Finite element analysis of carbon fiber reinforced polymer emergency shelters

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A Thesis
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Finite Element Analysis of Carbon Fiber Reinforced Polymer Emergency Shelters
By
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This research combines novel lightweight, strong materials, sustainable mobile design solutions, and analysis of extreme wind forces on an emergency shelter. The goal of this research is to develop a versatile, lightweight, strong, and economic emergency shelter which could withstand category five hurricane forces using linear finite element analysis (FEA).

The calculation takes into account that the emergency shelter should withstand wind gusts of 157 mile per hour (mph) and missile projectiles. The Main Wind Force Resisting System (MWFRS) and Components and Cladding (C&C) force are used in the wind analysis. The model for the emergency shelter includes composite sandwich panels and steel cable wires. The underlying aim of this analysis is to develop a model to predict how these different parts will react in a static state, bending to the constant or pressure load of the model.

In proposing the design and analysis, two materials were taken into consideration. Expanded Polystyrene (EPS) foam core, at a thickness of five inches, and a Carbon Fiber Reinforced Polymer (CFRP) composite with a thickness of 0.047 inches on each side was
used as the face sheet material. The total combined thickness of the panel is projected to be 5.09 inches. The advantage to using CFRP over conventional components are measured in vulnerability assessments of both conventional and composite wall panel systems.

FEA software package ANSYS 15.0 has been utilized to carry out this design of different models of composite sandwich panel and steel cable structures under the wind pressure loading. The aim of proposing different models is to compare results.

The panels of the emergency shelter were modeled by shell as well as solid elements and the results of the panels were compared. In addition, various configurations of anchored steel cables were also investigated. One of the cable configurations was analyzed only with the emergency shelter. The effect of both concentrated and pressure loads were also examined on the wall panels of the emergency shelters. Solid element was selected to model the emergency shelter for future investigation.

Regarding the steel cables supporting the emergency shelter, the results demonstrated that the optimal and appropriate positioning of the pre-tensioned loading of cables had significant influence on the performance of the shelter. In the future, this research could direct future applications in relation to steel truss design and configuration.

Through combining panels and steel cables, the emergency shelter could sustain the pressure load. The highest deflection detected was 3.2 inches for the Components and Cladding (C&C) model. The results recorded a deflection of 1.1 inches for the Main Wind Force Resisting System (MWFRS) model. These results demonstrated that the emergency shelter design may possibly withstand other types of extreme loading. However, the use of this system in other applications will depend upon the deflection thresholds of the design.
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First and foremost, I would like to pass along my deep gratitude and thanks to my professors in the Civil Engineering department at the University of Toledo. This process has been difficult on the path to the Master’s Degree, but I want to thank the faculty for extending their expertise and wisdom in the classroom and beyond.

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the seed together and reaped it together and we will be together the entirety of our lives,
God willing.
Contents

Abstract iii
Acknowledgements vi
Contents viii
List of Tables xiii
List of Figures xv
List of Graphs xx
List of Abbreviations xxii

1 Introduction 1

1.1 Background ......................................................... 1
1.2 In the Danger Zone - Seismic activity .......................... 2
1.3 In the Danger Zone: Hurricanes .............................. 4
1.4 Natural conditions faced in a hurricanes .................... 6
1.5 Hurricane Hugo ................................................... 7
1.6 In the Danger Zone - Charleston, South Carolina .......... 8

2 Literature Review 9

2.1 Literature Review Summary ................................. 9
2.2 How structures fail in hurricanes ............................ 10
2.3 Fiber Reinforced Polymer in Construction ................ 11
2.4 Background of Temporary Disaster Housing ............. 14
2.4.1 Background ...................................................................................................14
2.4.2 Predicting Earthquakes .................................................................................15
2.4.3 Planning for Emergency Shelter Placement ...............................................16
2.4.4 Current Designs and Manufactures...............................................................18
2.4.5 Transportation ...............................................................................................22
2.4.6 Assembly on Site ..........................................................................................23
2.4.7 Addressing Post Hurricane or Earthquake Aftershock Concerns .................23
2.4.8 How the panels are connected.......................................................................25
2.4.9 Emergency Shelter Design and Manufacture ..............................................26

2.5 Background of FRP modular construction and emergency shelters ..............28
2.5.1 Background ..................................................................................................28
2.5.2 FRP Materials for Increased Strength and Durability.................................30
2.5.3 Ballistic testing of CFRP .............................................................................33
2.5.4 Expanded Polystyrene Foam (EPS) .............................................................34
2.5.5 Previous Finite Element Models of Emergency Shelters ..............................35

2.6 Collective Conclusions and Emergent Concepts ............................................38
2.7 Original Contribution and Significance of Project .........................................39

3 Methodology .......................................................................................................40
3.1 Problem Statement .........................................................................................40
3.2 Design process ...............................................................................................41
3.2.1 Improved Portability ..................................................................................41
3.2.2 Improved Safety .........................................................................................41
3.2.3 Improved ease of Construction .................................................................41
3.2.4 The Finite Element Method of Analysis ...................................................44
List of Tables

1.1 Magnitude vs. Ground Motion and Energy .................................................................3
1.2 Tropical Cyclone History, from National Weather Service .................................5
1.3 Saffir-Simpson Hurricane Wind Scale (SSHWS) ..................................................7
2.1 Carbon Fiber Reinforced Polymer (CFRP) properties .........................................32
2.2 Physical and Mechanical Properties of EPS foam ...............................................35
2.3 Summarized Wind Loads for Internal Pressure Coefficients of ±0.55 .............36
3.1 Proposed Solution Methodology ...........................................................................48
4.1 Figure 27.4-1, ASCE 7-10 ....................................................................................58
4.2 Table 26.11-1, ASCE 7-10 ....................................................................................59
5.1 Fiber layup for non-symmetric and symmetric ...................................................67
5.2 Mechanical Properties of CFRP and EPS Foam Core ......................................73
5.3 Summary maximum displacements of 2D shell element model .........................82
5.4 Summary stress of 2D shell element for non-symmetric fiber layers ...............93
5.5 Summary stress of 2D shell element for symmetric fiber layers ....................93
5.6 Summary deflection values of all cases along span for 2D shell element ...........95
5.7 Summary of deflection value vs. load for 2D shell element ...............................96
5.8 Summary of stress values in X-Direction along the span for 2D shell element ....97
5.9 Summary of stress values in Z-Direction along the span for 2D shell element ....98
5.10 Summary of shear stress XZ values along span for 2D shell element ..........99
5.11 Summary maximum displacement of 3D solid element model ......................110
5.12 Summary of Stress Values in X-Direction along the span ...........................................115
5.13 Summary of stress values in Z-Direction along the span ...........................................120
5.14 Shear stress values of cases 1, 2, and 3, along the span.............................................124
5.15 Summary stress of 3D solid element for non-symmetric fiber layers .......................126
5.16 Summary stress of 3D solid element for symmetric fiber layers ..............................126
5.17 Summary of deflection values of cases 1, 2, 3, along the span.................................128
5.18 Summary of deflection values vs. load of cases 1, 2, and 3 .................................129
6.1 Mechanical Properties................................................................................................138
6.2 Comparison of displacements of the three models B, C, and D ...............................140
6.3 Comparison and summary of axial force and axial stress per cable .........................143
7.1 Summary of Flexural Stress, Shear Stress, and Principal Stress of the emergency shelter model for MWFRS .................................................................157
7.2 Summary of Flexural Stress, Shear Stress, and Principal Stress of the emergency shelter model for C&C .................................................................157
7.3 Summary of Load vs. Deflection ..............................................................................158
7.4 Summary of the Deflection along the wall .............................................................159
7.5 Summary of Flexural Stress and Shear Stress along the wall for both models of MWFRS and C&C .................................................................160
List of Figures

1-1 seismic zone ...........................................................................................................................................2
1-2 U.S. Geological Survey National Seismic Hazard Map .........................................................................3
1-3 Damages of Hurricane Hugo on The Ben Sawyer Bridge ....................................................................5
1-4 GOES-7 Infrared Satellite Loop of Hugo ............................................................................................8
2-1 Forces on a building due to wind moving around the Structure .........................................................10
2-2 Piece of Plywood driven through a palm tree ..................................................................................11
2-3 Emergency Shelter ............................................................................................................................12
2-4 Panel joints ............................................................................................................................................13
2-5 Picture of FRP emergency Shelter ...................................................................................................14
2-6 Linear System .....................................................................................................................................17
2-7 Central Settlement Model ..................................................................................................................17
2-8 Sample of a Linear-Central Settlement Model ................................................................................18
2-9 Durakit panels assembly ..................................................................................................................20
2-10 Durakit parts before assembly .........................................................................................................20
2-11 LEEP Design being assembled .......................................................................................................21
2-12 Cross Section of panel .....................................................................................................................21
2-13 Finished futuristic worldwide homes ...............................................................................................22
2-14 Notice the crane at top ......................................................................................................................22
2-15 example of wind testing diagnostic ..................................................................................................24
2-16 Roof Edge Connectors .....................................................................................................................25
5-10 Total Maximum Displacement Contour Plot of Shell Element, Concentrated Load on 32 nodes ...........................................................................................................................................80

5-11 Total Maximum Displacement Contour Plot of Shell Element, Pressure Load......81

5.12 - Flexural Stress Distribution in X-Direction of Shell Element, Concentrated Load on 8 nodes ...........................................................................................................................................83

5-13 Flexural Stress Distribution in X-Direction of Shell Element, Concentrated Load on 32 nodes ...........................................................................................................................................84

5-14 Flexural Stress Distribution in X-Direction of Shell Element, Pressure Load on the Surface ...........................................................................................................................................85

5-15 Flexural Stress Distribution in Z-Direction of Shell Element, Concentrated Load on 8 nodes ...........................................................................................................................................87

5-16 Flexural Stress Distribution in Z-Direction of Shell Element, Concentrated Load on 32 nodes ...........................................................................................................................................88

5-17 Flexural Stress Distribution in X-Direction of Shell Element, Pressure Load on the Surface ...........................................................................................................................................89

5-18 Shear Stress Distribution in XZ Plane for Shell Element, Concentrated Load on 8 nodes ...........................................................................................................................................90

5-19 Shear Stress Distribution in XZ Plane for Shell Element, Concentrated Load on 32 nodes ...........................................................................................................................................91

5-20 Shear Stress Distribution in XZ Plane for Shell Element, Pressure Load.............92

5-21 Deflection data across the span of all cases for 2D shell element.......................95

5-22 Deflection vs Load of 2D Composite Sandwich Panel ...........................................96

5-23 Comparison of Bending Stress in X-Direction Along the Span.........................97

5-24 Comparison of Bending Stress in Y-Direction Along the Span .........................98

5-25 The Comparison of Shear Stress XY Along the Span .........................................99

5-26 Unmeshed Modeling Geometry of 3D Solid Model .............................................102
<table>
<thead>
<tr>
<th>Page</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-24</td>
<td>Mesh plot for analysis 3D model</td>
</tr>
<tr>
<td>5-28</td>
<td>Boundary Conditions and Load Application</td>
</tr>
<tr>
<td>5-29</td>
<td>Total Maximum Displacement Contour Plot of Solid Element, Concentrated Load on 8 nodes</td>
</tr>
<tr>
<td>5-30</td>
<td>Total Maximum Displacement Contour Plot of Solid Element, Concentrated Load on 32 nodes</td>
</tr>
<tr>
<td>5-31</td>
<td>Total Maximum Displacement Contour Plot of Solid Element, Pressure Load</td>
</tr>
<tr>
<td>5-32</td>
<td>Flexural Stress Distribution in X-Direction of Solid Element, Concentrated Load on 8 nodes</td>
</tr>
<tr>
<td>5-33</td>
<td>Flexural Stress Distribution in X-Direction of Solid Element, Concentrated Load on 32 nodes</td>
</tr>
<tr>
<td>5-34</td>
<td>Flexural Stress in X-Direction of Solid Element, Pressure Load</td>
</tr>
<tr>
<td>5-35</td>
<td>Comparison of Bending Stress in X-Direction Along Span of 3D Solid Element</td>
</tr>
<tr>
<td>5-36</td>
<td>Flexural Stress Distribution in Z-Direction of Solid Element, Concentrated Load on 8 nodes</td>
</tr>
<tr>
<td>5-37</td>
<td>Flexural Stress Distribution in Z-Direction of Solid Element, Concentrated Load on 32 nodes</td>
</tr>
<tr>
<td>5-38</td>
<td>Flexural Stress in Z-Direction of Solid Element, Pressure Load</td>
</tr>
<tr>
<td>5-39</td>
<td>Comparison of bending stress in Z-Direction along the span</td>
</tr>
<tr>
<td>5-40</td>
<td>Shear Stress Distribution in XZ Plane for Solid Element, Concentrated Load on 8 nodes</td>
</tr>
<tr>
<td>5-41</td>
<td>Shear Stress Distribution in XZ Plane for Solid Element, Concentrated Load on 32 nodes</td>
</tr>
<tr>
<td>5-42</td>
<td>Shear Stress Distribution in XZ Plane, Solid Element, Pressure Load</td>
</tr>
<tr>
<td>5-43</td>
<td>The Comparison of shear stress XZ along the span</td>
</tr>
<tr>
<td>5-44</td>
<td>Predicted deflection data across the span of 3D model</td>
</tr>
</tbody>
</table>
5-45 Deflection vs load summary and comparison of 3D solid cases ......................... 128
5-46 Percentage changes of displacement between cases........................................... 129
5-47 Deflection vs load comparison of 2D shell and 3D solid elements .................... 129
5-48 Deflection data across the span of 2D shell and 3D solid elements .................... 130
5-49 Comparison of bending stress in X-Direction along the span of 2D shell element and 3D solid element .......................................................................................................................... 130
5-50 Comparison of bending stress in Z-Direction along the span of 2D shell element and 3D solid element .......................................................................................................................... 131
5-51 The Comparison of shear stress XZ along the span of 2D shell element and 3D solid element .......................................................................................................................... 131
6-1 Schematic of the Beam and cable ....................................................................... 135
6-2 LINK180 Geometry ............................................................................................. 137
6-3 BEAM188 Geometry ........................................................................................... 138
6-4 Total Maximum Displacement Contour Plot ....................................................... 140
6-5 The Maximum Stress in the Local X-Direction .................................................... 142
6-6 Load vs. Deflection .............................................................................................. 143
7-1 The Geometry of the Shelter .............................................................................. 146
7-2 Directions of fiber layer ....................................................................................... 148
7-3 Meshing of CFRP face sheets and EPS Foam Core ............................................ 149
7-4 Pressure Loading Transferred to Finite Element .............................................. 150
7-5 Deformed configuration ....................................................................................... 151
7-6 Total Maximum Displacement Contour Plot for MWFRS .................................. 151
7-7 Total Maximum Displacement Contour Plot for C&C ...................................... 152
7-8 Maximum stress in the Local Y-Direction for MWFRS ....................................... 153
7-9 Maximum stress in the Local Y-Direction for C&C ............................................ 154
7-10 Maximum stress in the Local Z-Direction for MWFRS ........................................154
7-11 Maximum stress in the Local Z-Direction for C&C ...........................................155
7-12 Maximum Shear Stress distribution in YZ Plane for MWFRS ...............................155
7-13 Maximum Shear Stress distribution in YZ Plane for C&C ....................................156
7-14 Load vs. Deflection ...............................................................................................158
7-15 Deflection along the panel of the wall of the emergency shelter .......................159
7-16 Comparison of bending stress in Y-Direction along the span of MWFRS Model and C&C model ......................................................................................................................160
7-17 Comparison of bending stress in Z-Direction along the span of MWFRS Model and C&C model ......................................................................................................................161
7-18 The Comparison of shear stress YZ along the span of MWFRS Model and C&C model ......................................................................................................................161
List of Abbreviations

ACI ........................................................................................................ American Concrete Institute
ATC ........................................................................................................ Applied Technology Council
ASCE ................................................................................................. American Society of Civil Engineering
C&C ...................................................................................................... Components and Cladding
CFD ...................................................................................................... Computational Fluid Dynamic
CFRP .................................................................................................... Carbon Fiber Reinforced Polymer
DOF ...................................................................................................... degrees of freedom
EPS ...................................................................................................... Expanded Polystyrene
FE ........................................................................................................ Finite Element
FEA ...................................................................................................... Finite Element Analysis
FEM ...................................................................................................... Finite Element Method
FEMA ................................................................................................. Federal Emergency Management Agency
FRP ...................................................................................................... Fiber Reinforced Polymer
LEEP ................................................................................................. Leading Edge Earth Products
LRFD ................................................................................................. Load Resistance Factor Design
mph ..................................................................................................... Mile per hour
MWFRS ............................................................................................. Main Wind Force Resisting System
USUM ................................................................................................. Total Max Displacement in Y or X-Direction
Chapter I

Introduction

1.1 Background

Natural disasters have plagued civilizations and populations since the dawn of time. How structures are designed to withstand the forces of nature, whether it be extreme wind shear force, seismic activity, or other forms of devastation. This is a problem that engineers have grappled with for years. Fiber Reinforced Polymers have revolutionized how prefabricated emergency shelters can be mass produced at light weights and low cost. These advances have made the units cheaper to produce, move, and erect as needed on disaster sites.

As populations grow across the world, and as forecasted extreme weather systems become a more frequent phenomenon faced by both the developed and developing world, a light-weight, cost efficient, portable and sustainable emergency shelter system is needed (Felix et al., 2013). Considering this need, testing is necessary to determine the structural elements of the design will fare in natural conditions such as those of a hurricane. To replicate these conditions, it is necessary to test the structural capacities of the design, specifically the anchored steel cables that are integral to the integrity of the design. To perform such an analysis, through finite element modeling, is the basis of this research.
1.2 In the Danger Zone - Seismic activity

Though there are many areas in the world where seismic activity and potential hurricane damage cross paths to create a particularly risky zone for potential natural disasters, in the United States these areas are few. In the Southeastern United States, along the coast between Georgia and North Carolina, lies the city of Charleston, South Carolina. In 1886, a 7.0 magnitude earthquake struck the city, resulting in many deaths and millions of dollars in damage to the city. As can be seen above, in an image taken from the United States Geological Survey (USGS), the destruction was immense. This event ranks as the largest seismic activity along the east coast of the United States in recorded history. The
city, though long since recovered from the devastation of the earthquake, still lies in an area where increased risk for seismic activity is predicted. As can be seen to the left side of Figure 1-2, there are numerous fault lines crisscrossing the area adjacent to Charleston. The dark black lines represent fault lines in the immediate area. The United States Geological Survey places South Carolina in the top 16 states likely to have an earthquake. Most recently, seismic activity was detected in Edgefield, South Carolina, where a 4.1 magnitude earthquake was recorded in February of 2015. As can be seen below, the majority of the state of South Carolina lies in a highly dangerous zone of potential seismic activity.

Figure 1-2: U.S. Geological Survey National Seismic Hazard Map

Table 1.1: Magnitude vs. Ground Motion and Energy

<table>
<thead>
<tr>
<th>Magnitude Change</th>
<th>Ground Motion Change (Displacement)</th>
<th>Energy Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>10.0 times</td>
<td>about 32 times</td>
</tr>
<tr>
<td>0.5</td>
<td>3.2 times</td>
<td>about 5.5 times</td>
</tr>
<tr>
<td>0.3</td>
<td>2.0 times</td>
<td>about 3 times</td>
</tr>
<tr>
<td>0.1</td>
<td>1.3 times</td>
<td>about 1.4 times</td>
</tr>
</tbody>
</table>
In the table above, it can be seen that a 7.2 magnitude earthquake gives off more than ten times the ground motion that is seen in a 6.2 magnitude earthquake, releasing around 32 times the amount of energy. This tremendous amount of energy tells of the destructive power that an earthquake can produce. It is estimated by the U.S. Geological survey (2015) that millions of earthquakes in varying magnitudes occur each year. Most of these earthquakes happen in remote areas of the world and go undetected. One organization that looks at earthquake magnitude and their total numbers is the National Earthquake Information Center (NEIC). This center finds around 20,000 notable earthquakes on a yearly basis. As the number of seismographs around the world increases, more earthquakes can be detected. Given the high number of earthquakes, the percentage of large earthquakes with a larger than 6 magnitude may increase on a yearly basis.

