si) = \begin{pmatrix} -1.4 \\ 0 \end{pmatrix}$$

$$GC_p(A_{eff}, c) = \begin{pmatrix} -1.4 \\ 0 \end{pmatrix}$$

$$p_k := \left(GC_p(A_{eff}, k)_0 + GC_{pi_0}\right) q_h$$

p =

| | 0 |
|---|---|
| 0 | -32.678 |
| 1 | -32.678 |
| 2 | -39.781 |
| 3 | -39.781 |
| 4 | -39.781 |
| 5 | -28.181 |
| 6 | -28.231 |
| 7 | -26.994 |
| 8 | -34.098 |
| 9 | -26.994 |

psf

$\phi := 1.0$

Negative pressures for r, ri, si, se and c zone

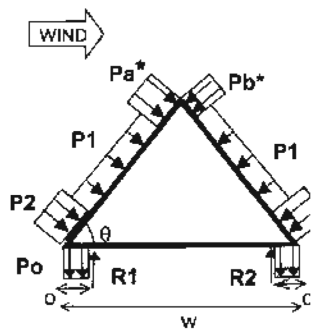
STRAP RESISTANCE
used in ARA model

WIND Perpendicular to Ridge at section A-A

Sum Moments (note that in the mathcad formulas p0 is zone r pressures and p2 is zone si pressure)

$$p_a := \begin{cases} p_r & \text{if } \theta < 10 \cdot deg \\ p_{si} & \text{otherwise} \end{cases}$$

$$p_b := \begin{cases} p_r & \text{if } \theta < 10 \cdot deg \\ p_{si} & \text{otherwise} \end{cases} \qquad p_c := p_r$$




$$
\begin{aligned}
R_0 := \frac{1}{W - 2 \cdot o} \cdot \Bigg[ & p_{so} \cdot \frac{o}{\cos(\theta)} \cdot \Delta \cdot \cos(\theta) \cdot \left( W - o - \frac{o}{2} \right) \ldots \\
& + p_{si} \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \cdot \Delta \cdot \cos(\theta) \cdot \left( W - o - \frac{a - o}{2} - o \right) \ldots \\
& + p_b \cdot a_\theta \cdot \Delta \cdot \cos(\theta) \cdot \left( \frac{W}{2} - o - \frac{a_\theta}{2} \cdot \cos(\theta) \right) \ldots \\
& + p_a \cdot a_\theta \cdot \Delta \cdot \cos(\theta) \cdot \left[ W - o - \left( l_r - \frac{a_\theta}{2} \right) \cdot \cos(\theta) \right] \ldots \\
& + \left[ p_c \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \cdot \Delta \cdot \cos(\theta) \cdot \left[ \frac{1}{2} \left( a_\theta - \frac{o}{\cos(\theta)} \right) \cdot \cos(\theta) \right] \right] \ldots \\
& + \left[ p_r \cdot (l_r - 2 \cdot a_\theta) \cdot \Delta \cdot \cos(\theta) \cdot \left( \left( W - o - \frac{l_r}{2} \cdot \cos(\theta) \right) \right) \right] \ldots \\
& + p_r \cdot (l_r - 2 \cdot a_\theta) \cdot \Delta \cdot \cos(\theta) \cdot \left( \frac{W}{2} - o - \frac{l_r}{2} \cdot \cos(\theta) \right) \ldots \\
& + -p_{so} \cdot \frac{o}{\cos(\theta)} \cdot \Delta \cdot \cos(\theta) \cdot \left( \frac{o}{2} \right) \ldots \\
& + -\left[ p_{so} \cdot \frac{o}{\cos(\theta)} \cdot \Delta \cdot \sin(\theta) \cdot \left( \frac{o}{2 \cdot \cos(\theta)} \cdot \sin(\theta) \right) \right] \ldots \\
& + -\left[ p_{si} \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \cdot \Delta \cdot \sin(\theta) \cdot \left[ \left[ a_\theta - \frac{1}{2} \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \right] \cdot \sin(\theta) \right] \right] \ldots \\
& + -\left[ p_r \cdot (l_r - 2 \cdot a_\theta) \cdot \Delta \cdot \sin(\theta) \cdot \left( \frac{l_r}{2} \cdot \sin(\theta) \right) \right] \ldots \\
& + -\left[ p_a \cdot a_\theta \cdot \Delta \cdot \sin(\theta) \cdot \left( l_r - \frac{a_\theta}{2} \right) \sin(\theta) \right] \ldots \\
& + p_{so} \cdot \frac{o}{\cos(\theta)} \cdot \Delta \cdot \sin(\theta) \cdot \left( \frac{o}{2 \cdot \cos(\theta)} \cdot \sin(\theta) \right) \ldots \\
& + p_r \cdot (l_r - 2 \cdot a_\theta) \cdot \Delta \cdot \sin(\theta) \cdot \left( \frac{l_r}{2} \cdot \sin(\theta) \right) \ldots \\
& + p_b \cdot a_\theta \cdot \Delta \cdot \sin(\theta) \cdot \left( l_r - \frac{a_\theta}{2} \right) \cdot \sin(\theta) \ldots \\
& + p_c \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \cdot \Delta \cdot \sin(\theta) \cdot \left[ \left[ a_\theta - \frac{1}{2} \cdot \left( a_\theta - \frac{o}{\cos(\theta)} \right) \right] \cdot \sin(\theta) \right] \ldots \\
& + \phi \cdot DL_{roof} \cdot \Delta \cdot W \cdot \left( \frac{W}{2} - o \right) \Bigg]
\end{aligned}
$$

