Design optimization of Frp composite panel building systems: Emergency shelter applications

Nicholas M. Bradford

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Design Optimization of Frp Composite Panel Building Systems:

Emergency Shelter Applications

by

Nicholas M. Bradford

A dissertation submitted in partial fulfillment
of the requirements for the degree of
Doctor of Philosophy
Department of Civil and Environmental Engineering
College of Engineering
University of South Florida

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Date of Approval:
August 24, 2004

Keywords: interlocking, fiber reinforced polymers, high wind design, hurricane construction, rapid deployment buildings

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ACKNOWLEDGMENTS

The research reported was funded by a grant from the University of South Florida’s Center for Disaster Management and Humanitarian Assistance (CDMHA) through the Office of Naval Research. Dr. Mike Conniff, founding co-director of CDMHA initiated the project and served as project manager during the initial phase. Dr. Tom Mason, co-director CDMHA took over as project manager for the later phase. We are very grateful for their support and contribution to the project. We also thank Mr. Eric Matos, Deputy Director, CDMHA for his keen interest and close interaction with the research team.

Finally, the principal investigators wish to acknowledge the contribution of the other members of the research team: Dr. Gray Mullins, Steve Cooke, School of Architecture and Community Design, Dr. Jose Danon, Florida Department of Transportation and adjunct professor, Dr. Andres Torres Acosta currently with Instituto Mexicano del Transporte, Queretaro, Mexico and graduate students Mr. Timothy Kimball, School of Architecture and Community Design and Dr. Niranjan Pai, Department of Mechanical Engineering, University of South Florida.
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DESIGN OPTIMIZATION OF FRP COMPOSITE PANEL BUILDING SYSTEMS: EMERGENCY SHELTER APPLICATIONS

Nicholas M. Bradford, PE SE

ABSTRACT

Using advanced composites, an emergency shelter system has been designed. The system parameters are hurricane resistance to 138 mph wind velocity, simple erection, light weight, high durability and rapid construction. The project involves the solicitation of design proposals from several building system manufacturers and the development of an optimized emergency shelter system. The usage is well suited to pultruded members made from fiber reinforced polymers (FRP). Due to the anisotropic nature of FRP composites, a limited amount of research has been conducted to develop design optimization techniques for panels used in construction.

This project allows for the development of optimization techniques for use in pultruded FRP panel members. The Project consisted of a detailed literature review conducted of emergency building industry to assess the validity of existing shelter systems, a state of the art review of connection design in FRP structures with an emphasis on non-standard types of connectors (ie...snap type), systemic structural optimization of emergency shelter for building geometry, roof
configuration, foundation anchorage and building envelop, development of statistical methods for evaluation of viable existing emergency shelter systems.

Subsequent to the initial phase of the investigation, an interlocking FRP composite panel system was developed. The system was analyzed for local buckling, first ply failure and global deflection criteria using modified equations originally developed for open section members. The results were verified using Finite Element Methods analysis software.

The findings from the study indicate the need for a second phase in which the most promising available systems and the concept developed are fully tested to verify their capacity to withstand high wind forces including impact of wind borne debris.
1. GENERAL INTRODUCTION AND OVERVIEW

1.1 Introduction

In October of 1998, the nations of the Caribbean and Central America experienced an event that directly affected more than 3,000,000 people. This event was called Hurricane Mitch (Figure 1.1), and it resulted in more than 10,000 dead, with ten-fold that number injured and in need of medical treatment. As for the housing stock, Mitch was no less devastating, with damage or destruction of 335,823 homes throughout the Caribbean region.
In some areas, up to 90% of the agriculture and 80% of the potable water resources were lost (OFDA Report, 1998). Hurricane Mitch provides a stark reminder as to the mission of the United States in the post cold war era. In areas like Kuwait, Africa and Central America, we have become our brother's keeper, working to relieve the pain caused by the hands of fate and man. In the aftermath of Hurricane Mitch, more than $300 million was spent by the United States Government in relief to the Caribbean nations. Of that relief, $150 million was allotted to the Department of Defense to facilitate the conveyance of this aid (OFDA, 1998).

Primary to the facilitation of this relief is the issue of shelter (Figure 1.2). Shelter can come in many forms, be it as the canvas tent carried in battle, the mud hut on an open plain, or the wood framed box with a back yard and a white picket fence. In times of emergency, provision of shelter reduces to several questions: What materials are needed to build it? How long does it take to build?
How much does it cost? What environmental conditions can it withstand? How many people can it house? This project attempts to address some of these questions.

### 1.1.1 Project History

This report summarizes the findings of a one year study conducted by the University of South Florida to investigate emergency shelters suitable for hurricane-devastated regions in the Caribbean and Latin America. The study commenced on January 1, 2000 ending on December 31, 2000. Originally, the goal of the study was to develop a modular, light-weight, wind-resistant Fiber Reinforced Polymer (FRP) structure that could be fabricated using existing facilities at the Lemay Center for Composites Technology (LCCT), St. Louis, MO. Although LCCT collaborated actively with the University of South Florida during preparation of the proposal, they were unable to continue with the study beyond the first quarter. In view of this, fabrication aspects of the study were dropped and the primary focus became a review of what was currently available in terms of both FRP and traditional materials. An important secondary focus was the development of an in-house emergency shelter concept using FRP. Both developments are described in this report.
1.1.2 Project Goal

The overall goal of the project was to identify emergency shelters that utilize a “house in a box” concept in which all unassembled components can be packaged in a crate and conveyed to the site for erection by relief workers with minimal skills. The emergency shelter was to be designed to withstand hurricane force winds. Given the variability of soil conditions, focus was primarily on the building shell. One significant problem with this type of system is due to the magnitude and nature of the forces it must resist. Hurricane winds create pockets of wind pressure that can cause individual components to failure or the building to fall at the foundation connections, wall connections and roof connections (Figure 1.3). High wind pressures are typically resisted with heavy and/or stiff construction systems. These systems run contradictory to the restrictions placed on this investigation.

![Figure 1.3 Wind Pressure Effect on Buildings](image)

1.1.3 Methodology

Two methods were used to identify the best solution. First, a detailed search was conducted of all viable building systems currently available in the construction market. A “Request for Proposal” was sent to all interested parties and subsequent proposals were reviewed for compliance with shelter parameters provided by the military. Concurrently, an emergency shelter system was
developed by the research team using interconnected building panels made of FRP (Fiber Reinforced Polymers) as shown in Figure 1.4. All building submissions were reviewed using shelter parameters developed by the military and structural/architectural parameters developed by the research team.

Figure 1.4 FRP Panel Emergency Shelter

Design equations will be developed to calculate localized plate buckling, global member deflection and first-ply failure loads. Design equations will be dependent on two criteria; ply orientation and laminate stacking sequence. The fixed parameters in all equations will include panel geometry, laminate thickness, laminae thickness and material properties of individual plies. Objective functions
will be developed using each of the three performance equations as sub functions. The objective functions will then be optimized to provide the optimum laminate lay-up. Once the panel section has been optimized, it will be reviewed against the applied loads encountered in the emergency shelter building.

1.2 Shelter System Geometry

In order to develop an optimal emergency shelter system, a suitable building geometry was developed by the research team. The purpose of the building geometry was two-fold:

1) To provide a referential basis for conducting a side-by-side evaluation of available building systems during the industry review phase of the project.

2) To allow for an in-depth structural analysis of the environmental conditions (i.e., hurricane wind velocities, exposures) so as to develop a detailed picture of the required building component performance.

Initially, the base geometric parameters proposed by the United States Southern Command (USSOUTHCOM) called for a 24'-0" x 36'-0" (7.32 m x 10.97 m) rectangular box, having a wall height of 8'-0" (2.44 m). A preliminary wind analysis, for a wind velocity of 138 mph (222 km/hr) and design pressures developed using ASCE 7-98, “Minimum Design Loads for Buildings and Other Structures”, was conducted to ascertain the magnitude of forces exerted on the structure (Figure 1.5). As a result of this analysis, it was found that severe stress concentrations occurred at all building corners with magnitudes as high as 30,000 lbs (13,608 kg) of uplift force at some locations.
Based on these findings, it was decided that the standard shelter geometry should be reduced in size so as to minimize the forces experienced by component members and connections. Subsequent modifications led to the development of a 12'-0"x 24'-0" " (3.66 m x 7.32 m) standard box geometry with 8'-0" (2.44 m) wall heights. Further modifications led to the adoption of a 4:12 roof pitch and the use of 36" (0.914 m) wide openings on each of the long walls to facilitate the installation of doors and windows (Figure 1.6).

Figure 1.5 Preliminary Wind Analysis
Window and door placement were developed so as to allow for building adaptability according to usage. Specifically, the size and locations of openings allowed for the construction of two or more building units back to back, thus resulting in the creation of several interconnected ‘rooms’. Further, the development of the 4:12 monoslope roof was developed so as to allow for proper roof drainage and overhangs on the structure. It should be noted that the proposed roof slope constitutes a maximum and that most likely the finalized roof slope shall be much less in the field of operation. This roof slope facilitates the use of 12'-0" (3.66 m) and 8'-0" (2.44 m) members throughout the majority of the building.
The modifications to the basic geometry of the emergency shelter resulted in a significant reduction in the forces and moments experienced by members and connections throughout the structure. Specifically, the maximum forces at the worst case members and reactions were reduced by a factor of 3, bringing these forces within workable magnitudes.

1.3 Alternative Building Systems

A detailed review was conducted of the building industry to develop a list of viable candidates for use as emergency shelter structures. To facilitate this review, a side-by-side analysis of each system had to be conducted. This analysis was performed as a “Request for Proposal” (RFP) in which the prototype building shown in Figure 1.6 was submitted to each interested group. Each party was then required to submit a detailed proposal for review by the research team.

Throughout the RFP, the need for confidentiality was stressed with regard to all proprietary information and content. The research team stated a willingness to sign agreements regarding all proprietary information. It was pointed out that the research team had no interest in entering the composite manufacturing industry. It was stated that the sole interest was in the fulfillment of the design objectives for Office of Naval Research and the Military.
1.4 Shelter Industry Systems

Subsequent to the detailed industry wide review conducted by the research team, several types of emergency shelters systems were found to provide viable alternatives. Specifically, as a result of our investigation, it was concluded that the viable emergency shelters fell into three types of construction.

1) Standard Construction - New Materials: These systems emphasize the improved performance gained through the use of new materials. Such materials offer the user improved mechanical properties (on a localized basis), light weight, non-corrosive and non-metallic performance. Further, these systems attempt to use the new materials as direct substitutes for standard components in building systems. An example of this type of construction would be substituting FRP studs in a wood framed stud wall system or the use of styrofoam molds in lieu of masonry blocks in a filled masonry wall system. An RFP was sent to two manufacturers that fall into this category.

2) New Construction - New Materials: These systems develop new construction systems in an attempt to best utilize the performance characteristics of the new materials. Typical examples of this construction consist of the development of panelized wall and roof systems which are fabricated using FRP systems. An RFP was sent to six manufacturers in this category.

3) Alternate Systems: These systems within this category constitute a fully alternate system of construction, based on geometry, materials and
construction. Typical examples of this construction include monolithic domes and Yurts. An RFP was sent to two manufacturers that fall into this category.

1.5 Design Optimization

Concurrent to the industry-wide search for viable solutions to the emergency shelter problem, the research team is developing an optimized structural system for use in this situation. During our development of a design strategy, the following basic principles arose as being primary to the successful fulfillment of our design goals. These principles include Structural Performance, Erection Simplicity, Cost and Durability/Adaptability.

1) Structural Performance is not a simple question when it comes to emergency shelters. One must design the components to withstand extreme conditions without failure. In our situation, the primary environmental situation involves hurricane force winds and flooding. The complexity of the problem is aggravated by the weight restrictions placed upon the shelter to facilitate manual erection.

2) Erection Simplicity is assessed by the speed of erection and the skill requirements of the workers. Further complicating the issue are the questions of connection and foundation requirements. The optimal solution involves an integration of several functions into a single component.
3) Cost runs in an inversely proportional relationship to all other issues addressed during the design optimization process. This question can only be circumvented through innovation in either the areas of the materials or construction techniques used during erection.

4) Durability/Adaptability address the possible long term usage of the final buildings as ‘safe houses’ where primary facilities can be maintained during future disasters.

Based on the four primary issues listed above, it was decided that the optimum solution would address the problem as a material issue and a building component issue. In order to answer both questions, our focus was turned to the use of Fiber Reinforced Polymer materials, which offer design customization for specific applications.

1.6 Fiber Reinforced Polymers

Fiber Reinforced Polymer (FRP) materials provide an incredible opportunity for structural engineers. They are light weight, non-magnetic and corrosion resistant. They offer mechanical properties similar to those found in standard engineered materials such as steel, aluminum and concrete. Most important is the versatility of FRP materials. Specifically, by allowing engineers to vary both fiber and matrix parameters, FRP materials provide theoretically “exact” solutions to real-life structural problems.
For these reasons, FRP materials have experienced wide and varied utilization throughout several engineering fields. First developed for use by the Aerospace industry, FRP materials have become the standard material in components ranging from landing gear to jet engines to heat protection on space shuttles. Within the automobile industry, these materials have replaced metal components in vehicle body frames and engines.

Within the civil engineering industry, acceptance of FRP materials has not been quite as widespread. The reason for this lack of utilization may be found in the direct relationship placing performance against cost. In the construction industry, this relationship is often of primary interest to Engineer, Contractor and Owner. Further, the complex nature of FRP materials, which are anisotropic, nonhomogeneous and viscoelastic, prohibits their evolution as a viable alternative to the relatively simpler construction materials such as concrete or steel.

As a result, FRP materials have been relegated to performing specialized structural tasks where either the non-mechanical properties of FRP (ie...nonmetallic, corrosion resistant) are of primary concern or in retro-fitting applications where the utilization of steel or concrete is prohibitive. Further, the aesthetic versatility of FRP materials has led to their utilization in auxiliary structural systems such as building facade panels, handrails and fixtures.
1.7 Dissertation Contents

The remainder of this dissertation documents the full review and development process of the “Design & Optimization of FRP Composite Panel Building Systems” investigation. The sections of this dissertation are as follows:

1) BACKGROUND - EMERGENCY SHELTERS provides the reader with a detailed overview of the viable building systems involved in the evaluation process.

2) EXISTING SYSTEM REVIEW provides the reader with a review of four of the available building systems who responded to the RFP. The review is with respect to the parameters developed by the military and the structural/architectural parameters developed by the research team.

3) WIND ANALYSIS & BUILDING DESIGN provides the reader with an overview of the wind analysis conducted during the investigation as well as the preliminary systemic optimization that resulted in the prototype shelter geometry.

4) USF SYSTEM - ASSEMBLY provides the reader with step-by-step instructions for assembling the USF panelized FRP building as well as detailed component geometries and performance descriptions.

5) FIBER REINFORCED POLYMERS provides the reader with a detailed overview of advanced composites with an emphasis in the design of FRP materials.
6) DESIGN OPTIMIZATION OVERVIEW provides the reader with an overview of design optimization procedures for advanced composite materials.

7) DEVELOPMENT OF PANEL PERFORMANCE CRITERIA develops each of the objective functions to be used in the investigation.

8) COMPOSITE PANEL DESIGN - ANALYSIS / RESULTS reviews the optimization results using an ANSYS generated Finite Element Model for verification.

9) CONCLUSIONS AND RECOMMENDATIONS provides the reader with a finalized analysis of the viable system developed during the investigation.
2. BACKGROUND - EMERGENCY SHELTERS

2.1 Introduction

A detailed review was conducted of the building industry to develop a list of viable candidates for use as emergency shelter structures. To facilitate this review, a side-by-side analysis of each system had to be conducted. This analysis was performed as a “Request for Proposal” (RFP) in which the prototype building shown in Figure 1.6 was submitted to each interested group. Each party was then required to submit a detailed proposal for review by the research team.

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2.2 Emergency Shelter Industry Systems

Subsequent to the detailed industry wide review conducted by the research team, several types of emergency shelters systems were found to
provide viable alternatives. Specifically, as a result of our investigation, it was concluded that the viable emergency shelters fell into three types of construction.

1) **Standard Construction - New Materials:** These systems emphasize the improved performance gained through the use of new materials. Such materials offer the user improved mechanical properties (on a localized basis), light weight, non-corrosive and non-metallic performance. Further, these systems attempt to use the new materials as direct substitutes for standard components in building systems. An example of this type of construction would be substituting FRP studs in a wood framed stud wall system or the use of styrofoam molds in lieu of masonry blocks in a filled masonry wall system. An RFP was sent to three manufacturers that fall into this category.

2) **New Construction - New Materials:** These systems develop new construction systems in an attempt to best utilize the performance characteristics of the new materials. Typical examples of this construction consist of the development of panelized wall and roof systems which are fabricated using FRP systems. An RFP was sent to six manufacturers in this category.

3) **Alternate Systems:** These systems within this category constitute a fully alternate system of construction, based on geometry, materials and construction. Typical examples of this construction include monolithic domes and Yurts. An RFP was sent to two manufacturers that fall into this category.
To further illustrate the variety and nature of the systems investigated during this project, we have enclosed the following overview of each emergency shelter system manufacturer, involved in the initial Request for Proposals. Please note that not all of the manufacturers provided below submitted proposals at the current time.

2.2.1 CoreFlex

FRP shapes, manufactured through the pultrusion process and used as individual structural members in a panelized construction system. Specifically, the FRP panels are hollow, narrow box shapes with internal ribs.

2.2.1.1 Main Contacts:

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Corflex International, Inc.
P.O. Box 830
Wilsonville, OR 97070
Phone (503) 582 - 8593 Fax (503) 582 - 9373

2.2.2 Dr. Ayman Mosallam, P.E.

Panelized construction system consisting of FRP composite sandwich materials. Adjacent panels to be interconnected using both epoxy adhesives and mechanical connections facilitated with universal connectors.

2.2.2.1 Main Contacts:

Dr. Ayman Mosallam, P.E.
Department of Civil & Env. Eng. & Mechanical Eng.
California State University, Fullerton
Phone (714) 278 - 2297 Fax (714) 278 - 3916
2.2.3 Leading Edge Earth Products, Inc. (Leep)

The system consists of composite panels composed of steel face sheet bonded to a foam core using sandwich construction. The panelized construction is supplemented using a metal frame system in which the panel sections are inserted. The building anchorage is provided through a system of manually placed deep set earth anchors.

![Figure 2.1 LEEP System](from www.Leepinc.com)

2.2.3.1 Main Contacts:

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Leading Edge Earth Products, Inc.  
P.O. Box 38636  
Greensboro, NC 27438  
Phone (336) 288 - 5668  Fax (336) 288 - 4407  
Website: [www.leepinc.com](http://www.leepinc.com)  E-Mail: [rama@nr.infi.net](mailto:rama@nr.infi.net)
2.2.4 DuraKit Shelters

The system consists of corrugated fiberboard that is factory-coated and treated to make a durable shelter with a fireproof interior and a weatherproof exterior. The fiberboard (similar to cardboard construction) is assembled as composite panels. The panels are connected to adjacent members using an adhesive system. Roof and floor connections are facilitated through the use of mechanically connected track systems.