1.3 In the Danger Zone: Hurricanes

According to the National Weather Service (2016), a branch of the National Oceanic and Climate Administration, 47 hurricanes have made landfall in Georgia and South Carolina since they began keeping records in 1851. Of these 47 hurricanes, 27 have directly affected the Charleston, SC County warning area, as seen in Figure 1-3 below on the graphic taken from the National Weather Service. The varying severity of these storms can be seen below, and in the Figure below, taken during hurricane Hugo in 1989. The varying wind speeds assigned to the hurricane categories give an idea of potential winds in the case of the most devastating, category 5 hurricane.
Figure 1-3: Some Damage of Hurricane Hugo on The Ben Sawyer Bridge Connecting Sullivan's Island to Charleston, S.C. National Oceanic And Atmospheric Administration (NOAA).

Table 1.2: Tropical Cyclone History, from National Weather Service

<table>
<thead>
<tr>
<th>Decade</th>
<th>Number of Storms Making Landfall in the NWS Charleston, SC 'County Warning Area'</th>
<th>Landfall Intensity (H-Hurricane; TS-Tropical Storm; TD-Tropical Depression)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1900-1909</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1910-1919</td>
<td>7</td>
<td>3 H; 3 TS; 1 TD</td>
</tr>
<tr>
<td>1920-1929</td>
<td>2</td>
<td>1 H; 1 TS</td>
</tr>
<tr>
<td>1930-1939</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>1940-1949</td>
<td>3</td>
<td>2 H; 1 TS</td>
</tr>
<tr>
<td>1950-1959</td>
<td>4</td>
<td>3 H; 1 TD</td>
</tr>
<tr>
<td>1960-1969</td>
<td>1</td>
<td>1 TS</td>
</tr>
<tr>
<td>1970-1979</td>
<td>4</td>
<td>1 H; 1 TS; 2 TD</td>
</tr>
<tr>
<td>1980-1989</td>
<td>4</td>
<td>2 H; 1 TS; 1 TD</td>
</tr>
<tr>
<td>1990-1999</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>2000-2009</td>
<td>3</td>
<td>2 H; 1 TS</td>
</tr>
<tr>
<td>2010-2015</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
1.4 Natural conditions faced in a hurricane

Though there are different names for hurricanes in different parts of the world, the storms are rated dependant on the scale developed by Saffir and Simpson (Table 1.3) (FEMA 361, 2000). This scale, developed in the 1970s, ranks hurricanes on a spectrum of 1 to 5, depending on the intensity of the storm. The intensity of the hurricane was determined through both barometric pressure readings and wind speed (Bradford, 2004). According to the website of National Hurricane Center (2012) states that:

"The Saffir-Simpson Hurricane Wind Scale (SSHWS) is a 1 to 5 rating based on a hurricane's sustained wind speed. This scale estimates potential property damage. Hurricanes reaching Category 3 and higher are considered major hurricanes because of their potential for significant loss of life and damage. Category 1 and 2 storms are still dangerous, however, and require preventative measures. In the western North Pacific, the term "super typhoon" is used for tropical cyclones with sustained winds exceeding 150 mph."
Table 1.3: Saffir-Simpson Hurricane Wind Scale (SSHWS)

<table>
<thead>
<tr>
<th>Category</th>
<th>Sustained Winds</th>
<th>Types of Damage Due to Hurricane Winds</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>74-95 mph</td>
<td><strong>Very dangerous winds will produce some damage:</strong> Well-constructed frame homes could have damage to roof, shingles, vinyl siding and gutters. Large branches of trees will snap and shallowly rooted trees may be toppled. Extensive damage to power lines and poles likely will result in power outages that could last a few to several days.</td>
</tr>
<tr>
<td></td>
<td>64-82 kt.</td>
<td>119-153 km/h</td>
</tr>
<tr>
<td></td>
<td>111-129 mph</td>
<td><strong>Extremely dangerous winds will cause extensive damage:</strong> Well-constructed frame homes could sustain major roof and siding damage. Many shallowly rooted trees will be snapped or uprooted and block numerous roads. Near-total power loss is expected with outages that could last from several days to weeks.</td>
</tr>
<tr>
<td></td>
<td>96-110 mph</td>
<td>83-95 kt.</td>
</tr>
<tr>
<td></td>
<td>154-177 km/h</td>
<td>96-112 kt.</td>
</tr>
<tr>
<td></td>
<td>178-208 km/h</td>
<td><strong>Devastating damage will occur:</strong> Well-built framed homes may incur major damage or removal of roof decking and gable ends. Many trees will be snapped or uprooted, blocking numerous roads. Electricity and water will be unavailable for several days to weeks after the storm passes.</td>
</tr>
<tr>
<td>3 (major)</td>
<td>111-129 mph</td>
<td><strong>Catastrophic damage will occur:</strong> Well-built framed homes can sustain severe damage with loss of most of the roof structure and/or some exterior walls. Most trees will be snapped or uprooted and power poles downed. Fallen trees and power poles will isolate residential areas. Power outages will last weeks to possibly months. Most of the area will be uninhabitable for weeks or months.</td>
</tr>
<tr>
<td></td>
<td>96-112 kt.</td>
<td>178-208 km/h</td>
</tr>
<tr>
<td></td>
<td>130-156 mph</td>
<td><strong>Catastrophic damage will occur:</strong> A high percentage of framed homes will be destroyed, with total roof failure and wall collapse. Fallen trees and power poles will isolate residential areas. Power outages will last for weeks to possibly months. Most of the area will be uninhabitable for weeks or months.</td>
</tr>
<tr>
<td></td>
<td>113-136 kt.</td>
<td>209-251 km/h</td>
</tr>
<tr>
<td>5 (major)</td>
<td>157 mph or higher</td>
<td>137 km/h or higher</td>
</tr>
<tr>
<td></td>
<td>137 km/h or higher</td>
<td>252 km/h or higher</td>
</tr>
</tbody>
</table>

1.5 Hurricane Hugo

According to the National Weather Service website, most recently, the Charleston area was hit by Hurricane Hugo in 1989. Hugo was a category four storm which hit
Sullivan’s Island first. This was the first storm to make landfall in South Carolina since Hurricane Gracie in 1959. The strongest winds reached 135 miles per hour, and happened in Charleston County’s northern areas. The airport in North Charleston reported gusts of up to 98 miles per hour, while in the downtown areas of the city, the top gust reached 108 miles per hour. On the water, a Coast Guard ship reported a wind gust of 138 miles per hour. The storm surge from the hurricane near Cape Romain rose 10-12 feet above the average sea level. Overall, rain total were between 5 and 10 inches, which the highest recorded rainfall of 10.28 inches was at Edisto island.

![Figure 1-4: GOES-7 Infrared Satellite Loop of Hugo (September 22, 1989), (NWS, NOAA)](image)

**1.6 In the Danger Zone - Charleston, South Carolina**

Given the unique situation of the potential seismic and hurricane damage in Charleston, South Carolina, the area certainly in a position for advanced preparedness in terms of disaster relief. Given the possibility of both seismic and high wind forces in the city, it will serve as the context for the study and analysis of the CFRP emergency shelter system.
Chapter II

Literature Review

2.1 Literature Review Summary

Given the background of Fiber Reinforced Polymer (FRP) construction methods and the emergent field enabled by technological advances in science, FRP techniques are impacting many areas of what have long been structures made of steel, wood, or other materials. How these modular, reusable, strong and durable construction materials can be combined with carbon reinforced fibers in a moveable, lightweight and easily constructed emergency shelter system is the focus of this literature review. Through tracing FRP modular construction trends, and combining these ideas with carbon reinforced polymer materials, a surface able to withstand significant point loads such as those experienced through tornado and hurricane debris can be addressed in adequate ways. Also, this literature review looks at how these panels can be connected in ways which align with the design aspirations for the shelter. Furthermore, this literature review traces how designs have advanced over the past decades in response to challenges and lessons learned from previous disasters. Examples of current models are cited, as well as pricing, strengths, and weaknesses for each piece.
2.2 How structures fail in hurricanes

Buildings fail in numerous ways when under duress from a major force, such as hurricane winds. The major ways in which this failure can occur are racking, sliding, and the failure of integral components. The complete failure of a structure usually involves more than one of the aforementioned causes, as illustrated below in Figure 2-1. A failure due to sliding happens when a force hits a structure from the lateral side of the foundation.

![Corrected Image](Image)

Figure 2-1: Forces on a building due to wind moving around the Structure (Qiao, 2003)

The most common type of failure is due to component failure. This occurrence most often happens when wind speeds are especially high. Though this is a common thread which connects all of these different structural failures, this type of failure is likely to be due to the presence of different objects apart from the structure being blown about, such as projectiles, which destroys integral structural components. These disasters may inflict individual component failures or cover the entire structure of system. As can be seen below in Figure 2-2 taken in the aftermath of Hurricane Andrew in Florida during the early 1990s, projectiles take many forms and can cause significant structural damage in hurricane winds. (NOAA, NWS)
In order to provide a degree of safety for any occupants of structures during windstorms of hurricane strength, it is important to anchor the structure to counteract the forces that are trying to move the structure. This means that all of the components of the structure should be strong enough to work against the uplift and overturning forces. Specifically, the parts which connect the roof, doors, and walls must be able to deflect airborne debris that can cause the most common types of structural failure. However, it is difficult to account for the conditions such as projectiles, though some of this research related to blast impacts on Fiber reinforced polymers and the resistance to various force loads in that area could be applicable to the proposed structure (Chun, 2004).

2.3 Fiber Reinforced Polymer in Construction

FRP building systems are used in a wide variety of construction projects around the globe due to their strength and increasing ease of fabrication. Recent developments in
modular building technology, combined with the light weight and cost efficient design properties of these systems have been emerging in many industries. Recent developments of FRP emergency shelter structures currently are joined by research into bulletproof walls, improved bridge deck systems, masonry materials and modular home construction (Bradford, 2004).

Current FRP emergency shelter systems demonstrate numerous cutting edge advances in FRP panel building systems. Figure 2-3 below shows the dimensions of an FRP panel emergency shelter, while Figure 2-4 shows how the panels are joined in construction.

Figure 2-3: Emergency Shelter (Bradford, 2004)
The advances in the FRP technology have meant lighter, sustainable and stronger building materials which are now being considered for modular applications, such as emergency shelters which require heavy load bearing capabilities. These are considered new materials and new construction systems which are designed to capture the performance capabilities of the FRP panelized wall and roofing systems. Emergency
systems such as these are guided by principals related to structural integrity, cost, durability, and construction simplicity (Bradford, 2004). During emergency conditions, such as a natural disaster, solid structures that can withstand hurricane winds and other abnormal conditions are necessary.

Figure 2-5: Picture of FRP emergency Shelter (StyroHome Review, 2010)

In conclusion, this paper will examine, discuss the pros and cons of current FRP panel emergency shelter systems and the possibilities for future improvements in a rapidly developing disaster relief industry.

2.4 Background of Temporary Disaster Housing

2.4.1 Background

In the past, post-disaster temporary housing has been the cause of many problems in the recovery efforts of nations hoping to cope with problems related to displaced families and communities. Specifically, these disaster areas have been faced with an array of
problems in temporary disaster housing (El-Anwar, 2009). These range from an initial delay in the acquisition of these structures and a logistical delay in how the temporary shelters are distributed to the areas. Further problems related to cultural problems, building problems, and the breakdown of necessary social services have caused very high crime rates in areas where temporary disaster shelters have been erected (El-Anwar, 2009). New models of data analysis have helped relief agencies in the developed world to solve some of these problems. For example, in the United States, the Federal Emergency Management Agency (FEMA) significantly overhauled their models of emergency relief following what many saw as a failure regarding emergency shelters in the aftermath of Hurricane Katrina in 2005 (El-Anwar, 2009).

2.4.2 Preparing for Earthquakes

Further research by the Chinese authorities into large scale disaster relief mobilization has focused on simulating responses to various aspects of emergency recovery efforts which have been overlooked in the past. Following the Sichuan earthquake of 2008, many Chinese authorities realized problems in building code enforcement and in disaster response relief (Tang & Wen, 2009). Following this perceived failure on behalf of the government authorities, a new strategy for disaster preparation began. In response to this, risk analyses for certain areas near fault lines have been performed. As a result of these analyses, increased preparation measures have been called for including large scale earthquake simulations (Tang & Wen, 2009).
2.4.3 Planning for Emergency Shelter Placement

The topic of disaster relief housing recently became a topic of debate in Geneva, where the international Red Cross, World Vision, and Habitat for Humanity worked together to find a solution for the design and logistical problems inherent in finding solutions for disaster relief housing. From this meeting, the basic necessities for constructing a shelter that can be built quickly, economically, and still provide adequate protection were outlined (Booth, 2012). These outcomes have framed designs which the consortium felt served as a traditional house that allowed citizens to collaboratively rebuild structures in the wake of disaster in ways which brought the community closer and which aligned which culturally appropriate ideas of what a house should look like (Booth, 2012). Currently, there are a number of different manufacturers who have researched designs and the manufacturing possibilities of fiber-reinforced polymer (FRP) composite panel systems, or “shelters in a box”.

From the 14th World Conference on Earthquake Engineering, a consensus was reached concerning the three types of shelters that are often required in the aftermath of an earthquake disaster. These shelters, however, need to be adaptable for protection from Hurricanes as well. The three types of shelters required were determined to be: a) emergency shelters, which were loosely defined as tents b) temporary shelters, which were constructed for a term of 1 to 2 years, and c) shelters that were designed for long term permanent housing. The engineers determined that the second category of shelter was the most problematic in terms of design and engineering because it had to suffice in providing the same characteristics of a permanent home even though it was not a permanent structure (Forouzandeh, 2008).
According to numerous researchers, there are certain criteria that must be considered in assessing emergency shelter considerations. These include the number of refugees, the functionality of the site, the infrastructures available at the site, the configuration of the site, and the accessibility (Forouzandeh, 2008).

Beyond the suitability of a site, further research into how these various sites are arranged is needed to appease the cultural context in which the emergency shelters are to be placed. There are many different ideas for how shelters should be arranged. Some examples, such as a linear system, a linear-central system, and a central system are depicted below:

Figure 2-6: Linear System (Forouzandeh, 2008).

Figure 2-7: Central Settlement Model (Forouzandeh, 2008).
An important aspect of choosing emergency shelter design for any context should involve the participation of the people in the planning process and development of an appropriate course of action. Evidence from many prior disasters has documented how disaster recovery efforts have been significantly hindered by overlooking the will of the people being most affected by the relief efforts. In assessing the applicability of an emergency shelter for a disaster site, this central process is important.

The focus of this research is to design a shelter which would fit most closely with type b from the 14th conference on earthquake engineering, called temporary housing, which is constructed for a term of one or two years. It could fit in any of the shelter site systems mentioned in the figures above.

2.4.4 Current Designs and Manufacturers

Zipflat, a manufacturer of disaster relief housing, cited statistics of 230 million people who are rendered homeless in an average year due to natural disasters, 100 million of whom
are left without shelter in the wake of the destruction. The design tried to keep in mind the best transitional house solution to protect inhabitants from natural elements, to be affordable, and to employ local labor in construction, which are essential components of the Zipflat construction vision (Bradord, 2004; Zipflat, 2014).

The design for the Zipflat shelter is based upon a snap together system where foam is poured into the walls to provide stability. Simple hand tools, and simple assembly steps that unskilled laborers can follow to erect a 102 square foot house in just a few hours.

The Zipflat shelter is earthquake resistant and does not use large wood, metal, or other cement materials and involves anchoring methods for wind resistance. The foam insulation makes the walls well insulated and leak-proof, while their initial opening makes the addition of electrical wiring or plumbing (in-wall) possible (Zipflat, 2014).

Another company, DuraKit, uses a shelter design that is composed of a resin saturated cardboard composite material. The shelters are constructed on site and are highly portable. The different parts are assembled using an epoxy resin adhesive (Bradford, 2004).

The benefits of the Durakit system are the price and simplicity of construction. The disadvantages of these designs are the permanency in terms of building. They cannot be taken down easily and have had problems with durability in high temperature and humid environments. The Durakit emergency shelter system can be seen below:
Another company invested in the manufacture and design of FRP emergency shelters is Leading Edge Earth Products (LEEP), a corporation that uses a design composed of composite panel systems, corrugated steel skins, and a foam filled core. The benefits of these systems relate to their strength. Furthermore, it is possible to build these units in two story structures because they are built on site. But if another disaster came, this building would collapse. Also, the design is simple to construct and there is excellent supporting data in terms of testing the components. The weakness in relation to this design is the reliance on the supporting steel frame in terms of the design. Because of this steel structure, issues with corrosion have occurred in areas of high humidity and temperature (Bradford, 2004).
The next manufacturer of emergency FRP shelters is CoreFlex, a company which manufactures pultruded FRP panel systems which snap fit into connection ends. These panels can be foam filled to provide more insulation and stiffness. The benefits of this design are the ease by which it is constructed and the ease by which it is disassembled as well. The negative side of the design is the necessity of the supporting structural system. There are issues related to durability in high temperature and humidity. There is very little supporting research into the design of Coreflex shelter, and the engineering calculations are incomplete (Bradford, 2004).

The last manufacturer is Futuristic Worldwide Homes. This design employs FRP members substituted for wood members in a wood framed wall stud system. One benefit related to this design is the ease with which it is assembled. Negatives related to this design include requiring a crane to erect the shelter and the need for a supportive structural system. There are also extra structural connections required, and there are issues with durability in high temperature and humid environments. Supporting documentation concerning the design is also lacking (Bradford, 2004).
These manufacturers all used designs which depended upon underlying structures and a degree of standard construction materials, tools, and construction knowledge that will not exist in abundance in areas where hurricanes or earthquakes have devastated the surrounding area.

2.4.5 Transportation

In terms of transportation, the standard device used to measure how many units can be moved is based upon the standard MILVAN containers and types used aboard container ships. Due to different parameters of airlift capability, how many units can be transported are also estimated in how many military C130 pallets would be needed to carry the FRP systems (Bradford, 2004). Dimensions of a MILVAN container are eight feet by eight feet by 20 feet (2.44m x 2.44 m x 6.1m).
2.4.6 Assembly on Site

Time and training are needed to assemble these products on site depending upon the different designs. According to CoreFlex documentation, the design takes five workers five hours to assemble the shelter, for a total of 25 staffing hours. For the CoreFlex system, five units can be transported in a standard Milvan container (Bradford, 2004). According to Leading Edge Earth products, the system takes 4 workers 8 hrs. each, for a total of 32 staffing hours to assemble. In terms of transportability, one unit fits in a standard MILVAN container. According to DuraKit documentation, it takes two trained workers 30 hrs. for a total of 60 staffing hours to assemble their design. It takes 80 staffing hours for unskilled workers. In a standard MILVAN shipping container, five units can be transported. Lastly, according to Futuristic Worldwide Homes, it takes eight workers 16 hours for a total of 128 staffing hours to assemble one of their units. Regarding transportation, one unit fits on the C130 (Bradford, 2004). It takes time and also a lot of staffing to assemble the current designs mentioned. This research focuses on reducing the time of assembly and staffing.

