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Executive Summary
After the recent earthquake in Haiti, the world was shocked by the dramatic destruction it caused. Thousands of people were killed, and millions lost their homes. In response, many aid organizations constructed shelters for displaced Haitian people, but the issue of safe permanent housing has yet to be solved. In most cases, the people of Haiti cannot afford to rebuild according to the seismic design practices followed in developed countries, leading to reliance on donations or handouts. However, for safe, low-cost housing to be a sustainable option for the people of Haiti there needs to be a construction system that does not rely upon such financial donations. Using local materials, local businesses, and local labor would benefit the local economy and therefore, improve the quality of life in Haiti. At approximately $2,000, our design is a low-cost solution to the housing problem in Haiti, and it is strong enough to withstand both earthquakes and hurricanes.

As a response to this need, the design team Homes for Haiti has worked to design a new building material and simple home design for Haiti over the course of the year. We considered many materials, including straw bales, earthbags, bamboo-reinforced concrete walls, and ferrocement panels. Following research into these options and team discussions relating to the focus of the design, we decided to pursue a ferrocement panel design. A previous design team completed a similar project designing ferrocement panels, and we worked to optimize their design by reducing the cost of their design and aligning the strength requirements to the market in Haiti.

To gain confidence that our home design would be able to handle hurricane and earthquake loading, multiple methods were used. Computer modeling, theoretical calculations, and physical testing were all incorporated in the completion of the design. Computer modeling was completed through Algor Finite Element Analysis. Theoretical calculations were used to determine geometric properties of our panel. Physical testing included compression testing to determine the optimal concrete mix, beam moment testing to compare reinforcing material options, and panel bending testing to determine the ultimate strength of the beams. Results for these tests are summarized later in the report.

After the initial decision to design ferrocement panels, two main design options were considered. The first was a relatively standard ferrocement panel, using rebar and chicken wire as reinforcing materials. The second design option replaced the rebar with splints of bamboo as the main reinforcement. Also included in the design report is a cost estimate for our home built in Haiti.

The final design decision was between the two types of reinforcement: bamboo and rebar. Though both types of reinforcement provided the required strength for the design, rebar was chosen based on the desired longevity of the design. Although the cost of the bamboo home was approximately $200 less than the cost of the rebar home, the risk of the bamboo biodegrading within the concrete and losing its strength is too high to accept the bamboo design. Also, the bamboo splints tended to expand while the concrete cured, leading to cracking and improper bondage issues. We are confident that the below detailed rebar reinforced panel home is be able to withstand the forces caused by a Category 3 hurricane or a 7.0 on the Richter scale earthquake.
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1 Introduction

On January 12, 2010 an earthquake measuring 7.0 on the Richter scale, along with at least 50 aftershocks, devastated the nation of Haiti on Hispaniola. Estimates placed the death toll at 230,000 and the number of homeless at 1.2 million. Many buildings collapsed, including the Presidential Palace, due to improper construction practices. Because Haiti is the poorest country in the western hemisphere, most Haitians cannot afford the building materials and skilled labor required to build an earthquake resistant building that would meet building codes in earthquake zones within developed countries.

Given these circumstances, Homes for Haiti has worked to develop a low cost building design that would preferably use local materials from Haiti and unskilled labor to construct a small dwelling to house a family that can withstand an earthquake of equal magnitude to that which crippled the country. Within this report, the project is described in detail, specifically broken into the management of the project, the research completed, the design requirements, the materials testing, structural analysis, computer analysis, and construction details.

1.1 Course Overview

The Homes for Haiti project is being completed as a senior design project for the Calvin College Engineering Department in the 2011-2012 academic year. The two semester course serves as a capstone for the engineering program. In the first semester, teams narrow down project ideas through research and design decisions, ending with a statement of feasibility. In the second semester, the design is refined, ending with a prototype, a design, a construction manual, and complete design drawings. This report is the final report following the design phase of the project.

1.2 Project Overview

As described in the introduction, there is a need for a low-cost, high-strength building component for Haiti. We designed ferrocement construction panels, testing the use of local materials such as bamboo and bagasse for reinforcement. Ferrocement is a composite material that uses wire mesh as the principle reinforcement within concrete, allowing for greater build flexibility. The panel configuration allows for easier home construction in Haiti. This allows locals to purchase panels one at a time, accruing their home over time. Each panel contains an internal support system of wire mesh and reinforcing material, such as bamboo or rebar in the flanges. The internal support system is surrounded by thin layers of concrete. The panels are made separately and later connected to form the walls of a home. Further descriptions of this material and its selection are in Design Criteria, Alternatives, and Decisions section.