![Figure 2.2 DuraKit Shelters](image)

System Photographs: (from www.DuraKit.com)

2.2.4.1 Main Contacts:

Tim Wimsatt  
DuraKit Shelters  
2785 Hwy #27, P.O. Box 200  
Bond Head, Ontario, Canada L0G 1B0  
Phone (905) 778 - 0005  
Fax (905) 778 - 0054  
Website: [www.DuraKit.com](http://www.DuraKit.com)  
E-Mail: [shelters@DuraKit.com](mailto:shelters@DuraKit.com)
2.2.5 Monolithic Dome Institute

Dome shaped construction that is conducted through the use of an air filled permanent plastic form. Once the form has been inflated on-site, concrete is sprayed on the exterior and exterior to create a monolithic structural system.

![Monolithic Dome Institute Diagram](image)

Figure 2.3 Monolithic Dome Institute

System Photographs: (from www.Monolithicdome.com)

2.2.5.1 Main Contacts:

David B. Smith, Monolithic Dome Institute
177 Dome Park Place
Italy, Texas 76651
Phone (972) 483 - 7423 Fax (972) 483 - 6662
Website: [www.monolithicdome.com](http://www.monolithicdome.com) E-Mail: mail@monolithic.com
2.2.6 Pacific Yurt, Inc.

Cylindrical shaped building geometry with a conical roof system. This system consists of a treated canvas material installed over a wood and plexiglass frame system. Anchorage consists of mechanical connections from roof to wall and wall to floor.

System Photographs: (from www.yurts.com)

2.2.6.1 Main Contacts:

Alan Bair  
Pacific Yurt, Inc.  
77456 Hwy 99, South  
Cottage Grove, Oregon 97424  
Phone (800) 944 - 0240 Fax (541) 942 - 0508  
Website: www.yurts.com E-Mail: pacyurt@yurts.com
2.2.7 Ambiente Housing Systems, Inc.

The house consists of a system of panels for the walls and roof, which are structurally reinforced with a comprehensive network of flexible glass fiber-rods throughout the entire structure and anchored to the ground through a structural concrete slab foundation in such a way as to withstand hurricane force winds.

Figure 2.5 Ambiente Homes

System Photographs: (from www.ambientehomes.com)

2.2.7.1 Main Contacts:

Wayne DeWald  
P.O. Box 70005, Suite 266  
Fajardo, P.R. 00738 - 70005  
Phone (787) 889 - 1362 Fax (787) 889 - 2944  
Website: www.ambientehomes.com E-Mail: ambientehomes.com
2.2.8 American Structural Composites, Inc.

The system consists of six foot wide panels incorporating phenolic fiberglass laminated sheets that are bonded with epoxy adhesives to an internal frame made up of extruded I-beams, wall to wall connectors and a base insert.

System Photographs: (from www.asc-housing.com)

2.2.8.1 Main Contacts:

Max Weir
American Structural Composites
905 Southern Way, Suite 201
Sparks, NV 89431
Phone (775) 355 - 4444 Fax (775) 355 - 4455
Website: www.asc-housing.com Email: info@asc-housing.com
2.2.9 Modular Engineering Company

The system consists of composite panels composed of steel face sheet bonded to a foam core using sandwich construction. The panelized construction is supplemented using a metal frame system in which the panel sections are inserted.

Figure 2.7 Modular Engineering

System Photographs: (from www.modularengineering.com)

2.2.9.1 Main Contacts:

Bob McGee
Modular Engineering Company
P.O. Box 8241
Erie, PA 16505
Phone (814) 838 - 6551 Fax (814) 833 - 2577
Website: www.modularengineering.com E-Mail: info@modularengineering.com
2.2.10 Royal Building System

The system consists of stay-in-place formwork constructed of Fiber-Reinforced-Plymers. The structure is provided using a post and beam reinforced concrete system.

System Photographs: (from www.rbsdirect.com)

2.2.10.1 Main Contacts:

Royal Landmark Structures L.L.C.
16701 Greenspoint Park Drive
Suite 120
Houston, Texas 77060
Phone: (281) 872-0200
Fax: (281) 875-8935
Website: www.rbsdirect.com
E-Mail: info@rbsdirect.com
2.2.11 Futuristic Worldwide Homes / Lemay Center

FRP shapes, manufactured through the pultrusion process and used as individual structural members in a standard construction system. Specifically, the overall system mirrors typical wood framed house construction. This system substitutes the FRP members for the wood studs, top plate and bottom plate. The system also includes styrofoam board insulation that is inserted between each stud to enhance the insulating and structural performance of the system.

System Photographs: (from www.lemay.umr.edu)

2.2.11.1 Main Contacts:

Advanced Composite Structures, LLC
2171 Eagle Creek Road
Barnhart, MO 63012
Phone (314) 475 - 4928 Fax (314) 475 - 3317
2.3 Summary

Eleven viable building systems were found during an extensive review of the shelter manufacturing industry. *Standard Construction - New Material* were two viable systems that emphasized the improved performance gained through the use of new materials. *New Construction - New Material* systems constituted the bulk of the buildings which developed new construction systems in an attempt to best utilize the performance characteristics of the new materials. *Alternate* systems were two shelter systems that constituted a fully alternate system of construction, based on geometry, materials and construction.
3. EXISTING SYSTEM REVIEW

3.1 Introduction

A total of eleven available emergency building systems were identified in Chapter 2. To evaluate their suitability for use in hurricane devastated regions and to facilitate direct comparison of the disparate systems, a "Request for Proposal" (RFP) was sent to all manufacturers. In the RFP, the plan, elevation and wind load information was provided along with a request to address specific non-structural and structural parameters that would be used in the evaluation. This chapter reviews the responses that were submitted to the research team.

The evaluation presented in this chapter is based on military, structural and architectural parameters. The non-structural parameters were developed by the military while the structural and architectural parameters were developed by the research team. Sections 3.2-3.7 cover the non-structural and structural reviews and subsequent conclusions reached with respect to each viable building system.
3.2 Military Parameters

The nine military parameters (Table 3.1-3.9) focused on the on-site construction of the building shell, including the installation of all windows and doors. These parameters did not address ancillary systems such as electrical or mechanical that are included in the architectural review. Nor did the parameters evaluate construction of the foundation systems (ACT, 2000). In all cases, a higher numerical value signified better performance, e.g. on a scale of 1-5, 5 was the best.

3.2.1 Cost

The cost of the structure includes the cost of all materials, training, special tools and vendor representatives. Cost also includes all work/materials necessary to erect a waterproof structural shell as specified in the building plans. Cost were to be provided both in terms of the specific project as well as on a per square foot basis. The weighting parameter for cost in the overall evaluation was 1.

Table 3.1 Military Parameters - Cost

<table>
<thead>
<tr>
<th>Value</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>$0 - $17.36 / ft² ($0 - $186.87 / m²)</td>
</tr>
<tr>
<td>4</td>
<td>$17.37 - $34.72 / ft² ($186.87 - $373.52 / m²)</td>
</tr>
<tr>
<td>3</td>
<td>$34.73 - $52.08 / ft² ($373.52 - $560.12 / m²)</td>
</tr>
<tr>
<td>2</td>
<td>$52.09 - $69.44 / ft² ($560.12 - $746.68 / m²)</td>
</tr>
<tr>
<td>1</td>
<td>$69.45 and up / ft² ($746.68 and up / m²)</td>
</tr>
</tbody>
</table>
3.2.2 Erection Time

The amount of time required for erection of the structure is crucial. Erection time is estimated as the fastest time a trained crew erects a structure that provides weather tight, hurricane-resistant protection. The weighting factor for this parameter in 3.

<table>
<thead>
<tr>
<th>Value</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>&lt; 20 man hours</td>
</tr>
<tr>
<td>4</td>
<td>21 - 40 man hours</td>
</tr>
<tr>
<td>3</td>
<td>41 - 60 man hours</td>
</tr>
<tr>
<td>2</td>
<td>61 - 80 man hours</td>
</tr>
<tr>
<td>1</td>
<td>&gt; 81 man hours</td>
</tr>
</tbody>
</table>

3.2.3 Constructibility

This is measured in terms of the number of untrained personnel required to erect the structure. Further, no single component should be too heavy; maximum weight 80 lb (36.29kg) to be maneuvered on-site by two female personnel. The weighting factor for this parameter is 3.
Table 3.3 Military Parameters - Constructibility

<table>
<thead>
<tr>
<th>Value</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>&lt; 20 man hours</td>
</tr>
<tr>
<td>4</td>
<td>21 - 40 man hours</td>
</tr>
<tr>
<td>3</td>
<td>41 - 60 man hours</td>
</tr>
<tr>
<td>2</td>
<td>61 - 80 man hours</td>
</tr>
<tr>
<td>1</td>
<td>&gt; 81 man hours</td>
</tr>
</tbody>
</table>

3.2.4 Durability

This measures the manufacturer’s confidence in the components. Typically, it is based on the manufacturer’s warranty (in years). For the purpose of the parametric evaluation, this issue is given a parameter weight of 1.

Table 3.4 Military Parameters - Durability

<table>
<thead>
<tr>
<th>Value</th>
<th>Durability</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>5 years or more</td>
</tr>
<tr>
<td>4</td>
<td>4 to 5 years</td>
</tr>
<tr>
<td>3</td>
<td>3 to 4 years</td>
</tr>
<tr>
<td>2</td>
<td>2 to 3 years</td>
</tr>
<tr>
<td>1</td>
<td>1 year to 2 years</td>
</tr>
</tbody>
</table>

3.2.5 System Complexity

This is based on the number of days required to train construction personnel, the number of trainers required, and the number of building components utilized in the construction. This parameter was given a weight of 1.
Table 3.5 Military Parameters - Complexity

<table>
<thead>
<tr>
<th>Value</th>
<th>Complexity</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Excellent</td>
</tr>
<tr>
<td>3</td>
<td>Good</td>
</tr>
<tr>
<td>2</td>
<td>Fair</td>
</tr>
<tr>
<td>1</td>
<td>Poor</td>
</tr>
</tbody>
</table>

System complexity is best described as a combination of the Erection Time and Constructibility parameters. The rating system for system complexity is clarified as:

1) **Excellent**: System can be constructed from component form with less than one hour of instruction. No specialized tools and construction skills are required for erection.

2) **Good**: System construction requires one to four hours of instruction to complete erection and fabrication. Minimal specialized tools (ie...electric drills, pneumatic tools) and construction skills (ie...rough carpentry) are required for erection and fabrication.

3) **Fair**: System construction requires five to eight hours of instruction to complete erection and fabrication. Specialized tools (ie...electric drills, pneumatic tools) and construction skills (ie...rough carpentry, flat masonry work) are required for erection and fabrication.

4) **Poor**: System construction requires more than eight hours of instruction to complete erection and fabrication. Specialized tools and equipment...
(ie...cranes, concrete pumps) and construction skills (ie...survey work, masonry, steel erection) are required for erection and fabrication.

**3.2.6 Material / Construction Adaptability**

This is based on the materials required to be supplied by the host nation (ie...concrete, wood, etc... may not be available in host country). This parameter was given a weight of 1.

<table>
<thead>
<tr>
<th>Value</th>
<th>Material Availability</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Excellent</td>
</tr>
<tr>
<td>3</td>
<td>Good</td>
</tr>
<tr>
<td>2</td>
<td>Fair</td>
</tr>
<tr>
<td>1</td>
<td>Poor</td>
</tr>
</tbody>
</table>

Table 3.6 Military Parameters - Material Availability

Material Availability describes the dependence of the shelter construction on materials and services provided by the host nation. This parameter is of crucial importance since the proposed post-disaster usage would prohibit the production and conveyance of construction materials to the job site. The rating system for system complexity is clarified as:

1) Excellent: System can be constructed using no supplemental materials provided by the host nation. System can be constructed using no supplemental construction services provided by the host nation.

2) Good: System can be constructed with or without minor supplemental materials (ie.. electrical wiring, concrete) provided by the host nation.
System can be constructed using no supplemental services provided by the host nation.

3) Fair: System construction requires minor supplemental materials (i.e., electrical wiring, concrete) provided by the host nation. System can be constructed with or without supplemental construction services, depending on extent of shelter finishes.

4) Poor: System construction requires significant supplemental materials (i.e., steel framing, wood framing, concrete) provided by the host nation. System requires supplemental construction services, provided by the host nation.

### 3.2.7 Transportability

This is based on the number of standard MILVAN containers and/or C130 pallets required to transport the components. Each container has the storage dimensions of 8'-0"x 8'-0"x 20'-0" (2.44m x 2.44m x 6.1m). For the purpose of the parametric evaluation, this issue was given a parameter weight of 2.

<table>
<thead>
<tr>
<th>Value</th>
<th>Transportability</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>4 Units or more / C130</td>
</tr>
<tr>
<td>4</td>
<td>2 - 3 Units / C130</td>
</tr>
<tr>
<td>3</td>
<td>1 Unit / C130</td>
</tr>
<tr>
<td>2</td>
<td>1 - 2 C130 / Unit</td>
</tr>
<tr>
<td>1</td>
<td>3 or more C130 / Unit</td>
</tr>
</tbody>
</table>
3.2.8 Building System Flexibility

This measures how flexible the building system is to adapt to changes in building geometry, wall height, roof configuration, etc. For the purpose of the parametric evaluation, this issue was given a parameter weight of 1.

Table 3.8 Military Parameters - Flexibility

<table>
<thead>
<tr>
<th>Value</th>
<th>Flexibility</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Excellent</td>
</tr>
<tr>
<td>3</td>
<td>Good</td>
</tr>
<tr>
<td>2</td>
<td>Fair</td>
</tr>
<tr>
<td>1</td>
<td>Poor</td>
</tr>
</tbody>
</table>

Building System Flexibility describes the ability of the system to adapt to changes in the roof geometry, wall heights, opening locations, and building footprint. The rating system for system complexity is clarified as:

1) Excellent: Roof system can be modified with no engineering and changes to the members sizes and connections, and Wall system can be modified with no engineering and changes to the members sizes and connections, and Building geometry can be modified with no engineering and changes to the members sizes and connections.

2) Good: Roof system can be modified with minimal engineering and changes to the members sizes and connections, or Wall system can be modified with minimal engineering and changes to the members sizes and connections, or
Building geometry can be modified with minimal engineering and changes to the members sizes and connections.

3) Fair: Roof system can be modified with significant engineering and changes to the members sizes and connections, or Wall system can be modified with significant engineering and changes to the members sizes and connections, or Building geometry can be modified with significant engineering and changes to the members sizes and connections.

4) Poor: Roof system cannot be modified, or wall system cannot be modified, or building geometry cannot be modified.

3.2.9 Ease of Maintenance

Measures the ease with which the erected structures can be maintained and repaired. Further, this parameter addresses the special tools and training required to facilitate the maintenance and repair of these structures. For the purpose of the evaluation, this issue was given a parameter weight of 1.

Table 3.9 Military Parameters - Maintenance

<table>
<thead>
<tr>
<th>Value</th>
<th>Maintenance</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Excellent</td>
</tr>
<tr>
<td>3</td>
<td>Good</td>
</tr>
<tr>
<td>2</td>
<td>Fair</td>
</tr>
<tr>
<td>1</td>
<td>Poor</td>
</tr>
</tbody>
</table>
Maintenance describes the amount of monitoring and upkeep required to facilitate the usable life of the building system. The rating system for system complexity is clarified as:

1) **Excellent**: System requires less than one hour per month of monitoring and upkeep during the usable lifetime of the building system. Requires no major maintenance job (ie...roof replacement) during the usable lifetime of the shelter.

2) **Good**: System requires between one and two hours per month of monitoring and upkeep during the usable lifetime of the building system. Requires one major maintenance job (ie...roof replacement) during the usable lifetime of the shelter.

3) **Fair**: System requires between two and four hours per month of monitoring and upkeep during the usable lifetime of the building system. Requires one major maintenance job (ie...roof replacement) during the usable lifetime of the shelter.

4) **Poor**: System requires more than four hours per month of monitoring and upkeep during the usable lifetime of the building system. Requires more than one major maintenance job (ie...roof replacement) during the usable lifetime of the shelter.

### 3.3 Structural Parameters

A significant part of the review concerned structural performance (see Figure 3.1) of the building under wind loading. The specific conditions addressed was 138 mph (222 km/hr) hurricane wind loads, as determined by The American Society of
Civil Engineers (ASCE7 - 98), “Minimum Design Loads for Buildings and Other Structures”. Further, the buildings were considered essential structures subjected to the worst environmental and geographical conditions. Each of the structural parameters is viewed as a pass / fail screening criteria. To conduct this review, the following structural information was required:

3.3.1 Flexural Capacity

This information required structural calculations and/or experimental test data showing flexural performance up to component failure. Further, information on load - deflection performance had to be submitted. This information was used to verify wind induced suction force resisting capacity of roof members and wall members. Construction details were also required showing standard member profiles, material strengths and section properties.
3.3.2 In-Plane Shear

This information required structural calculations and/or experimental test data showing in-plane shear performance up to failure of standard wall assembly section (i.e., pounds per linear foot). This information was used to verify wind shear resisting capacity of system.

3.3.3 Connection Capacity - Roof to Wall

This information required structural calculations and/or experimental test data showing the performance up to failure of standard roof to wall connections. Construction details were also required to show how the connection is built on-site.

3.3.4 Connection Capacity - Wall to Foundation

This information required structural calculations and/or experimental test data showing the performance up to failure of standard wall to foundation connections. Construction details were also required to show how the connection is built on-site.

3.3.5 Connection Capacity - Member to Member

This information required structural calculations and/or experimental test data showing the performance up to failure of all other connections. Construction details were also required to show how the connection is built on-site.
3.4 Screening Criteria

As per the investigation conducted by the Military, four non-structural pass / fail criteria were set as performance minimums. The four criteria were:

3.4.1 Construction Time

This criteria is based on the crucial aspect of rapid construction. The construction time excludes foundation work and is based on the complete erection of the building envelop. The pass / fail criteria is 15 days.

3.4.2 Transportability

Due to the need to use these systems on a global basis, it is crucial that any system be able to be flown into the theater of operation. The pass / fail criteria is Air transport.

3.4.3 Personnel Resources

The criteria is based on the availability of Military personnel to facilitate the erection of the buildings. The pass / fail criteria is based on standard company size and is set at a maximum of 12 soldiers and 96 man hours.

3.4.4 Durability

Due to the emergency nature of usage, it is crucial that shelters last for a minimum time period. The criteria is based on a minimum 1 year warrantee.
3.5 Decision Matrix

The building systems were evaluated using the eighteen parameters reviewed in the two previous sections. The evaluation methodology is based on the process utilized by USSOUTHCOM in a similar project conducted to evaluate proposals for Sea Hut building to be constructed in Yugoslavia (ACT, 2000). These parameters were developed for the Theater Contingency Operation (TCO) and were for a 16'-0" x 32'-0" (4.88m x 9.75m) shelter building. While the current investigation utilizes a 12'-0" x 24'-0" (3.66m x 7.32m) building, it is reasonable to assume that the two projects are similar in usage and design exposure.