2.4.7 Addressing Post Hurricane or Earthquake Concerns

A concern among any government agency or relief organization is the potential for earthquake aftershocks or residual damage resulting from hurricane rains (mudslides, flooding, etc.). For these emergency shelters to survive these unfortunate events, a certain degree of logistical planning and wise building location is needed in the case of both hurricanes and earthquakes. Level and open areas away from potential flash flood zones or landslides should be considered for emergency shelter construction. This very simple strategy can help these shelters and people they are protecting avoid further hardship due
to the disaster and its after affects. Thus, an initial survey of the area is important for future logistical purposes. Shelters need to be constructed near a water source, but not too close to a flash flood area. Emergency shelters need to have proper means of maintaining sanitation given the often long process of reconstructing the stricken communities. Also, a system of drainage or a higher elevation should be sought to ease in removing dangerous sewage.

Another important aspect of shelters is their large, one story design. While the LEEP design can include two stories, there are not any secure designs with two or more stories because of the increased likelihood of collapse in an earthquake aftershock or the increased amount of surface area exposed to hurricane force wind shear. Through effective anchoring of emergency shelters and designs which minimize wind shear, hurricane force winds can be accounted for in emergency shelter design systems. An example of such testing is shown in Figure 2-15 (Bradford, 2004):

![Figure 2-15: example of wind testing diagnostic (Bradford, 2004).](image)
In terms of protection from earthquake aftershocks, a strong anchoring system and reinforced joints, combined with a relatively lightweight building material, can ensure that building collapse does not occur.

2.4.8 How the panels are connected

There are examples of rhombus shaped triacontahedron assemblies in which panels are connected along the edges (Lipson, 2001). These hinges can swing, slide, or snap into place to create structures which are strong and easily connected. These structures have been used for shelters, as temporary housing, and for storage containers as well. When walls are vertical, structures can be nested to be used with doors, windows, and other fixtures. Hinges can be made from plastic that is cast or from formed steel as well. The CFRP can also be specialized to be used in a variety of different laminate panels. Essentially, there are ten wall panels and ten nearly similar roof panels joined together by thirty-five, 144-degree edge connections as can been seen in Figure 2-16. The plastic edge hinges are easy to make, and are, therefore, not all that expensive. This system, through its light weight panels, increases efficiency, decreases cost, and makes the assembly and disassembly simple (Lipson, 2001).

![Figure 2-16: Roof Edge Connectors (Lipson, 2001).](image)
2.4.9 Emergency Shelter Design and Manufacture

There are many ways in which FRP emergency shelters can be designed and manufactured. Aside from prefabricated kits designed and molded remotely, innovative new solutions for custom design kits, cut by mobile factories transported in shipping containers, can also be used, as shown in the Figure 2-17 (Global Mobile Factory, 2014):

![Figure 2-17: Global Mobile Factory (Global Mobile Factory, 2014)](image)

The global mobile factory places hardware and building supplies in the location where they are needed. Instead of shipping many containers or airlifting emergency shelter kits, the global mobile factory brings the means of production to the site where they are needed. This innovation is economically wise, as the costs of maintaining fixed buildings, transporting shelters, and logistical demands of many disaster prone areas are eased through the global mobile factory setup. This system allows for adaptable design planning and the fabrication of parts or pieces that may be missing. Oftentimes, with other structures
prefabricated remotely, parts go missing. With the global mobile factory, parts can be
produced if missing (Global Mobile Factory, 2014).

There are many different designers of FRP emergency shelters, ranging from highly
complex to relatively simple. Of the most complex, the Diawa Lease design ranks high in
terms of cost and efficacy. Around the size of a shipping container as can be seen in Figure
2-18, the design can extend through automated control to twice its size through a switch
(Loomans, 2013). Equipped with solar panels and a filter for water, the shelter can survive
without outside support. Beds and an office are built into the design. There is a shower,
toilet, supplies, and equipment.

Figure 2-18: Diawa Lease Design (Loomans, 2013)

The most important aspect for many designers regarding emergency shelter systems is
their portability, ease of construction, and overall cost. The differences between the various
manufacturers are many, but the overall concerns and bottom line are the same.
2.5 Background of FRP modular construction and emergency shelters

2.5.1 Background

It is a given that buildings are the cornerstone of society. Many of these structures, or buildings, have historically been made from brick, wood, or other building supplies. Fiber reinforced polymer (FRP) composites were initially used for small parts of larger structures such as windows, canopies, doors, profiles, and other lesser features. However, these materials are not used to construct entire buildings. Modular FRP houses are easy to maintain and do not deteriorate over time due to weathering or other environmental effects (Bradford, 2004). Furthermore, these materials ensure lower heating and cooling costs due to their inherent insulating properties. FRP panels retain the following: 1) high structural strength (the curved surface of the panels reduce wind resistance, enabling the house to withstand hurricanes); 2) quick construction (modular design for ease of erection); 3) earthquake resistance (the FRP panels flex instead of breaking); 4) water resistance (completely sealed from the ground up); and 5) portability (a FRP house can be easily disassembled and relocated to a new site). These buildings suit as disaster-resistant shelters, military barracks, school buildings, industrial factories and warehouse buildings, large dormitory settings for workers in remote locations, greenhouses, etc.

Every year, extreme windstorms such as tornadoes and hurricanes have imposed disastrous consequences to the United States community, economy, and population. Having a community shelter can protect civilians from these dangerous events. This study focused on providing detailed design guidance in addition to demonstrating the design and analysis of a safe, on-ground, stand-alone community shelter to resist extreme wind forces. The storm shelter must be able to resist 300 miles per hour wind gusts and flying debris.
impacts of such storms. The primary objective was to develop an innovative and cost-effective Fiber Reinforced Polymer (FRP) panelized construction system. The technique allows the shelter to be sub-divided into basic elements that can be prefabricated and shipped to the construction site, where they can be assembled into the finished structure. The design must meet the requirements recommended by the Federal Emergency Management Agency (FEMA) with life safety as the primary consideration. The wind pressures design for the community shelter was calculated per the American Society of Civil Engineering (ASCE) 7-05 code provision and modeled in the Computational Fluid Dynamic (CFD) software named HYBRID 3D. Comparison was made between the ASCE 7-05 and CFD wind pressures to validate the accuracy of the ASCE 7-05 design methodology. Two different FRP materials were proposed for the design and analysis of the FRP panels. The design of FRP panels was based on the performance and minimum wind load design criteria of the ASCE 7-05. The finite element analysis (FEA) software package called ANSYS 11.0 was used to carry out the design of the FRP panels under wind pressure loading. In addition, the FEA was conducted on the bolted joint connections of the shelter. Further analysis was carried out on the bolted joint connections per the Load Resistance Factor Design (LRFD) code to determine if the bolts had sufficient strength to resist applied external loadings (Bradford, 2004).

The design of shelter structures has received little attention from the engineering community since the days of nuclear fallout shelters; this attention increased once guidance for community shelters for cases of extreme wind events was released by FEMA in July 2000 (FEMA 361). To respond to the recent demand for community shelters, many states are designating existing schools or other public buildings, such as community centers or
multipurpose buildings, as public shelter areas. In most cases, these buildings, or portions of these buildings, were never designed for use as shelters. Most of the designated shelters were designed and constructed according to older local building codes that do not include requirements for extreme wind pressures and uplift. Even recently designed structures have been found to have inadequate features for a high-wind shelter, particularly with respect to cladding and architectural features that are vulnerable to damage from high winds and windborne debris. Damage to the cladding is often the beginning of building failure and occupant injury during an extreme wind event. This paper identifies critical issues and gaps in present available technology for evaluating proposed shelters and providing retrofit guidance for building owners. The writer's' experience with inspections of designated shelters, proposed retrofit recommendations, and damage investigations of buildings affected by hurricanes or tornadoes is summarized. Recommendations for design considerations that include current standards of practice as outlined in FEMA 361, ASCE 7-98, and the Florida Shelter Evaluation Guidelines are given (Bradford, 2004).

2.5.2 FRP Materials for Increased Strength and Durability

Given that hurricane and tornado winds are not the major concern in the case of emergency shelter, it's the wind-borne debris that poses a greater hazard. For this reason, fiberglass reinforced polymer as a material would be too brittle. To withstand storm debris, there is need for something more elastic, in the material sense (Qureshi, 1988). For these conditions, a material is needed that has a high tensile strength, but also can withstand impact load. For this purpose, Carbon reinforced polymer could serve as a carbon fiber mesh overlay on a light weight steel/aluminum structure fiber mesh. Carbon fibers have a
very high tensile strength and elastic modulus (see Table 2.1). Carbon fiber also has an elastic modulus almost the same as steel. Using these high modulus fibers in the aerospace industry has become commonplace, due to the weight to strength ratio. These normal modulus carbon fibers are used with CFRP in infrastructure (Qureshi, 1988).

Advanced composites are high strength, high modulus materials are finding increasing use as structural components in aircraft, automotive, and sporting goods applications. Typically, they comprise structural fibers such as carbon fibers, in the form of woven cloth or continuous filaments embedded in a thermosetting resin matrix.

Most advanced composites are fabricated from “prepreg", a ready-to-mold sheet of reinforcement impregnated with uncured or partially cured resin. Resin systems containing an epoxide resin and aromatic amine hardener are often used in prepreg since they possess the balance of properties required for this composite fabrication process. State-of-the-art epoxy/carbon fiber composites have high compressive strengths, good fatigue characteristics, and low shrinkage during cure (Qureshi, 1988).

In this sense, it would act somewhat like a "net" for debris that could puncture the skin, but the underlayment would give structural stability and allow for higher overall strength. Metal skins are easy to pierce with point loads, such as debris or missiles, found in hurricane and tornado winds. However, a blanket of something on both sides might help. To create the most supportive structure, a lattice like structure on the inside of the panels would brace the skin from all angles (Qureshi, 1988). The mechanical properties of Carbon Fiber Reinforced Polymer are listed in Table 2.1.
Table 2.1: Carbon Fiber Reinforced Polymer properties (ACI 440.2R-08 Code)

<table>
<thead>
<tr>
<th>Young's modulus</th>
<th>At Ultimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{frp}$ (MPa)</td>
<td>$\varepsilon_{fu}$</td>
</tr>
<tr>
<td>165,000</td>
<td>0.0170</td>
</tr>
</tbody>
</table>

CFRP are made of two different parts: reinforcement and the matrix. The CFRP is given strength by the carbon fiber; the matrix is often of an epoxy which reinforces the resin. Most properties are dependent on the two elements.

Stress and elastic modulus provide strength and rigidity. Some materials such as steel and aluminum, which are isotropic like carbon fiber reinforced polymer, have directional strength properties. Depending on layouts of fibers, properties of the CFRP differ in relation to the polymer. The equations of the net elastic modulus of composite materials employing the dimensions of carbon fibers and polymer matrix can be seen in the following equation:

$$E_c = V_m E_m + V_f E_f$$  \hspace{1cm} \text{Eq. 2.1}

This equation works for composite materials with fibers parallel to the direction of the load. This composite modulus is denoted as $E_c$ while the volume fractions of the matrix are $V_m$ and $V_f$, and $E_m$ and $E_f$ are the denoted elastic modules of the matrix and fibers. In order to determine the elastic modulus for the load of the carbon fibers transverse to the applied load, Equation 2.2 is used (Boboulos, 2010).

$$E_c = \frac{V_m}{E_m} + \frac{V_f}{E_f}$$  \hspace{1cm} \text{Eq. 2.2}

Fracture toughness of the CFRP is dictated by the mechanism of debonding between the polymer matrix and fibers and when fibers pull apart as the sheets move. When the CFRP is held together by epoxy, there is little plasticity, with 0.5 % strain to failure.
Because of this brittleness, a high failure rate occurs. There have been developments seeking to toughen CFRPs with new epoxy and alternative polymer matrices. One material, known as PEEK, is tougher with a similar strength and elastic modulus. The downside to PEEK is its cost and difficulty in production.

Regardless of the high strength to weight ratio, CFRP lacks a definable fatigue endurance limit. This means that stress cycle failure is a possibility. Though steel, and other structural metals have fatigue endurance limits that can be estimated, failure models of composites are difficult to design. Due to this, critical cyclic load applications need to be designed with strength safety margins to give better reliability of components (Boboulos, 2010).

Temperature and humidity can affect polymer based composites such as CFRPs. Although these materials do resist corrosion very well, moisture and wide ranging temperatures can degrade the mechanical properties of CFRPs where the matrix and fibers meet. The fibers themselves are not bothered by the moisture, but the moisture can plasticize the polymer (Boboulos, 2010).

2.5.3 Ballistic testing of CFRP

Windborne missiles can cause severe damage to structures. If wind speeds are high, a missile can have enough force to go through walls, a roof, and, surely, a window. Both glass and epoxy systems of graphite, which are laminated with polycarbonate laminate, have undergone ballistic testing. Magnetic sensors were used to measure the velocity of the projectile at striking the surface and at exiting. The limit of ballistics for these tests and the absorbed energy were calculated with the equation for the conservation of momentum.
The two components in the composite provided ideals for protection against missiles in the shelter (Uddin & Vaidya, 2005).

A missile in hurricane force winds can certainly penetrate a conventional house. A masonry wall that is reinforced can also be penetrated unless specifically designed to withstand such conditions. Due to the fact that debris can go through roofs and walls, structures as well as people inside of those structures are threatened by debris. Issues related to emergency shelters involve how impact properties are designed. FRP sandwich panels can be used for cladding or load bearing applications (Schafer, Destefanis, Riedel & Lambert, 2005).

Projectiles, such as tools at low velocities and blast fragments at high velocity, can indent and deform sandwich panels. FRP-epoxy panels have different reactions to projectiles than graphite-epoxy panels (Uddin & Vaidya, 2005).

### 2.5.4 Expanded Polystyrene Foam (EPS)

EPS foam is an affordable, lightweight, and fire resistant material. Due to this, it is an effective insulator as well. The R-value of EPS will not go down with age. This material can withstand extreme cold without losing its power of insulation or integrity of structure. As an inert material, it is not degradable in the sense that organic material may be. It is closed cellular plastic resin developed from crude oil. The blowing agent of pentane is used for introducing the resin into the mold (Horvath, 1994). EPS foam has a strength for compression of 10 to 60 psf. This makes it ideal for many construction applications. In the early 1950s, builders began to understand the role that these types of materials could play with their insulation and strong properties (Horvath, 1994). Soon, these thermoplastics
became the status quo in the building industry. Given the high R value of EPS compared to the amount of money it costs, it is a value. The physical and mechanical properties of EPS foam being used in this research are listed in Table 2.2, as provided from the thesis of (Nguyen, 2009). These mechanical properties are based on R CONTROL manufacturer.

Table 2.2: Physical and Mechanical Properties of EPS foam (Nguyen, 2009).

<table>
<thead>
<tr>
<th>Property</th>
<th>US Units (Psi)</th>
<th>SI Units (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1.0 pcf</td>
<td>1.6E7 mg/m³</td>
</tr>
<tr>
<td>Young’s Modulus</td>
<td>180 - 260</td>
<td>1.2 – 1.5</td>
</tr>
<tr>
<td>Flexural Modulus</td>
<td>25 - 30</td>
<td>0.1 – 0.2</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>280 - 320</td>
<td>1.9 – 2.2</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>18 - 22</td>
<td>0.1 – 0.15</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

2.5.5 Previous Finite Element Models of Emergency Shelters

One such shelter, a fixed shelter made of reinforced concrete, was analyzed through Finite Element modeling and subjected to an extreme wind event in which the winds speeds were recorded at 112 meters per second (250 mph). The wind pressures on the shelters were obtained using the provisions of ASCE 7-98 ASCE 1998 (Budek, Zain, Qiao, & Phelan, 2006). The structure had to survive the testing - this analysis of the structure took into account how everything performed in the building, from the foundation to the roof, to the doors and to the wall. After this, the shelter could not turn over. In remaining right side up and not turning over, the structure signals integrity in the foundational connection needed. After this, the debris impact that may have affected the walls and the doors had to be accounted for.
Table 2.3: Summarized Wind Loads for Internal Pressure Coefficients of ±0.55  
(Budek et al., 2006)

<table>
<thead>
<tr>
<th>Velocity (mph)</th>
<th>Windward wall net pressure (psf)</th>
<th>Leeward wall net pressure (psf)</th>
<th>Side wall net pressure (psf)</th>
<th>Front roof net pressure (psf)</th>
<th>Back roof net pressure (psf)</th>
</tr>
</thead>
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<td>-521.57/-36.87</td>
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</table>
Figures 2-19(a) and 2-19(b) show a representative set of loads applied on the shelter model. Table 2.4 shows the load cases that were applied to each model presented in this study. The loads associated with different wind speeds are determined by *Method 2-Analytical procedure* presented in ASCE 7-02 ASCE 2002, with coefficients and factors specified by FEMA 361. Linear stress analysis was used. Element specifications were defined. The mass density and the modulus of elasticity of the elements were required to perform this type of analysis (Budek et al., 2006).

Several panel specimens were tested to determine the stress in the orthotropic directions (the strong direction parallel to the studs/joists, and the weak direction normal to them). As can be seen in the Figures 2-19(a) and 2-19(b), regarding the walls, the modulus of elasticity used in the strong direction was 48,000 Mpa (7,009,000 psi). In the weak direction, the modulus of elasticity was used 5020 Mpa and (727,800 psi). Regarding the roof, the modulus of elasticity was used 137,000 Mpa and (19,830,000 psi) in the strong direction and 6400 Mpa and (927,800psi) in the weaker direction (Budek et al., 2006).
The results from these analyses show that the FA method can be successfully used to model extreme wind pressures on a storm shelter.

Figure 2-20(a) and 2-20(b): Displaced Shape of the shelter model (20.a), stress in Z direction in the walls (20.b) (Budek et al., 2006).

2.6 Collective Conclusions and Emergent Concepts

Emergency shelters need to be easy to assemble. If a trained team of laborers is needed to make these structures, there will be little practical application in a large scale disaster recovery situation.

If the shelters are prefabricated and shipped via containers, the size and weight of the materials is important. Due to the distance between manufacturers and recovery sites, this is an important factor in the process. Emergency shelters must be constructed with high strength, lightweight, inexpensive material.

Emergency shelters need to incorporate a culturally appropriate appearance of what a house looks like. In many contexts, if the shelter does not resemble what they consider to be a livable house, people may not use it in such a way.
2.7 Original Contribution and Significance of Project

In this project, a FEM analysis of an emergency shelter (built with CFRP) will be performed. The material used in the analyses of the emergency shelter will be fiber reinforced polymer, a strong and lightweight material which can be prefabricated into panels which operate by hinges to easily assemble and reinforce the design. This innovation, a hinged, prefabricated assembly system made from CFRP panels, is the central strength of the newly improved design.
Chapter III

Methodology

3.1 Problem Statement

Given the large number of fiber composite designs used in emergency shelters, there are numerous areas in which designs can be improved. These problems, related to the difficulty with construction techniques, the time involved in transport, the stability of anchor systems, and aesthetic shortcomings of emergency shelter drives the underlying research question for this proposal: How can emergency shelters which use carbon fiber composite materials be improved in terms of safety, portability, ease of construction, and cost?