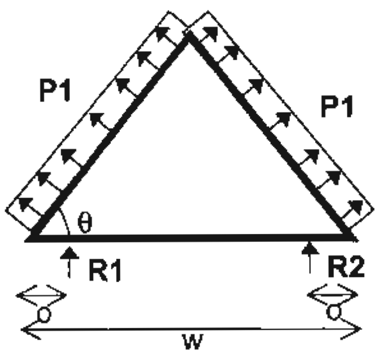
$$R_0 = -951.036\,\text{lbf}$$

Sum Forces in Vertical

$$R_1 := \left[\left[\begin{aligned}&2\cdot\left(p_{so}\cdot\frac{o}{\cos(\theta)}\cdot\cos(\theta)\cdot\Delta\right)\ ... \\ &+\left[p_{si}\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\right]\ ... \\ &+\left(p_b\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\right)\ ... \\ &+2\cdot p_r\cdot\left(l_r-2\cdot a_\theta\right)\cdot\cos(\theta)\cdot\Delta\ ... \\ &+p_a\cdot a_\theta\cdot\cos(\theta)\cdot\Delta\ ... \\ &+p_c\cdot\left(a_\theta-\frac{o}{\cos(\theta)}\right)\cdot\cos(\theta)\cdot\Delta\end{aligned}\right]+\phi\cdot DL_{roof}\cdot(\Delta\cdot W)\right]-R_0$$

$$R_1 = -902.446\,\text{lbf}$$

WIND Parallel to Ridge at Section A-A




WIND Parallel to Ridge

$$R_2 := \frac{\Delta}{W-2\cdot o}\cdot\left[\begin{aligned}&p_r\cdot l_r\cdot\cos(\theta)\cdot\left[\left(W-o-\frac{l_r}{2}\cdot\cos(\theta)\right)+\left(\frac{l_r}{2}\cdot\cos(\theta)-o\right)\right]\ ... \\ &+\phi\cdot DL_{roof}\cdot W\cdot\left(\frac{W}{2}-o\right)\end{aligned}\right]$$

$$R_3 := 2\cdot p_r\cdot l_r\cdot\Delta\cdot\cos(\theta)-R_2+\phi\cdot DL_{roof}\cdot\Delta\cdot W$$

$$R_2 = -899.746\,\text{lbf}$$

$$R_3 = -899.746\,\text{lbf}$$

**Summary of Strap Design**

Strap Design of interior zone truss:

Components and Cladding:
Interior Truss

$$R = \begin{pmatrix}-951.036\\-902.446\\-899.746\\-899.746\end{pmatrix}\text{lbf} \qquad \min(R) = -951.036\,\text{lbf} \qquad R_{design} := \min(R)$$