We performed materials testing of several panels to determine their strength and other properties. The panels were tested according to strength and displacement criteria. This builds on work completed by a senior design team nine years ago. The other project, Positive Reinforcement, created a building panel using ferrocement reinforced with multiple layers of welded steel mesh and steel rebar. Though a successful project in terms of meeting strength requirements, the panel designed by Positive Reinforcement was too expensive for the market in Haiti and was overdesigned. The cost estimates given by Positive Reinforcement were not accurate for Haiti, and therefore they cannot be compared to a realistic target cost. The Homes for Haiti team designed a home made of panels that meets strength requirements but is lower in cost.

1.3 Introduction to the Team

As pictured in Figure 1, the Homes for Haiti team is composed of four members: Derek Bandstra, Willem Both, Sarah Pennema, and Joe Westerbeke. All team members are senior engineering students in the civil and environmental engineering concentration.
Figure 1: Team members: Derek Bandstra, Joe Westerbeke, Willem Both, and Sarah Fennema

Derek Bandstra is from Terrace, British Columbia. He spent the previous summer as an engineering intern for Johann Bunte GmbH in Magdeburg, Germany. Derek is a member of the hockey team at Calvin College. He will be returning to Canada upon graduation from Calvin to work with Maple Reinders, Inc. in Calgary, Alberta.

Willem Both is from Owen Sound, Ontario. Last summer, Willem worked with Professor Sykes doing engineering research here at Calvin. Willem is a member of the track team at Calvin College. Willem has one semester left and is planning on completing it abroad. Upon graduation from Calvin, Willem plans on finding a job. His parents were missionaries in Haiti and are good contacts for general knowledge about the country, cultural appropriateness, and finding more contacts that have lived and live in the country currently.

Sarah Fennema is from Brookfield, Wisconsin. Last summer, Sarah worked for RMD Architects designing roof framing systems and drawing buildings in AutoCAD. Sarah runs both cross country and track at Calvin. Upon graduation from Calvin, Sarah will be pursuing a Master’s degree in Structural Engineering.

Joe Westerbeke comes from Oostburg, Wisconsin. During the previous summer, Joe worked at the Christman Company doing construction management for a project downtown Grand Rapids. Upon graduation, Joe will move to the Chicago area for a job at V3 Companies.

2 Project Management
To achieve the design goals described previously, we has remained focused and motivated. The project was primarily self-managed, with some supervision from our advisor Professor Wayne Wentzheimer. Other course instructors were invited to share insight as well.
2.1 Team meetings
Team meetings have occurred at least twice per week during the project. Scheduled team meeting occurred on Tuesday and Friday during the first semester. During the second semester, scheduled meetings have occurred on Tuesdays, but the hours of outside work have increased greatly. For all meetings, minutes are taken by Sarah Fennema and recorded in a project folder. The project folder also includes a record of all material testing and documentation of the research completed.

2.2 Document Organization
We used Google Docs to share important notes on research and contact lists. All other files, such as reports, test results, modeling files, and the budget, are in a folder on the Shared drive of the Novell Network, as provided by Calvin College, which can be accessed using Novell Netstorage. In this way, all team members are able to access any file from a computer connected to the internet.

2.3 Team Organization
The following gives a description of the role of each team member.

2.3.1 Derek Bandstra
In the first semester, Derek Bandstra was in charge of the business plan, transforming the design to a product and service that can be used by local entrepreneurs to implement this design across the country in a sustainable and economical way. Derek has been in charge of the budget, as it is connected to the business plan. In the second semester, Derek has been focused on the constructing the testing panels and completing supporting calculations for the design as well as updating our website.

2.3.2 Willem Both
Willem Both has contacts in Haiti and elsewhere that he was in constant conversation with to ensure correct design constraints and feasibility. He has been working with Joe to create and test finite element analysis (FEA) models for our proposed designs and purchasing materials. During the second semester, Willem has been focusing on the physical testing of the panels, as well as the generation of Inventor models for the FEA modeling and the construction manual.

2.3.3 Sarah Fennema
In the first semester, Sarah Fennema was at the head of research, looking for various material options and previous successful and unsuccessful designs. She also designed spreadsheets to calculate the design wind load and the seismic loading conditions. During the second semester, she has continued her work with the loading conditions, as well as designing the floor plans and structural details for the design of the home. As she is taking a course in vibration analysis, Sarah has been working to improve the computer models used for the project as well.

2.3.4 Joe Westerbeke
Joe Westerbeke has been working with FEA because of his previous knowledge in this area from taking a course in FEA. Using FEA, Joe has been modeling the material on the computer and applying forces in the computer model to ensure the material would not fail. Joe has also served as the project manager, making sure all deadlines are met. During the second semester, Joe has continued his role as project manager and worked with Willem and Derek on the physical testing of the panels.

2.4 Schedule
The schedule for the project was created in Microsoft Office Project 2007 (see Appendix I for a detailed schedule of tasks).