The CFRP emergency shelter system will be anchored to the ground with steel cables employing an arrowhead earth anchor. This anchor is operationalized when a load is applied to the retriever cable. In terms of six supporting columns underneath the FRP emergency shelter, these will not be fixed to an anchor, though do provide structural support. These columns are not fixed due to the design’s portable and reusable nature. This is why this research will deal specifically with how the cables which reinforce the CFRP emergency shelter system fare under hurricane wind force during a finite element modeling simulation.
3.2 Design process

3.2.1 Improved Portability

The issue of portability seemed to be something which affected most of the designs, as the bulky dimensions, weight, and reliance on cargo containers and military planes hampered where these emergency shelters could be erected and the means by which emergency shelters arrived at the disaster site. The first innovation for the design demonstrates how retractable wheels make the emergency shelter portable. In shipping, via cargo container, wheels are retracted and the design resembles a long rectangular box. When emergency shelters arrive at the disaster site, wheels can be deployed and the unit made transportable to ease in reaching difficult to access locations. This aspect of the improved emergency shelter system relates also to sustainability; the mobile aspects make it easier to reuse.

3.2.2 Improved Safety

The second original aspect of the proposed design relates to safety. The improved emergency shelter will have increased support from both load bearing columns and steel cable reinforcement attached to stakes to protect against high winds. Furthermore, there will be extra space underneath the design to allow for flooding or high water levels.

3.2.3 Improved ease of Construction

The most innovative aspect of the design relates to the ease of construction. On shipment the design resembles a long rectangular box. This rectangular shape measures 7.5 ft. x 7.5 ft. 18 ft. to fit easily into a cargo container.
Upon arrival at the site, the wheels descend for improved portability as shown in Figure 3-1. Next, Part 1 is laid down horizontally to create more space. After this, Part 2 is lifted vertically to create the first wall. Following this, part 3 extends out from part two, lifted upwards to form what will be the slanted roof. Next, part 4 is laid down horizontally, as part 5 forms the supporting wall for the roof. After this, parts 6 and 7 rise vertically to form the other two supporting walls:

![Figure 3-1: Drawing of how to improve ease of construction](image)

Finally, six supporting columns, which are made of the composite materials and wheels in the center anchor the shelter. Embedded within the walls are aspects related to basic survival needs, such as a simple stove and sink, a shower, a bed/table, and windows which can open and close. Furthermore, the electricity and the water and plumbing systems will also be embedded into the walls. A facade view of the erected emergency shelter showing the door, stairs, and windows.
The various emergency structures currently in use are secured to the ground through a number of means as shown in Figure 3-2. Some methods involve submerging the base of the panel into a concrete footing, or equally stable footing, during drying of the concrete. After the concrete has dried, a way to uniformly connect between the shelter and the foundation has been created. In most cases of emergency shelters, there will not be an opportunity to mix concrete, and the natural ground will have to suffice in connecting the structure to a natural foundation. If the emergency shelter needs to have a clean floor, as in the case of a hospital, the side panels can be used as a rudimentary floor (Bradford, 2004).
3.2.4 The Finite Element Method of Analysis

There are only two different types of structural analysis: numerical and analytical. The difference between these two methods are many, but can be summarized as follows: Analytical methods make available closed-form solutions for nuanced designs. Therefore, it is difficult to use such an analysis on a complicated design or structure. Within numerical analysis, there are also two sub-categories for analysis. Numerical solutions can involve either answers to diminished differential equations or Finite Element Methods (FEM). Regarding diminished differential equations, approximated solutions related to elasticity calculations limit the use of the approach, limiting its use to basic structures. On the other hand, FEMs can account for the intricacies of highly complicated structures. In short, FEMs are dynamic and allow a broader scope of possibilities for structural analysis (Rust & Schweizerhof, 2003).

FEMs use computerized models to predict how structures will react to catastrophic possibilities in the real world. It is this possibility which makes finite element analysis necessary for the design of FRP emergency shelters. This analysis is a software based model for seeing how different potential natural forces, such as seismic activity, fire, wind shear, or tsunami, will affect proposed structures. The software is advanced to the point that it can predict the time frame of how certain materials will break down or perform. The analysis is, in fact, a prediction of how the design or structure will perform. This prediction is possible through myriad calculations which de-compartmentalize the entire structure into thousands upon thousands (and potentially millions) of moving parts, each with their own properties. How each of these properties behaves is determined by mathematical laws and theorems -- this is the basis of finite element methodology. It is breaking down all of the
various parts of a structure to basic elements which can be calculated in concert with each other. The growth in computing power over the past 30 years has led to major breakthroughs in the development of finite element modeling and analysis. When it comes to seismic activity and the behavior of built structures, Finite Element analysis is now a necessary part of any rigorous testing process.

There are numerous software programs which aid in the research of the FEM or analysis, such as ALGOR and ANSYS (FEMPRO, 2002; Rust, 2003). Programs such as these are designed to bring together complex machinations of the tremendous calculating power of the FEM with real world applications for determining locations of structures, steady and time-dependent load distributions, and other information related to solutions that might be sought. For this research, ANSYS software was used.

3.2.5 ANSYS for Finite Element Modeling

As a software provider of engineering simulation systems, ANSYS offers a finite element modeling software package for static and dynamic analysis of structures. Some of the special features of this program are its corner nodes and mid-side nodes for shell element. Mindlin kinematics inform the bending theory which is implemented in the modeling. The low order Mindlin elements, and the formulation for shear strains, are improved by the assumed strain formulation. Following this, the plane stiffness can be reduced for integration, and problems with rotation and other big stresses can be accounted for. Regarding the elements related to the shell, material models like hyperelastic, plastic creep behavior can be employed (Rust, 2003).
When the user of the program denotes two contact surfaces, the contact analysis is put into rotation. How thick the shell is, depending on the transverse contraction, is always a part of the calculation. The methods of Lagrange and Penalty can be selected to make contact conditions (Rust & Schweizerhof, 2003). If discretization is handled in an imprecise manner, the non-modeled interferences, or gaps, can be accounted for by the parameters or the options. The bonded contact is employed in order to link surfaces and meshes (Rust & Schweizerhof, 2003).

Nonlinear equation solutions in ANSYS are found with the Newton-Raphson method. This uses direct and iterative solutions sequencing the equations of linear systems (Rust, 2003). Finally, the command language used in the software, also known as parametric design language, has many common links to other higher programmatic coding schemas. For what is being considered for analysis in this paper, the FRP emergency system, this coding relationship can be employed to put the data and coding into other formats for further analysis and graphical display (Rust, 2003).

3.2.6 Soft Walled Emergency Shelter Shortcomings

The need for Carbon Fiber Reinforced Polymer (CFRP) emergency shelters is apparent from the literature, as the most common form of structural failure has been linked to projectiles being launched at high speeds during hurricanes, causing catastrophic structural damage (Bradford & Sen, 2005).

Emergency shelters made from inflatable plastic or tents do not stand up to the types of debris which cause significant damage. These shelters, of the soft variety, can be compromised with very little force, sometimes even tearing due to simple tree branches.
These structures cannot withstand significant wind forces or any other force for that matter (especially extreme heat or cold) (Owens, 2005).

However, on the opposite side of that argument, soft walled structures are light to transport, whereas metal frames are heavier and more difficult to move (Waugh, 2006). Even the most basic steel structures are difficult to transport because of their weight, and one truck is usually required for transporting each structure to the emergency location. In most regions, mobile homes are not considered temporary structures, and thus require permits and land dedicated to “trailer parks.” FEMA spent over $1 billion to purchase mobile homes in the wake of Hurricane Katrina, but because of logistical difficulties of establishing mobile homes, only 105 families could be placed in that type of housing (DuraKit 2005). No type of steel structure has the combination of economic viability and transportability necessary to meet housing needs in emergency situations within the United States, let alone in more remote locations around the world (Waugh, 2006).

3.2.7 Finite Element Modeling for FRP Emergency Shelter System

Table 3.1 shows the proposed solution methodology for testing the steel pinned cables for the fiber reinforced polymer emergency shelter system involve first developing the design to be analyzed. After this design has been formulated, the process for inputting the design data into the finite element modeling software will begin. From this point, the analysis of the design through the software will initiate, followed by the output report. After analyzing the output data for weaknesses or failures, the design will be remodeled for future testing.
Table 3.1: Proposed Solution Methodology

The mockup in Figure 3-3 shows the proposed aspects of the design to be tested under finite element modeling. As can be seen, steel cables which extend from the corners of the design to the ground are proposed to provide a solution for the potential shear forces anticipated as part of a hurricane.

Figure 3.3 - Proposed aspects of the design
3.2.8 Anchor System

As can be seen below, the anchor system for the improved FRP emergency shelter system is explained. It is a system of support for potential racking or shifting due to shear stress. As can be seen in Figure 3-4, the anchors are secured to the ground though being hammered into the ground.

The anchor will utilize an arrowhead earth anchor to reduce risk of movement, and to increase the potential levels of strength. As can be seen in Figure 3-4, the anchor unfolds for a toe line to spring the guard into action when load is applied.

Figure 3-4: Arrowhead anchoring system from MILSPEC ANCHORS

3.2.9 Hypothesis

Given the relative lack of research in the area, the expected results are that the structure will experience similar shear forces as demonstrated in the FEM analysis above (Budek et al., 2006). The shelter evaluated in this thesis is built of different material than the Budek shelter (FRP versus steel and plywood). The shelter evaluated has a larger space beneath the structure compared to the Bedeck shelter. And this shelter is not fixed to the foundation; thus, anchoring cables need to be evaluated.
3.2.10 Potential Benefits

Hurricanes are forces which affect millions of people each and every year. For the most unfortunate, this can mean the very shelter in which they live. For others, subjected to different natural disasters, the results can be similar, in the destruction of their living space. Through advances in lightweight materials and advanced simulations for modeling extreme conditions, it is now possible to test design systems before governments or international aid organizations invest large sums of money to protect those who have lost everything. The proposed system is mobile, lightweight, and reusable. For further research into the design, it is necessary to simulate the types of conditions that this emergency shelter may face during hurricane conditions through ANSYS simulation.

3.3 Objectives of the Methodology

The development of the improved FRP emergency shelter outlined in this paper is a conceptual design directed at addressing the weaknesses of current designs. There are several objectives of this first conceptualization of the design. First, the building should use an FRP panel system that is automated to make the ease of construction complete. Second, the unit should be mobile, equipped with wheels in some manner. Third, the design must utilize interlocking of different members to reinforce the design. The FRP composition of the panels will allow for a unique combination of strength and diversity of design.

3.4 Recommendations for Future Work

Future research should explore how emergency shelter systems can become more responsive to the context of the disaster in terms of community needs and traditions. This
could mean more mobile emergency shelter units, or could investigate future analysis of community needs before certain emergency shelters are deployed through surveying and collecting data in earthquake and hurricane prone areas of the world.

Other research could be directed towards investigating more simple methods of construction, such as proposed by the improved design described in this paper. An automated construction, which does not require a team of laborers or days to finish, is an ideal that emergency shelter systems should strive for.
Chapter IV

Building Geometry and Wind Load Design

4.1 Introduction

Using guidelines for the “Minimum Design Loads for Buildings and Other Structures” (ASCE 7-10), the following wind pressure parameters were used.

4.2 Description and size of the building

The footprint of the building was set at 7.5' ft. x 18 ft. x 9.5ft after a first round of structural analysis. This was determined to be optimal for both stresses and member forces. The structure height is 9 feet (7.5 ft. wall, 1.5 ft. column). These Figures were chosen for the single story shelter due to standard construction wall heights. The four walls were equal in height, leading to a flat roof to be used on the shelter. In Figure 4-1, the proposed design and dimensions are described.
If we assume that the risk category IV and the terrain exposure C were applied to the emergency shelter, the wind speed for Charleston, South Carolina was determined to be 157 mph (253 km/hr.) by utilizing the Applied Technology Council (ATC), “Windspeed by Location” calculator (see Figure 4-2). This speed correlates to a category IV hurricane. In viewing the emergency shelter as “primary,” it is classified as category IV, (ASCE 7-10). The ATC calculator utilizes the Basic Wind Speeds Contour Map found in ASCE 7-10, Figure 25.6-1A (see Figure 4-3).
Figure 4-2: Wind Speed Location (ATC)
Figure 4-3: Wind Velocity Hazard Contour Map from ASCE 7-10
4.3 Wind Load Calculation

By calculating the base wind pressure, the first step is taken in evaluating the overall design wind pressure. This base wind pressure depends upon building importance, exposure, wind velocity, and the topography around the structure. To calculate the base wind pressure, the equation below is used (ASCE 7-10):

(1). **Velocity pressure equation:** The equation provided by ASCE 7-10 for determining the velocity pressure is the following

\[ q = 0.00256 \cdot (K_z) \cdot (K_{zt}) \cdot (K_d) \cdot (V^2) \text{ lb/ft}^2 \]  
(Equation 4.1)

- Wind directionality factor: \( K_d = 0.85 \)  
  (Table 26. 6-1)

- Assume Exposure Category C: open terrain with obstructions < 30 ft. in height, \( K_z = 1.0 \)  
  (ASCE 7-10)

\( K_z \) = velocity pressure coefficient, at height \( z \)

\( K_h \) = velocity pressure coefficient, at \( z = h \) = mean roof height

- Assume Topographic factor:

\[ K_{zt} = 1.0 \text{ no hills, ridges, nor escarpments} \]  
(ASCE 7-10, Figure 26.8-1)

\[ q = 0.00256 \cdot (K_z) \cdot (1) \cdot (0.85) \cdot (157^2) \text{ lb/ft}^2 = 53.6(K_z) \text{ psf} \]

- Velocity Pressure Exposure Coefficient: for Exposure Category C: (ASCE 7-10, Table, 27.3-1)

\( K_z \) or \( K_h = 0.85 \) for \( z = 0 \rightarrow 15 \text{ft} \)

\[ q_{z=0→9\text{ft}} = 53.6(0.85) = 45.6 \text{ psf} \]
On Leeward and Side walls and Roof, pressure is constant from ground to top

\[ q_{LW} = q_{SW} = q_{Roof} = q_{z=0-9\text{ft}} = 45.6 \text{ psf} \]

The corresponding wind pressures were calculated for each of the leeward and windward surfaces. For the overall testing of the design, the worst-case scenario wind pressures were calculated for all leeward and windward points. The applied wind was also split into the perpendicular and parallel direction in relation to the roof. Factors related to specific gusts were considered in relation to Charleston, South Carolina for all structural members.

The final design pressure for each member was determined by taking into account the local gusts and the interior pressure variables in relation to the base design wind pressures. The equations below demonstrates the Main Wind Force Resisting Systems and Components and Cladding systems, respectively (MWFRS and CC) (ASCE 7-10).

The main wind force resisting system is made of every structural component that helps in the transfer of force from the wind to the foundation system. This means any columns, beams, or connecting components within the design. To assess the cladding and component
forces, the separate members are analyzed in light of the wind loads that are directly applied.

(2). Design wind pressure for Main Wind Force Resisting System, MWFRS:

\[ P = qG_{C_p} - q_i(G_{C_p_i}) \quad \text{lb/ft}^2 \]  
(Equation 4.2)

- Gust Effect factor: \( G = 0.85 \)  
  (ASCE 7-10, 26.9.1)

- For Windward wall: \( C_p = 0.8 \) use with constant \( q = q_{z=0\rightarrow9ft} \)

Since the wind could come from any direction, either wall could become the leeward wall. Thus, the controlling \( C_p \) for the leeward wall will be the largest value obtained from the calculation of the two possible aspect ratios of the building; \( \frac{L}{B} = \frac{18\text{ft}}{7.5\text{ft}} = 2.4 \) and \( \frac{L}{B} = \frac{7.5\text{ft}}{18\text{ft}} = 0.4167 \)

- For Leeward wall: \( \frac{L}{B} = \frac{18\text{ft}}{7.5\text{ft}} = 2.4 \), for this building \( C_p = -0.3 \)

- For Leeward wall: \( \frac{L}{B} = \frac{7.5\text{ft}}{18\text{ft}} = 0.4167 \), for this building \( C_p = -0.5 \)

Consequently, \( C_p = -0.5 \) controls

- For Side walls: \( C_p = -0.7 \), use with constant \( q = q_h \)

Table 4.1: Figure 27.4-1, ASCE 7-10

<table>
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<th>L/B</th>
<th>( C_p )</th>
<th>Use with</th>
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<td>( q_z ) varies with height above 15 ft</td>
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<td>( q_h ) mean roof height</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-0.3</td>
<td>( q_h ) mean roof height</td>
</tr>
<tr>
<td></td>
<td>( \geq 4 )</td>
<td>-0.2</td>
<td>( q_h ) mean roof height</td>
</tr>
<tr>
<td>Side wall</td>
<td>All values</td>
<td>-0.7</td>
<td>( q_h ) mean roof height</td>
</tr>
</tbody>
</table>

- Positive: acting toward surface
- Negative: acting away from surface
Table 4.2: Table 26.11-1, ASCE 7-10

<table>
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<th>Enclosure Classification</th>
<th>GC_{pi}</th>
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<tr>
<td>Open Building</td>
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</tr>
<tr>
<td>Partially Enclosure Building</td>
<td>±0.55</td>
</tr>
<tr>
<td>Enclosure Building</td>
<td>±0.18</td>
</tr>
</tbody>
</table>

- For enclosed buildings: GC_{pi} = ±0.18

![Figure 4-4: Direction of applied force, (Pickett, fall 2015)]

For Windward Wall: \( p = (45.6 \text{ psf})(0.85)(0.8) - (45.6 \text{ psf})(-0.18) = 39.2 \text{ psf} \)

For Leeward Wall: \( p = (45.6 \text{ psf})(0.85)(-0.5) - (45.6 \text{ psf})(+0.18) = -27.6 \text{ psf} \)
For Windward Wall:

\[ R(7.5) = (39.2 \text{ psf})(7.5) \left( \frac{7.5}{2} \right) = 1102.5 \text{ psf} \]

\[ R = \frac{1102.5}{7.5} = 147 \frac{\text{lb}}{\text{ft}} \]

\[ V_{\text{windward wall}} = \left( 147 \frac{\text{lb}}{\text{ft}} \right) \times (18 \text{ ft}) = 2646 \frac{\text{lb}}{\text{ft}} \]

For Leeward wall:

\[ R(9) = (-27.6 \text{ psf})(7.5) \left( \frac{7.5}{2} \right) = -776.25 \text{ psf} \]

\[ R = \frac{-776.25}{7.5} = -103.5 \frac{\text{lb}}{\text{ft}} \]

\[ V_{\text{Leeward wall}} = \left( -103.5 \frac{\text{lb}}{\text{ft}} \right) \times (18 \text{ ft}) = -1836 \frac{\text{lb}}{\text{ft}} \]

\[ V_{\text{Total}} = \left( 2646 \frac{\text{lb}}{\text{ft}} \right) - \left( -1836 \frac{\text{lb}}{\text{ft}} \right) = 4509 \frac{\text{lb}}{\text{ft}} \]

(3). Design WALL ELEMENTS using Components and Cladding provisions:

- **Effective Wind Area**, \( A = (\text{span length})(\text{effective width} \geq 1/3 \text{ of span}) \)

\[ A = (7.5) \left( \frac{7.5}{3} \right) = 18.75 \text{ ft}^2 \]

- **Obtain External Pressure coefficients**

\[ P = q_h \left[ (G_{C_p}) - (G_{C_{pl}}) \right] \]  