Convert from 5%ile of Ultimate Distribution to Mean and SD of Ultimate

$$ratio_{5\%UltMean} := 1.196$$

$$ratio_{5\%UltSD} := 0.1196$$

Ultimate Failure Capacity

$$R_U := \frac{R_{design}}{1.6}\cdot\left[3\cdot\begin{pmatrix}ratio_{5\%UltMean}\\ratio_{5\%UltSD}\end{pmatrix}\right] \qquad R_U = \begin{pmatrix}-2132.697\\-213.27\end{pmatrix}\text{lbf}$$

# SUMMARY:

Design Parameters: $V = 110\,\text{mph}$ $IntPressure = 0$

Nail Spacing: $nailsize = 8$ $s_e = 6\,\text{in}$ edge of plywood $s_i = 16\,\text{in}$ interior of plywood

Straps: C&C loads $R_{design} = -951.036\,\text{lbf}$ $R_U = \begin{pmatrix} -2132.697 \\ -213.27 \end{pmatrix}\text{lbf}$

Window Design Pressure: $\max(DP) = 33.246\,\text{psf}$ $\min(DP) = -33.246\,\text{psf}$

# SBC 76

Wind Loads by SBC 1976 version

**Variables for Enclosed/Part Encl.**

$Enclosed \equiv 0$

$PartEnclosed \equiv 1$

**Design Parameters**

$in0 := (110 \ \ Enclosed \ \ 0)$

$V := \left|in0^{\langle 0 \rangle}\right| \cdot mph$

$V = 110 mph$

$IntPressure := \left|in0^{\langle 2 \rangle}\right|$

$IntPressure = 0$

Table 1606: Use factor

$Use := 1.0$

**Geometry of Building:** Building Name: 0023 - condo project

$h := \left(\frac{22.18 + 22.18}{2}\right) \cdot ft$ ht of building

$\theta := atan\left(\frac{0}{12}\right)$ $\theta = 0 deg$

$o := 0.0 \cdot ft$ overhang width

$o_g := 0 \cdot ft$

$W := 38ft + 2 \cdot o$ dimensions of building

$L := 192ft + 2 \cdot o$

$\Delta := 24 \cdot in$ Truss spacing

Roof cover: Shingle

$h_{wall} := 9 \cdot ft$ Height of Wall, single story

**Dead load of roof**

$DL_{roof} := 9 \cdot psf$ Hip roof, Tile, trusses, underlayment (from SBC Appendix A)

$DL_{sheath} := (0.5 \cdot in) \cdot \left(\frac{0.4psf}{.125 \cdot in}\right)$ $DL_{sheath} = 1.6psf$

Dead load of 17 psf is composed of following: Truss/Sheathing (7 psf), Tile (10psf). If shingles are used, use 2 psf instead of 10 psf.

$L_{attic} := 30 \cdot psf$ SBC Table 1604.1

$L_{floor} := 40 \cdot psf$

$L_{roof} := 16 \cdot psf$

$DL_{wall} := \begin{pmatrix} 10 \\ 55 \end{pmatrix} \cdot psf$ Wood Frame wall weight / Masonry Wall Weight

$DL_{misc} := 15 \cdot psf$ Miscellaneous: Contents, carpet, cabinets, fixtures)

Dynamic Wind Pressure

$h_{min} := 30 \cdot ft$

$$q_h := \begin{cases} \left[ .00256 \cdot V^2 \cdot \left(\frac{h}{30 \cdot ft}\right)^{\frac{2}{7}} \cdot \frac{slug}{2.15111 \cdot ft^3} \right] & \text{if } (h > h_{min}) \\ \left[ .00256 \cdot V^2 \cdot \left(\frac{h_{min}}{30ft}\right)^{\frac{2}{7}} \right] \cdot \frac{slug}{2.15111 \cdot ft^3} & \text{otherwise} \end{cases}$$

Dynamic Wind Pressure( Table 1606.2A)