2.4.1 First Semester
Our focus during the first semester was on assessing the feasibility of the project. The major tasks that we hoped to accomplish are summarized in Table 1.
Table 1: Tasks to be Completed Within the First Semester

<table>
<thead>
<tr>
<th>Task Name</th>
<th>Deadline Date</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Proposal Report</td>
<td>9/14</td>
<td>Complete</td>
</tr>
<tr>
<td>Selection of Construction Materials</td>
<td>11/30</td>
<td>Complete</td>
</tr>
<tr>
<td>Selection of Configuration of Materials</td>
<td>11/30</td>
<td>Complete</td>
</tr>
<tr>
<td>Project Website Posted</td>
<td>10/26</td>
<td>Complete</td>
</tr>
<tr>
<td>Project Proposal and Feasibility Study Draft</td>
<td>11/18</td>
<td>Complete</td>
</tr>
<tr>
<td>Business Plan</td>
<td>12/9</td>
<td>Complete</td>
</tr>
<tr>
<td>Project Proposal and Feasibility Study Report</td>
<td>12/9</td>
<td>Complete</td>
</tr>
</tbody>
</table>

2.4.2 Second Semester
We focused on implementing design decisions in the second semester. Several major tasks for the second semester are listed in Table 2.

Table 2: Tasks Completed Within the Second Semester

<table>
<thead>
<tr>
<th>Task Name</th>
<th>Deadline Date</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pour Cylinders of Different Mix Designs</td>
<td>3/9</td>
<td>Complete</td>
</tr>
<tr>
<td>Compression Testing for Cylinders</td>
<td>3/16</td>
<td>Complete</td>
</tr>
<tr>
<td>Develop Decision Matrix to Choose Optimal Mix Design</td>
<td>3/16</td>
<td>Complete</td>
</tr>
<tr>
<td>Construction of Small Panels for Testing</td>
<td>2/14</td>
<td>Complete</td>
</tr>
<tr>
<td>Material Testing on Small Panels</td>
<td>3/27</td>
<td>Complete</td>
</tr>
<tr>
<td>Construction of Ferrocement Panels</td>
<td>4/25</td>
<td>Complete</td>
</tr>
<tr>
<td>Design Typical Home and Structural Details</td>
<td>5/5</td>
<td>Complete</td>
</tr>
<tr>
<td>Create Construction Manual for Panels</td>
<td>4/20</td>
<td>Complete</td>
</tr>
<tr>
<td>Preliminary Presentation of Design (CEAC)</td>
<td>4/20</td>
<td>Complete</td>
</tr>
<tr>
<td>Design Report Draft</td>
<td>4/20</td>
<td>Complete</td>
</tr>
<tr>
<td>Senior Design Night</td>
<td>5/5</td>
<td>Complete</td>
</tr>
<tr>
<td>Final Design Report</td>
<td>5/9</td>
<td>Complete</td>
</tr>
</tbody>
</table>

2.5 Budget
Since Willem is the most familiar with the procedures involved with ordering parts, he was in charge of ordering all materials. The budget itself has been managed by Derek, since he was the most familiar with the business plan. All major purchasing decisions were discussed with the whole team prior to sending orders through Bob DeKraakker, the Engineering Lab Manager.

2.6 Method of Approach
The following section describes the way in which we approached the problem. Specific procedures, complete results, and overall conclusions are included later in the report.

2.6.1 Design Methodology
The flow chart in Figure 2 portrays the structure of all of the design tasks for the year. Tasks are broken down into “Class Tasks” and “Design Tasks”.
2.6.1.1 Class Tasks
Class tasks were completed in accordance with the deadlines laid out on the syllabi for Engineering 339 and 340.
2.6.1.2 Design Tasks

2.6.1.2.1 Computer Modeling
The design of the panels has been completed as follows. The computer modeling involved three main cases. First, a base case computer model has been designed in Algor. The base case has been of the homes that existed in Haiti prior to the 2010 earthquake. The main goal of the base case model was to prove that current construction practices were not adequate and to gain confidence in the Algor model. The wind load calculations, and seismic load calculations are included in Appendix II: Wind Load Calculations. All computer models require both wind and seismic loading conditions. The second model tested the whole building structure. As in all tasks of design, iteration was necessary.

The design drawings for the house were completed at the end of the project. The panel drawings and the building drawings are detailed enough that a skilled worker can understand them and could construct a house, but simple enough that an unskilled laborer could understand the majority of the design. Figure 3 is a preliminary design drawing for the ferrocement panels.