Table 3.10 summarizes the decision matrix for this study. Note that the weighting factors used in this evaluation are identical to those used in the TCO evaluation, with the exception being the weighting factor used for Constructibility (3.0 instead of 1.0). The additional weighting value was used based on the importance of erection speed and simplicity in the overall success of the system.
Table 3.10 Decision Matrix

<table>
<thead>
<tr>
<th>Construction Category</th>
<th>DuraKit</th>
<th>LEEP</th>
<th>CoreFlex</th>
<th>Futuristic Homes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Screening Criteria</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constructibility: &lt; 15 Days</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Transportability: Air</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Resources: &lt; 12 Soldiers</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Durability: 1 Year</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td><strong>Structural Criteria</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural / Bending Capacity</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td>In-Plane Shear Capacity</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td>Roof - Wall Connections</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Fail</td>
</tr>
<tr>
<td>Wall - Foundation Connections</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Fail</td>
</tr>
<tr>
<td>Adjacent Panel Connections</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
<td>Pass</td>
</tr>
<tr>
<td><strong>Military Parameters</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cost</td>
<td>5</td>
<td>3</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Time (3x)</td>
<td>3</td>
<td>4</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Constructibility (3x)</td>
<td>2</td>
<td>3</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Durability</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Complexity</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Material Availability</td>
<td>5</td>
<td>3</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Transportability (2x)</td>
<td>5</td>
<td>3</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td>Flexibility</td>
<td>4</td>
<td>5</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Maintenance</td>
<td>4</td>
<td>5</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>53</td>
<td>51</td>
<td>50</td>
<td>25 (Fail)</td>
</tr>
</tbody>
</table>
3.6 System Proposal Information

Proposals were solicited from twelve alternate construction manufacturers. The solicited systems represented both alternate and standard construction materials. Of the twelve solicited companies, proposal packages were received from four companies. The following information represents the full proposal process with respect to each of the alternate building systems.

3.6.1 CoreFlex

FRP shapes, manufactured through the pultrusion process and used as individual structural members in a panelized construction system. Specifically, the FRP panels are hollow, narrow box shapes with internal ribs. Adjacent panels are connected to each other using interlocking connector shapes. The system allows for the hollow portions of the panels to be used for conduit installation or foam insulation.

3.6.1.1 Main Contacts:

Richard J. Alli, Sr.  Pierre Jordan
Corflex International, Inc.  Jordex Engineering
P.O. Box 830
Wilsonville, OR 97070
Phone (503) 582 - 8593  Phone (562) 590 - 7334

A summary of the information received relating to the military and structural parameters may be found in Table 3.11 and Table 3.12. Inspection of these tables indicate that much of the required information relating to structural performance was not provided.
### Table 3.11 Military Parameter Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1. Cost</td>
<td>$18,300 or $63.54 / sq.ft.</td>
<td>Includes Roof, Wall, and Uplift Anchors. No door/window, foundation or ancillary systems.</td>
</tr>
<tr>
<td>A2. Erection Time</td>
<td>5 workers for 5 hrs. 25 man hours</td>
<td>No Comments</td>
</tr>
<tr>
<td>A3. Constructibility</td>
<td>No submission (see A2. parameter)</td>
<td>No Comments</td>
</tr>
<tr>
<td>A4. Durability</td>
<td>35 year warrantee on panels</td>
<td>No Comments</td>
</tr>
<tr>
<td>A5. System Complexity</td>
<td>One hour of training required</td>
<td>No Comments</td>
</tr>
<tr>
<td>A6. Mat Constr.</td>
<td>No information</td>
<td>Floor panels can be provided in areas where concrete is unavailable.</td>
</tr>
<tr>
<td>A7. Transportability</td>
<td>5 units fit in 8'-0&quot;x 8'-0&quot;x 20'-0&quot;</td>
<td>Panels weigh under 80 lbs.</td>
</tr>
<tr>
<td>A8. Building System Flexibility</td>
<td>No information</td>
<td>Panels may be added and size expanded without changing the design.</td>
</tr>
<tr>
<td>A9. Ease of Maintenance</td>
<td>Maintenance free materials.</td>
<td>Class 1 fire retardant, insensitive to mildew, termite &amp; rodent proof.</td>
</tr>
</tbody>
</table>

### Table 3.12 Structural Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Capacity</th>
<th>Method of Verification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1. Panel Bending</td>
<td>160 mph wind with 2.8&quot; deflection.</td>
<td>No Calculations Received</td>
<td>Acceptable panel bending based on foam filled core system. This is not part of the base system and has additional costs involved.</td>
</tr>
<tr>
<td>B2. Wall Shear</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
<tr>
<td>B3. Roof to Wall Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>Provided using (8) marine grade stainless steel cables from roof to foundation.</td>
</tr>
<tr>
<td>B4. Wall to Footing Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>Each cable rated for 5000 lbs tensile load.</td>
</tr>
<tr>
<td>B5. Adjacent Panel Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
</tbody>
</table>
3.6.2 Leading Edge Earth Products, Inc. (Leep)

The system consists of composite panels composed of steel face sheet bonded to a foam core using sandwich construction. The panelized construction is supplemented using a metal frame system in which the panel sections are inserted. The building anchorage is provided through a system of manually placed deep set earth anchors. Table 3.13 and Table 3.14 summarize the information submitted for review.

3.6.2.1 Main Contacts:

Bill Oakes
Leading Edge Earth Products, Inc.
P.O. Box 38636
Greensboro, NC 27438
Phone (336) 288 - 5668 Fax (336) 288 - 4407
Website: www.leepinc.com E-Mail: rama@nr.infi.net
### Table 3.13 Military Parameter Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1. Cost</td>
<td>$13,091 or $45.45 / sq.ft.</td>
<td>Includes Roof, Wall, Doors, Windows and Uplift Anchors. No foundation or ancillary systems.</td>
</tr>
<tr>
<td>A2. Erection Time</td>
<td>4 workers for 8 hrs. 32 man hours</td>
<td>No Comments</td>
</tr>
<tr>
<td>A3. Constructibility</td>
<td>No submission (see A2. parameter)</td>
<td>Each additional 2 man team reduces erection time by 2 hours.</td>
</tr>
<tr>
<td>A4. Durability</td>
<td>5 year warrantee on panels</td>
<td>No Comments</td>
</tr>
<tr>
<td>A5. System Complexity</td>
<td>Factory 2 man team conducts all training.</td>
<td>No Comments</td>
</tr>
<tr>
<td>A6. Mat/Constr. Tech. Adaptability</td>
<td>All materials are provided.</td>
<td>No Comments</td>
</tr>
<tr>
<td>A7. Transportability</td>
<td>1 unit fits in 8'-0&quot;x 8'-0&quot;x 20'-0&quot;</td>
<td>Panels weigh under 80 lbs.</td>
</tr>
<tr>
<td>A8. Building System Flexibility</td>
<td>No information</td>
<td>Panels may be added and size expanded without changing the design.</td>
</tr>
<tr>
<td>A9. Ease of Maintenance</td>
<td>Minimal maintenance (painting).</td>
<td>No Comments</td>
</tr>
</tbody>
</table>

### Table 3.14 Structural Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Capacity</th>
<th>Method of Verification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2. Wall Shear</td>
<td>2,000 lbs per ft. of wall</td>
<td>Testing (Univ. of Washington, 1999)</td>
<td>No Comments</td>
</tr>
<tr>
<td>B3. Roof to Wall Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>Uses steel skeleton frame anchor bolted to the foundation.</td>
</tr>
<tr>
<td>B4. Wall to Footing Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>Uses steel skeleton frame anchor bolted to the foundation.</td>
</tr>
<tr>
<td>B5. Adjacent Panel Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
</tbody>
</table>
3.6.3 DuraKit Shelters

The system consists of corrugated fiberboard that is factory-coated and treated to make a durable shelter with a fireproof interior and a weatherproof exterior. The fiberboard (similar to cardboard construction) is assembled as composite panels. The panels are connected to adjacent members using an adhesive system. Roof and floor connections are facilitated through the use of mechanically connected track systems. Table 3.15 and Table 3.16 summarize the submitted information.

Figure 3.3 DuraKit Shelters

3.6.3.1 Main Contacts:

Tim Wimsatt
DuraKit Shelters
2785 Hwy #27, P.O. Box 200
Bond Head, Ontario Canada L0G 1B0
Phone (905) 778 - 0053 Fax (905) 778 - 0054
Website: www.DuraKit.com E-Mail: shelters@DuraKit.com
Table 3.15 Military Parameter Summary

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1. Cost</td>
<td>$6,624 or $23.00 / sq.ft.</td>
<td>Includes Roof, Wall, and Uplift Anchors. No door/window, foundation or ancillary systems.</td>
</tr>
<tr>
<td>A2. Erection Time</td>
<td>2 workers for 30 hrs. 60 man hours</td>
<td>No Comments</td>
</tr>
<tr>
<td>A3. Constructibility</td>
<td>80 man hours for unskilled workers</td>
<td>No Comments</td>
</tr>
<tr>
<td>A4. Durability</td>
<td>5 year warrantee on panels.</td>
<td>No Comments</td>
</tr>
<tr>
<td>A5. System Complexity</td>
<td>20 - 25 students / trainer for 3 days.</td>
<td>No Comments</td>
</tr>
<tr>
<td>A6. Mat/Constr. Adaptability</td>
<td>No information</td>
<td>Floor beams can be provided in areas where concrete is unavailable.</td>
</tr>
<tr>
<td>A7. Transportability</td>
<td>5 units fit in 8'-0&quot;x 8'-0&quot;x 20'-0&quot;</td>
<td>Panels weigh 72 lbs maximum.</td>
</tr>
<tr>
<td>A8. Building System Flexibility</td>
<td>No information</td>
<td>System has been designed specifically for geometry shown.</td>
</tr>
</tbody>
</table>

Table 3.16 Structural Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Capacity</th>
<th>Verification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1. Panel Bending</td>
<td>52.6 psf (141 mph wind velocity, BOCA)</td>
<td>Testing (Univ. of Western Ontario) 1999</td>
<td>Acceptable panel bending based on gravity and lateral wind loading.</td>
</tr>
<tr>
<td>B2. Wall Shear</td>
<td>36.4 N/mm In-plane shear (2494 lb/foot)</td>
<td>Testing (Univ. of Western Ontario) 1999</td>
<td>No Comments</td>
</tr>
<tr>
<td>B3. Roof to Wall Connections</td>
<td>86.7 psf</td>
<td>Testing (Univ. of Western Ontario) 1999</td>
<td>Provided using screws between \ component panels.</td>
</tr>
<tr>
<td>B4. Wall to Footing Connections</td>
<td>86.7 psf</td>
<td>Testing (Univ. of Western Ontario) 1999</td>
<td>Failure caused by delamination of composite layers at connections.</td>
</tr>
<tr>
<td>B5. Adjacent Panel Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
</tbody>
</table>
3.6.4 Futuristic Worldwide Homes / Lemay Center

FRP shapes, manufactured through the pultrusion process and used as individual structural members in a standard construction system. Specifically, the overall system mirrors typical wood framed house construction. This system substitutes the FRP members for the wood studs, top plate and bottom plate. Table 3.17 and Table 3.18 summarize the submitted information.

3.6.4.1 Main Contacts:

Advanced Composite Structures, LLC
2171 Eagle Creek Road
Barnhart, MO 63012
Phone (314) 475 - 4928  Fax (314) 475 - 3317

Table 3.17 Military Parameter Summary

50
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1. Cost</td>
<td>$26,000 or $30.09 / sq.ft.</td>
<td>Includes Roof, Wall, and Uplift Anchors. No door/window, foundation or ancillary systems.</td>
</tr>
<tr>
<td>A2. Erection Time</td>
<td>8 workers for 16 hrs. 128 man hours</td>
<td>No Comments</td>
</tr>
<tr>
<td>A3. Constructibility</td>
<td>No submission (see A2 parameter)</td>
<td>No Comments</td>
</tr>
<tr>
<td>A4. Durability</td>
<td>35 year warrantee on panels</td>
<td>No Comments</td>
</tr>
<tr>
<td>A5. System Complexity</td>
<td>One hour of training required</td>
<td>No Comments</td>
</tr>
<tr>
<td>A6. Mat/Constr. Tech. Adaptability</td>
<td>No information</td>
<td>Floor panels can be provided in areas where concrete is unavailable.</td>
</tr>
<tr>
<td>A7. Transportability</td>
<td>1 unit fits in C130</td>
<td>Panels weigh under 80 lbs.</td>
</tr>
<tr>
<td>A8. Building System Flexibility</td>
<td>No information</td>
<td>Panels may be added and size expanded without changing the design.</td>
</tr>
<tr>
<td>A9. Ease of Maintenance</td>
<td>Maintenance free materials.</td>
<td>Class 1 fire retardant, insensitive to mildew, termite &amp; rodent proof.</td>
</tr>
</tbody>
</table>

Table 3.18 Structural Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Capacity</th>
<th>Method of Verification</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1. Panel Bending</td>
<td>70 mph wind with 2.8&quot; deflection.</td>
<td>Testing</td>
<td>No Comments</td>
</tr>
<tr>
<td>B2. Wall Shear</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
<tr>
<td>B3. Roof to Wall Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
<tr>
<td>B4. Wall to Footing Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
<tr>
<td>B5. Adjacent Panel Connections</td>
<td>No information</td>
<td>No Calculations Received</td>
<td>No Comments</td>
</tr>
</tbody>
</table>
3.7 Existing Systems Structural Conclusions

Of the twelve system manufacturers contacted to participate in this project, four complete proposals were submitted for review. Four expressed interest in the project, but could not provide the necessary support calculations and information. Two manufacturers expressed a great deal of interest, but did not provide the requested information. All potential manufacturers were contacted several times prior to and after the submission deadline. The four submitted building systems are ranked and described as follows:

3.7.1 DuraKit Shelters

Resin saturated cardboard composite shelter constructed on-site. Components fastened to adjacent members using epoxy resin adhesive.

1) Rating: 53

2) Pros: Very inexpensive, Economical, Simple Construction. Excellent supporting data (full scale testing and component testing)

3) Cons: Permanent construction (no disassembly), Durability issues in high temperature and humidity environment.

3.7.2 Leading Edge Earth Products, Inc.

Composite panel systems consisting of corrugated steel skins with a foam in-fill core.

1) Rating: 51

2) Pros: Very strong, Ability to have two story construction. Simple Construction. Good supporting data (component testing)
3) **Cons:** Relies on supporting structural frame system, Durability issues in high temperature and humidity environment due to steel corrosion.

### 3.7.3 CoreFlex

Pultruded FRP panel members with snap fit connection ends. Can be foam filled to provide added insulation and stiffness.

1) **Rating:** 50

2) **Pros:** Simple Construction. Ability to disassemble easily.

3) **Cons:** Supporting structural system required, Extra structural connections required, Durability issues in high temperature and humidity environment. Supporting data is very poor and is suspect (no component testing).

### 3.7.4 Futuristic Worldwide Homes

Pultruded FRP members used in substitution for wood members in a wood framed stud wall system.

1) **Rating:** 25 (Fail structural calculations)

2) **Pros:** Ability to disassemble.

3) **Cons:** Building requires cranes for erection. Supporting structural system required, Extra structural connections required, Durability issues in high temperature and humidity environment. Supporting data is very poor and is suspect (no component testing).
Aside from the performance rating results, the building systems reviewed during this phase of the investigation illustrated several areas of concern with regard to systemic performance. Most notable of these was the dependency of the systems on some type of supporting structural system. Specifically, we noted several of the systems utilized structural steel framework and connections designed to resist the high wind forces. While this is an acceptable design concept, it illustrates the complexity of the problem at hand. Further, the reliance of the systems on standard construction materials and methods becomes a serious impediment when one addresses the areas where these shelters will be constructed. In most cases, the shelters will be erected in areas where standard construction tools and techniques are not applicable. Further, long-term corrosion and maintenance becomes an issue in the case of systems utilizing steel support structures. Specifically, some conclusions made as a result of the investigation are as follow,

1) The optimal system would be one which would require no additional materials and tools to be provided on-site during the construction process.

2) Construction workers would require minimal experience and skill to erect the emergency shelter building.

3) The optimal system would be adaptable so that different building geometries and configurations could be constructed depending upon parameters such as usage, erection time, etc...
4) The optimal system would be able to be built, taken down and re-built without significant damage to the building components or connections.

5) The optimal system would consist of materials that were corrosion resistant and require very little maintenance.
4. WIND ANALYSIS & BUILDING GEOMETRY

4.1 Introduction

The design wind pressures for this investigation were developed using the 1998 edition of The American Society of Civil Engineers Standard 7, “Minimum Design Loads for Buildings and Other Structures”. The following parameters were utilized to develop all necessary design wind pressures:

1) Design Wind Velocity: The wind speed was specified by the military as 138 mph (222km/hr) that corresponds to a Category IV hurricane.

2) Building Size: The building footprint was selected to be 12’- 0” x 24’- 0” (3.66m x 7.32m) on the basis of preliminary structural analysis to optimize the structure for member forces and stresses.

3) Building Wall Height: The building height was selected as 8 ft (2.44m). It was assumed that the shelter was single-story with standard construction wall heights.

4) Roof Slope: A standard roof slope 4:12 (18.4°) was selected use in the shelter. Such a roof slope provides for transfer of wind uplift forces as well as roof drainage.
5) Mean Roof Height: Based on the assumed roof slope, the mean roof height taken at the mid-height of the roof is 10 ft (3.05m).

6) Building Importance: The structure is viewed as a primary emergency shelter and is therefore Category IV, \( I = 1.15 \) (ASCE7-98, Table 6-1).

7) Exposure D: The structures may be erected in any geographic location. It is also assumed that ground scour may have occurred prior to the erection of the building. Thus, \( K_z = 1.03 \) (ASCE7-98, Table 6-5).

8) Directionality: The probability of high wind occurrence on exposed main force systems and components and cladding sets \( K_d = 0.85 \) (ASCE7-98, Table 6-6).

9) Partially Enclosed Building Envelope: It is assumed that the shelters may be used as temporary buildings, lacking door and window assemblies rated for wind borne debris and forces. Consequently, \( GC_{pi} = \pm 0.55 \) (ASCE7-98, Table 6-7).

10) Hills or Escarpments: It is assumed that the shelters are erected in “camps” where grading and excavation of surrounding lands may be conducted. The absence of hills and escarpments presents the most conservative geographic situation setting parameter \( K_{zt} = 1.00 \) (ASCE7-98, Figure 6-2).
4.2. Wind Design Process

The first step in evaluating design wind pressures is to calculate the base wind pressure. This base wind pressure is dependent upon the wind velocity, building importance, building exposure and the geography of the surroundings. The base wind pressure is calculated using the following equation (ASCE7-98, Equation 6-13):

\[ q_z = 0.00256 \times 1.03 \times (1.00)(0.85)(138)^{2}(1.15) = 49.09 \text{ psf} (2.35\text{kPa}) \]  

(4.1)

where the variables are taken from ASCE7-98, Table 6-5, Table 6-6 and Table 6-1.

The design wind pressure is further refined based on the geometry of the building (roof slope, wall height, corner proximity) and the area of wind surface supported by the member to be designed. The building is first divided into end and interior zones. The determination of these zones are based on the following equations (ASCE7-98, Note 7 of Figure 6-4, Note 6 of Figure 6-5a, Note 7 of Figure 6-5b, Note 5 of Figure 6-7a);

**Main Wind Force Resisting Systems**

- \( a \) is smaller of \( 0.1 \times (12'-0'') \) or \( 0.4 \times (8'-0'') \) but >3.0'
- End Zone = \( 2a \) = 6.0' (1.83m)
- Interior Zone = 24.0' - (2)*6.0' = 12.0' (3.66m)
- Interior Zone = 12.0' - (2)*6.0' = 0.0' (short side)

**Component and Cladding Systems**

- End Zone = \( a \) = 3.0' (0.91m)
- Interior Zone = 24.0' - (2)*3.0' = 18.0' (5.49m)
- Interior Zone = 12.0' - (2)*3.0' = 6.0' (1.83m)
The building is further subdivided into windward and leeward surfaces, for which individual design wind pressures are calculated. For the purposes of design, worst case design wind pressures are chosen by comparing each member subjected to either windward or leeward locations. The direction of the applied wind is also subdivided into wind applied perpendicular to the roof ridge line and wind applied parallel to the roof ridge line. Specific gust factors are then developed, based on location and type, for each structural member being designed.