\hspace{10cm} \text{(ASCE 7-10, eqn. 30.4-1)}
- Velocity Pressure Exposure Coefficient: for Exposure Category C: (ASCE 7-10, Table, 30.3-1)

\[ K_z \text{ or } K_h = 0.85 \text{ for } z = 0 \rightarrow 15\text{ft} \]

\[ q_h = 45.6 \text{ psf} \quad P = 45.6[(GC_p) - (GC_{pi})] \]

For Windward Wall:

\[ GC_p = +0.9 \text{ inward on Windward wall, Zone } = 5 \]

\[ GC_{pi} = \pm 0.18 \text{ for Internal Pressure Coefficient} \]

\[ P = 45.6[(GC_p) - (GC_{pi})] \]

\[ P = 45.6[(+0.9) - (-0.18)] \]

\[ P = +41.04 - (-8.208) = 49 \text{ psf} \]

For Leeward wall:

\[ GC_p = -1.8 \text{ inward on Leeward wall, Zone } = 5 \]

\[ GC_{pi} = \pm 0.18 \text{ for Internal Pressure Coefficient} \]

\[ P = 45.6[(GC_p) - (GC_{pi})] \]

\[ P = 45.6[(-1.8) - (+0.18)] \]

\[ P = -82.08 - (+8.208) = -90.3 \text{ psf} \]
The design Wall Elements either for:

\[ P = 49 \text{ psf acting inward or } p = 90.3 \text{ psf acting outward} \]

\[ P = -90.3 \text{ psf} \quad \text{control} \]

- **Design ROOF using Components and Cladding provisions:**
  - Effective Wind Area,
    
    \[ A = (\text{span length})(\text{effective width} \geq 1/3 \text{ of span}) \]
    
    \[ A = (18 \text{ ft}) \left( \frac{18 \text{ ft}}{3} \right) = 108 \text{ ft}^2 \]
    \[ A = (7.5 \text{ ft}) \left( \frac{7.5 \text{ ft}}{3} \right) = 18.75 \text{ ft}^2 \]
- External pressure coefficients, for Zone 1 Roof area = 18.75 ft\(^2\)

\[
G_{C_p} = -1.3 \quad \text{For Roof Zone (1)}
\]

\[
G_{C_{pi}} = \pm 0.18 \quad \text{For internal Pressure Coefficient}
\]

\[
P = q \left[ (G_{C_p}) - (G_{C_{pi}}) \right]
\]

\[
q_{z=0\to9ft} = 45.6 \text{ psf} = q_h
\]
\[ P_{\text{roof}} = 45.6\left[-1.3 \right] \left[\pm 0.18\right] \]
\[ P_{\text{roof}} = \left[-59.28\right] \left[\pm 8.208\right] \]
\[ P_{\text{roof}} = -67.5 \text{ psf} \]

- **For Roof Zone 3, least horizontal:**

\[ a = 0.1 \text{ (dimension horizontal = 7.5 ft)} = 0.75 \text{ ft} < 3 \text{ ft} \]

\[ 2a = 6 \text{ ft} \quad (2a)^2 = 36 \text{ ft}^2 \]

- **External pressure coefficients, For Zone 3 Roof area = 36 \text{ ft}^2**

\[ GC_{p} = -2.9 \quad \text{For Roof Zone (3)} \]

\[ GC_{p_i} = \pm 0.18 \quad \text{For internal Pressure Coefficient} \]

\[ P_{\text{roof}} = 45.6\left[-2.9\right] \left[\pm 0.18\right] \]

\[ P_{\text{roof}} = \left[-132.24\right] \left[\pm 8.208\right] = -140.5 \text{ psf} \]
4.4 Summary

The wind pressure load was calculated by using ASCE 7-10. The footprint of the building was set based on the dimensions of a MILVAN container mentioned in the literature review, which are eight feet by eight feet by 20 feet (2.44m x 2.44 m x 6.1m). The emergency shelter was set at 7.5 ft. x 18 ft. x 9ft. The calculation takes into account that the emergency shelter should withstand wind gusts of 157 mph and missile projectiles. The Main Wind Force Resisting System (MWFRS) and Components and Cladding (C&C) force are used in the wind analysis.
Chapter V

Finite Element Modeling

5.1 Introduction

Understanding the structural behavior of the composite under loading are essential to later understanding the entire structure. Thus, the model for the emergency shelter should include the composite sandwich panel, steel cable wire and columns. The underlying aim of this is to develop a model to predict how these different parts will react in a static state, bending to the constant or pressure load of the model.

5.2 Modeling of CFRP Composite Sandwich Panels

By starting with the composite sandwich panel, simulations have been performed to compare two different geometries: shell element model and solid element model with isotropic and orthotropic material properties were attempted in finite element software ANSYS 15.0. Each of the models has been used for different types of fiber orientation in order to study the effect of share of fiber in a given direction. This was categorized into symmetric and non-symmetric orientation for convenience. Table 5.1 below shows the fabric arrangement for each layer. The faceplate laid stacking sequence is [45 / -45 / 0 / 90]. The fiber orientation for each fabric layer is shown in Figure 5-1 below. All the FE models are in the following sections.
Figure 5-1: Orientations of multi-axial fibers

Table 5.1: Fiber layup for non-symmetric and symmetric

<table>
<thead>
<tr>
<th></th>
<th>Layers</th>
<th>1st</th>
<th>2nd</th>
<th>3rd</th>
<th>4th</th>
<th>5th</th>
<th>6th</th>
<th>7th</th>
<th>8th</th>
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<tbody>
<tr>
<td>Non-Symmetric</td>
<td>Direction</td>
<td>45</td>
<td>-45</td>
<td>0</td>
<td>90</td>
<td>45</td>
<td>-45</td>
<td>0</td>
<td>90</td>
</tr>
<tr>
<td>Symmetric</td>
<td>Layers</td>
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<td>2nd</td>
<td>3rd</td>
<td>4th</td>
<td>5th</td>
<td>6th</td>
<td>7th</td>
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</tr>
<tr>
<td></td>
<td>Direction</td>
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<td>90</td>
<td>90</td>
<td>-45</td>
<td>45</td>
<td>0</td>
</tr>
</tbody>
</table>

5.2.1 Sketching Methods of Loads and Boundary Conditions

The Figure 5-2 below is the sketching methods of the load application and boundary condition for different cases of concentrated load and pressure load

(a). Concentrated Load of 1000 lb. on 8 nodes (125 lb/node), Fixed Support all sides
(b). Concentrated Load of 1000 lb. on 32 nodes (31.25 lb./node), Fixed Support all sides

(c). Pressure Load of (39.2 psf./144 in²) on the top area, Fixed Support all sides

Figure 5-2: Load Applications and Boundary Condition

5.2.2 Composite Sandwich Panels Description

Both geometries of the shell element model and solid element model composite sandwich panel have the size of width = 4.5 ft. x Length = 7.5 ft. x thickness = 0.425 ft., and after it was converted to inches, we get width = 54 inches x Length 90 inches x thickness = 5.094 inches. The composite sandwich panel consists of eight layers of CFRP at top face sheet and eight layers at bottom face sheet; each of the layers has thickness of
0.006 inches, and the EPS Foam core has a thickness of 5 inches which present between the top and bottom face sheets.

5.2.3 Materials and Size thickness

For composite sandwich panels, EPS Foam core is used as a core material (thickness 5 inches) and carbon fiber reinforced polymer (CFRP) composite (thickness 0.05 inches each side) is used as face sheet material. The total size will be 5.09 inches.

5.2.4 Parameters and Type of Loads

The loads are applied based on the potential risks that may cause by the wind and two types of potential risks were considered in this paper to define the types of loads. First risk was considered, is the wind force that pushing any side of the building and might cause racking, sliding, and potential overturning and then collapse. Second risk was considered, is the wind-borne debris which might cause a serious damages to the buildings or lives. A concentrated load and a pressure load were applied to the building in response of these two risks of wind mentioned.

Each section of shell element model and solid element model has been used for different type of loads in order to study the effect of share of fiber in a given direction. This was categorized into two types of loads. The first type is a concentrated load which acts over a small distance of the panel. Because of concentration over a small distance, this load may be considered as acting on a point. Point load is denoted by P and symbol of point load is arrow heading downward. The second type is when the distribution of pressure adjusts the loads of nodes for balanced loading. In this instance, pressure is determined as the force spreads over an area in which it is acting. The reason for using different types of
loads is to compare the resulting data on composite sandwich panel bending properties to help in establishing an appropriate modeling approach and bring it in to the main model presented later in this paper.

5.2.5 The ANSYS Elements Utilized

Determining the type of element used in both sections of the shell element model and solid element model is very important and, in this analysis, shell and solid elements have been used. In the section of shell element model, a shell 281 element (with 8 nodes) is used for modeling of thick sandwich structures. In the section of solid element model, a solid 186 element (with 20 nodes) is used for the core, and a solid 185 element (with 8 nodes) is used for the top and bottom face sheets.

5.2.5.1 SHELL281 Element Description

The SHELL281 element is able to have a maximum of 100 layers and is useful for thick shell structures. There are eight nodes in the element, and there are six degrees of freedom (DOF) in each node. This is well suited for linear and large rotations, or for large strain nonlinear applications. The facets of this element are shown in Figure 5-3 below (ANSYS 15.0, manual).
5.2.5.2 SOLID185 Layered Structural Solid Element Description

This is used for layered solid model thick shells or solids. The layered section definition is given by ANSYS section (SECxxx) commands.

Figure 5-4: SOLID185 Layered Structural Solid Geometry (ANSYS 15.0, manual).
5.2.5.3 SOLID186 Element Description

A higher order element, this three dimensional 20-node solid has quadratic displacement behavior. SOLID186 has 20 nodes possessing three DOF per node, in the X, Y, and Z directions. The element has plasticity, hyperelasticity, creep, stress stiffening, large deflection, and large strain capabilities. Furthermore, it has the mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials (ANSYS 15.0, manual).

Figure 5-5: SOLID186 Structural Solid Geometry (ANSYS 15.0, manual).

5.2.6 Chosen Element

After performing model verification of the 2D shell element model and 3D solid element model and comparing the resulting data, the 3D solid element model was chosen to be used in the main geometry. The SOLID185 element was used to model the top and bottom face sheets, representing the skin of the composite. SOLID186 was used to model the EPS foam core. Figures 5-4 and 5-5 above show the geometries of the SOLID185 element and SOLID186 element.
5.2.7 Mechanical Material Properties

The material properties are assigned as linear isotropic and linear orthotropic. The linear isotropic material properties were used for defining the EPS Foam Core, and the linear orthotropic material properties were used for defining face sheets of CFRP. Linear isotropic Young’s Modulus, \( E \), and the Poisson’s Ratio are required to represent elastic properties. For example, an orthotropic material has elastic properties which are symmetric for each of three mutually perpendicular symmetry planes. This requires nine independent elastic constants for complete characterization of its elastic properties including: Young’s Modulus (\( E_x - E_y - E_z \)) directions, Poisson’s Ratio (\( \nu_{xy} - \nu_{yz} - \nu_{xz} \)) planes, and Shear Modulus (\( G_{xy} - G_{yz} - G_{xz} \)) planes. Table 5.2 below shows the material properties for CFRP face sheets and EPS Foam Core of composite sandwich panel. The Young’s Modulus of CFRP is from the ACI 440.2R-08 Code, while the shear modulus and Poisson’s ratio are from the literature.

<table>
<thead>
<tr>
<th>Material</th>
<th>CFRP (MPa)</th>
<th>CFRP (psi)</th>
<th>EPS Foam (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Young’s Modulus</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( E_x )</td>
<td>165000</td>
<td>23931226</td>
<td></td>
</tr>
<tr>
<td>( E_y )</td>
<td>10000</td>
<td>1450377</td>
<td>200</td>
</tr>
<tr>
<td>( E_z )</td>
<td>10000</td>
<td>1450377</td>
<td></td>
</tr>
<tr>
<td><strong>Poisson’s Ratio</strong></td>
<td></td>
<td></td>
<td>0.3</td>
</tr>
<tr>
<td>( \nu_{xy} )</td>
<td>0.34</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>( \nu_{yz} )</td>
<td>0.38</td>
<td>0.38</td>
<td></td>
</tr>
<tr>
<td>( \nu_{xz} )</td>
<td>0.016</td>
<td>0.016</td>
<td></td>
</tr>
<tr>
<td><strong>Shear Modulus</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( G_{xy} )</td>
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<td>1015264</td>
<td></td>
</tr>
<tr>
<td>( G_{yz} )</td>
<td>7000</td>
<td>1015264</td>
<td></td>
</tr>
<tr>
<td>( G_{xz} )</td>
<td>7000</td>
<td>1015264</td>
<td></td>
</tr>
</tbody>
</table>
5.2.8 Meshing, Boundary Conditions, and Loading

Both isotropic and orthotropic cases were analyzed and meshing was accomplished based on convergence study. The mesh size 1.5 inches was used for CFRP and EPS Foam. The number of subdivisions on selected lines was successively increased, generating a mesh during each step. Figure 5-6 shows the meshing of the 2D panel model.

The support conditions were set as fixed support on all edges of the panel. A vertical load in the form of concentrated load of 1000 lb. was applied in negative Y-direction at the top surface of the composite sandwich panel, the load was simulated as point loads. The load was divided into concentrated load applied on eight nodes, concentrated load applied on thirty-two nodes, and pressure load applied on the whole area. Figure 5-7 shows the boundary conditions and loading application of the 2D shell element model panel.

5.2.9 Symmetric and non-symmetric Direction of Layer Fiber

Non-symmetric and symmetric direction of the fiber layers are used for both the 2D model and the 3D model in this analysis. The concentrated load and pressure load were applied in different cases (A and B) where case A represents the non-symmetric fiber layer directions and case B represents the symmetric fiber layer directions. In the first method of analysis, a concentrated load of 1000 lb. was applied on 8 nodes in the middle of the panel. For the second method of analysis, a concentrated load of 1000 lb. was applied on 32 points. For the third method of analysis, a pressure load of 1000 lb. was applied on the area of the panel.
5.3 Section 1: Design and FEM of 2D Shell Element Model

A sandwich panel is modeled as two dimensional elements with three layers represented by top and bottom face sheets and core materials in the middle. The first layer corresponds to the top face sheet or laminate. The next layer corresponds to core material, and the last layer represents the bottom face sheet or laminate. Each layer of the top and bottom face sheets contain eight sub-layers stacked together in different directions as explained above and in Figure 5-8 below. The above approach corresponds to an equivalent single-layer classical lamination theory for sandwich structures to study their deformations. Figure 5-2 above shows the load and boundary conditions used for the 2D shell element sandwich panel model. The material properties used for the 2D sandwich panel model are listed in Table 5.2.

Figure 5-6: Meshing of CFRP face sheets and EPS Foam Core for both symmetric and non-symmetric of layer CFRP
(a). Concentrated Load of 8 nodes

(b). Concentrated Load of 32 nodes
Figure 5-7 shows the boundary conditions and load application for non-symmetric and symmetric fiber layers. Concentrated load (force/nodes) and pressure load (force/area) were applied to the panel. Figure 5-7(a) shows that a total force of 1000 lb. was applied (125 lb. force equally applied to each of the 8 nodes) at the middle of the panel. In Figure 5-7(b) a total force of 1000 lb. was applied to the panel (31.25 lb. distributed to each of the 32 nodes) of the panel. The pressure load in Figure 5-7(c) shows that the load of \( \frac{39.2 \text{ psf}}{144 \text{ in}^2} = 0.27 \text{ psi} \) for W and \( \frac{90 \text{ psf}}{144 \text{ in}^2} = 0.6 \text{ psi} \) was applied on the whole area and transferred to the finite element.
The stacking sequences for the different cases were looked at for a select few composite panels. For this, the symmetric and non-symmetric sequences are included. The Figure 5-8 shows how the fibers of the layer were distributed on each side of the EPS foam core.
The Layer Stacking Sequence (LSS) are denoted. The CFRP face sheets and EPS Foam were assigned with materials numbered 1 and 2, respectively.

### 5.3.1 Result: Solution and Discussion for 2D Shell Element Model

#### (a). non-symmetric fiber layers. (Inches)

#### (b). Symmetric fiber layers. (Inches)

Figure 5-9: Total Maximum Displacement Contour Plot of Shell Element, Concentrated Load on 8 nodes
Figure 5-10: Total Maximum Displacement Contour Plot of Shell Element, Concentrated Load on 32 nodes

(a). non-symmetric fiber layers. (Inches)

(b). Symmetric fiber layers. (Inches)
Figures 5-9 through 5.11 show the total maximum displacement of the panel made up of the CFRP and EPS Foam core materials, with the load of 1000 lb. for all cases 1, 2, and 3, A and B. The FEA result shows that the red hotspot is the maximum displacement which
occurred at the center of the panel as expected. However, the loads were applied as a concentrated load of \([1000 \text{ lb./(number of nodes)}]\) for case 1 and case 2 and a pressure load of \((1000 \text{ lb./area})\) for case 3. Table 5.3 below demonstrates the result of FE for two dimensional shell element for all cases including non-symmetric fiber layers and symmetric. It shows the total maximum displacement in the Y-direction, USUM, for load cases 1, 2, and 3.

Table 5.3: Summary maximum displacements of 2D shell element model

<table>
<thead>
<tr>
<th>Non-symmetric fiber layers</th>
<th>Symmetric fiber layers</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cases</td>
<td>USUM (in.)</td>
<td>USUM (in.)</td>
</tr>
<tr>
<td>1</td>
<td>0.482945</td>
<td>0.48295</td>
</tr>
<tr>
<td>2</td>
<td>0.25362</td>
<td>0.253622</td>
</tr>
<tr>
<td>3</td>
<td>0.158927</td>
<td>0.158928</td>
</tr>
</tbody>
</table>

1 = Concentrated Load on 8 Nodes
2 = Concentrated Load on 32 Nodes
3 = Pressure Load distributed on the surface area
USUM = Total Max Displacement in Y-Direction

The FEA results in Table 5.3 obtained a small changes between the non-symmetric fiber layers and symmetric fiber layers, where the percentage changing of displacement in case 1 was only 0.001\%, case 2 was only 0.0008\%, and case 3 was 0.0006\%. Consequently, no need to focus on what we should use for the final model in this paper whether non-symmetric fiber layers or symmetry fiber layers because the comparison between them were almost the same.
Figure 5-12: Flexural Stress Distribution in X-Direction of Shell Element, Concentrated Load on 8 nodes (Force / node)
Figure 5-13: Flexural Stress Distribution in X-Direction of Shell Element, Concentrated Load on 32 nodes (Force / node)
As seen in Figure 5-12(a), the maximum stress in the local X-Direction for concentrated load on 8 Points was 688 psi for the non-symmetric fiber layer. Figure 5-12(b) shows that the maximum stress was 358 psi for symmetric. Both (5-27.a) and (5-27.b) stress occurred
away from the middle toward the edges of the panel and the maximum was represented by the red hotspot, which occurred at edges.

Figure 5-13(a) shows the maximum stress in the local X-Direction for concentrated load on 32 points was 227 psi. The maximum stress in Figure 5-13(b) was 175 psi.