$q_h = 30.976\,psf$

length of top chord of truss

$l_r := \frac{W}{2 \cdot \cos(\theta)}$ $\qquad l_r = 19\,ft$

## Shape Factors: Tables 1205.2 to 1205.6

$$GCp := \begin{cases} \begin{bmatrix} \begin{pmatrix} -1.1 \\ 1.1 \end{pmatrix} \\ \begin{pmatrix} -0.55 \\ 1.1 \end{pmatrix} \\ \begin{pmatrix} -1.0 \\ 0 \end{pmatrix} \\ \begin{pmatrix} -0.75 \\ 0 \end{pmatrix} \\ \begin{pmatrix} -1.5 \\ 0 \end{pmatrix} \end{bmatrix} & \text{if IntPressure = Enclosed} \\ \begin{bmatrix} \begin{pmatrix} -1.5 \\ 1.1 \end{pmatrix} \\ \begin{pmatrix} -0.55 \\ 1.1 \end{pmatrix} \\ \begin{pmatrix} -1.5 \\ 0 \end{pmatrix} \\ \begin{pmatrix} -1.25 \\ 0 \end{pmatrix} \\ \begin{pmatrix} -1.5 \\ 0 \end{pmatrix} \end{bmatrix} & \text{otherwise} \end{cases}$$

$$\begin{pmatrix} extwall \\ window \\ hor\_windward \\ hor\_leeward \\ overhang \end{pmatrix} \equiv \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \end{pmatrix}$$

## Window Design Pressure

DP is not a function of location on building, exposure, or size of opening: Single DP can be calcuated

$GCp_{window} = \begin{pmatrix} -0.55 \\ 1.1 \end{pmatrix}$ $\qquad q_h = 30.976\,psf$

$DP := q_h \cdot GCp_{window}$ $\qquad DP = \begin{pmatrix} -17.037 \\ 34.074 \end{pmatrix} psf$

# Design of Nailing Pattern for Roof Deck

Load on one nail: use 10 SF as effective area

$$GCp_{hor\_windward} = \begin{pmatrix} -1 \\ 0 \end{pmatrix}$$

Design Load: windward zone of horizontal surface

$p_{single} := q_h \cdot \left(GCp_{hor\_windward}\right)$ $\qquad p_{single} = \begin{pmatrix} -30.976 \\ 0 \end{pmatrix} psf$

Tributary Area of single sheet of plwood: (4ftx8ft)

$p_{panel} := p_{single}$ $\qquad p_{panel} = \begin{pmatrix} -30.976 \\ 0 \end{pmatrix} psf$

## Resistance of Single Nail

6d common nail

$q_r := 35 \cdot \frac{lbf}{in}$ 6d common nail, Southern Pine (specifig gravtiy =0.55) NDS 1997-S Table 12.2A

$l_{nail} := 2.0in$ length of nail, 6d

$t := .5 \cdot in$ Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 1.5\,in$ penetration length

$C_D := 1.6$ Duration factor for short term loads - wind = 10 minutes

$C_m := 1.0$ Condition Factor = assume that wood moisture content at time of construction is same as long term value

$R_{nail_0} := q_r \cdot l_p \cdot C_D \cdot C_m$

8d common nail

$q_r := 41 \cdot \frac{lbf}{in}$ $\quad l_{nail} := 2.5in$ length of nail, 8d, Southern Pine (SG=0.55), NDS 97-S Table 12.2A

$t := .5 \cdot in$ Plywood thickness = 1/2" (min thickness of code)

$l_p := l_{nail} - t$ $\quad l_p = 2\,in$ penetration length

$R_{nail_1} := q_r \cdot l_p \cdot C_D \cdot C_m$ $\qquad R_{nail} = \begin{pmatrix} 84 \\ 131.2 \end{pmatrix} lbf$ Resistance of single Nail, 6d and 8d respectively