The final design pressure for each member is calculated by applying both localized gust and interior pressure factors to the base design wind pressure. The appropriate equation, shown below, applies for both Main Wind Force Resisting Systems (MWFRS) and Components and Cladding Systems (CC) (ASCE7-98, Eq. 6-16):

\[
p = q_h \left[ \left( GC_{pf} \right) - \left( GC_{pi} \right) \right]
\]  

(4.2)

The Main Wind Force Resisting System consists of all structural members that facilitate the transfer of wind induced forces to the foundation system and include any beams, columns or connection members within the structure. Components and Cladding forces are used to evaluate the capacity of individual members to handle directly applied wind loads.

Wind codes develop different coefficients based on the building and roof geometry as well as the type of system (MWFRS or CC) being addressed. For the current research, a wind analysis was performed using the initial emergency
shelter geometry proposed by The Lemay Center. This geometry consisted of a gable roofed, rectangular building having a 24'- 0" x 36'- 0" (7.32m x 10.92m) footprint. As a result of the analysis, the critical design pressures, shown in Figure 4.1, were developed for use during the evaluation of this structure and in the subsequent design optimization of the emergency shelter system (Figure 4.2). A detailed breakdown of all wind design pressures and calculations have been provided in Appendix A.
Figure 4.2 Critical Wind Pressures - Mono Slope Roof
4.3 Emergency Shelter Geometric Design

A multitude of structures have been developed and fabricated by the emergency shelter industry. Their geometry range from cubicle shaped boxes to monolithic dome type structures (see Section 1.5). In order to develop an optimal geometric shape for use in this project, special attention was paid to the needs of the end user. These were classified as:

1) Structural Performance: Of primary concern is the capability of the structure to withstand hurricane force winds.

2) Anchorage Performance: Due to the wide variety of usage proposed for the emergency shelter, adequate foundation anchorage is required for a multitude of ground soil conditions.

3) Construction Simplicity: The emergency shelter should be simple to construct, both in terms of the constituent construction components and the technical skills of the laborers.

4) Geometric Adaptability: The emergency shelter must fit into a pod-like system of urban design. Specifically, the end user should be able to add or subtract shelter units to “build” configurations to suit specific usage needs.

4.3.1 Step One

An initial geometry based on previous shelters developed by the military was selected as a starting point for the emergency shelter. This geometry
corresponded to a rectangular box shape, 24'-0" x 36'-0" x 8'-0" (7.32m x 10.97m x 2.44m) in shape, with a single roof ridge line and gable ends in the shorter 24'-0" (7.32m) dimension. A preliminary structural analysis of the building exposed to the design wind design pressures indicated that very large force concentrations occurred at the corners of the building and at the transition between the roof and wall members. Due to the magnitude of these forces, it was concluded that the building geometry needed to be revised.

4.3.2 Step Two

In order to restrict the force concentrations to acceptable limits, the base geometry of the building was reduced to a 12'-0"x 24'-0"x 8'-0" (3.66m x 7.32m x 2.44m) box shape. A preliminary structural analysis of the reduced geometry showed that the force concentrations were reduced by a factor of three. Further, the reduced geometry provides for a usable area of 288 ft² (26.76m²). This reduced footprint area allows for a greater variety of uses. Specifically, this area is more acceptable for usage as sleeping area, office space, storage area or medical facility.

4.3.3 Step Three

The initial roof geometry called for a gable end roof system. This system was found to be unacceptable due to the inherent structural weakness that
occurs at the gable end wall connections. This weakness results in the development of a hinge joint failure in the gable end wall during high wind events.

A double hipped roof configuration was investigated as a first alternative roof system. While this roof configuration provides for adequate structural bracing at all wall roof transition points, it was concluded that the complexity of this construction prohibits its selection for use in a rapidly deployed emergency shelter where simplicity of the building construction is crucial to success.

The geometry of the roof system is crucial to structural performance since it directly affects the magnitude and application of wind load forces. Further, the roof system geometry affects erection speed and construction complexity. Roof optimization led to the selection of a low rise mono sloped roof configuration. This roof configuration has several positive aspects. Specifically, a mono sloped roof system provides structural stability at all roof wall transition points. Further, a mono sloped roof is simple to constructed, requiring one basic structural component (plank member). Additional positive aspects of the mono sloped roof system include the ability to use the same members as used in the wall system, and the ability to align adjacent shelter units so as to create a variety of roof configurations (gable roof, sawtooth roof, etc...).

4.3.4 Step Four

The initial design of the shelter utilized a variety of door and window sizes, placed in all elevations of the building. For the purpose of adaptability and simplicity, it was concluded that all door and window sizes should use the same
opening size. Further, it was concluded that openings should not be installed in either of the short dimension elevation wall, due to the need for lateral shear resisting members in these elevations. Subsequent design resulted in the use of a 36" (0.914m) wide nominal opening for all windows and doors. Further, the placement of these openings in the long dimension elevation of the shelter facilitates the placement of adjacent shelters to create “rooms” which can be arranged into usable building complexes.
5. USF SYSTEM - ASSEMBLY

5.1 Introduction

A lightweight FRP section was designed to overcome many of the shortcomings identified in available systems. This chapter provides step-by-step directions illustrating the assembly of FRP panel members to rapidly construct the emergency shelter.

Section 5.2 includes additional information related to the FRP panel developed. The assembly of the structure is described in Section 5.3. Concluding remarks are summarized in Section 5.4.

5.2 Component Geometry

The panel shape was developed to enhance structural performance. Specifically, the panel is comprised of a continuous truss system that helps stiffen the section and facilitates stress transfer between the upper and lower skins that also act to resist bending induced stresses in two directions. The single panel configuration is detailed in Figure 5.1. However, the panels were developed to be used in an opposing, interlocking fashion, as shown in Figure 5.3.
Trapezoid shaped ribs provide method for connection to adjacent members. Also, when panels are interlocked, ribs provide resistance against twisting along member length.

Members are slid together during construction.

Trapezoid shape reduces the stress concentration that would occur at this point in a triangle shaped rib.

FRP skin provides the flexural and in-plane shear capacity.

Panel lip locks into adjacent member, providing resistance to moisture / air intrusion.

Figure 5.1 Basic Panel Geometry and Characteristics (Units in Inches)
Aside from the structural performance characteristics inherent in the continuous truss configuration of the panel, several positive attributes develop as a result of the geometry. Specifically, the truss shape of the panel ribs allow for their usage as interlocking connectors. When opposing panels are connected in this fashion, the overall panel structure acts to restrict moisture and air infiltration.

![Figure 5.2 Alternate Panel Geometry - Transverse Web Stiffener](image)

Another attribute of the panel member design are the lip connectors that run along the perimeter of each panel. These connectors, while not designed to transfer structural stresses between members, are adequate to seal the joint that occurs between adjacent panel members. Such a lip connection is required to restrict moisture and air penetration through the system.

In addition to the basic panel geometry, an alternative geometry concept was developed for use in loading situations where the member is subjected to large concentrated out-of-plane loads. Such loading conditions are experienced in members used in bridge deck applications. The alternative geometry incorporates additional transverse ribs into the basic geometry to improve the
ability of the panel to transfer concentrated point loads throughout the member. The alternate panel geometry is shown in Figure 5.2. Note that both the standard panel and alternate panel geometries have been developed to perform in an interlocking fashion. Critical to their performance is the interlocking nature of the ribs. This attribute allows the user to transfer forces between adjacent panels while ensuring that building envelop integrity is maintained. The interlocking nature of the panel member is illustrated in Figure 5.3.

The interlocking connections of the panel system greatly simplifies the erection process due to the lack of a need for additional connections and members. This feature contrasts with one of the main observed weaknesses in existing building systems. Specifically, it was noted that all of the available systems required separate connector members, both for member to member connection and member to support frame attachment. It may be argued that for each supplemental connector / attachment, an increase occurs in both erection complexity and the time required to construct the building.

In summary, the member geometry was developed to provide the most efficient means for stress transfer. Further, the geometry was developed for use both in an interlocking panel, as shown in Figure 5.3, and in a one direction fashion where no interlocking performance occurs in the construction. The geometry also works to restrict moisture and air penetration through the system. Finally, the simplicity of the geometry directly addresses the non-structural performance parameters of erection time, system complexity, system adaptability and durability.
5.3 Construction Process

The following construction sequence is provided to illustrate the simplicity and speed with which the optimized panel system can be erected in the field. It should be noted that all of the panel members are made of FRP and will be light enough to be handled by one to two personnel. Further, the construction sequence assumes that the foundation system is already in place at the time of erection. This assumption is based on the need for the emergency shelters to be used in a wide variety of environments. Such usage precludes the development of one specific type of foundation system. In view of this, foundation development was excluded from the scope of this investigation.

5.3.1 Step One

As stated previously, no specific design for the foundation system was developed in this investigation. Nonetheless, several observations may be made with regard to the optimized panel system. Firstly, the panel system can be utilized to provide direct anchorage between a concrete foundation system and the shelter system.

This connection, shown in Figure 5.3, utilizes the interlocking ribs of the standard member. To accomplish this connection, the base row of panel members is submerged into the concrete footing prior to curing of the concrete. Once the concrete has cured, a mechanism for uniform connection between the
A basic assumption with regard to the emergency shelter is that an earth floor system will be used to facilitate rapid construction. In cases where such a floor is inadequate, such as in a hospital or clean room facility, panel members can be installed as a floor system. Specifically, individual panel members can simply span between the exterior walls of the shelter.
5.3.2 Step Two

The construction of the wall system involves sliding opposing panels into each other in an interlocking fashion as shown in Figure 5.4. Note that the while the panels are shown to be erected in a horizontal fashion in this example, it can also be assembled vertically. Further, note that stiffening columns have been shown at the termination of each panel length. These columns are used at changes in geometry and to frame out wall openings. The dimensions of the FRP panel were selected so that they could be used in conjunction with standard pultruded FRP sections.

Figure 5.4 Second Step: Wall Assembly
5.3.3 Step Three

As the walls continue to be assembled, openings are located and framed out using the standard FRP pultruded members shown in Figures 5.5. These members also provide the user with the ability to relocate opening locations and sizes as needed to facilitate each specific usage situation. It should be noted that, while the walls are shown to be constructed in a linear fashion, with each section erected from the ground to the roof, the user also has the option to construct the wall panels on the ground, tilting each section up into place between the column members.
5.3.4 Step Four

After the walls have been completed, the roof members are prepared for assembly and installation onto the main structure. Once again, it should be noted that the roof members can be installed in an opposing, interlocking fashion, or in a non-interlocking fashion. The type of construction depends upon the types and magnitude of the roof loads to be experienced by the system. Further, it should be noted that the connection between the wall and roof members is provided by interlocking anchors, developed to slide into the panel rib members. These connectors, shown in Figure 5.7, provide direct transfer of wind induced uplift and shear forces between the roof and wall members.

Figure 5.6 Fourth Step: Wall Completion
The use of a separate connector allows the engineer to customize the anchorage system to accommodate specific loading conditions. Furthermore, as each connector member is adjustable, it allows the engineer to develop pretensioned connections within the structural framework of the building. The offset panel construction shown in Figure 5.1 and Figure 5.3 provides an extra groove into which the connector member can slide. The connector strap is then tightened, locking the connector into place. This type of connection facilitates multiple construction and disassembly processes without damage to the system members.

Figure 5.7 Hurricane Connector Component
5.3.5 Step Five

Each of the roof members are slid into place, creating a roof system which performs as a structural plate system transferring forces to the perimeter walls shown in Figure 5.8. The interlocking, opposing nature of the system prevents both moisture and air penetration. This importance of this factor is evident with the subsequent conclusion that no additional roof preparation is necessary prior to usage.
5.3.6 Step Six

The building shell is completed and ready for the installation of window and door assemblies. Due to the wide variety of high wind rated products available, neither assembly has been developed at this time. It is assumed that the frame out column members are adequate to receive a standard window / door assembly and will provide sufficient anchorage for these members.
5.4 Conclusions

The preceding sections attempted to illustrate the power of the optimized panel system developed in this study. The versatility of section geometry facilitates alternative applications. Thus, while the panel shape was developed for use in an emergency shelter system, it can also provide a viable solution for reinforced concrete roof/floor applications in lieu of corrugated metal deck systems. Furthermore, variation of the panel geometry permits its use in bridge deck applications.

In summary, a major strength of the optimized panel system developed is the simplicity and speed it offers the user. Both issues are critical for the success of emergency shelters, where untrained workers must build shelters in the worst of conditions. Reducing the construction process to the six basic steps shown in Figure 5.10 allows the response team the best possible solution, in terms of training, simplicity and erection speed.
Figure 5.10 Complete Construction Process
6. FIBER REINFORCED POLYMERS

6.1 Introduction

The combination of high strength, lightweight, corrosion resistance and the facility with which it can be fabricated into complex shapes has made FRP the material of choice for the aerospace industry. This section provides basic information on the manufacturing process of FRP pultruded sections and also on the material properties assumed in the analysis. More detailed information may be found in texts, e.g. Mallick 1988.

6.2 Manufacturing of FRP Composites

Pultrusion is a economical method of fabricating FRP shapes having a constant cross-section. In the process, the constituent materials are pulled through a heated steel die, which forms the resulting laminate material into the desired shape. The pultrusion process shown in Figure 6.1 consists of,

1) spools of uncured fiber rovings and mats,
2) Pre-form blocks which combine and coordinate the fiber orientations prior to saturation,
3) Resin bath which saturates the fibers prior to final shaping and
4) Heated steel die which provides the final shaping and curing of the shape.
6.3 Laminate Lay-up and Material Properties

A pultruded FRP composite laminate consists of four specific component layers:

1) Thin layer of randomly oriented chopped fibers, heavily saturated with resin located on the surface of the shape. This layer, sometimes referred to as the Nexis, provides a smooth surface and protection for the inner layers.

2) Unidirectional rovings, which contain fiber bundles running longitudinally down the axis of the pultruded shape. These layers provide tensile strength along the axis of the member, as would be required in flexural or tension applications.

3) Stitched Fabric Mat (SF) layers consist of unidirectional fiber bundles, woven into mats of off-axis angular orientation. These layers, typically at 30, 45, 60 or 90 degree orientation to the longitudinal axis of the member,
provide shear and weak direction flexural strength for the members.

4) Cross Stitch Mats (CSM) consist of continuous or short randomly oriented fibers. These layers can be of various weights and attempt to simulate isotropic material behavior within the plane of the layer. Figure 6.2 shows the placement of the surface veil, the cross-stitched fabric mat and the rovings in the pultrusion of a flanged section.

![Figure 6.2 Laminate Lay-Up Convention](image)

### 6.3.1 Material Properties

Pultruded FRP shapes consist of a series of interconnected thin-walled plate and shell elements. The constituent parts of the laminate are the fiber and matrix. E-glass fiber, which provides the strength and stiffness characteristics for the laminate, is used in the pultrusion process. Polyester or Vinylester resin, which provides a protective matrix for the fibers and a medium for stress transferral, is used in the pultrusion process. For the purposes of this
investigation, the elastic modulus, the shear modulus, Poisson’s ratio and density assumed are listed in Table 6.1. These were taken from prior investigations and represent a good sampling of current industry standard materials (Qiao, 1997). Note the order of magnitude difference in relative strengths of the fiber and the resin. The ply stiffness values shown in Table 6.2 were developed by Qiao (Qiao, 1997) for the roving, SF and CSM laminae using micromechanics for composites with periodic microstructure (Luciano, 1994). These ply stiffness values will be used during the design procedure in Chapter 9.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E \times 10^6$ psi</th>
<th>$G \times 10^6$ psi</th>
<th>$\nu$</th>
<th>$\rho$(lb/in$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-glass Fiber</td>
<td>10.5</td>
<td>4.18</td>
<td>0.255</td>
<td>0.092</td>
</tr>
<tr>
<td>Vinylester resin</td>
<td>0.42</td>
<td>0.2</td>
<td>0.30</td>
<td>0.041</td>
</tr>
</tbody>
</table>

Table 6.1 Material Properties - Constituents

<table>
<thead>
<tr>
<th>Ply</th>
<th>$E_1 \times 10^6$psi</th>
<th>$E_2 \times 10^6$psi</th>
<th>$G_{12} \times 10^6$psi</th>
<th>$\nu_1$</th>
<th>$t_k$</th>
<th>$X_{ct} \times 10^6$psi</th>
<th>$Y_{ct} \times 10^6$psi</th>
<th>$S \times 10^3$psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4 oz. CSM</td>
<td>1.710</td>
<td>1.710</td>
<td>0.610</td>
<td>0.402</td>
<td>0.015</td>
<td>21.35</td>
<td>8.512</td>
<td>4.2</td>
</tr>
<tr>
<td>17.7 oz. SF</td>
<td>4.207</td>
<td>1.202</td>
<td>0.465</td>
<td>0.294</td>
<td>0.026</td>
<td>96.8</td>
<td>8.434</td>
<td>4.2</td>
</tr>
<tr>
<td>54 roving (62 yield)</td>
<td>5.7</td>
<td>1.24</td>
<td>0.540</td>
<td>0.28</td>
<td>0.0355</td>
<td>48.4</td>
<td>4.217</td>
<td>2.1</td>
</tr>
</tbody>
</table>
6.4 FRP Connection Review

A critical limitation to the widespread usage of fiber reinforced polymers in structural framework centers on the design of connections between adjacent members. Within connection components, force concentrations and paths develop a highly complex map of stress and strain. In materials such as steel, the complexity of the stress and strain can be simplified due to the isotropic nature of the material. Such is not the case in connections fabricated using FRP composite materials, where component performance varies greatly depending upon the direction of stress being applied (see Figure 6.3 through Figure 6.5).

A significant amount of research has been conducted during the past two decades in the area of FRP connection design (Mosallam 1997, Mottram, 1997). Specifically, extensive research has been conducted to investigate connections which mirror shear and moment resisting connections in steel. “Steel type” connections typically consist of beam and column members as shown in Figure 6.6. Major work in this area is characterized in the following sections of this chapter.
Figure 6.3 Effect of Joint Flexibility (Mosallam 1990)

Figure 6.4 Failure of FRP Angle (Mosallam 1994)

Figure 6.5 FRP Thread Failure (Mosallam 1995)
6.4.1 FRP Connection Performance

Several tests have been conducted which investigate the performance of bolted connections which directly mirror those found in steel construction. Typically the work would entail full-scale testing of a moment frame constructed from standard W-shape or tube shaped FRP members (Bank, 1991; Bruneau, 1994; Mosallam, 1992). This body of research investigated a wide variety of connections, ranging from the typical clip angles shown in Figure 6.4 to the use of web stiffeners and thru-bolts that extend through both flanges of the column member. Several variations of the test connections have been shown below in Figure 6.6 through Figure 6.9.