The maximum stresses in the local X-Direction for pressure load as shown in Figure 5-14 was 128 psi (Tension) for both non-symmetric and symmetric fiber layers, but the maximum Compression stress (-116 psi) occurred along the span length, about 20 in. from the fixed edge, as shown in Graph 5-3. It can be seen from Figure 5-14 that stress occurred out of the center to the edges of the panel which are represented by green, light green, yellow, orange, and red colors.
Figure 5-15: Flexural Stress Distribution in Z-Direction of Shell Element, Concentrated Load on 8 nodes (Force / node)

(b). Symmetric fiber layers (psi)

(a). non-symmetric fiber layers (psi)
Figure 5-16: Flexural Stress Distribution in Z-Direction of Shell Element, Concentrated Load on 32 nodes (Force / node)

(b). Symmetric fiber layers (psi)

(a). non-symmetric fiber layers (psi)
Figure 5-17: Flexural Stress Distribution in X-Direction of Shell Element, Pressure Load on the Surface (Force / area)

Figure 5-15(a) shows that the maximum stress in the Local Z-Direction for concentrated load on 8 points was 286 psi. The FEA result showed that the minimum stress is located at the center of the panel and the maximum occurred at the edges of the panel. Figure 5-15(b) shows that the maximum stress was 348 psi and note that the red hotspot represents the maximum and is also located at the edges of the panel.

In Figure 5-16(a) and 5-16(b) the maximum stress in the local Z-Direction for concentrated load on 32 points was 191 psi.

The maximum stress in the local Z-Direction for pressure load in Figure 5-17(a) and 5-17.b were 143 psi and the FEA result showed that the red hotspot of maximum stress was located along the edges of the boundary conditions of the panel where the minimum was located at the center as expected.
Figure 5-18: Shear Stress Distribution in XZ Plane for Shell Element, Concentrated Load on 8 nodes

(a). non-symmetric fiber layers (psi)

(b). Symmetric fiber layers (psi)
Figure 5-19: Shear Stress Distribution in XZ Plane for Shell Element, Concentrated Load on 32 nodes
Figure 5-20: Shear Stress Distribution in XZ Plane for Shell Element, Pressure Load

The FEA shown in Figures of 5-18 through 5-20 demonstrates that for both non-symmetric and symmetric fiber layers, the maximum shear stress occurs at the middle of
the panel. The maximum shear stress distributed in XZ plane for pressure load were 69 psi for non-symmetric and 121 psi symmetric.

The maximum stress of all flexural stress, shear stress, and principal stress of the 2D shell element sandwich composite model obtained from the FEA analysis are presented in Table 5.4 and 5.5. The maximum 1st principal stress that the CFRP face sheets sustained above 680 psi for a concentrated load and above 250 psi for a pressure load. The first principal stress was higher than the second and third principal stress, because it was responsible for taking up the main stress directed along the thickness direction.

Table 5.4: Summary stress of 2D shell element for non-symmetric fiber layers

<table>
<thead>
<tr>
<th>Cases A</th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SX</td>
<td>SZ</td>
<td>SXZ</td>
</tr>
<tr>
<td>1</td>
<td>T</td>
<td>688</td>
<td>286</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-511</td>
<td>-347</td>
</tr>
<tr>
<td>2</td>
<td>T</td>
<td>227</td>
<td>191</td>
</tr>
<tr>
<td></td>
<td>C</td>
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<td>128</td>
<td>143</td>
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<tr>
<td></td>
<td>C</td>
<td>-331</td>
<td>-83</td>
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</tbody>
</table>

Table 5.5: Summary stress of 2D shell element for symmetric fiber layers

<table>
<thead>
<tr>
<th>Cases B</th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
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<tr>
<td></td>
<td>SX</td>
<td>SZ</td>
<td>SXZ</td>
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<td>348</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-358</td>
<td>-348</td>
</tr>
<tr>
<td>2</td>
<td>T</td>
<td>175</td>
<td>191</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-175</td>
<td>-191</td>
</tr>
<tr>
<td>3</td>
<td>T</td>
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<td>142</td>
</tr>
<tr>
<td></td>
<td>C</td>
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</tr>
</tbody>
</table>
5.3.2 Results Comparison and Summary of 2D shell element Model

For this application, composite sandwich panels of size 90 inches x 54 inches x 5.09 inches were tested under two and eight static Load points for the 8-node and 32-node models, respectively, and analyzed by the FEM. Pressure load distribution on the surface was also tested and analyzed by the FEM. All edges of the exterior were fixed supported. Two concentrated load cases were applied, symmetrically. In the first case, equal concentrated loads were applied to 8 nodes. In the second case, equal concentrated loads were applied to 32 nodes. In the third case, a pressure load was applied. For each of the model cases, deflections were recorded at the mid span with corresponding stresses.

In the FEA, a sandwich panel with three equivalent layers, including top face, bottom face, and core, is modeled. For simplicity and verification purposes, the equivalent properties obtained for face laminates and core are used directly in the model. In this section of the 2D shell element sandwich panel model, all properties of the face laminates and core are each modeled in a single layer, using an 8 nodded shell element called SHELL281.

Figure 5-21 shows the calculated deflection data across the span of the 2D shell element model composite sandwich panel. Figure 5-22 shows the curve of deflection vs the load applied on the 2D model for all three cases. The curve deflection values are extracted from the time history postprocessor and exported as a note file. It is then imported to an excel file and the curve is plotted.

Bending stresses and shear stresses are calculated from the FEA. The comparison of bending stress (X and Z direction along the span) and shear stresses (XZ along the span)
from the section of the two dimensional element for three cases of non-symmetric and symmetric fiber layers are shown below in Figure 5-23, Figure 5-24, and Figure 5-25.

Table 5.6: Summary deflection values of all cases along span for 2D shell element

<table>
<thead>
<tr>
<th>Span L (in.)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20</td>
<td>1.36E-17</td>
<td>1.36E-17</td>
<td>1.24E-17</td>
</tr>
<tr>
<td>30</td>
<td>-2.07E-16</td>
<td>-2.07E-16</td>
<td>-1.34E-16</td>
</tr>
<tr>
<td>40</td>
<td>-5.77E-17</td>
<td>-5.77E-17</td>
<td>-4.67E-17</td>
</tr>
<tr>
<td>50</td>
<td>-5.77E-17</td>
<td>-5.77E-17</td>
<td>-4.67E-17</td>
</tr>
<tr>
<td>60</td>
<td>2.08E-16</td>
<td>2.08E-16</td>
<td>1.35E-16</td>
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<tr>
<td>80</td>
<td>2.83E-17</td>
<td>2.83E-17</td>
<td>3.99E-17</td>
</tr>
<tr>
<td>90</td>
<td>2.19E-32</td>
<td>2.19E-32</td>
<td>3.69E-32</td>
</tr>
</tbody>
</table>

Figure 5-21: Predicted deflection data across the span of all cases for 2D shell element
Table 5.7: Summary of deflection value vs. load for 2D shell element

<table>
<thead>
<tr>
<th>Load (lb.)</th>
<th>Case 1</th>
<th></th>
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<th>Case 3</th>
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<tr>
<td></td>
<td>Deflection (in.)</td>
<td></td>
<td>Deflection (in.)</td>
<td></td>
<td>Deflection (in.)</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>1.89E-02</td>
<td>1.89E-02</td>
<td>1.27E-02</td>
<td>1.27E-02</td>
<td>7.85E-03</td>
<td>7.85E-03</td>
</tr>
<tr>
<td>100</td>
<td>3.78E-02</td>
<td>3.78E-02</td>
<td>2.54E-02</td>
<td>2.54E-02</td>
<td>1.57E-02</td>
<td>1.57E-02</td>
</tr>
<tr>
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<td>7.55E-02</td>
<td>7.55E-02</td>
<td>5.07E-02</td>
<td>5.07E-02</td>
<td>3.14E-02</td>
<td>3.14E-02</td>
</tr>
<tr>
<td>300</td>
<td>0.1133</td>
<td>0.1133</td>
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<td>7.61E-02</td>
<td>4.71E-02</td>
<td>4.71E-02</td>
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<tr>
<td>400</td>
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<td>0.1511</td>
<td>0.1011</td>
<td>0.1015</td>
<td>6.28E-02</td>
<td>6.28E-02</td>
</tr>
<tr>
<td>500</td>
<td>0.1889</td>
<td>0.1889</td>
<td>0.1268</td>
<td>0.1268</td>
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<td>7.85E-02</td>
</tr>
<tr>
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<td>0.2266</td>
<td>0.1522</td>
<td>0.1522</td>
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<td>9.42E-02</td>
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<td>0.1099</td>
</tr>
<tr>
<td>800</td>
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<td>0.2029</td>
<td>0.2030</td>
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</tr>
<tr>
<td>900</td>
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<td>0.3399</td>
<td>0.2283</td>
<td>0.2283</td>
<td>0.1413</td>
<td>0.1413</td>
</tr>
<tr>
<td>1000</td>
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<td>0.3777</td>
<td>0.2536</td>
<td>0.2536</td>
<td>0.1570</td>
<td>0.1570</td>
</tr>
</tbody>
</table>

Figure 5-22: Deflection vs load of 2D composite sandwich panel
Table 5.8: Summary of stress values in X-Direction along the span for 2D shell element

<table>
<thead>
<tr>
<th>Span Length (in)</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Case 2</td>
</tr>
<tr>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>0</td>
<td>-8</td>
</tr>
<tr>
<td>10</td>
<td>-6</td>
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<td>20</td>
<td>-10</td>
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<tr>
<td>30</td>
<td>-12</td>
</tr>
<tr>
<td>40</td>
<td>-12</td>
</tr>
<tr>
<td>50</td>
<td>-12</td>
</tr>
<tr>
<td>60</td>
<td>-12</td>
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<td>80</td>
<td>-6</td>
</tr>
<tr>
<td>90</td>
<td>-8</td>
</tr>
</tbody>
</table>

Figure 5-23: Comparison of bending stress in X-Direction along the span
Table 5.9: Summary of stress values in Z-Direction along the span for 2D shell element

<table>
<thead>
<tr>
<th>Span L (in)</th>
<th>Stresses (psi)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
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</tr>
<tr>
<td>10</td>
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<td>-19</td>
<td>-166</td>
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<td>-29</td>
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<td>-25</td>
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<td>-6</td>
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<td>-31</td>
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<tr>
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<td>-1</td>
<td>-2</td>
<td>-0.6</td>
<td>-2</td>
</tr>
</tbody>
</table>

Figure 5-24: Comparison of bending stress in Z-Direction along the span
Table 5.10: Summary of shear stress XZ values along the span for 2D shell element

<table>
<thead>
<tr>
<th>Span L (in.)</th>
<th>Shear Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case 1</td>
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<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>0</td>
<td>0.4</td>
</tr>
<tr>
<td>10</td>
<td>17.0</td>
</tr>
<tr>
<td>20</td>
<td>22.0</td>
</tr>
<tr>
<td>30</td>
<td>13.0</td>
</tr>
<tr>
<td>40</td>
<td>1.0</td>
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<tr>
<td>50</td>
<td>-1.0</td>
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<tr>
<td>60</td>
<td>-13.0</td>
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<tr>
<td>70</td>
<td>-22.0</td>
</tr>
<tr>
<td>80</td>
<td>-17.0</td>
</tr>
<tr>
<td>90</td>
<td>-0.4</td>
</tr>
</tbody>
</table>

Figure 5-25: The Comparison of shear stress XZ along the span for 2D shell element

5.4 Section 2: Design and FEM of 3D Solid Element Model

Solid element model accounts for a three dimensional nature of the test sample and uses a solid 185 element to model two face sheets and a solid 186 element to model the
EPS Foam core material. This model was able to accommodate both isotropic and orthotropic material properties, but isotropic material properties were initially applied to the model as reported in the present section of the 2D shell element model. Furthermore, one can also build a model with a core of solid shell and face sheets with a two dimensional panel, but a model with all solid elements offers better compatibility among the nodes and elements. Load applications and boundary conditions used for the 3D solid element model is the same as the one presented in the section of the 2D shell element model as shown in Figure 5-2 above. Material properties assigned for the 3D solid element model are linear isotropic and linear orthotropic as listed in Table 5-2 (Mechanical Properties of CFRP and EPS Foam Core), which require Young’s Modulus, $E$, and the Poisson’s for linear isotropic and nine independent elastic constants for orthotropic materials including: Young’s Modulus ({$E_x - E_y - E_z$}) directions, Poisson’s Ratio ({$v_{xy} - v_{yz} - v_{xz}$}) planes, and Shear Modulus ({$G_{xy} - G_{yz} - G_{xz}$}) planes. The fiber direction was categorized into symmetric and non-symmetric orientations.

The 3D orthotropic solid element model exploits advantages of the isotropic solid element model and orthotropic panel model. The top and bottom skin in the model were built with solid geometry and assigned to be orthotropic material properties and the core was assigned to be isotropic material properties. In the course of study, this model was first constructed as shown in Figure 5-26, where the geometry is modeled by creating the skin face sheet in the XZ plane. In addition, the face sheet was extruded in Y direction with dimensions of 0.05 inches, but the core had a different width of 5 inches. Hence, XZ became the principal plane for material properties, instead of XY. Different cases were
modeled in terms of type of load and direction of fiber as reported in the previous section of the 2D model. These cases were designated as follows:

- Case 1-A = concentrated load on 8 nodes and non-symmetric
- Case 1-B = concentrated load on 8 nodes and symmetric
- Case 2-A = concentrated load on 32 nodes and non-symmetric
- Case 2-B = concentrated load on 32 nodes and symmetric
- Case 3-A = pressure load on 8 nodes and non-symmetric
- Case 3-B = pressure load on 8 nodes and symmetric
(b) CFRP sheets and Core

Figure 5-26: Unmeshed Modeling Geometry of 3D Solid Element Model

The geometry of the 3D solid is modeled for orthotropic and isotropic materials, and Figure 5-26 above shows that the CFRP sheets represent orthotropic material and the core represents isotropic material. In the ANSYS software, Foam and the skin of CFRP sheets (top and bottom) were constructed with blocks that represent volume. The CFRP sheets, material properties, thickness, and orientation of fibers are defined using build-up thin area sections. The CFRP sheets were created as an area and modified with shell section properties.
Different material models and element types are available in the FEA software program, ANSYS. Specific material models and element types had to be selected to model the panel core and CFRP sheets. Material models, element type, mesh generation, boundary conditions, and linear finite element analysis solutions are presented in detail in this section. CFRP sheets were meshed with a mesh size such that the nodes of CFRP sheets coincide with the nodes of the panel. Finally, all nodes were merged to simulate the perfect bonding between the CFRP sheet and panel.

The mesh was defined through mesh attributes and the Foam Core, and CFRP sheets were selected and were assigned by their respective element type and material properties, as shown in Figure 5-27. The size element edge length of 1.5 inches has been made through mesh tools. The mesh of 3D solid element models was done by using the command of
volume sweep by meshing horizontally from one end to the next end of each volume and the mesh was generated.

(a) Concentrated Load of (1000 lb./8 nodes)

(b) Concentrated Load of (1000 lb./32 nodes)
(c) Pressure load of (1000 lb./area)

Figure 5-28: Boundary Conditions and Load Application

The above Figure 5-28 shows the boundary conditions and load application for both symmetric and non-symmetric fiber layers. Support conditions at the edges were set to be fixed support, and different cases of a static concentrated load and pressure load were applied at the top of the panel. The respective nodes of the vertical load were selected and the loads were applied.

Before the problem was solved, the load was analyzed and the load step was selected for iteration and proper convergence. The same load of 1000 lb. that was applied on the top of the panel was also set up to be the end load step. The load steps settings were set such that if non-convergence occurred, after the problem was solved, the load steps were automatically divided into eleven load steps, depending upon the previous response of the structure.
5.4.1 Result: Solution and Discussion for 3D Solid Element Model

(a). non-symmetric fiber layers (Inches)

(b). Symmetric fiber layers (Inches)

Figure 5-29: Total Maximum Displacement Contour Plot of Solid Element, Concentrated Load on 8 nodes
Figure 5-30: Total Maximum Displacement Contour Plot of Solid Element, Concentrated Load on 32 nodes
Figure 5-31: Total Maximum Displacement Contour Plot of Solid Element, Pressure Load

Figures 5-29, 5-30, and 5-31 are the total maximum displacement of the panel made up of the CFRP and EPS Foam core materials, with the load of 1000 lb. for all cases 1, 2, and
3, A and B. The FEA result shows that the red hotspot is the maximum displacement which occurred at the center of the panel as expected. However, in Figure 5-29 the load was applied as a concentrated load of 1000 lb. on 8 nodes. This Figure is represented by case (1-A and 1-B) and shows the total maximum displacement of the panel, which is 0.182 inches. That displacement is true for both non-symmetric fiber layer and symmetric fiber layer.

Figure 5-30 represents by case (2-A and 2-B) shows that the total maximum displacement of the panel is 0.129 inches. However, the load was applied as a concentrated load of 1000 lb. on 32 nodes.

The load in Figure 5-31 was applied as a pressure load on the top area of the panel. According to the FEA the case (3-A and 3-B), the total maximum displacement of the panel is 0.092 inches for both non-symmetric fiber layer and symmetric fiber layer. Table 5.11 below demonstrates the result of FE for three dimensional solid element for all cases including non-symmetric fiber layers and symmetric. It shows the total maximum displacement in the Y-direction, USUM, for load cases 1, 2, and 3.

<table>
<thead>
<tr>
<th>Cases</th>
<th>Non-symmetric fiber layers USUM (in.)</th>
<th>Symmetric fiber layers USUM (in.)</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.182103</td>
<td>0.182101</td>
<td>0.0011</td>
</tr>
<tr>
<td>2</td>
<td>0.128881</td>
<td>0.128880</td>
<td>0.0008</td>
</tr>
<tr>
<td>3</td>
<td>0.092272</td>
<td>0.092271</td>
<td>0.0011</td>
</tr>
</tbody>
</table>

1 = Concentrated Load on 8 Nodes
2 = Concentrated Load on 32 Nodes
3 = Pressure Load distributed on the surface area
USUM = Total Max Displacement in Y-Direction

The FEA results in Table 5.11 obtained a small changes between the non-symmetric fiber layers and symmetric fiber layers, where the percentage changing of displacement in case 1 was only 0.0011%, case 2 was only 0.0008%, and case 3 was 0.0011%.
Consequently, no need to focus on what we should use for the final model in this paper whether non-symmetric fiber layers or symmetry fiber layers because the comparison between them were almost the same.

The following FEA is the flexural stress distribution in X and Z directions, and shear stress distribution in XZ plane.

(a). non-symmetric fiber layers (psi)
Figure 5-32: Flexural Stress Distribution in X-Direction of Solid Element, Concentrated Load on 8 nodes (Force / node)

(b). Symmetric fiber layers (psi)

(a). non-symmetric fiber layers (psi)
b). Symmetric fiber layers (psi)

Figure 5-33: Flexural Stress Distribution in X-Direction of Solid Element, Concentrated Load on 32 nodes (Force / node)

(a). non-symmetric fiber layers (psi)
Figure 5-34: Flexural Stress Distribution in X-Direction of Solid Element, Pressure Load (Force / area)

Figures 5-32 through 5-34 show the maximum stress in X-Direction and the load in this FEA was applied as a concentrated load and as a pressure load. Different cases were analyzed to have different results and to see how the panel responded in load with different directions of fiber layers. The maximum stress of the 3D solid element model in all cases above appear on the top and bottom along the edges of the panel where the boundary conditions were applied.