**Maximum Spacing for nails:**

$$A_t := \frac{R_{nail}}{\left(\left|p_{single_0} + DL_{sheath}\right| \cdot 2 \cdot ft\right)} \qquad A_t = \begin{pmatrix} 17.157 \\ 26.797 \end{pmatrix} in$$

maximum allowable spacing of fasteners

Select nailing pattern that meets max spacing criteria

*Check 6d nail first*

number of nails that meets nailing pattern criteria for Zone si

$$ceil\left(linterp\left(s_{possible}, N_{possible}, A_{t_0}\right)\right) = 4$$

lookup nailing pattern to meet Zone2/3

$$II_s := floor\left(linterp\left(s_{possible}, II, A_{t_0}\right)\right) \qquad II_s = 8$$

$$s_{i6} := s_{possible_{II_s}} \qquad s_{i6} = 16\,in$$

*check 8d nail*

$$ceil\left(linterp\left(s_{possible}, N_{possible}, A_{t_1}\right)\right) = 3$$

$$II_s := floor\left(linterp\left(s_{possible}, II, A_{t_1}\right)\right) \qquad II_s = 9$$

$$s_{i8} := s_{possible_{II_s}} \qquad s_{i8} = 24\,in$$

spacing, nails

NailSched =

| | |
|---|---|
| 4.364 | 12 |
| 4.8 | 11 |
| 5.333 | 10 |
| 6 | 9 |
| 6.857 | 8 |
| 8 | 7 |
| 9.6 | 6 |
| 12 | 5 |
| 16 | 4 |
| 24 | 3 |
| 48 | 2 |

**USE the following spacing:**

edge spacing $s_e := 6in$

interior spacing

$$s_i := \begin{vmatrix} s_{i8} \text{ if } s_{i6} < 12 \cdot in \\ s_{i6} \text{ otherwise} \end{vmatrix} \qquad nailsize := \begin{vmatrix} 8 \text{ if } s_{i6} < 12 \cdot in \\ 6 \text{ otherwise} \end{vmatrix}$$

$$nailsize = 6 \qquad s_i = 16\,in$$

Spacing cannot exceed 12 inches: $s_i := min\left(s_i, 12 \cdot in\right)$ $\quad s_i = 12\,in$

Check whole panel resistance

$$N_{nails} := 2 \cdot \left(\frac{48in}{s_e} + 1\right) + 3 \cdot \left(\frac{48in}{s_i} + 1\right) \qquad N_{nails} = 33$$

$$L_{panel} := \left(\left|p_{panel_0} + DL_{sheath}\right|\right) \cdot 32ft^2 \qquad L_{panel} = 940.033\,ft^2\,psf$$

uplift

$$R_{total} := R_{nail_{\left(\frac{nailsize-6}{2}\right)}} \cdot N_r\, R_{total} = 2772\,lbf$$

$$Statns_{RoofNail} := R_{total} > L_{panel} \qquad Status_{RoofNail} = 1$$

PASS = 1, FAIL = 0

## ROOF STRAPS DESIGN

Roof Truss Design should be based on Components and Cladding loads from Table 1205.5 which refers to Table 1205.3

External Gust Factors

$$GCp_{hor\_windward} = \begin{pmatrix} -1 \\ 0 \end{pmatrix} \qquad k := 0..4$$

$$GCp_{hor\_leeward} = \begin{pmatrix} -0.75 \\ 0 \end{pmatrix}$$

Section 1205.3: Stability Indicates: (c) the uplift forces calculation from wind pressure shall not exceed two-thirds of the resisting dead load.

This is interpreted as limiting Dead load contribution to 67%

$$\phi := \frac{2}{3}$$

$$p_{hor\_windward} := q_h \cdot GCp_{hor\_windward}$$

$$p_{hor\_leeward} := q_h \cdot GCp_{hor\_leeward}$$

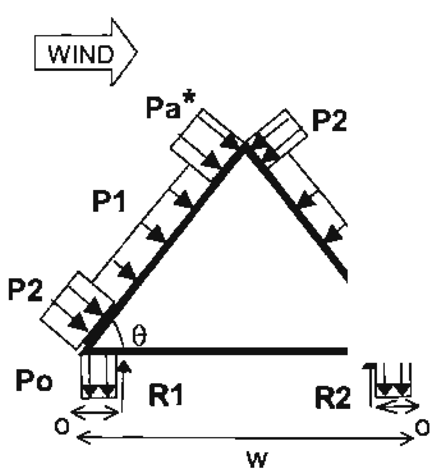
$$p_{hor\_windward} = \begin{pmatrix} -30.976 \\ 0 \end{pmatrix} psf$$