![Figure 6.6 Typical Steel Type Connector - FRP](image-url)
Figure 6.7 Clip Angle Connections. Clip angle shear and moment connections constructed using thru-bolting. In some cases, adhesives were also installed to provide supplemental connection mechanism. Testing entails rotation induced in the beam element with rotation measured through failure of the connection. Moment - rotation curves are used to develop analytical models. (Bank 1991; Mottram 1994; Bruneau 1994)
Figure 6.8 Stiffener Connections. Additional web stiffeners installed to improve connection performance. Stiffeners reduce the level of stress concentrations around bolt locations. The addition of web stiffeners were shown to account for a 33.6% increase in moment - rotation capacity. (Bank 1991; Mottram 1994; Smith 1999)
Figure 6.9 Full Thru-Bolting. Based on previous testing, which illustrated that connection failures were often due to localized shearing at the location of thru-bolts, connections were developed where bolts extended through both flanges. This connection ensures greater joint stiffness by transferring forces more evenly into the connected members. These connections were shown to provide up to 200% strength increases and 270% stiffness increases with respect to simple clip angle connections (Mosallam 1997; Smith 1999).
The testing of steel type connections has centered around the ability to transfer moment through the connection. As a result of these investigations, a variety of analytical models were developed by the researchers. Several observations can be made, based on the body of work previously done in this area,

1) Bolted connections exhibit non-linear moment - rotation performance as the frame is loaded to failure. This non-linearity is due to the semi-rigid nature of the connection (Mosallam, 1992).

2) Typical failures of the simple clip angle connections involve localized buckling and separation of the column flange and web (Bank, 1994).

3) In connections where the members are braced using stiffeners or full thru-bolting, typical failures occur due to delamination and buckling of the clip angle member (Bank, 1994).

4) A significant impediment to the capacity of FRP connections is an inherent weakness in pultruded W-shaped members at the web - flange intersection. Specifically, an under-reinforced triangle shaped zone at this intersection significantly reduces the capacity of connections members (Mosallam, 1994).

5) Significant increases in connection performance, both in terms of stiffness and strength, can be gained through the development of new types of connectors, such as the Universal Connector and the installation of stiffeners and thru-bolts (Mosallam, 1997; Smith, 1999).
As a result of these observations, it was concluded that steel type connections of FRP frame members can be developed which provide joint characteristics similar to steel frame systems. But, the improved connection combinations (shown in Figure 6.9) are highly complex with respect to similar connections found in steel frames and would act to deter the use of such frames in typical construction situation.

### 6.5 Coordinate Systems and Sign Conventions

The FRP shapes to be investigated are panelized pultruded members that, for the purposes of analysis will be designed as a series of interconnected flat panels. Global (X, Y, Z) and local (x, y, z) coordinate systems are defined as shown in Figure 6.10. The moment and force conventions used in the development of material properties within individual laminae is also provided below in Figure 6.11.
6.6 Laminate Design

Anisotropic materials such as FRP create a challenge for engineer due to the complexity of the material in comparison to homogeneous materials such as steel. One aspect of this complexity is the increased number of material constants required for analysis. Generalized Hooke’s Law for anisotropic materials requires 21 independent stiffness coefficients. While this makes the design of anisotropic materials in 3-dimensions very complex, the characterization of composites as plate elements consisting of orthotropic laminae, or layers, can be utilized to greatly reduce the number of independent coefficients.
The following sections describe Classical Laminate Theory, beginning with the principle equations for stress and strain within an orthotropic laminae. These equations are then developed into stress and strain equations for the entire laminate based on the location and stress conditions of individual laminae. An orthotropic material is classified as one in which the material properties are identical in all three directions. The number of material coefficients reduces to 9 in an orthotropic material with the axis of orthotropy 1-2, with \( q = 0 \). The stress-strain relationship in such a laminae is given in Equation 6.1.

\[
\begin{align*}
\begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \sigma_{23} \\ \sigma_{13} \\ \sigma_{12} \end{bmatrix} &= \begin{bmatrix} C_{11} & C_{12} & C_{12} & 0 & 0 & 0 \\ C_{12} & C_{22} & C_{23} & 0 & 0 & 0 \\ C_{12} & C_{23} & C_{33} & 0 & 0 & 0 \\ 0 & 0 & 0 & C_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & C_{66} & 0 \\ 0 & 0 & 0 & 0 & 0 & C_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \gamma'_{23} \\ \gamma'_{13} \\ \gamma'_{12} \end{bmatrix} \\
\end{align*}
\]

(6.1)

This relationship is further simplified through the assumption that each layer exists in a plane stress state where,

\[
\sigma_3 = \tau_{23} = \tau_{31} = 0
\]

(6.2)

this condition reduces Equation 6.1 to,

\[
\begin{align*}
\begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \gamma'_{12} \end{bmatrix} &= \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ Q_{12} & Q_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \gamma'_{12} \end{bmatrix} \\
\end{align*}
\]

(6.3)

where the reduced stiffness coefficients \( Q_{ij} \) are given by four independent engineering constants in the principle material directions as follow,
The principle material axes 1-2 within each laminae are typically rotated with respect to the overall laminate reference axes x-y. To account for this rotation, transformed reduced stiffness coefficients are developed using the following equations,

\[
\begin{align*}
Q_{11} &= \frac{E_1}{1 - v_{21}v_{12}} \\
Q_{12} &= \frac{v_{12}E_2}{1 - v_{21}v_{12}} = \frac{v_{21}E_1}{1 - v_{21}v_{12}} \\
Q_{22} &= \frac{E_2}{1 - v_{21}v_{12}} \\
Q_{66} &= G_{12}
\end{align*}
\] (6.4)

Subsequently, the stress-strain relationship for each laminae, given in terms of the laminate reference axes x-y, is as follows,

\[
\begin{align*}
\bar{Q}_{11} &= Q_{11}\cos^4\psi + Q_{22}\sin^4\psi + 2(Q_{12} + 2Q_{66})\sin^2\psi\cos^2\psi \\
\bar{Q}_{22} &= Q_{11}\sin^4\psi + Q_{22}\cos^4\psi + 2(Q_{12} + 2Q_{66})\sin^2\psi\cos^2\psi \\
\bar{Q}_{12} &= (Q_{11} + Q_{22} - 4Q_{66})\cos^2\psi\sin^2\psi + 2(Q_{12} + 2Q_{66})\sin^2\psi\cos^2\psi + Q_{66}(\sin^4\psi + \cos^4\psi) \\
\bar{Q}_{66} &= (Q_{11} + Q_{22} - 2Q_{12} - 2Q_{66})\cos^2\psi\sin^2\psi + Q_{66}(\sin^4\psi + \cos^4\psi)
\end{align*}
\] (6.5)

The transformed reduced stiffness Equations 6.5 can be simplified with respect to the angular orientation of the principle laminae axes 1-2 and the laminate reference axes x-y. The first step of this simplification is the development of invariant coefficients $U_k$ as follows,
Substitution of Equation 6.7 into Equation 6.5 results in the following set of equations which are simpler than those shown in Equation 6.5 with respect to laminae orientation. This simplification is helpful during the optimization process.

\[
\begin{align*}
U_1 &= \frac{1}{8} (3Q_{11} + 3Q_{22} + 2Q_{12} + 4Q_{66}) \\
U_2 &= \frac{1}{2} (Q_{11} - Q_{22}) \\
U_3 &= \frac{1}{8} (Q_{11} + Q_{22} - 2Q_{12} - 4Q_{66}) \\
U_4 &= \frac{1}{8} (Q_{11} + Q_{22} + 6Q_{12} - 4Q_{66}) \\
U_5 &= \frac{1}{8} (Q_{11} + Q_{22} - 2Q_{12} + 4Q_{66})
\end{align*}
\]  

\text{(6.7)}

\[
\begin{align*}
\overline{Q}_{11} &= U_1 + U_2 \sin 2\varphi + U_3 \cos 4\varphi \\
\overline{Q}_{12} &= U_4 - U_3 \cos 4\varphi \\
\overline{Q}_{22} &= U_1 + U_2 \sin 2\varphi + U_3 \cos 4\varphi \\
\overline{Q}_{66} &= U_5 - U_3 \cos 4\varphi
\end{align*}
\]  

\text{(6.8)}

Classical Laminate Theory (CLT) allows the engineer to create a relationship between the stress and strain characteristics of individual laminae and the performance of the entire laminate. In order to accomplish this, CLT makes the assumption that the laminate consists of N orthotropic layers, perfectly bonded to each other, that the bond line between layers is infinitely thin and non-shear deformable and that Kirchhoff Plate Theory is used, in which in-plane displacements vary linearly through the thickness of the layer. Specifically,
\[ \mu = \mu_0 - z \frac{d\omega_0}{dx} \]
\[ v = v_0 - z \frac{d\omega_0}{dy} \]
\[ \varepsilon_z = \gamma_{xz} = \gamma_{yz} = 0 \]
\[ \omega = \omega_0 \]

where \( \mu \) and \( v \) are the in-plane displacements; \( \varepsilon_z, \gamma_{xz}, \gamma_{yz} \) and \( \omega \) describe the out of plane deformation and \( z \) is the distance from the layer to the laminate mid-plane as shown below in Figure 6.12,
Subsequent to these assumptions, the strain distribution within the laminate becomes,

\[
\begin{bmatrix}
\varepsilon_x \\
\varepsilon_y \\
\gamma_{xy}
\end{bmatrix} = \begin{bmatrix}
\varepsilon_x^0 \\
\varepsilon_y^0 \\
\gamma_{xy}^0
\end{bmatrix} + Z \begin{bmatrix}
K_x \\
K_y \\
K_{xy}
\end{bmatrix}
\] (6.10)

where \( K \) are the midplane curvatures and \( \varepsilon^0 \) and \( \gamma^0 \) are the strain components at midplane of the laminate. Substitution of Equation 6.10 into Equation 6.6 results in the following stress-strain relationship,

\[
\begin{bmatrix}
\sigma_x \\
\sigma_y \\
\tau_{xy}
\end{bmatrix} = \begin{bmatrix}
Q_{11} & Q_{12} & 0 \\
Q_{21} & Q_{22} & 0 \\
0 & 0 & Q_{66}
\end{bmatrix} \begin{bmatrix}
\varepsilon_x^0 \\
\varepsilon_y^0 \\
\gamma_{xy}^0
\end{bmatrix} + Z \begin{bmatrix}
K_x \\
K_y \\
K_{xy}
\end{bmatrix}
\] (6.11)

Resultant force and moment equations are developed using the laminae stress values derived in Equation 6.11 and summed through the thickness of the laminate as shown in Equations 6.12 and 6.13,

\[
\begin{bmatrix}
N_x \\
N_y \\
N_{xy}
\end{bmatrix} = \int_{-h/2}^{h/2} \begin{bmatrix}
\sigma_x \\
\sigma_y \\
\tau_{xy}
\end{bmatrix} dz = \begin{bmatrix}
A_{11} & A_{12} & A_{16} \\
A_{21} & A_{22} & A_{26} \\
A_{61} & A_{62} & A_{66}
\end{bmatrix} \begin{bmatrix}
\varepsilon_x^0 \\
\varepsilon_y^0 \\
\gamma_{xy}^0
\end{bmatrix} + \begin{bmatrix}
B_{11} & B_{12} & B_{16} \\
B_{21} & B_{22} & B_{26} \\
B_{61} & B_{62} & B_{66}
\end{bmatrix} \begin{bmatrix}
K_x \\
K_y \\
K_{xy}
\end{bmatrix}
\] (6.12)

\[
\begin{bmatrix}
M_x \\
M_y \\
M_{xy}
\end{bmatrix} = \int_{-h/2}^{h/2} \begin{bmatrix}
\sigma_x \\
\sigma_y \\
\tau_{xy}
\end{bmatrix} zdz = \begin{bmatrix}
B_{11} & B_{12} & B_{16} \\
B_{21} & B_{22} & B_{26} \\
B_{61} & B_{62} & B_{66}
\end{bmatrix} \begin{bmatrix}
\varepsilon_x^0 \\
\varepsilon_y^0 \\
\gamma_{xy}^0
\end{bmatrix} + \begin{bmatrix}
D_{11} & D_{12} & D_{16} \\
D_{21} & D_{22} & D_{26} \\
D_{61} & D_{62} & D_{66}
\end{bmatrix} \begin{bmatrix}
K_x \\
K_y \\
K_{xy}
\end{bmatrix}
\] (6.13)

where \( A_{ij}, B_{ij} \) and \( D_{ij} \) are the extensional, shear coupling and bending coefficients, respectively. These coefficients are defined as follow,
\[ A_{ij} = \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z_k - z_{k-1}) \]  \hspace{1cm} (6.14)

\[ B_{ij} = \frac{1}{2} \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z^2 - z_{i+1}^2) \]  \hspace{1cm} (6.15)

\[ D_{ij} = \frac{1}{3} \sum_{k=1}^{N} (\bar{Q}_{ij})_k (z^3 - z_{i+1}^3) \]  \hspace{1cm} (6.16)

In order to simplify the number of variables and the complexity of the equations, shear coupling can be eliminated through the symmetric placement of layers with respect to the mid-plane of the laminate. This allows for the elimination of the \( B_{ij} \) coefficients from the engineering calculations. Equations 6.14 through 6.16 can be modified into a form more conducive to design optimization. This form uses laminae invariant coefficients \( V_i \) which are defined below (Gurdal, 1999),

\[ V_{0(AB,D)} = \left\{ h_0, \frac{h^3}{12} \right\} \]  \hspace{1cm} (6.17)

\[ V_{1(AB,D)} = \sum_{k=1}^{N} \cos 2\phi_k \left\{ t_k, t_k z_k, t_k \left( z_k^2 - 2z_k z_{k-1} + z_{k-1}^2 \right) \right\} \]  \hspace{1cm} (6.18)

\[ V_{1(AB,D)} = \sum_{k=1}^{N} \cos 2\phi_k \left\{ t_k, t_k z_k, t_k \left( z_k^2 - 2z_k z_{k-1} + z_{k-1}^2 \right) \right\} \]  \hspace{1cm} (6.19)

\[ V_{2(AB,D)} = \sum_{k=1}^{N} \sin 2\phi_k \left\{ t_k, t_k z_k, t_k \left( z_k^2 - 2z_k z_{k-1} + z_{k-1}^2 \right) \right\} \]  \hspace{1cm} (6.20)

\[ V_{3(AB,D)} = \sum_{k=1}^{N} \cos 4\phi_k \left\{ t_k, t_k z_k, t_k \left( z_k^2 - 2z_k z_{k-1} + z_{k-1}^2 \right) \right\} \]  \hspace{1cm} (6.21)

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where \( t_k = z_k - z_{k-1} \) represents the layer thicknesses and \( z_k \) is the z coordinate of the mid-plane of the \( k \text{th} \) layer with reference to the mid-plane of the laminate.

Use of the \( V_i \) and \( U_i \) laminae invariants allows for a simplified representation of the \( A, B \) and \( D \) matrices, as shown below (Gurdal, 1999),

<table>
<thead>
<tr>
<th>( (A_{11}, B_{11}, D_{11}) )</th>
<th>( V_0(A,B,D) )</th>
<th>( V_1(A,B,D) )</th>
<th>( V_2(A,B,D) )</th>
<th>( V_3(A,B,D) )</th>
<th>( V_4(A,B,D) )</th>
</tr>
</thead>
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<td>( U_2 )</td>
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<tr>
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<td>( 0 )</td>
<td>( 0 )</td>
<td>( U_2 )</td>
<td>( 0 )</td>
<td>( -2U_3 )</td>
</tr>
</tbody>
</table>

### 6.7 Summary

This chapter has provided an extensive foundation with which the design optimization and analysis will be performed in Chapter 7. The initial sections of this Chapter were intended to provide insight as to the scope of the problem at hand. Specifically, the following observations can be made based on the background work conducted in the area of hurricane winds design and emergency shelters,

1) Advanced composite materials offer a highly adaptable solution to the emergency shelter problem.

2) A significant weakness in the design of advanced composite structures occurs at member connections. This weakness is due to the use of “steel
type” connectors which do not fully utilize the strengths of composite materials.
7. DESIGN OPTIMIZATION OVERVIEW

7.1 Introduction

Previous chapters of this investigation have dealt the pragmatic application of the shelter problem, issues of construction cost, availability and adaptability. Further the investigation has provided an overview in Classical Laminate Theory. The purpose of this chapter is to provide a general overview of structural design optimization techniques. While not intended to be a detailed review, this chapter shall provide the reader with an understanding of the typical terms, process and characteristics of design optimization.

7.2 Design Optimization - General Procedure

Design optimization in structural engineering can best be described as an educated trial and error procedure, where performance functions are first developed and then analyzed to ascertain maximum or minimum values. Composite materials are well suited for design optimization, based on the extensive range of configurations available to the engineer. Similarly, the complex nature of composite materials can result in cumbersome performance functions which may be costly in terms of computation time (Haftka, 1990).
Several design optimization procedures have been developed to locate the maximum and minimum values of design functions. While the procedure followed by each optimization technique may differ, all design optimization techniques follow the same basic format of

1) Selecting a group of initial trial values,
2) Calculating a value for the objective performance function for each of the trial values,
3) Removing the least successful trial values from the solution pool,
4) Re-calculation values using permutations of the most successful trial values from the previous round of optimization.

This four step process is typically undertaken until a global solution is found. While there are myriad variety of design optimization techniques, the following terms and assumptions are generally found in most accepted methods and are described here to inform the reader.

7.2.1 Objective Function

An objective function can be any performance function to be optimized. For example, the compression buckling formula developed by Qiao in his work with FRP beam optimization could be chosen as a objective function. This equation, shown below, is dependent on the $D_{11}$, $D_{12}$ and $D_{22}$ within the laminae, which are in turn dependent on the orientation of the plies and their stacking sequence (Qiao, 1997).
\[ \lambda_b N_x = \frac{2\pi^2}{b^2} \left[ \sqrt{D_{11}D_{22}} + (D_{12} + 2D_{66}) \right] \] (7.1)

Note the coefficient \( \lambda_b \) located at the beginning of Equation 7.1. This is a buckling factor and acts as the value to be optimized. In general notation, Equation 7.1 can be described by the objective function,

\[ \lambda_{\text{optimum}} = (1 - p)\min(\lambda_b) \] (7.2)

Another aspect of the objective function is the application of the penalty factor \( p \). The penalty factor acts to penalize the proposed solution when certain design constraints, such as maximum laminate thickness, are violated. This factor is typically applied as a percentage reduction in the calculated value of \( \lambda_b \).

### 7.2.2 Constrained Optimization

Sometimes it is necessary to limit the search based on constraints placed on performance criteria related to the design variables within the objective function. For example, a design constraint may be placed on the laminate that requires \( G_{xy} \) to be below a certain value. Since the value of \( G_{xy} \) directly affects Equation 7.1, it can be seen that restricting this variable will result in constraint of the solution pool. Typical notation for Constraint is as follows,

\[ 0 \leq G_{xy} \leq 0.5 \times 10^6 \text{ psi} \] (7.3)
7.2.3 Neighborhood Searches

Neighborhood search techniques are often used in conjunction with integer programming. In these situations, the neighborhood is defined as all single unit variations of the initial trial solution (Pai, 2001). For example, if the initial trial solution is described by the integer sequence \{1201\}, the neighborhood would entail the following variations: \{0201\}, \{2201\}, \{1001\}, \{1101\}, \{1221\}, \{1211\}, \{1200\} and \{1202\}. Note that each variation is gained by shifting one variable a single position.