Table 5.12 below shows the stress values in X-direction along the span. All case values with non-symmetric and symmetric fiber layers are listed below. Figure 5-35 shows the comparison of bending stress in X-Direction along the span for cases 1, 2, and 3.
Table 5.12: Summary of Stress Values in X-Direction along the span

<table>
<thead>
<tr>
<th>Span L (in)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>90</td>
<td>364</td>
<td>364</td>
<td>306</td>
</tr>
</tbody>
</table>

Figure 5-35: Comparison of bending stress in X-Direction along the span of 3D solid element
Figure 5-36: Flexural Stress Distribution in Z-Direction of Solid Element, Concentrated Load on 8 nodes (Force / node)
Figure 5-37: Flexural Stress Distribution in Z-Direction of Solid Element, Concentrated Load on 32 nodes (Force / node)
Figure 5-38: Flexural Stress Distribution in Z-Direction of Solid Element, Pressure Load (Force / area)

The maximum stress in the local Z-Direction of concentrated load and pressure load for both non-symmetric and symmetric fiber layer directions are shown in Figures 5-36
through 5.38. According to the FEA result above, the stress is represented by the colors red, orange, yellow, and light green which can be seen in one side of the panel whether in top or bottom. All The CFRP face sheets and EPS Foam Core materials in all cases have stress in the same position with different values. The maximum stress is represented by red hotspots and takes place on the edges. The minimum can be seen with negative values on the center of the panel. The stress values in Z-Direction along the span of the 3D solid element model for all cases are listed below in Table 5.13. Based on the FEA result Figure 5-39 below shows the comparison of bending stress in Z-Direction along the span for all cases 1, 2, and 3. The FEA results show that the non-symmetric and symmetric fiber layer directions of 3D solid element models closely matched each other, which draw a symmetric diagrams.
Table 5.13: Summary of stress values in Z-Direction along the span

<table>
<thead>
<tr>
<th>Span L (in)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
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<td>-2</td>
<td>-2</td>
<td>-2</td>
</tr>
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<td>-96</td>
</tr>
<tr>
<td>90</td>
<td>-2</td>
<td>-2</td>
<td>-2</td>
</tr>
</tbody>
</table>

Figure 5-39: Comparison of bending stress in Z-Direction along the span
Figure 5-40: Shear Stress Distribution in XZ Plane for Solid Element, Concentrated Load on 8 nodes
(a). non-symmetric fiber layers (psi)

(b). Symmetric fiber layers (psi)

Figure 5-41: Shear Stress Distribution in XZ Plane for Solid Element, Concentrated Load on 32 nodes
Figure 5-42: Shear Stress Distribution in XZ Plane for Solid Element, Pressure Load

The maximum shear stress in XZ plane of concentrated load and pressure load for both non-symmetric and symmetric fiber layer directions is shown in Figures 5-40 through 5-
42. According to the FEA result, the shear stress for cases 1, 2, and 3 are 71 psi, 46 psi, and 25 psi. Tables 5.14 is the shear stress values in XZ plane along the span of 3D models.

Table 5.14: Shear stress values of cases 1, 2, and 3, along the span

<table>
<thead>
<tr>
<th>Span L (in.)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
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<td>13</td>
<td>7</td>
</tr>
<tr>
<td>30</td>
<td>8</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>40</td>
<td>0.6</td>
<td>0.6</td>
<td>2</td>
</tr>
<tr>
<td>50</td>
<td>-0.6</td>
<td>-0.6</td>
<td>-2</td>
</tr>
<tr>
<td>60</td>
<td>-8</td>
<td>-8</td>
<td>-5</td>
</tr>
<tr>
<td>70</td>
<td>-13</td>
<td>-13</td>
<td>-7</td>
</tr>
<tr>
<td>80</td>
<td>-10</td>
<td>-10</td>
<td>-7</td>
</tr>
<tr>
<td>90</td>
<td>1.15E-13</td>
<td>1.13E-13</td>
<td>3.74E-13</td>
</tr>
</tbody>
</table>

Figure 5-43: The Comparison of shear stress XZ along the span

Figure 5.43 is based on Table 14 which gives some idea where the shear stresses are located along the span for all cases 1, 2, and 3. The FEA results show that the non-
symmetric and symmetric fiber layer directions of 3D solid element models closely matched each other, which draw a symmetric diagrams.

The maximum stress of all flexural stress, shear stress, and principal stress of the 3D solid element model obtained from the FEA analysis are presented in Tables 5.15 and 5.16. According to the FEA results for all cases in the first principal, the tensile stresses were carried by the CFRP face sheets and located along the edges where the boundary conditions were applied. In case of a concentrated load on 8 nodes for both non-symmetric and symmetric fiber layers have a maximum tensile stress value of 2208 psi. In the case of a concentrated load on 32 nodes for both non-symmetric and symmetric fiber layers have a maximum tensile stress value of 1489 psi. Finally, in the case of a pressure load for both non-symmetric and symmetric fiber layers have a maximum tensile stress value of 1114 psi. The first principal stress was higher than the second and third principal stress, because it was responsible for taking up the main stress directed along the thickness direction (Nguyen, 2009). Third principal is the compressive stress distributions on the top and bottom face sheets of CFRP.
Table 5.15: Summary stress of 3D solid element for non-symmetric fiber layers

<table>
<thead>
<tr>
<th>Cases A</th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SX</td>
<td>SZ</td>
<td>SXZ</td>
</tr>
<tr>
<td>1</td>
<td>T</td>
<td>619</td>
<td>189</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-737</td>
<td>-189</td>
</tr>
<tr>
<td>2</td>
<td>T</td>
<td>617</td>
<td>136</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-566</td>
<td>-137</td>
</tr>
<tr>
<td>3</td>
<td>T</td>
<td>440</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-390</td>
<td>-97</td>
</tr>
</tbody>
</table>

Table 5.16 – Summary stress of 3D solid element for symmetric fiber layers

<table>
<thead>
<tr>
<th>Cases B</th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SX</td>
<td>SZ</td>
<td>SXZ</td>
</tr>
<tr>
<td>1</td>
<td>T</td>
<td>619</td>
<td>189</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-737</td>
<td>-189</td>
</tr>
<tr>
<td>2</td>
<td>T</td>
<td>617</td>
<td>136</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-566</td>
<td>-137</td>
</tr>
<tr>
<td>3</td>
<td>T</td>
<td>440</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>-390</td>
<td>-97</td>
</tr>
</tbody>
</table>

5.4.2 Results Comparison and Summary of 3D Solid Element Model

For this application, the 3D solid element model composite sandwich panels mentioned before were tested under different cases and analyzed by the FE method. All edges of the exterior panel were set as fixed support and given to a specific load. Case 1 has a two points loading condition, which was applied at the mid span to simulate a symmetric condition. Case 2 has an eight points loading condition, which was applied and divided equally along the span length. Case 3 has a pressure loading condition, which was applied at the top area of the panel carried out by the skin of the composite. All of the loads were applied in the negative Y-direction for all three cases. In this section of the 3D sandwich panel model the FE method was conducted at three modeling cases: the first case was under a concentrated load on 8 nodes (case 1) where the second case was under a concentrated load on 32 nodes.
(case 2), and the third case was under a pressure load (case 3). For each of the model cases, the deflections were recorded at the mid span with corresponding stresses.

In the FEA, a sandwich panel with three layers including top face, bottom face, and core is modeled. For simplicity and verification purposes, the properties obtained for face laminates and core are used directly in the model. In this section of the 3D sandwich panel model, all cases of the face laminates and core are each modeled in three main layers, using an 8-noded solid element called SOLID185 to model the two face sheets and using a 20-noded solid element called SOLID186 to model the EPS Foam core material. The top and bottom layers of the main three layers represent the fiber layers, and each one of them has eight sub-layers. The middle layer of the three main layers represents the EPS Foam core material.

The bending stresses and shear stresses are predicted from FEA. The summary and comparison of bending stress (including X and Z direction along the span) and shear stress (XZ along the span) from the section of the three dimensional model for the three cases of non-symmetric and symmetric fiber layers are reported in this section and shown in Figure 5-35, Figure 5-39, and Figure 5-43 above under the stresses Figures.
Table 5.17: Summary of deflection values of cases 1, 2, 3, along the span

<table>
<thead>
<tr>
<th>Span L (in.)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>40</td>
<td>1.93E-17</td>
<td>1.93E-17</td>
<td>1.51E-17</td>
</tr>
<tr>
<td>60</td>
<td>8.48E-18</td>
<td>8.48E-18</td>
<td>5.26E-18</td>
</tr>
<tr>
<td>80</td>
<td>1.03E-18</td>
<td>1.03E-18</td>
<td>1.52E-18</td>
</tr>
<tr>
<td>90</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 5-44: Predicted deflection data across the span of 3D model
Table 5.18: Summary of deflection values vs. load of cases 1, 2, and 3

<table>
<thead>
<tr>
<th>Load (lb.)</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td>50</td>
<td>3.99E-02</td>
<td>3.99E-02</td>
<td>2.42E-02</td>
</tr>
<tr>
<td>100</td>
<td>7.97E-02</td>
<td>7.971E-02</td>
<td>4.84E-02</td>
</tr>
<tr>
<td>200</td>
<td>0.1594</td>
<td>0.1595</td>
<td>9.70E-02</td>
</tr>
<tr>
<td>300</td>
<td>0.2391</td>
<td>0.2392</td>
<td>0.1453</td>
</tr>
<tr>
<td>400</td>
<td>0.3188</td>
<td>0.3189</td>
<td>0.1938</td>
</tr>
<tr>
<td>500</td>
<td>0.3985</td>
<td>0.3986</td>
<td>0.2422</td>
</tr>
<tr>
<td>600</td>
<td>0.4782</td>
<td>0.4784</td>
<td>0.2906</td>
</tr>
<tr>
<td>700</td>
<td>0.5579</td>
<td>0.5581</td>
<td>0.3391</td>
</tr>
<tr>
<td>800</td>
<td>0.6376</td>
<td>0.6378</td>
<td>0.3875</td>
</tr>
<tr>
<td>900</td>
<td>0.7174</td>
<td>0.7176</td>
<td>0.4359</td>
</tr>
<tr>
<td>1000</td>
<td>0.7971</td>
<td>0.7973</td>
<td>0.4844</td>
</tr>
</tbody>
</table>

Figure 5-45: Deflection vs load summary and comparison of 3D solid cases

5.5 Results Comparison of 2D Shell Element and 3D Solid Element

Figure 5-46 shows the percentage changes of displacement between case 1-2, case 1-3, and case 2-3. There is a significant improvement among cases 1, 2, and 3. The percentage
changing decreased between case 1 to case 2 was (47 % - 29 %), while the percentage changing decreased between case 1 to case 3 was (67 % - 49 %), and the percentage changing decreased between case 2 to case 3 was (37 % - 29 %).

Figure 5-46: Percentage changes of displacement between cases

Figure 5-47: Deflection vs load comparison of 2D shell and 3D solid elements
Figure 5-48: Deflection data across the span of 2D shell and 3D solid elements

Figure 5-49: Comparison of bending stress in X-Direction along the span of 2D shell and 3D solid elements
Figure 5-50: Comparison of bending stress in Z-Direction along the span of 2D shell and 3D solid element

Figure 5-51: The Comparison of shear stress XZ along the span of 2D shell and 3D solid elements
5.6 Summary of 2D shell element and 3D solid element

To conclude, through combining this FE Modeling analysis and the CFRP composite sandwich panels described in this chapter, three different cases were assessed from the composite sandwich panel response. The second layer of the model was an EPS foam core between the top and bottom laminate in the model. This includes the concentrated load on eight nodes, the concentrated load on 32 nodes, and the pressure load. The FEA emphasized the assessment of deflection, shear response under different point bending conditions, and bending stress including the two and eight point conditions. At the end of the chapter, the 2D and 3D predictive models were compared. In this comparison, the predicted deflection which closely matched the CFRP composite sandwich panels. Under static loading condition, the 2D shell element model closely matched the resulting 3D solid element model. Given the 3D solid element representation of the composite sandwich panel, it is concluded that this should be used in the main construction of the emergency shelter.
Chapter VI

Finite Element Analysis of Steel Cable Structures

6.1 Introduction

This chapter presents one of the important parts of the main design presented in this paper. It is necessary to test the structural capacities of the design, specifically the anchored steel cables that are integral to the integrity of the design. To perform such an analysis, through finite element modeling, is the basis of this research project proposal. This reason to deal specifically with how the cables which reinforce the CFRP emergency shelter system fare under hurricane wind force during a finite element modeling simulation is that the six supporting columns underneath the design. These supporting columns will not be fixed to an anchor, though do provide structural support. These columns are not fixed due to the designs portable and reusable nature.

For this research, how the cable properties and pretensions affect the trusses with varying tensions and models in the focus. These cable truss configurations are often simple and can be analyzed accordingly (Liao & Du, 2010)

Four different models were proposed for design and analysis of modeling steel cable structures. One of these proposed design was modeled to analysis with no cables, but just truss and the reason for this to see if the truss can stands on the load was applied and prevent
failure with no support of cables. This specific model was failed to give any solution. The other three models were modeled and analyzed with different design of cables. Specific software was taken into consideration in which Finite Element packages ANSYS 15.0 has been utilized to carry out this design of steel cable structures under the wind pressure loading. The aim of proposing three different models is to compare results.

Comparison was made between model B, model C, and model D. The results achieved with all models are compared concerning simulation time and quality in terms of displacement, axial force and axial stress per element (cables).

6.2 Sketching Methods of Loads and Boundary Conditions

Figure 6-1 shows the sketching of the load application and boundary condition for different models. It shows the proposed aspects of the steel cable design to be tested under finite element modeling. As can be seen, the steel cables which extend from the corners of the design to the ground are proposed to provide a solution for the potential shear forces anticipated as part of a hurricane. An external force of 1000 lbs. is applied at the top of each node along positive Z-direction. All of the bottom nodes on the ground for all models are constrained. The six supporting columns underneath the design will not be fixed to an anchor.
6.3 Description and size

In Figure 6-1 half model of the actual main design of the emergency shelter was taken into consideration (W = 90 inches x L = 90 inches H = 108 inches). The green elements represent the truss and the black line elements represent the cables. The three models have the same length and cross-sectional size of truss elements. The three models will have the same cross-sectional, but each model has different length of cable and different location of nodes. As shown in Figure 6-1, all models are consists of 20 truss members. Models B and
D are consists of only 4 cables extend from each corner of the structure to the ground, also model C consists 4 cables, but each one extends from the top side of the structure to the ground.

There are 8 elements of truss in Y-direction, 4 of them in the bottom which they have a size of (18 inches x 6 inches x 6 inches) and the other 4 in top of them have size of (90 inches x 6 inches x 6 inches). In X-direction there are four elements have size of (90 inches x 6 inches x 6 inches) which they are connected with the elements in Y-direction from bottom and top. Z-direction also has 4 elements running positive which is also connected with the elements running on positive Y-direction. The elements running in X-direction and Z-direction have a size of (108 inches x 6 inches x 6 inches).

6.4 The ANSYS Elements Utilized

It is very important to choose the appropriate type of elements to be used in all model B, C, and D. In this analysis LINK and BEAM elements have been used. For all three models, LINK 3D finit stn 180 element is used for modeling of steel cable structures and BEAM 2 node 188 is used for modeling the truss elements. LINK and BEAM elements have different description which presented below.

6.4.1 LINK180 Element Description

Used in a variety of engineering applications, LINK180 is a three dimensional spar that can model trusses, links, sagging cables, and springs. The uniaxial tension compression element has three degrees of freedom for each node as translations in the nodal directions of x, y, and z. supported within this are the cable (tension only) and gap (compression-only). No bending of the element is considered, similar to a pin-jointed structure. Included
in this are large strain capabilities, deflection, rotation, creep and plasticity (Ansys 15.0 manual).

Figure 6-2: LINK180 Geometry, (Ansys 15.0 manual).

6.4.2 BEAM188 Element Description

To look at slender to medium thick beam structures, BEAM188 is used. This is based upon Timoshenko beam theory which looks at shear deformation effects. This provides options for warping that is unrestrained and the warping in restrained means for cross-sections. This element is quadratic, linear or alternately two-node cubic beam elements in three dimensions. For each node in BEAM188 there are six or seven degrees of freedom. Translations in the x, y, and z directions and rotations about the same directions are included in this scenario. Optional in this is the seventh degree of freedom, which is the warping magnitude. The ideal element for this is a linear, large rotation or large strain nonlinear applications (Ansys 15.0 manual).
6.5 Mechanical Material Properties

The material properties assigned as linear isotropic. The linear isotropic material properties were used for defining for both truss and cable elements. For linear isotropic Young’s Modulus, $E$, and the Poisson’s Ratio were required to represent elastic properties. Table 6.1 below is displayed the material properties for both truss which is assigned with BEAM188 element and steel cable which is assigned with LINK180 elements.

Table 6.1 - Mechanical Properties (Liao, 2010)

<table>
<thead>
<tr>
<th>Element</th>
<th>Young’s Modulus E (psi)</th>
<th>Poisson’s Ratio</th>
<th>Area (in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Truss</td>
<td>2E6</td>
<td>0.3</td>
<td>$6 \times 6 = 36$</td>
</tr>
<tr>
<td>Cable</td>
<td>1E6</td>
<td>0.3</td>
<td>$0.5 \times 0.5 \times \pi = 0.79$</td>
</tr>
</tbody>
</table>
6.6 Result and Discussion

Model B

Model C
Figure 6-4 above shows the total maximum displacement of cable-truss structures of all models B, C, and D. It can be seen that the FEA result showed that the red hotspot is the maximum displacement which occurred along all truss members and cables structures.

An external load of 1000 lb. was applied on the top of each node of the structure in Z-direction. The Figure shows the total maximum displacement of model B is 1.25 inches, model C has total maximum displacement of 0.4 inches, and model D has total maximum displacement of 2.2 inches. Table 6.2 below shows the comparison of USUM displacements of the three models presented above. The Figures above show that the cables prevented the structure from sliding or overturning.

**Table 6.2:** Comparison of displacements of the three models B, C, and D

<table>
<thead>
<tr>
<th></th>
<th>Model B</th>
<th>Model C</th>
<th>Model D</th>
</tr>
</thead>
<tbody>
<tr>
<td>USUM (in.)</td>
<td>1.2</td>
<td>0.4</td>
<td>2.2</td>
</tr>
</tbody>
</table>

USUM = Total Max Displacement in Z-Direction
Figure 6-5: The Maximum Stress in the Local X-Direction (psi)

As it can be seen in Figure 6-5 the maximum stress in the Local X-Direction of cable-truss structures was 3547 psi for model B, 1685 psi for model C, and model D has the highest maximum stress which is 6621 psi. All models have the stress occurred on cables elements. Tables 6.3 shows the comparison and summary of axial force and axial stress per cable. The result of FAE in the Figures of stress above and the Table below show that the axial force and axial stress carried out by the cables of each model.

In terms of axial force, the result shows that 2785.7 lb. was carried out by 2 cables (1 and 2) of model B. However, 1323.3 lb. was carried out by 2 cables (3 and 4) of model C, and 5200 was carried out by only one cable (1) of model D.

In terms of axial stress, the result shows that cable 1 and 2 of model B has maximum stress of 3457 psi (Tension). Cables 1 and 2 of model C has maximum stress of 1685 psi
(Tension). Model D has maximum tension stress of 6621 psi which only carried out by cable 1.

6.7 Results Comparison and Summary

Table 6.3: Comparison and summary of axial force and axial stress per cable

<table>
<thead>
<tr>
<th>Cables No.</th>
<th>Model B Axial Force (lb.)</th>
<th>Model C Axial Force (lb.)</th>
<th>Model D Axial Force (lb.)</th>
<th>Model B Axial Stress (psi.)</th>
<th>Model C Axial Stress (psi.)</th>
<th>Model D Axial Stress (psi.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2785.7</td>
<td>-</td>
<td>5200</td>
<td>3547</td>
<td>-</td>
<td>6621</td>
</tr>
<tr>
<td>2</td>
<td>2785.7</td>
<td>-</td>
<td>3547</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>-</td>
<td>1323.3</td>
<td>-</td>
<td>1685</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>-</td>
<td>1323.3</td>
<td>-</td>
<td>1685</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 6.6: Load vs. Deflection

To conclude, the analysis provided above tests two different models of truss structures using pre-tensioned cables. ANSYS 15.0, a software package for FEA was used to look at how the cable truss structures for both models behaved under static conditions. The truss element forces, nodal displacements, and cables tension were reported and give a general
idea of configuration design and analysis of trusses. From the results, there is a significant
effect on the performance of cable supported trusses when optimally positioned, pre-
tensioned cables are appropriately used.