$$p_{hor\_leeward} = \begin{pmatrix} -23.232 \\ 0 \end{pmatrix} psf$$

WIND Perpendicular to Ridge at section A-A

STRAP RESISTANCE used in ARA model

Sum Moments (note that in the mathcad formulas p0 is zone r pressures and p2 is zone si pressure)




$$\begin{aligned} R_0 := \frac{\Delta}{(W - 2\cdot o)} \cdot \Bigg[ & p_{hor\_windward} \cdot \frac{W}{3\cdot\cos(\theta)} \cdot \cos(\theta) \cdot \left(W - o - \frac{W}{3}\cdot 0.5\right) \ldots \\ & + p_{hor\_leeward} \cdot \left(l_r - \frac{W}{3\cdot\cos(\theta)}\right) \cdot \cos(\theta) \cdot \left(W - o - \frac{W}{3} - \frac{W}{12}\right) \ldots \\ & + p_{hor\_leeward} \cdot (l_r) \cdot \cos(\theta) \cdot \left(\frac{W}{4} - o\right) \ldots \\ & + -p_{hor\_windward} \cdot \left(\frac{W}{3\cdot\cos(\theta)}\right) \cdot \sin(\theta) \cdot \left(\frac{W}{3\cdot\cos(\theta)} \cdot \frac{\sin(\theta)}{2}\right) \ldots \\ & + -p_{hor\_leeward} \cdot \left(l_r - \frac{W}{3\cdot\cos(\theta)}\right) \cdot \sin(\theta) \cdot \left[\left(l_r - \frac{W}{12\cdot\cos(\theta)}\right) \cdot \sin(\theta)\right] \ldots \\ & + p_{hor\_leeward} \cdot (l_r) \cdot \sin(\theta) \cdot \left(\frac{l_r}{2}\right) \cdot \sin(\theta) \ldots \\ & + \phi \cdot DL_{roof} \cdot W \cdot \left(\frac{W}{2} - o\right) \Bigg] \end{aligned}$$

$$R_0 = \begin{pmatrix} -818.301 \\ 228 \end{pmatrix} lbf$$

Sum Forces in Vertical

$$R_1 := \left[\Delta \cdot \left[p_{hor\_windward} \cdot \frac{W}{3\cdot\cos(\theta)} \cdot \cos(\theta) + \left(p_{hor\_leeward}\right) \cdot \left(\frac{2\cdot W}{3\cdot\cos(\theta)}\right) \cdot \cos(\theta)\right] - R_0 + \phi \cdot DL_{roof} \cdot \Delta \cdot W\right]$$

$$R_1 = \begin{pmatrix} -687.513 \\ 228 \end{pmatrix} lbf$$

Design value:

$$R = \begin{bmatrix} \begin{pmatrix} -818.301 \\ 228 \end{pmatrix} \\ \begin{pmatrix} -687.513 \\ 228 \end{pmatrix} \end{bmatrix} \text{lbf}$$

$R_{design} := \min(R_0)$

$R_{design} = -818.301\,\text{lbf}$

Ultimate Failure Strength

Convert from 5%ile of Ultimate Distribution
Mean and SD of Ultimate distribution

$ratio_{5\%U} := 1.196$

$ratio_{5\%SD} := 0.1196$

$$R_U := \frac{R_{design}}{1.6} \cdot 3 \cdot ratio_{5\%U} \qquad R_U = -1835.04\,\text{lbf}$$

$$R_{Us} := \frac{R_{design}}{1.6} \cdot 3 \cdot ratio_{5\%SD} \qquad R_{Us} = -183.504\,\text{lbf}$$

# <u>SUMMARY:</u>

Design Parameters: $V = 110\,\text{mph}$ $IntPressure = 0$

Nail Spacing: $nailsize = 6$ $s_e = 6\,\text{in}$ edge of plywood $s_i = 12\,\text{in}$ interior of plywood

Straps: C&C loads $R_{design} = -818.301\,\text{lbf}$ $R_U = -1835.04\,\text{lbf}$

Window Design Pressure: $\max(DP) = 34.074\,\text{psf}$ $\min(DP) = -17.037\,\text{psf}$