7.3 Linear Integer Programming

Integer programming is significant to optimization in that it allows the engineer to simplify the representation and manipulation of design variables within each trial solution. In the case of optimization for composite laminates, integer programming is typically used to represent the ply orientations. Specifically, 0°, +45°, -45° and 90° ply orientations can be assigned integer values of 1, 2, 3 and 4, respectively. Therefore a laminate having the stacking sequence \{0,+45,-45,90,90,-45,+45,0\} could be represented as \{1,2,3,4\}_s^\varepsilon. Note the subscript s has been applied to represent symmetry. To further simplify the representation, the engineer can represent the +45° and -45° as a stacked pair, thus reducing the number of variables by one. The subsequent reduced notation would be \{1,2,3\}_s^\varepsilon, where the 90° ply is represented by the integer 3.
The use of integer programming allows the designer to avoid the problems involved when design variables are not consecutive in nature, as in a variable set consisting of 0°, 45° and 90° composite layers. Assigning these layer orientations integer values allows the designer to develop the objective and constraint functions into linear functions of the design values. The standard form of an Integer Linear Programming problem (ILP) is,

\[
\begin{align*}
\text{minimize} & \quad f(x) = c^T x \\
\text{such that} & \quad Ax = b, x \geq 0
\end{align*}
\]

where \( c \) is an \( n \times 1 \) vector of constant coefficients, \( A \) is an \( m \times n \) matrix of constraint coefficients, and \( b \) is an \( m \times 1 \) vector of constants (Gurdal, 1999).

In optimizing composite laminates, the designer sometimes uses integers to describe certain laminate characteristics (such as layer orientation) while allowing other characteristics (such as layer thickness) to be continuous. This is referred to as Mixed Integer Linear Programming (MILP) and has the standard form,

\[
\begin{align*}
\text{minimize} & \quad f(x) = c_1^T x + c_2^T y \\
\text{such that} & \quad A_1 x + A_2 y = b,
\end{align*}
\]

where \( x \) is an integer greater than or equal to one and \( y \) is any number greater than or equal to one (Gurdal, 1999).
7.4 Genetic Algorithms

Genetic Algorithms (GA) is a well known optimization procedure which is based on the principles of evolution found in nature. Specifically, GA utilizes the observation that survival of the fittest tends to propagate desirable characteristics while eliminating unwanted characteristics from a subject pool. GA perform design optimization using similar techniques (Gurdal, 1999).

The basic GA procedure begins with an initial population of trial solutions evaluated using the objective performance function. The fitness of each trial solution measures how desirable the results of this evaluation. The possibility that a trial solution will be used in subsequent optimization runs is proportional to the level of its fitness. Therefore, the better a trial solution performs during the evaluation, the better its chances to remain in the solution pool as a potential parent for the subsequent generation of trial solutions.

Subsequent generations of solutions are developed through the pairing and combination of the trial solutions, based on their evaluated fitness. The process is based on the evolution of a gene pool that occurs in nature. In addition to this basic process of evolution, mutations and permutations are introduced into the GA process to prevent premature loss of solution characteristics which might be significant to the final solution.
8. DEVELOPMENT OF PANEL PERFORMANCE CRITERIA

8.1 Introduction

The design and optimization information presented within Chapter 7 will eventually be utilized in Chapter 9 to optimize the composite laminate panel member. Before this optimization can be conducted the performance functions must be developed in a form conducive to the eventual optimization process. To that end, the following tasks will be performed within this chapter,

1) Design equations will be developed to calculate localized plate buckling, global member deflection and first-ply failure loads.

2) Localized plate buckling equations will be developed by dividing the panel member into discrete plate elements.

3) Global member deflection will be developed based on the combined stiffness and geometric properties of the panel element.

4) First-Ply Failure will be developed using the Tsai-Hill Failure Criteria to determine the strength envelope for the panel element,

5) Design equations will be dependent on two criteria; ply orientation and laminate stacking sequence,

6) The fixed parameters in all equations will include panel geometry, laminate thickness, laminae thickness and material properties of plies.
8.2 Local Buckling Performance

Pultruded FRP beam and panel members consist of a series of interconnected plate members. During loading conditions in which members are exposed to axial compression and bending, premature failure of the member due to local buckling of these plates can occur. The local buckling capacity of the constituent plates can be modeled using a series of discrete plates subjected to in-plane compression and shearing forces. Through variation of the discrete plate boundary conditions, the local buckling can be characterized (Qiao, 1997).

To clarify the previous paragraph, the panel unit shown in Figure 8.1 has been divided into a series of discrete plates for the purpose of subsequent analysis in this chapter.

The individual plate elements, including in-plane loading and retraint conditions, have been illustrated in Figure 8.2. Division of the panel into component plate elements results in two load conditions. In each flange elements, the in-plane loading consists of compression ($N_x$) applied along the
longitudinal axis of the panel. In the web elements, shear stress \( (N_{xy}) \) is induced in each plate.

In Figure 8.2, (a) and (b) are the dimensions of each plate element and \( \zeta \) is the restraint coefficient characterizing the fixity of the plate boundary. \( \zeta \) is based on the stiffness of the adjoining plate element and is developed in the following section. The following sections will present general solutions for thin plate buckling due to axial compression and shearing force. Subsequent to the development of the solutions, equations for the calculation of the restraint coefficient will be presented.
8.2.1 General Buckling Equations - Axial Compression

Local buckling in thin plates under axial compression is governed by Equation 8.1, developed by Whitney for a symmetric anisotropic plate (Whitney, 1987). Within this equation, $D_{ij}$ are the plate bending stiffness coefficients; $N_x$ is the uniform axial stress resultant and $w(x,y)$ describes the buckled shape of the plate. This equation has been simplified based on the assumption that the laminate consists of balanced off-axis laminae, resulting in the elimination of bending-twisting coupling ($D_{16} = D_{26} = 0$).

$$D_{11} \frac{\partial^4 w}{\partial x^4} + 2D_{12} \frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} + 4D_{66} \left( \frac{\partial^4 w}{\partial x \partial y} \right)^2 + D_{22} \frac{\partial^2 w}{\partial x^2} + N_x \frac{\partial^2 w}{\partial x^2} = 0 \quad (8.1)$$

The general solution to this equation can be written in the form (Bleich, 1952),

$$w(x,y) = \sin \left( \frac{n \pi x}{a} \right) \left( C_1 \cosh k_2 y + C_2 \sinh k_2 y + C_3 \cos k_2 y + C_4 \sin k_2 y \right) \quad (8.2)$$

where $k_1$ and $k_2$ are (Webber, 1985),

$$k_1 = \frac{n \pi}{a} \sqrt{\alpha - \sqrt{\alpha^2 - \beta + \mu \ell^2}} \quad (8.3)$$

$$k_2 = \frac{n \pi}{a} \sqrt{-\alpha + \sqrt{\alpha^2 - \beta + \mu \ell^2}} \quad (8.4)$$

$$\mu \ell^2 = \frac{N_x}{D_{22}} \left( \frac{a}{n \pi} \right)^2; \quad \alpha = \frac{D_{12} + 2D_{66}}{D_{22}}; \quad \beta = \frac{D_{11}}{D_{22}} \quad (8.5)$$
where the constants $C_i$ are determined based on the specific boundary conditions for each plate element along the edges described by (a).

The general solution listed above can be further simplified by assuming that the deflection of the plate results in a symmetric function of $y$ as the buckling load is approached. Specifically, it is assumed that the plate forms a symmetric sin wave shape along the $y$-axis. The second assumption is that equal restraint exists along both of the unloaded edges of the plate. These assumptions result in the following form of Equation 8.2,

$$w(x, y) = \sin\left(\frac{n\pi x}{a}\right)(C_i \cosh k_y + C_3 \cos k_y)$$

Qiao used Equation 8.6 to develop the following equation for the critical axial buckling stress resultant, $N_x$, for long simply supported plates (Qiao, 1997).

$$\min(N_x) = \frac{2\pi^2}{b^2} \left[ \sqrt{D_{11}D_{22} + (D_{12} + 2D_{66})} \right]$$

Equation 8.7 was further developed to account for elastic edge restraint. This condition occurs in Flange I, II and III in Figure 8.1. The boundary conditions for this condition are that no local deflection ($w_{b/2,-b/2} = 0$) occurs along the boundary and that the rotation along the boundary for the plate in question ($\varphi$) is identical to the rotation in the adjoining plate which provides the elastic restraint ($\varphi = \varphi_r$). To represent the effect of this elastic restraint, Bleich derived
restraint coefficients $p$ and $q$. The solution to the buckling equation for axial compression with elastic restraint is (Qiao, 1997),

$$\min(N_x) = \frac{\pi^2}{b^2} \sqrt{q\left(\sqrt{2D_{12}D_{22}} + p(D_{12} + 2D_{66})\right)}$$  \hspace{1cm} (8.8)

Note that the above equation constitutes the local buckling equation to be used for Flanges I, II and III of the panel member.

Buckling load equations were also developed for biaxial loading conditions for a composite laminate plate by Liu. The buckling equations were developed under the assumption that the composite laminate could buckle into $m$ and $n$ half-waves in the $x$ and $y$ directions (Liu, 2004). Subsequent to this assumption, they proposed the following critical buckling equation for a laminate plate under axial loading (no shearing),

$$\frac{\lambda_n^{(m,n)}}{\pi^2} = \frac{D_{11}\left(\frac{m}{a}\right)^4 + 2\left(D_{12} + 2D_{66}\right)\left(\frac{m}{a}\right)^2\left(\frac{n}{b}\right)^2 + D_{22}\left(\frac{n}{b}\right)^4}{\left(\frac{m}{a}\right)^2 N_x + \left(\frac{n}{b}\right)^2 N_y}$$ \hspace{1cm} (8.9)

### 8.2.2 Elastic Restraint

The presence of elastic restraint along the unloaded boundaries of the plate element (see Figure 8.2) significantly increases the complexity of the buckling equations. Elastic restraint is addressed through the development of a restraint constant, $\zeta$, which is based directly on the material and geometric properties of all plates that occur at that boundary.
The development of this constant is based on the assumption that rotation about the boundary will be transferred without losses. The restraint constant was first developed by Bleich for isotropic materials using a uniformly loaded column (Bleich, 1952). These equations were modified by Qiao to account for material anisotropy and the effect of compressive stresses in thin walled sections. These modifications result in the following equation for the restraint constant, $\zeta$, for a box section in which elastic restraint occurs along both boundaries of the plate (Bleich, 1952; Qiao, 1997).

$$\zeta = \frac{b^w D_{22}^f}{b^f D_{22}^w} r$$  \hspace{1cm} (8.10)

where $w$ and $f$ refer to the web and flange plate elements and $r$ is a modification factor introduced by Bleich and modified by Qiao,

$$r = \frac{1}{1 - \left(\frac{b^w}{b^f}\right) \left(\frac{\sqrt{D_{11}^w D_{22}^w + D_{12}^w + 2D_{66}^w}}{\sqrt{D_{11}^f D_{22}^f + D_{12}^f + 2D_{66}^f}}\right)}$$  \hspace{1cm} (8.11)

Both of the equations above are dependent on the Bending Coefficients, $D_{ij}$, of the laminate and the width, $b$, over which the plate would experience bending. It should be noted that for this investigation, Equations 8.10 and 8.11 simplify to be solely dependent upon the web and flange width geometries since all members are made of the same laminate.
8.2.3 General Buckling Equations - Shearing

Local buckling of the webs in the panel member is controlled by the development of shearing forces as flexural moment is induced on the panel. Web member buckling is illustrated in Figure 8.2. For the purpose of this investigation, the web is modeled as a simply supported plate subjected to shear forces only. The flanges are assumed to restrain the web and deflection is assumed to be zero. For these conditions, the restraint coefficient, $\zeta_s$, was developed as (Qiao, 1997),

$$\zeta_s = \frac{2D_{f22}}{b^2}$$  \hspace{1cm} (8.12)

where $f$ denotes the flange that restrains the web plate element being analyzed.

The general equation for an anisotropic thin plate under shear loading was developed using the first variation of the total potential energy equation (Barbero, 1993),

$$\int_0^a \int_0^b \left( D_{11} \frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} + D_{12} \frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial x \partial y} + \frac{\partial^2 w}{\partial x \partial y} \frac{\partial^2 w}{\partial y^2} + 4D_{16} \frac{\partial^2 w}{\partial x \partial y} \frac{\partial^2 w}{\partial x \partial y} \right) + \left( \zeta_s \left( \frac{\partial w}{\partial y} \right)_{y=0} + \zeta_s \left( \frac{\partial w}{\partial y} \right)_{y=b} \right) \left( \frac{\partial w}{\partial x} \right)_{x=0} \, dx \, dy = \int_0^a \int_0^b \left( \zeta_s \left( \frac{\partial w}{\partial y} \right)_{y=0} + \zeta_s \left( \frac{\partial w}{\partial y} \right)_{y=b} \right) \left( \frac{\partial w}{\partial x} \right)_{x=0} \, dx \, dy$$  \hspace{1cm} (8.13)

A solution for the displacement $w(x,y)$ which satisfies the boundary conditions defined by Equation 8.12 is defined as (Qiao, 1997),

$$w = \sum_{i=1}^{m} \sum_{j=1}^{n} A_{ij} \sin \frac{i \pi x}{a} \sin \frac{j \pi y}{b}$$  \hspace{1cm} (8.14)
The critical buckling shear stress, $N_{xy}$, can be calculated for the web member as a linear eigenvalue problem using Equations 8.13 and 8.14.

In an attempt to simplify the shear buckling equations shown above, the laminate panels can be assumed to have infinite length in the x direction (Whitney, 1985). Based on this assumption, critical shear buckling is defined by,

$$\lambda_s = \frac{4\beta_1 \sqrt{D_{22}(D_{12} + 2D_{66})}}{b^2 N_{xy}} \text{, for } 0 \leq \Gamma \leq 1$$  \hspace{1cm} (8.15)

$$\lambda_s = \frac{4\beta_1 (D_{11}D_{22})^{1/4}}{b^2 N_{xy}} \text{, for } 1 \leq \Gamma \leq \infty$$  \hspace{1cm} (8.16)

where variables are as defined below,

$$\Gamma = \frac{\sqrt{D_{11}D_{22}}}{D_{12} + 2D_{66}}$$  \hspace{1cm} (8.17)
Table 8.1 Buckling Factors

<table>
<thead>
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<th>B₁</th>
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<tr>
<td>0.0</td>
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</tr>
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</tr>
<tr>
<td>40.0</td>
<td>8.25</td>
</tr>
<tr>
<td>Infinite</td>
<td>8.13</td>
</tr>
</tbody>
</table>

Under simultaneous loading conditions, the critical axial and shearing buckling interaction can be approximated through the following interaction equation (Lekhnitskii 1968, Liu 2004),

\[
\frac{1}{\lambda_c^{(m,n)}} = \frac{1}{\lambda_n^{(m,n)}} + \frac{1}{\lambda_s^2} \]  \hspace{1cm} (8.18)

### 8.3 Global Deflection Performance

The global performance properties of the panel member; axial compression, member bending and shear, have been developed in the past as a summation of the constituent plate elements. Using beam theory with no torsion and assuming that the off-axis plies are balanced symmetric (no bending -
twisting coupling), Qiao developed simplified equations for the axial \((A_i)\), bending \((D_i)\) and shear \((F_i)\) stiffness coefficients (Qiao, 1997). Specifically, the stiffness coefficients of individual plate elements are,

\[
A_i = \left(\frac{E_x}{t_i}\right)
\]

\[
D_i = \frac{\left(\frac{E_x}{t_i}\right)}{12}
\]

\[
F_i = \left(\frac{G_{xy}}{t_i}\right)
\]

where \((E_x)\) and \((G_{xy})\) are the engineering properties of the \(i^{th}\) plate and \(t_i\) is its thickness. The stiffness coefficients are then combined to provide the beam axial \((A_i)\), bending \((D_i)\) and shear \((F_i)\) stiffness Equation 8.20,

\[
A_z = \sum_{i=1}^{n} A_i b_i
\]

\[
D_z = \sum_{i=1}^{n} \left[ A_i \left(\frac{b_i^2}{12} \sin^2 \theta_i \right) + D_i \cos^2 \theta_i \right] b_i
\]

\[
F_z = \sum_{i=1}^{n} F_i b_i \sin^2 \theta_i
\]

where \(b_i\) is the plate width, and \(\theta_i\) is the cross sectional orientation of the \(i^{th}\) plate. Subsequently, these stiffness coefficients can be used in conjunction with general formulas for maximum bending and shear deflection under uniform loading conditions. The resulting equation for the maximum deflection of a simply supported beam of length \(L\) and uniform load \(W\) is,

\[
\delta_{total} = \delta_{bending} + \delta_{shear} = \frac{5WL^4}{384D_z} + \frac{WL}{K_zF_z}
\]
where $K_z$ is a shear correction factor to account for the actual shear stress across the member cross section. For the purpose of design, this correction factor can be set as 1.0 (Davalos, 1996).

Further work by Giroux and Shao on FRP reinforced sheet piles resulted in the development of equivalent flexural rigidity properties of a panelized member (Giroux, 2003). The equivalent rigidity properties were developed utilizing Timoshenko’s beam theory and are as follows,

\begin{equation}
\begin{align*}
(EI)_{\text{shape}} &= \sum_{\text{flange}} \frac{b}{3} \sum_{j=1}^{n} \left( E_x \right)_j \left( z_j^3 - z_{j-1}^3 \right) \\
&\quad + \sum_{\text{web}} \left[ \sum_{i=1}^{m} \frac{(E_x)_i (t_w)_i (I_y)_{\text{web}}}{t_w} \right]
\end{align*}
\end{equation}

The equivalent flexural rigidity value generated from Equation 8.22 would then be utilized in Equation 8.21 in place of $D_z$ for the calculation of deflection due to bending. It should be noted that none of the global deflection calculation shown above account for closed sections where stress sharing occurs between adjacent connected panel members.

### 8.4 First Ply Failure (FPF) Performance

When a laminate material is loaded, different stresses develop in each of the layers, depending on the orientation of the fibers and the location of the layer with respect to the laminate mid-plane. As a result of these stress differences, it
is likely that some laminae plies will fail prior to others. This phenomena is called first-ply failure (FPF). In FRP composites, the brittle nature of the laminate materials prevents strength performance past FPF (Gurdal, 1999).

The strength of the laminate is dependent upon the FPF. Several failure envelops have been developed to ascertain the stress levels at which FPF will occur. For the purposes of this investigation, the Tsai-Hill failure criterion shall be used and is defined as follows (Gurdal, 1999),

\[
\frac{P}{P_{cr}} = \sqrt{\left(\frac{\sigma_1}{X}\right)^2 + \left(\frac{\sigma_2}{Y}\right)^2 - \left(\frac{\sigma_1\sigma_2}{X^2}\right) - \left(\frac{r_{12}}{S}\right)^2} \leq 1
\]  

(8.22)

where \(\sigma_1, \sigma_2\) and \(r_{12}\) are the laminae principle stresses; \(X, Y\) and \(S\) are the corresponding ply strengths; \(P\) is the applied load and \(P_{cr}\) is the critical load.

8.5 Summary

This chapter has developed four primary performance related optimization functions. These are the performance functions of local buckling (axial and shearing), global deflection and laminate ply failure. Additionally, an overall cost function, represented as the thickness of individual plies, will be optimized for the development of the best solution set.
9. COMPOSITE PANEL DESIGN - ANALYSIS / RESULTS

9.1 Introduction

The preceding chapters have laid a groundwork through which the reader was first introduced to the problem of temporary shelters needed for disaster relief and response. The basic requirements necessitate erection speed, weight minimization and strength. These design requirements led to the conceptual development of an interlocking panel system made of composite materials.