This research provides new insights into how pre-tensioned cables and configuration
affects trusses. The geometry or the cables and their properties affect the performance of
the trusses. An increase in elastic modulus of the cables has the outcome of less
deforation of the truss. Important in the prediction of truss performance is the cable
pretension, and the setting of cable pretensions at appropriate setting also decreased truss
deforation. Under static loading conditions, model C’s finite element model has the
smallest displacement compared with the results of model B’s and D’s finite element
model. However, as can be seen in the results, model B demonstrates the cables in the ideal
configuration and have been chosen to be applied to the specifications of the emergency
shelter. Due to less length of cable elements, the cost effects of the model will be minimized
and access to an exit in a comfortable manner will be allowed.
Chapter VII

Finite Element Analysis of the Emergency Shelter

7.1 Introduction

This provides an overview of the design and analysis involved in the wind pressure loading on the emergency shelter. Through FEA, the shelter panels were calculated along with the results of the stress testing.

7.2 Modeling Geometry of Shelter and Description

The shelter geometry was modeled as: W=7.5 ft. x H=9 ft. x L=18 ft. (90 in x 108 in x 216 in). The model was built from the bottom up by connecting key points, lines, areas and volumes. In the beginning, the geometry was created as shell models from the different materials. In this model the shell elements were pictured as the mid-surface of the actual shells that were placed midway through the thickness. The thickness of each was a composite, as the real constant, without modeling the real geometry. By designing a model with shell elements, it is desirable to model thin plates of composite materials. This is due to the fact that the thickness of plates can be changed simply through assigning real constants to represent the actual physical depth of the plates. However, by changing the fiber volume fraction or constituents, they cannot be replaced by actual constants with solid elements because the actual depth of the geometry has to be crafted physically. The
The downside to using shell elements is that the results of this process are often less accurate than what is used with solid elements because there are fewer nodes in the shell elements than there are in the solid elements. This gap does not capture with accuracy the real situation. Due to this, it was determined to create solid models.

In the analysis of the panel design for the connection geometry and endplates were not included. The glue option in ANSYS allowed the pieces to be held together as if they were welded. For this, each panel was glued to the other panel of the shelter as if it were welded. This was responsible for holding the parts together during the wind pressure loading analysis.

![Figure 7-1: The Geometry of the Shelter.](image)

### 7.3 The used ANSYS Elements and Description

Solid186 and Solid185 are Linear solid element were used in this chapter to model the emergency shelter and Chapter Five has been explained these element in detail. Solid186 element is a higher order element, this three dimensional 20-node solid has quadratic
displacement behavior and defined by 20 nodes having 3 DOF at each node in the translations of the nodal x, y, and z directions. Solid185 element is used for three dimensional modeling of solid structures. It is defined by 8 nodes having 3 DOF at each node in the translations of the nodal x, y, and z directions (ANSYS 15.0 manual).

Each panel contains three layers, the first and third layers represents the top and bottom face sheets were modeled by using Solid185 and the second layer represents the EPS Foam Core was modeled by using Solid186 element.

7.4 Mechanical Material Properties and Size thickness

For the CFRP and EPS composite model, the isotropic properties were assigned to the EPS foam core, and the orthotropic materials were assigned to the top and bottom layers of the CFRP face sheets. Table eight in chapter five shows that the elastic modulus and Poisson’s ratio are the two constants required for input to represent the materials of the EPS foam core. Also elastic modulus (E_x - E_y - E_z) directions, Poisson’s ratios (\(\nu_{xy} - \nu_{yz} - \nu_{xz}\)) planes, and shear modulus (\(G_{xy} - G_{yz} - G_{xz}\)) planes have nine constants required for input to represent the materials of the CFRP face sheets.

Chapter five presented different types of fiber orientation has been used in order to study the effect of share of fiber in given direction. That was categorized into symmetric and non-symmetric orientation. In this final model of the emergency shelter, symmetric orientation has been used and Table 5.1 in chapter five shows the fabric arrangement for each layer. The faceplate laid stacking sequence is [45 / -45 / 0 / 90].

Each composite sandwich panel has the size of W = 4.5ft. x L = 7.5ft. x t = 0.425ft. and after it was converted to inches we get W = 54 inches x L = 90 inches x t = 5.09 inches.
Seventeen layers have been considered in this model for each panel. Sixteen layers represent the CFRP face sheets and one layer represents the EPS Foam Core. Each composite sandwich panel consists of eight layers of CFRP at top face sheet and eight layers at bottom face sheet, each one of the layers has thickness of 0.006 inches, and the EPS Foam core has a thickness of 5 inches which present between the top and bottom face sheets. The fiber orientation for each fabric layer is shown in Figure 7-2 below.

![Figure 7-2: Directions of fiber layers](image)

### 7.5 Meshing of Models

The geometry of the model was meshed with volume sweep with global and local mesh controls. Within the ANSYS meshing methods, there is a function known as sweep meshing. This can be used to make high solver accuracy while reducing mesh cell counts. This method begins by taking a source surface with automatic global settings, or other local sizing controls applied by the user. Next, it will sweep the source mesh through the body,
dividing it into equal pieces or by taking the side faces into a desired number of divisions (ANSYS 15.0 manual).

Global size control was used and the size element edge length was set up 1.5 inches. This technique of meshing gives accurate stress distribution and reasonable analysis time (ANSYS 15.0 manual). Figure 7-3 shows the mesh distribution of the CFRP and EPS sandwich composite model using volume sweep. It can be seen in the Figure below that the source face on the end of the model has been meshed, which was then brought through the swept section at specified intervals resulting in a square mesh on the side faces of the model.

Figure 7-3: Meshing of CFRP face sheets and EPS Foam Core

7.6 Loading & Boundary Conditions

In relation to the x, y, and z coordinates and displacements, each direction was restrained for particular nodes of the cable anchor to the ground. Restraining the displacements and rotations about the x, y, and z coordinates was needed to signify the
rigid connections at these points. The six supporting columns underneath the FRP emergency shelter will not be fixed to an anchor.

Discussed in chapter four, the wind pressure loading was applied to the surfaces area of the emergency shelter in a uniform and static pressure. There are positive and negative pressures that tell the way in which the wind pressure was applied. Figure 7-4 shows how the pressure load was applied. Figure 7-5 shows the deformed shape form the pressure load. Figure 7-6 shows the resultant displacement for pressure load of \( \frac{39.2 \text{ psf}}{144 \text{ in}^2} = 0.27 \text{ psi} \) (controlling load for MWFRS) that was applied on the whole area and transferred to the finite element. Figure 7-7 shows the resultant displacement for pressure load of \( \frac{90 \text{ psf}}{144 \text{ in}^2} = 0.6 \text{ psi} \) (controlling load for C&C) that was applied on the whole area and transferred to the finite element.

![Figure 7-4: Pressure Loading Transferred to FE](image-url)
7.7 Solution and Discussion of FEA Result

Figure 7-5: Deformed configuration

Figure 7-6: Total Maximum Displacement Contour Plot for Main Wind Force Resisting System (MWFRS), (Inches)
Figures 7-6 and 7-7 show the total maximum displacement of the made up of the CFRP and EPS Foam core materials with the pressure load of 39.2 psf./144 in² distributed on the surface area for Main Wind Force Resisting System (MWFRS), and 90 psf./144 in² distributed on the surface area for Components and Cladding (C&C). The result is recorded a maximum displacement of 1.1 inches for MWFRS model and 3.2 inches for C&C model. It can be seen that the FEA result showed that the red hotspot is the maximum displacement which occurred at the center of the side and the minimum displacement occurred at the bottom of the cables with value of 0 inches as excepted.

As discussed previously in this paper, the six supporting columns underneath the FRP emergency shelter will not be fixed to an anchor, though they do provide structural support. These columns are not fixed due to the design’s portable and reusable nature. According to the FEA result, these columns with the whole structure are displaced safely toward X direction which presented in green color around the structure with 0.48 inches
for MWFRS model and 1.4 inches for C&C model. But the panel starts to get displacement after 0.48 inches for MWFRS model and 1.4 inches for C&C model, which means the total value of displacement recorded to each model minus the displacement recorded in by green color. Thus, the resultant maximum panel displacement was (1.1–0.48=0.62 inches) for MWFRS model. The resultant maximum panel displacement was (3.27–1.45=1.82 inches) for C&C model. The cables worked well and prevent the structure from larger displacement or overturning due to the load was applied to the façade of the structure.

![Figure 7-8: Maximum stress in the Local Y-Direction for Main Wind Force Resisting System (MWFRS), (psi)](image)
Figure 7-9: Maximum stress in the Local Y-Direction for Components and Cladding (C&C), (psi)

Figure 7-10: Maximum stress in the Local Z-Direction for Main Wind Force Resisting System (MWFRS), (psi)
Figure 7-11: Maximum stress in the Local Z-Direction for Components and Cladding (C&C), (psi)

Figure 7-12: Maximum Shear Stress distribution in YZ Plane for Main Wind Force Resisting System (MWFRS), (psi)
Figures 7-8 through 7-11 present the maximum stress of Y and Z in local direction for both models of MWFRS and C&C. However, Figures 7-12 and 7-13 show the maximum shear stress in YZ Plane. The results summary of the lowest and highest value of stress are listed in Tables 7.1 and 7.2.

7.8 Results Comparison and Summary

Tables 7.1 and 7.2 show the summary flexural stress, shear stress, and principal stress for both models of MWFRS and C&C. It can be seen that all the values of the stress are increased in the C&C model by 66% and that because the applied load was increased about 66% from 39.2 psf./144 in$^2$ to 90 psf./144 in$^2$. And the principal stress 1 is the highest value of stress relative to other stress. The principal stress 1 had a value of 122672 psi for MWFRS model, however C&C model has a principal stress 1 of 368020 psi. The reason
of principal 1 had the highest value because it was responsible for taking up the main stress in the thickness direction (Nguyen, 2009).

Table 7.1: Summary of Flexural Stress, Shear Stress, and Principal Stress of the Emergency shelter model for MWFRS

<table>
<thead>
<tr>
<th></th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SX</td>
<td>SY</td>
<td>SZ</td>
</tr>
<tr>
<td>T</td>
<td>61644</td>
<td>41369</td>
<td>61549</td>
</tr>
<tr>
<td>C</td>
<td>61781</td>
<td>41372</td>
<td>61677</td>
</tr>
</tbody>
</table>

Table 7.2: Summary of Flexural Stress, Shear Stress, and Principal Stress of the emergency shelter model for C&C

<table>
<thead>
<tr>
<th></th>
<th>Flexural Stress (psi)</th>
<th>Shear Stress (psi)</th>
<th>Principal Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SX</td>
<td>SY</td>
<td>SZ</td>
</tr>
<tr>
<td>T</td>
<td>184930</td>
<td>124110</td>
<td>184650</td>
</tr>
<tr>
<td>C</td>
<td>185340</td>
<td>124120</td>
<td>185030</td>
</tr>
</tbody>
</table>
Table 7.3: Summary of Load vs. Deflection

<table>
<thead>
<tr>
<th>Load Steps</th>
<th>Load (lb.)</th>
<th>Deflection (in.)</th>
<th>Load (lb.)</th>
<th>Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>5.44E-02</td>
<td>600</td>
<td>0.163398</td>
</tr>
<tr>
<td>2</td>
<td>400</td>
<td>0.108834</td>
<td>1200</td>
<td>0.326796</td>
</tr>
<tr>
<td>3</td>
<td>800</td>
<td>0.217668</td>
<td>2400</td>
<td>0.653593</td>
</tr>
<tr>
<td>4</td>
<td>1200</td>
<td>0.326501</td>
<td>3600</td>
<td>0.980389</td>
</tr>
<tr>
<td>5</td>
<td>1600</td>
<td>0.435335</td>
<td>4800</td>
<td>1.30719</td>
</tr>
<tr>
<td>6</td>
<td>2000</td>
<td>0.544169</td>
<td>6000</td>
<td>1.63398</td>
</tr>
<tr>
<td>7</td>
<td>2400</td>
<td>0.653003</td>
<td>7200</td>
<td>1.96078</td>
</tr>
<tr>
<td>8</td>
<td>2800</td>
<td>0.761837</td>
<td>8400</td>
<td>2.28758</td>
</tr>
<tr>
<td>9</td>
<td>3200</td>
<td>0.87067</td>
<td>9600</td>
<td>2.61437</td>
</tr>
<tr>
<td>10</td>
<td>3600</td>
<td>0.979504</td>
<td>10800</td>
<td>2.94117</td>
</tr>
<tr>
<td>11</td>
<td>4000</td>
<td>1.08834</td>
<td>12000</td>
<td>3.26796</td>
</tr>
</tbody>
</table>

Figure 7-14: Load vs. Deflection
Table 7.4: Summary of the Deflection along the wall

<table>
<thead>
<tr>
<th>Load (lb.)</th>
<th>Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MWFRS</td>
</tr>
<tr>
<td>21.6</td>
<td>0.58574</td>
</tr>
<tr>
<td>43.2</td>
<td>0.71224</td>
</tr>
<tr>
<td>64.8</td>
<td>0.88637</td>
</tr>
<tr>
<td>86.4</td>
<td>1.0179</td>
</tr>
<tr>
<td>108</td>
<td>1.084</td>
</tr>
<tr>
<td>129.6</td>
<td>1.0803</td>
</tr>
<tr>
<td>151.2</td>
<td>1.0072</td>
</tr>
<tr>
<td>172.8</td>
<td>0.86963</td>
</tr>
<tr>
<td>194.4</td>
<td>0.69079</td>
</tr>
<tr>
<td>216</td>
<td>0.55739</td>
</tr>
</tbody>
</table>

Figure 7-15: Deflection along the panel of the wall of the emergency shelter
Table 7.5: Summary of Flexural Stress and Shear Stress along the wall for both models of MWFRS and C&C

| Span Length (in.) | Flexural Stress (psi) | | | Shear Stress (psi) | | |
|------------------|-----------------------|---|---|-------------------|---|
|                  | MWFRS | C&C | MWFRS | C&C | MWFRS | C&C |
| 21.6             | -497.12 | -1491.4 | -158.75 | -476.24 | 248.01 | 744.04 |
| 43.2             | 40.958 | 122.87 | 12.724 | 38.171 | -30.879 | -92.638 |
| 64.8             | 119.06 | 357.17 | 40.497 | 121.49 | -36.63 | -109.89 |
| 86.4             | 270.26 | 810.79 | 166.91 | 500.74 | -149.96 | -449.89 |
| 108              | 370.53 | 1111.6 | 360.48 | 1081.4 | -289.17 | -867.5 |
| 129.6            | 602.1 | 1806.3 | 548.57 | 1645.7 | -503.63 | -1510.9 |
| 151.2            | 579.9 | 1739.7 | 539.76 | 1619.3 | -493.63 | -1480.9 |
| 172.8            | 407.2 | 1221.6 | 409.27 | 1227.8 | -357.58 | -1072.7 |
| 194.4            | 190.62 | 571.86 | 231.37 | 694.11 | -187.01 | -561.02 |
| 216              | -27326 | -81978 | 10004 | 30013 | -11915 | -35744 |

Figure 7-16: Comparison of bending stress in Y-Direction along the span of MWFRS Model and C&C model
To conclude, the analysis provided above tests the whole proposal design of the emergency shelter including panels and steel cables. ANSYS 15.0, a software package for FEA was used to look at how the model behaved under the applied wind pressure loading.
The nodal displacements, stresses, and shear stresses were reported and gives a general idea of configuration design and analysis of panels and steel cables.

Table 7.5 shows that the maximum flexural stress in the interior of any panel for the C&C loading was 1806 psi. Near the edge of the panel, the maximum flexural stress for the C&C loading was approximately 82 ksi. These values compare favorably with the allowable stress of 400 ksi (2800 MPa) tabulated in Table 2.1 as recommended by ACI 440.2R-08.

From the results presented in this chapter, it is demonstrated that the steel cables and panels meet the conditions necessary for performance under the specified loading conditions. At 1.1 inches for MWFRS and 3.2 inches for C&C, the maximum deflection, and the design of the panel system performed in a such a way under high loading conditions, suggesting that there may be other application in which it can be used given the deflection requirements.
Chapter VIII

Conclusion

The risks posed by hurricanes and tornadoes are great. In this research, an innovative design was formulated that sought to protect people from the dangers posed by extreme wind forces. The Carbon Fiber Reinforced Polymer (CFRP) emergency shelter was modeled using FEA. The shelters were subjected to wind speeds of up 157 mph. The wind speed was taken from the ASCE 7-10. The emergency shelter was developed in such a way to take into the account the various variables involved in addressing post-disaster situations. The goal, in this development, was to create a lightweight, strong, versatile, and economic shelter that could withstand category five hurricane wind forces.

The effect of the wind load was investigated on the CFRP panels combined with steel cables. Comparison of these panels when modeled using shell and solid elements under various load cases showed that the finite element model predictions closely matched each other. Subsequently, the three dimensional solid elements were employed to model the emergency shelter.

The steel cables demonstrated that the optimal and appropriate positioning of the pre-tensioned loading of cables had significant influence on the performance of the shelter. In
the future, this research could direct future applications in relation to steel truss design and configuration.

Lastly, through combining panels and steel cables, the emergency shelter results demonstrated performance meeting criteria to fill the loading conditions needed for extreme wind speeds. The highest deflection detected was 3.2 inches for the components and cladding (C&C) model and a deflection of 1.1 inches for the main wind force resisting system (MWFRS) model. These results demonstrate that the emergency shelter design may withstand other types of extreme loading. However, the use of this system in other applications will depend upon the deflection thresholds of the design.

**Recommendations for Future Work**

(1) Expand the research work to investigate the type of soil for Charleston, South Carolina due to its location in a geographic location prone to both earthquakes and hurricanes. Based on this analysis, identify the size of arrowhead earth anchors needed for the emergency shelter and run a finite element analysis for the arrowhead earth anchors using ANSYS software.

(2) Expand the research work to investigate more about earthquakes in and around Charleston, South Carolina and test the emergency shelter through finite element analysis using ANSYS.

(3) To perform experiments in a suitable lab for every piece of the proposed design as has been discussed in this paper. This analysis was done through finite element means including individual panel under static different point bending load with different cases of load, steel cables in different categories, and panels connected with steel cables. After this,
future research would compare the experimental results with the results as they were predicted from ANSYS. Local manufacturers, professors, and groups of students could probably collaborate to get this work completed in a time efficient manner.

(4). Expand the design in terms of architecture and construct the emergency shelter on site using 3D max and Revit software. This would show how the emergency shelter arrives at the site, how the wheels descend for improved portability, and demonstrate how the automated emergency shelter parts complete construction without any help by using just one button to assemble it. This would show how the embedded wheels of the shelter would work and move the shelter from one site to another. This process would also take into account the exterior and interior design components such as the furniture (a simple stove and sink, a shower, a bed/table, and windows which can open and close). The Figure below was completed in AutoCAD software 2016. It gives an idea how the panel wall with steel cable would be designed to be experimented with in the lab.
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