The previous three chapters provide the reader an overview of the basics of Composite Laminate Design (Chapter 6), Design Optimization (Chapter 7) and the Development of Performance Functions (Chapter 8). This chapter will accomplish several things.

1) First, the reader will be re-introduced to the primary equations utilized in the design process. These will include equations for local buckling, first ply laminate failure and global deflection.

2) Second, the reader will be taken, step by step, through the laminate design and analysis process. This process will begin with an explanation of the loads exerted on the section and finish with the computation of the performance values for local buckling, first ply failure and global deflection.
3) The reader will be presented with all possible laminate solutions. Due to manufacturing restrictions, the number of possible laminates is restricted to nine (based on a eight ply balanced symmetric laminate).

4) The solution pool will be verified utilizing the Finite Element Software Ansys 5.7. The comparison will evaluate each of the performance criteria. The results will be discussed with reference to accuracy and significance.

The next section explains the restrictions that are placed on the design process prior to initiation. These restrictions are based on a pultrusion industry review and have to do with value engineering of the end product and elimination of interlaminar stress coupling in the laminate pool.

9.2 Design Restrictions

Prior to the analysis phase of the investigation, an exhaustive review was conducted of the composite manufacturing industry and of existing techniques for the design of composites. Subsequent to this investigation, it was found that several restrictions were necessary to facilitate the economical production of a composite member. Specifically, the following design parameters were restricted based on the need for overall economy,

1) Lay-up Restrictions; Based on a review of composite engineering / design, the candidate pool of composite laminates was restricted to symmetric laminates (laminates having symmetry about the mid-plane) made of
paired orientation stacked layers (ie...45/-45/30/-30). These restrictions eliminate bending - extension coupling from occurring within the laminate. Bending - extension coupling refers to the generation of bending stresses due to in-plane loads placed on the laminate.

2) Thickness Restrictions; Through discussion with several pultrusion composite manufacturers, we were informed as to the standard stock material used in their production. For the purposes of economy, the minimum layer thicknesses and maximum laminate thickness have been restricted to those typically utilized in the manufacturing process.

3) Laminate Uniformity; For the purposes of economy, all walls of the composite panel member are assumed to be the same thickness and laminate lay-up.

4) Layer Orientations; For the purposes of economy, all walls of the composite panel member are assumed to be the same thickness and laminate lay-up.

Subsequent to the restrictions noted above, the solution sample pool was limited to nine lay-up orientations for an eight ply laminate and twenty five lay-up orientations for a twelve ply laminate. Based on this small pool, it was decided that all possible solution sets would be analyzed and compared.
9.3 Performance Equations - Buckling

Local buckling in thin plates constitutes a primary area of design concern with respect to the strength characteristics of the laminate. The following buckling performance equations, first presented in Chapter 8, will be used in the design,

Local Buckling (Axial Compression)

\[
\frac{\lambda_{n}^{(m,n)}}{\pi^2} = \frac{D_{11} \left( \frac{m}{a} \right)^4 + 2 \left( D_{12} + 2D_{66} \right) \left( \frac{m}{a} \right)^2 \left( \frac{n}{b} \right)^2 + D_{22} \left( \frac{n}{b} \right)^4}{\left( \frac{m}{a} \right)^2 N_X + \left( \frac{n}{b} \right)^2 N_Y} \quad (9.1)
\]

Local Buckling (Shear Forces)

\[
\lambda_s = \frac{4\beta_1 \sqrt{D_{22} \left( D_{12} + 2D_{66} \right)}}{b^2 N_{XY}}, \text{ for } 0 \leq \Gamma \leq 1 \quad (9.2)
\]

\[
\lambda_s = \frac{4\beta_1 \left( D_{11} D_{22}^3 \right)^{1/4}}{b^2 N_{XY}}, \text{ for } 1 \leq \Gamma \leq \infty \quad (9.3)
\]

where the variables are as defined in Sections 8.2.1 and 8.2.3.

Local Buckling (Combined Forces)

\[
\frac{1}{\lambda_{c}^{(m,n)}} = \frac{1}{\lambda_{n}^{(m,n)}} + \frac{1}{\lambda_s^2} \quad (9.4)
\]

The factors \( m \) and \( n \) represent the number of half-sine waves that represent the deformed shape of the plate at buckling load. For this analysis, the maximum value of \( m \) and \( n \) was set to a maximum of 4. This restriction was based on initial analysis conducted to a maximum value of 20. It was noted during the
preliminary analysis that the minimum buckling factors were found at small values of m and n.

9.3.1 Restraint Factor and Load Distribution Factor

To represent the effect of interconnected panel members, the restraint factor developed by Bleich (Qaio 1997) was modified for use in a closed cell beam where more than two panels share a joint. The subsequent restraint factor is presented below,

\[
R_1 = \left( \frac{1}{b_1} \right) \left( \frac{1}{b_1} + \frac{1}{b_2} \cdots \frac{1}{b_i} \right)
\]

where \(b_i\) is the width of each panel member at the joint in question. Equation 9.5 also assumes all panel members to consist of the same laminate lay-up and thickness.

In addition to the strengthening characteristics of the interconnected panel members, this geometry allows for load distribution / sharing to be conducted throughout the panel. To represent this load distribution, the following equation was developed utilizing similar distribution methods used in moment distribution of structural frames,
9.4 Performance Equations - Laminate Failure

Laminate Failure constitutes another primary area of design concern with respect to the strength characteristics of the laminate. For this investigation, laminate failure is determined through the use of the Tsai - Hill failure criteria. The following laminate failure performance equation, first presented in Chapter 8, will be used in the design,

$$\frac{P}{P_{cr}} = \sqrt{\left(\frac{\sigma_1}{X}\right)^2 + \left(\frac{\sigma_2}{Y}\right)^2 - \left(\frac{\sigma_2 \sigma_1}{X^2}\right)^2 + \left(\frac{\tau_{12}}{S}\right)^2} \leq 1$$

(9.7)

where the variables are as defined in Section 8.4.

9.5 Performance Equations - Deflection

Global deflection constitutes a primary area of design concern with respect to the serviceability characteristic of the laminate. For this investigation, the deflection criteria developed by Qiao and Giroux and presented in Chapter 8 will be utilized. Specifically, the flexural rigidity equations developed by Giroux will be utilized to calculate global deflection due to bending and the torsional
rigidity equation developed by Qiao will be used to calculated the global deflection due to torsion. This criteria is as shown below,

\[ F_i = \left( G_{xy} \right)_i t_i \]  
\[ F_z = \sum_{i=1}^{n} F_i b_i \sin^2 \theta_i \]  
\[ (EI)_{shape} = \sum_{flange} \left[ \frac{b}{3} \sum_{j=1}^{n} (E_x)_j \left( z_j^3 - z_{j-1}^3 \right) \right] \]
\[ + \sum_{web} \left[ \sum_{i=1}^{m} \left( E_x \right)_i \left( t_w \right)_i \left( I_y \right)_i \right] \]  
\[ \delta_{total} = \delta_{bending} + \delta_{shear} = \frac{5WL^4}{384EI_{shape}} + \frac{WL}{K_f F_z} \]  

where the variables are as defined in Section 8.3.

### 9.6 Example Design Process

In order to best illustrate the design results, a step by step process will be presented using an example laminate lay-up. The numerical computations were conducted using Excel spreadsheets. These spreadsheets will be presented below for each primary step. The laminate to be presented will be a 0/0/45/-45/-45/45/0/0 eight ply laminate represented by 0/45 in the results tabulation.

The geometry of the shelter building is as shown in Figure 9.2. This building represents the geometric optimization performed in Chapter 4. Subsequent to the wind analysis also performed in Chapter 4, the resulting
design wind pressures are presented in Figure 9.1. Since these values vary widely throughout the structure, it was decided to design the panel for the worst case load condition. This case corresponds to 198.1 psf (9.49 kPa). Further, the composite shell panel members have designed for 151.8" (3.856 m) overall span length.
Figure 9.2 Emergency Shelter Footprint

Figure 9.3 Member Section
9.6.1 Design Step One

The first step in the process is to convert the wind pressure exerted globally on the building into $N_x$, $N_y$ and $N_{xy}$ forces exerted locally on the individual plates that constitute the composite panel section shown in Figure 9.3. The following assumptions were used to convert the wind pressure into local forces.

9.6.1.1 Axial Compression - Bending

The axial compression force generated as a result of bending in the member. As the member flexes, compressive and tensile forces build up in a moment couple about the centroid of the section. The worst case compression force due to bending is assumed to occur in the roof panels due to the wind exposure and span. Theses forces would be maximized on Plates 1, 2, 3 and 6, which constitute the extreme section components.

\[
\text{Load} = 198.1 \text{psf} \\
\text{Span} = 12.649 \text{ft} \\
M = \frac{198.1 \text{psf} (12.649 \text{ ft})^2 (12\text{in})}{8(12\text{in})} = 3961.9 \text{in} - \text{lb}/\text{in}
\]

The development of the bending stress into total bending induces compression force and subsequent distribution among component plates (as per Equation 9.9) is shown in Table 9.1.
9.6.1.2 Axial Compression - Wind

The axial compression force generated as a result of the downward roof pressure acting on the perimeter walls. This force will be evenly carried throughout the member cross section. In addition, the self-weight of the structure and wall system combine to produce axial loading. This loading is maximized at the base of the wall system.

\[
\text{Load} = 74.8 \text{psf (MainForce ResistingSystem)} \\
\text{SelfWeight} = 15 \text{psf (roofandwall)} \\
\text{Span} = 12 \text{ft} \\
N_{x(\text{axial})} = \frac{(74.8 \text{psf} + 15 \text{psf})(12 \text{ft})}{2(12\text{in})} + \frac{(15 \text{psf})(12 \text{ft})}{12\text{in}} \\
N_{x(\text{axial})} = 59.9 \text{lb / in}
\]

9.6.1.3 Transverse Compression - Wind

The compression force acting along the short axis of the panel member induced by the lateral shear forces generated globally by the wind. These forces would be maximized on Plates 1, 2, 3 and 6, which are arranged parallel to the orientation of the applied global force.

\[
\text{Load} = 61.8 \text{psf (MainForce ResistingSystem)} \\
\text{Height} = 12 \text{ft} \\
\text{Length} = 24 \text{ft} \\
N_{y(\text{axial})} = \frac{(61.8 \text{psf})(12 \text{ft})(24 \text{ft})}{2(12 \text{ft})(12\text{in})} = 61.8 \text{lb / in}
\]
9.6.1.4 Shearing Forces - Wind

The shearing induced in the panel section induced by the wind induced shear forces on the building. The shear forces would be maximized in Plates 1, 2 and 3, which constitute a uniform shear plate.

\[
\text{Load} = 618 \text{ psf (MainForce ResistingSystem)} \\
\text{Height} = 12 \text{ ft} \\
\text{Length} = 24 \text{ ft} \\
N_{xy} = \frac{(618 \text{ psf})(12 \text{ ft})(24 \text{ ft})}{2(12 \text{ ft})(12 \text{ in})} = 618 \text{ lb/in} \tag{9.13}
\]

At this time it would be wise to mention the conservative nature of this analysis. As is illustrated above in the description of the four forces, all of the constituent plates do not experience the maximum forces. Further, it could be assumed that plates do not simultaneously experience the maximum forces. It could therefore be concluded that the design forces used here are overly conservative. In response, it should be pointed out the these are building code specified wind design pressures, which themselves are equivalent static loads developed from a dynamic force (wind). The utilization of worst case load conditions in all applications provides the investigator with a level of conservatism that is necessary for guaranteed performance and subsequently, life safety.
9.6.2 Design Step Two

The design variables for the investigation include the geometric properties of the composite panel member. For the purposes of the analysis, the composite panel member was divided into the individual repetitive cells made of interconnected plates as shown in Figure 9.1. The geometric properties of this section are as shown below,

<table>
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<tr>
<th>Load Calculations (Taken using worst case pressures throughout)</th>
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<tbody>
<tr>
<td>Bending</td>
</tr>
<tr>
<td>Shearing</td>
</tr>
<tr>
<td>Axial</td>
</tr>
</tbody>
</table>

<table>
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<tr>
<th>Geometric Properties</th>
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<th>thickness</th>
<th>length</th>
<th>area</th>
<th>Y</th>
<th>AY</th>
<th>I</th>
<th>Ad^2</th>
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<td>2.848</td>
<td>0.809</td>
<td>0.213</td>
<td>0.172</td>
<td>0.005</td>
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</tr>
<tr>
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<td>0.284</td>
<td>1.241</td>
<td>0.352</td>
<td>0.213</td>
<td>0.075</td>
<td>0.002</td>
<td>0.928</td>
</tr>
<tr>
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<td>2.848</td>
<td>0.809</td>
<td>0.213</td>
<td>0.172</td>
<td>0.005</td>
<td>2.131</td>
</tr>
<tr>
<td></td>
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<td>0.284</td>
<td>4.463</td>
<td>1.267</td>
<td>1.937</td>
<td>2.455</td>
<td>2.104</td>
<td>0.013</td>
</tr>
<tr>
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<td>4.463</td>
<td>1.267</td>
<td>1.937</td>
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<td>2.104</td>
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<td>0.011</td>
<td>5.371</td>
</tr>
</tbody>
</table>

| Y bar | 1.836 |
| Ix Total | 14.818 |

| Bending Moment | 3962.000 in-lb / in |
| Nx (Bending)   | 490.897 lb / in |
| Ny (Wall Shear)| 61.800 lb / in |
| Nx (Walls)     | 59.900 lb / in |
| Nxy (Wall Shear)| 61.800 lb / in |
| Nx Force (Total)| 550.797 lb / in |

<table>
<thead>
<tr>
<th>Table 9.1 Composite Panel Geometric Properties - 8 Ply</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric Properties</td>
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<tr>
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<tr>
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</tr>
</tbody>
</table>

| Y bar | 1.836 |
| Ix Total | 14.818 |

| Bending Moment | 3962.000 in-lb / in |
| Nx (Bending)   | 490.897 lb / in |
| Ny (Wall Shear)| 61.800 lb / in |
| Nx (Walls)     | 59.900 lb / in |
| Nxy (Wall Shear)| 61.800 lb / in |
| Nx Force (Total)| 550.797 lb / in |

<table>
<thead>
<tr>
<th>Restraint Calculations</th>
<th>a</th>
<th>144.000</th>
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<tr>
<td>t</td>
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<td>0.275</td>
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<table>
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<th>I/b based</th>
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<th>Nxy</th>
<th>Nxy</th>
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<td>0.132</td>
<td>505.311</td>
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<td>56.696</td>
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<td>1.241</td>
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<td>0.058</td>
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<td>24.705</td>
</tr>
<tr>
<td>3</td>
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<td>0.165</td>
<td>0.132</td>
<td>505.311</td>
<td>56.696</td>
<td>56.696</td>
</tr>
<tr>
<td>4</td>
<td>4.463</td>
<td>0.105</td>
<td>0.207</td>
<td>791.854</td>
<td>88.847</td>
<td>88.847</td>
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<td>0.207</td>
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<td>88.847</td>
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<tr>
<td>6</td>
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<td>0.083</td>
<td>0.263</td>
<td>1006.363</td>
<td>112.915</td>
<td>112.915</td>
</tr>
</tbody>
</table>

132
Additionally, it should be noted that the laminae thickness was set to 0.0355 in (0.869 mm). This complies with restrictions placed by pultrusion manufacturers and presented in Table 6.2 for 54 roving E-glass mat. The length of the member has been taken to be 144 in (3.658 m) and represents the component application in the emergency shelter. The load values are taken using the design wind pressures listed in Figure 4.2. The load values result from wind induced member bending, compression and wind shear.

The restraint factors and load distribution factors are based on the fixed variables of member geometry and laminate thickness. Table 9.2 illustrates the development of these factors. The results of the [D] matrix were developed using Classical Laminate Theory and are presented below in Table 9.2 for the illustrative laminate. Additionally, Table 9.2 summarizes the initial design information to be used in the buckling / first ply failure investigations.

<table>
<thead>
<tr>
<th>Initial Design Information - Composite Panels 8 Ply</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>D11 1.03E+04</td>
<td>D22 2.70E+03</td>
</tr>
<tr>
<td>D12 9.24E+02</td>
<td>D66 1.28E+03</td>
</tr>
<tr>
<td>a 151.8</td>
<td>Nx 3820.879</td>
</tr>
<tr>
<td>b1 5.70</td>
<td>Ny 428.707</td>
</tr>
<tr>
<td>b2 1.24</td>
<td>Nxy 428.707</td>
</tr>
<tr>
<td>b3 2.85</td>
<td>4 791.854</td>
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<td>b6 5.67</td>
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</table>

**9.6.3 Design Step Three**

Local buckling factors under axial compression and shearing forces are developed as two separate values for each panel member as per Equations 9.3.
through 9.5. The cumulative buckling factors are then developed using restraint factors as described in previous sections. Note the resulting buckling factor describes how the applied load relates to the critical load at which localized buckling would occur. The table of these values is provided below,

**Table 9.3 Local Buckling - 0/45 Ply Laminate**

<table>
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<tr>
<th>Axial Buckling - 0/45 laminate</th>
<th>b1</th>
<th>b2</th>
<th>b3</th>
<th>b4</th>
<th>b5</th>
<th>b6</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>n</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
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</table>

Min Buckling 12.78 695.39 55.96 13.89 13.89 6.48

Shear Buckling

| Lambda | 1.510848 |
| Beta   | 1.20E+01 |

Buckling 98.07 4741.42 392.29 101.94 101.94 49.66

Buckling (Total) 10.14 537.67 43.54 10.91 10.91 5.14

Restraint Factor 0.15 0.67 0.15 0.56 0.56 0.44

Final Buckling Factor 8.85 322.25 38.00 7.00 7.00 3.57
9.6.4 Design Step Four

The First Ply Failure Criteria for the Laminates were developed using the Tsai-Hill Failure Equations. For the purpose of this investigation, the material flexural stiffness matrix, the axial compression and shearing loads listed in Table 9.2 were utilized. The Tsai - Hill criteria was found to provide the most conservative values for failure when compared to several other failure criteria. The table below provides the failure factors using the Maximum Stress, Maximum Strain, Tsai - Hill, Hoffman and Tsai - Wu failure criteria.

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<th>Layer</th>
<th>Max Stress</th>
<th>Max Strain</th>
<th>Tsai Hill</th>
<th>Hoffman</th>
<th>Tsai-Wu</th>
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<td>43.61</td>
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<td>35.76</td>
</tr>
</tbody>
</table>

---

Min 9.77 10.54 9.36 9.38 9.57

Table 9.4 First Ply Failure
9.6.5 Design Step Five

Global deflection for the composite panel member was developed through summation of the geometric stiffness properties of the individual panels. This development results in an equivalent member section. Deflection is then calculated using this equivalent section in standard equations for a simple span beam exposed to bending and shearing forces. The table of these values is shown below,

**Figure 9.5 Global Deflection 0/45 Laminate**

<table>
<thead>
<tr>
<th>Deflection Calculations - 0/45 laminate</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Ex</strong></td>
</tr>
<tr>
<td><strong>Gxy</strong></td>
</tr>
<tr>
<td><strong>t</strong></td>
</tr>
<tr>
<td><strong>w</strong></td>
</tr>
<tr>
<td><strong>ly (web)</strong></td>
</tr>
<tr>
<td><strong>EI (web)</strong></td>
</tr>
<tr>
<td><strong>EI (flange)</strong></td>
</tr>
<tr>
<td><strong>F</strong></td>
</tr>
<tr>
<td><strong>b1</strong></td>
</tr>
<tr>
<td><strong>b2</strong></td>
</tr>
<tr>
<td><strong>b3</strong></td>
</tr>
<tr>
<td><strong>b4</strong></td>
</tr>
<tr>
<td><strong>b5</strong></td>
</tr>
<tr>
<td><strong>b6</strong></td>
</tr>
<tr>
<td><strong>Flexure</strong></td>
</tr>
<tr>
<td><strong>Shear</strong></td>
</tr>
<tr>
<td><strong>Deflection</strong></td>
</tr>
<tr>
<td><strong>Total Deflection</strong></td>
</tr>
</tbody>
</table>
9.7 Results

The following results tabulation, shown in Table 9.6, represents all nine possible laminate lay-ups available for use in the panelized system. All calculations were developed using the same process as illustrated in Section 9.6. Further, the values shown in Table 9.6 are represented graphically in Figures 9.4 and 9.5. Figure 9.4 provides a comparison of each laminate lay-up iteration with respect to the strength characteristics of buckling and first ply failure. Figure 9.5 provides a comparison of each laminate lay-up iteration with respect to the serviceability characteristic of global deflection.

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Layups</th>
<th>Deflection</th>
<th>Buckling</th>
<th>Laminate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>1.13</td>
<td>3.15</td>
<td>14.00</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>1.72</td>
<td>3.57</td>
<td>9.36</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>1.84</td>
<td>4.35</td>
<td>6.17</td>
</tr>
<tr>
<td>4</td>
<td>45</td>
<td>1.72</td>
<td>7.56</td>
<td>9.36</td>
</tr>
<tr>
<td>5</td>
<td>45</td>
<td>3.79</td>
<td>8.07</td>
<td>6.48</td>
</tr>
<tr>
<td>6</td>
<td>45</td>
<td>3.80</td>
<td>9.26</td>
<td>3.16</td>
</tr>
<tr>
<td>7</td>
<td>90</td>
<td>1.84</td>
<td>10.55</td>
<td>6.17</td>
</tr>
<tr>
<td>8</td>
<td>90</td>
<td>3.80</td>
<td>11.03</td>
<td>3.16</td>
</tr>
<tr>
<td>9</td>
<td>90</td>
<td>5.17</td>
<td>11.35</td>
<td>2.25</td>
</tr>
</tbody>
</table>
Figure 9.4 Strength Factors - 8 Ply Laminate

Figure 9.5 Deflection Comparison - 8 Ply Laminate
9.8 Ansys Verification Model

A finite element model was developed to verify the results of the model for use in a more complex structural element. For the purposes of the verification, Ansys 5.7 was utilized to conduct the analysis. The verification model utilized Shell99 elements. This element type was selected to best represent the composite laminate properties of the material. Specifically, due to the manufacturing constraints presented in chapter 4, the shell thickness could not be greater than 0.284" (7.214 mm). This thickness would allow for a maximum of eight layers.

In order to reduce inter-laminar shearing stresses, the lay-up is further limited to a symmetric twelve layer lay-up using balanced orientation pairs. Further, through discussion with the manufacturing industry, we were informed that typical composite laminates consist of 0, 45 or 90 degree layer orientations. The geometry of the panel are as shown below in Figure 9.6.

To best represent the manufactured component, the analysis was performed on a 151.8" (3.856 m) long shell member. The analysis was performed on one repetitive unit of the panel system. The material constraints listed above accurately represent the manufacturing and performance limitations involved in the component system. The reduction of the possible lay-up configurations results in a solution set consisting of twenty five lay-up orientations.

A full analysis was performed on all possible solution lay-ups in Ansys 5.7. The results of this analysis are presented below in Table 9.7 and above in Figure
9.4 and Figure 9.5. Note the layer orientations shown are one quarter of the entire composite laminate thickness (i.e., 0/45 is 0/0/45/-45/-45/45/0/0).

Table 9.7 Ansys Results - 8 Ply Laminate

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Layups</th>
<th>Deflection</th>
<th>Stress (-)</th>
<th>Stress (+)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1.00</td>
<td>1674.00</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>45</td>
<td>1.25</td>
<td>2047.00</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>90</td>
<td>1.31</td>
<td>1749.00</td>
</tr>
<tr>
<td>4</td>
<td>45</td>
<td>0</td>
<td>1.25</td>
<td>4359.00</td>
</tr>
<tr>
<td>5</td>
<td>45</td>
<td>45</td>
<td>2.27</td>
<td>5211.00</td>
</tr>
<tr>
<td>6</td>
<td>45</td>
<td>90</td>
<td>2.19</td>
<td>5334.00</td>
</tr>
<tr>
<td>7</td>
<td>90</td>
<td>0</td>
<td>1.31</td>
<td>2426.00</td>
</tr>
<tr>
<td>8</td>
<td>90</td>
<td>45</td>
<td>2.19</td>
<td>2351.00</td>
</tr>
<tr>
<td>9</td>
<td>90</td>
<td>90</td>
<td>5.85</td>
<td>7270.00</td>
</tr>
</tbody>
</table>

Figure 9.6 Panel Geometry - Ansys
9.9 Comparison of Analysis Results

The values attained through the Ansys 5.7 run were used to verify the optimal solutions reached during the composite panel laminate design. Through this review and verification process, the optimal solution was selected based on each of the following criteria.

9.9.1 Strength Discussion

The localized buckling characteristics of the individual laminates were compared with respect to each other and to the maximum stress values attained through the Ansys verification model. All stresses and factors were linearized according to the maximum values. The comparison graph is as shown below, Through review of the buckling factor / maximum stress comparison shown in Figure 9.7, the following laminates are of principle interest,
1) 90/90 Laminate; This laminate provides the maximum buckling factor while exhibiting the minimum stress under the prescribed loading conditions. Deeper review explains the results, as this laminate contains only plies running perpendicular to the long axis of the composite panel. Such a laminate would provide the highest buckling factor since all plies run in the direction resisting thin plate buckling along the short axis. The low maximum stress values also are directly related to the ply orientation. Since no plies run in the direction of the principle bending stresses, the maximum stress would not be developed in the fibers, but in the matrix. The matrix offers less material stiffness and would be able to generate less stress. It would be expected that the deflection in this laminate would be high.

2) 0/0 Laminate; This laminate provides a low maximum stress value while also providing a low buckling factor. The stress value results from the high stiffness provided by a laminate having all plies oriented in the direction of the applied principle stresses. The low buckling factor is due to the lack of plies oriented to resist localized buckling along the short axis. It would be expected that this laminate would provide good resistance against deflection.

3) 90/0; 0/45 Laminates; These laminates provide a good combination of high buckling factors and low maximum stress values. These laminates should provide good all around performance due to the use of all available laminate orientations.
9.9.2 Deflection Discussion

The deflection results were compared directly to the verification model performed in Ansys 5.7. Both the composite panel and verification model were analyzed based on a simply supported single span beam. The comparison is shown previously in Figure 9.5.

It can be seen immediately that all of the computed values for global deflection are more conservative compared to the verification values provided by Ansys. This is to be expected based on the use of Equation 9.9 for deflection in single span beam under uniform loading. The overestimation of deflection is due to the inability of Equation 9.9 to account for the closed nature of the composite panel system. This equation was originally utilized for open section laminate beams (such as W-shaped) and does not accurately account for the interaction of the panels in a closed beam.

Review of the deflection results illustrates that the composite panel analysis follows the same deflection trends as the verification analysis. The following laminates are once again of interested and are discussed below,

1) 90/90 Laminate; As expected it can be seen that this laminate provides the worst deflection resistance of all the possible solutions. This is due to the lack of layer orientation in the direction of the flexural stresses.

2) 0/0 Laminate; This laminate provides the best overall deflection results, which follows with the understanding that all layers are oriented to resist the flexural stresses and provide the greatest amount of member stiffness.

3) 90/45; 90/0 Laminates; These laminates provide good performance with
respect to resisting global deflection.

In an attempt to improve on the accuracy of the deflection equations in estimating the behavior of a closed beam section, the restraint factors presented in Equation 9.5 have been used to modify the deflection equations as follows,

\[ F_z = \sum_{i=1}^{n} \left[ 1 + R_i \right] F b_i \sin^2 \theta_i \]  \hspace{1cm} (9.11)

\[ (EI)_{shape} = \sum_{flange} \left[ \frac{b}{3} \sum_{j=1}^{n} \left( E_s \right)_j \left( z_j^3 - z_{j-1}^3 \right) \right] \left[ 1 + R_i \right] \]

\[ + \sum_{web} \left[ \sum_{j=1}^{m} \left( E_s \right)_j \left( t_w \right)_j \left( I_y \right)_{web} \right] \left[ 1 + R_i \right] \]

where \( R_i \) is the restraint factor defined in Equation 9.5. Subsequent to this adjustment, the global deflection results are as shown below in Figure 9.8. The use of the restrained equations provides a better overall curve fit with the Ansys verification model.
9.10 Conclusions

The design process conducted in this Chapter resulted in several optimal solution sets, depending upon the specific criteria being investigated. While this provides for a variety of solution, it also illustrates the difficulty faced in the optimization of structural components for multiple performance criteria. The current investigation is a good example of this difficulty, with several laminates performing well against some performance criteria and doing poorly in other areas.

Typically, multi-variable optimization problems such as this are addressed through the selection of one performance function. This function then becomes
the objective of the optimization, while all other performance criteria are
developed as constraints on the process.

Regardless, the results presented herein illustrate that the panel members
are well suited to performance criteria necessary to fulfill the loading conditions in
a temporary shelter application. The maximum deflections ranged from $L/39$ to
$L/177$. The buckling factors ranged from 3.15 to 11.35. The first-ply failure
factors ranged from 2.25 to 14.0. The overall safety of this panel system at such
high residential loading conditions implies that the system may perform well in
other applications, such as rapidly deployed bridge decking. The use of this
panel system in other applications will be dependent upon the deflection
requirements of the project.
10. CONCLUSIONS AND RECOMMENDATIONS

10.1 Project Summary

When addressing the aftermath of a natural disaster, the aid worker is faced with three immediate tasks,

1) Provide protection from the environment,

2) Provide food and resources for the facilitation of life and

3) Provide health services for the treatment and prevention of illness.

The development of an emergency shelter is central to the facilitation of each of the tasks that occur in post-disaster situations. The goal of this project was to investigate hurricane-resistant shelters that could be easily transported, rapidly built on-site and required minimal tools and skill to construct.

In the study, a comprehensive search of the existing housing market was conducted to locate viable emergency shelter manufacturers (Chapter 2). Eleven candidates shelters were located and a Request for Proposal (RFP) prepared containing information on the geometry and wind loading of a simple structure that permitted a side-by-side comparison of the available systems. This was sent to all the manufacturers. Upon receipt of the completed proposals, four competing systems were evaluated (Chapter 3). In addition, an emergency shelter building concept was developed in-house. This building utilized lightweight, high strength,
corrosion resistant Fiber Reinforced Polymer (FRP) material and was optimized for global construction performance (Chapter 8 and Chapter 9). Assembly of this system is illustrated in Chapter 5.

10.2 Conclusions: Historical Precedent

A review of the history of emergency shelters (Chapter 1) showed that these structures are typically classified as either temporary shelters, that are rapidly constructed with an intended life span of less than six months, or temporary housing, that provide a more substantial type of construction and have a much longer life span. Further, foreign aid providers have noted that key to the success of emergency shelters is the level of cultural acceptance it facilitates in the area of deployment. As a result, structurally superior shelter types, such as the geodesic dome or the Quonset hut prove to be failures due to a lack of usage during disaster events. Thus, aesthetic familiarity is key to the successful implementation and usage of emergency shelters. This refers not only to building geometry, but also wall, roof and floor surfaces.

Since the 1950's, FRP materials have seen a wide range of use, from the aerospace industry to everyday ladders and tools. FRP materials provide a good fit for use in an emergency shelter, since their light weight and corrosion resistance facilitates storage, transportation and erection of the shelters. Further, their ability to be molded and designed for specific structural applications provides the engineer with structural shapes that can perform multiple structural tasks concurrently.
While a variety of different FRP structural members have been developed, our review noted a lack of development in the area of connections. Specifically, FRP framing systems have, for the most part, been limited to the use of connections that mirror those found in structural steel applications. Bolted connections were developed for use in homogeneous, isotropic materials like steel, but they are not as suited for anisotropic materials like FRP (see Figure 6.5).

10.3 Conclusions: Existing System Review

The existing emergency shelter industry was reviewed for viable candidates. Due to the stringent performance requirements, only eleven existing building systems met project requirements. Within this group the viable emergency shelters fell into three types of construction:

1) Standard Construction - New Materials emphasize improved performance gained through the use of new materials (Three manufacturers). Such materials offer the user improved mechanical properties (on a localized basis), light weight, non-corrosive and non-metallic performance.

2) New Construction - New Materials develop new construction systems in an attempt to best utilize the performance characteristics of the new materials (Six manufacturers).

3) Alternate Systems constitute a fully alternate system of construction, based on geometry, materials and construction (Two manufacturers). As mentioned earlier, only four building system manufacturers submitted systems for review
and evaluation. From this evaluation, the following conclusions were reached with respect to the existing building systems:

### 10.3.1 Best System: Durakit

![Figure 10.1 DuraKit Emergency Shelter](image)

The DuraKit building system consists of corrugated fiberboard that is factory-coated and treated to make a durable shelter with a fireproof interior and a weatherproof exterior. Fiberboard (similar to cardboard construction) is assembled as composite panels. The panels are connected to adjacent members using an adhesive system.

1) **Rating:** 53

2) **Pros:** Very inexpensive, Economical, Simple Construction. Excellent supporting data (full scale testing and component testing)

3) **Cons:** Permanent construction (no disassembly), Durability issues in high temperature and humidity environment.
10.3.2 Best System: Leading Edge Earth Products (LEEP)

The system consists of composite panels composed of steel face sheet bonded to a foam core using sandwich construction. The panelized construction is supplemented using a metal frame system in which the panel sections are inserted.

1) Score: 51

2) Strengths: Highest Wind Resistance, Transportability, Simple Erection, Testing

3) Weaknesses: Permanent Adhesive/Mechanical Connections, Corrosion Issues

The other two building systems, CoreFlex International and Futuristic Homes were eliminated from the review during the evaluation process for the following reasons:

1) Inadequate supporting structural information (no test data, no detailed calculations) - Coreflex.
2) Inadequate structural capacity of connections - Futuristic Homes.
3) Reliance on supplemental steel systems for connections and member stiffening - Futuristic Homes.
4) Reliance on supplemental steel systems for connections - Coreflex.
5) No building system components currently being manufactured - Coreflex.

As a result of the existing emergency shelter review, we concluded that while two systems appear to have met the base criteria of the project, all of the systems investigated exhibit similar weaknesses. Specifically, we concluded that all of the system are designed as one time usage buildings, since disassembly would constitute a significant amount of work and possible member damage. Further, we concluded that all of the systems emphasize localized member performance issues of bending and shear, while failing to fully develop the global issues of member connections and systemic performance under load.

10.4 Conclusions: USF System Design

The development of the USF system building started with a conceptual design to address non-structural issues such as building system simplicity and the ability to disassemble and rebuild the structure with minimal work or member damage. Two conclusions resulted from this first level of design. First, it was concluded that the building should utilize a system of panel components that incorporated structural connections into the standard member shape. Second, the section needed to be optimized.
The new system reduced the number of member types required during construction and facilitates systemic strength through interlocking of component members. Moreover, the use of FRP materials provided the greatest amount of design flexibility. The resulting interlocking panel member, developed using trapezoidal shaped open ribs fastened to a stiffening plate surface is shown in Figure 10.3.
The final level of design optimization uses Classical Laminate Theory (Jones, 1975) and Approximate Composite Laminate Properties (Nagraj, 1994) to develop composite laminate systems with the optimal $E_x$, $E_y$ and $G_{xy}$ developed previously. Based on these global material properties, several composite laminate lay-ups will be compared and optimized for volume, based on work conducted by Qiao, Haftka and Giroux. The resulting composite laminate represents the fully optimized structural system for the loading conditions proposed in the emergency shelter problem. Further, the proposed approach illustrates a truly powerful tool which enables the designer to separately design one member geometry for a multitude of loading situations and usage conditions.

10.5 Contributions

The main contributions of this study are summarized as follows.

1) A state of the art review was conducted of the emergency shelter industry. While similar reviews have been conducted within the area of emergency shelter construction, this review is the first to emphasize the performance of emergency shelters under extreme environmental conditions engendered by hurricanes.

2) A new type of structural panel system is developed. This uses new FRP material in a novel construction system that incorporates structural connections into the member section. This design development acts to improve the member’s structural efficiency while at the same time reducing the level of system complexity.
3) A design optimization process is developed in which member geometry and material properties can be optimized independent of each other. This process illustrates the wide variety of uses available to the design limited to a small number of member geometries.

10.6 Recommendations for Future Work

This investigation has shown that several building systems appear to adequately fulfill the design parameters laid out by the investigation team. However, this assessment is based on information provided and therefore needs independent verification. This is especially true with the new requirement in the forthcoming Florida Building Code that makes it mandatory to conduct missile impacts for structures where wind velocities are as large as assumed in this study; 138 mph (222km/hr). The purpose of future work is to take each shelter from “the drawing board” to the field, through on-site erection, testing and evaluation. This goals of the recommended work may be attained in the following manner:

1) Prototype shelter buildings should be purchased from each successful building system candidate. Two such structures, Durakit and LEEP were identified from available systems.

2) Prototype of the USF optimized FRP shelter building system should be fabricated and erected for testing and evaluation.

3) Prototype shelter buildings to be erected in a controlled environment, with all phases of construction reviewed and evaluated.
4) Prototype shelter buildings to be tested for global stability under laterally and vertically induced load conditions.

5) Prototype shelter buildings to be tested for wind-borne projectile impact (hurricane force winds).

Further, the optimization procedure developed within this body of work should be expanded to more accurately determine localized stresses developed in and around the integrated connection component of the FRP panel members. Such detailed stress analysis would allow the engineer to perform localized optimization in other applications, such as those found in more standard steel type bolted FRP connections.
REFERENCES


Nagraj, V. (1994). “Static and Fatigue Response of Pultruded FRP Beams Without and With Splice Connections,” MS Thesis, Department of Civil and Environmental Engineering, West Virginia University, Morgantown, WV.


Pultruded Member Image (2000). “Creative Pultrusion Website”


ABOUT THE AUTHOR

Nick M. Bradford graduated from The Pennsylvania State University with Bachelor’s and Master's of Science degrees in Architectural Engineering. Subsequent to graduation, Mr. Bradford worked as an intern for structural engineering firms in Virginia, New York and Florida. In 1998, Mr. Bradford became licensed as a Professional Engineer in Florida and opened a small structural engineering firm based in Tampa, Florida.

While in the Ph.D. program at the University of South Florida, Mr. Bradford has developed an impressive resume of engineering work, specifically in the areas of high wind construction and forensic engineering. Mr. Bradford served as an adjunct professor at University of South Florida during the spring of 2002 and has worked with the Civil and Environmental Engineering Department on investigations into high wind construction. He currently lives with a wife and two cats, spending as much time on the water as possible.