![Figure 3: Preliminary Design of Ferrocement Panel (dimensions in millimeters)](image)

2.6.1.2.2 Physical Modeling
The physical modeling of the project involved strength testing of both small beams to test the proposed reinforcing materials and the full panels with these materials. For the small beams, the addition of bagasse was analyzed as well. For example, in the small beam testing, several “control samples” were prepared, using only concrete. In this way, we were able to judge that the bagasse did not help the strength of the concrete. The full panels have been tested for strength and tested for deflection. The results of tests completed on the full panels were compared to those of Positive Reinforcement, the senior design team from 2003.
3 Requirements

3.1 Structural Performance
The main goal for the ferrocement panel was to ensure the individual panel and the structure as a whole did not experience ultimate failure under Category 3 hurricane wind loads or seismic loads of a 7.0 magnitude on the Richter Scale. The Category 3 hurricane loads were chosen after conversations with Larry Hulst, an experienced structural engineer who has worked in Haiti. He explained that designing for Category 3 hurricane wind loads was the standard practice for foreign nonprofit groups providing housing. Further research has justified this decision. Only six hurricanes stronger than Category 3 have hit Haiti in the last 150 years. None of these hurricanes have been Category 5 hurricanes, and only two were Category 4 hurricanes.6

The ferrocement panel must also withstand an earthquake ranking 7.0 on the Richter scale. This is the same magnitude earthquake that struck Haiti in 2010. In general, earthquake loads were less important because the weight of the panels was much lower than that of other building materials, such as concrete block.

The goal is to have the panels and overall structure strong enough that it would remain upright even after storm events, keeping the residents safe. The owners may need to replace panels after a severe weather event if there is visible cracking, but the panel would remain structurally sound. Even in developed areas of like the United States, a structural engineer’s goal is simply to prevent ultimate failure, though often buildings would need serious renovations to guarantee structural integrity for the next extreme event.

3.2 Cost and Materials
Another key requirement for the final design was its affordability. Since the average annual income in Haiti is about $400, the final design must be low cost to ensure it is affordable for Haitians.7 The two designs considered, a ferrocement home with rebar reinforcing or a ferrocement home with bamboo reinforcing, differed in cost. Without a contingency, the home using rebar would cost approximately $1,700, while a home using bamboo reinforcement would cost about $1,500. These estimates are higher than our initial estimates of $800 due to increased knowledge about the design. Even with this cost increase, our design is competitive with the current price of about $2,500 for homes in Haiti.8 These homes are constructed by the CRWRC using wood construction.

We aimed to use materials that are available in the country of Haiti, both to keep the cost down as well as build the Haitian economy. The use of local materials would make the design sustainable as well by using local resources. Also, the design would require less outside aid.

3.3 Cultural Considerations
The design norms of cultural appropriateness and trust are also important constraints. Before the earthquake, homeowners preferred houses that were made completely of concrete blocks, including concrete roofs. During the earthquake, many of these homes and other buildings collapsed, leaving many Haitians trapped under concrete roof pieces. Since the earthquake, many Haitians doubt the integrity of the concrete block structures and afraid to continue living in homes with concrete roofs. Thus the design must be sensitive to the concerns that post-earthquake citizens of Haiti have. Our design does not include a concrete roof, but instead recommends the use of a wooden truss system with tin sheets.

The type of building and the building material also have social status implications. Living in earthen homes would be seen as a reduction of status because concrete homes are more respected.9
4 Material Research
Initially, we chose four materials that appeared to be feasible construction alternatives for new building designs in Haiti, based on our research and evaluation. We researched each material extensively, considering strength, material properties, cost, and availability. Our initial research was not limited to these four materials, though the focus was on these materials because their material properties seemed well-suited for building design.

4.1 Straw Bale Construction

4.1.1 Background
Baled wheat, oats, barley or rye are the primary materials in a straw bale. These materials are typically waste products, sold by farmers for animal bedding or for landscaping. In recent years, straw bales have found a new use as a building material. Construction workers have created straw bale houses by stacking the straw bales within a wall, and then applying some sort of stucco as a plaster to ensure the walls are durable, and do not break down with time. People in developing countries as well as developed countries construct straw bale homes, though this construction occurs more often in developing countries because of the price.

Many are skeptical about straw bale construction because of moisture issues. If kept dry, the buildings can be extremely durable depending on the type of plaster used. Even if cracks appear in the straw bale walls, they are easily repaired with cement or plaster. According to the California Strawbuilding Association, the straw would not decompose if cared for correctly\textsuperscript{10}, though Dr. Owen Geiger, a team contact on straw bale construction, argued with this. Dr. Geiger, an author, engineer, licensed contractor, and consultant for international housing projects, responded to our questions saying that building straw bale homes in Haiti was not worth the risk with all the flooding, hurricanes, and earthquakes in Haiti. According to his statement, eventually water finds its way into the walls and the house would have to be demolished.\textsuperscript{11}

4.1.2 Material Properties
Most material properties are hard to quantify when it comes to straw bales. As compressed pieces of organic material, the modulus elasticity and the yielding stress are difficult to define. The material also performed better than a rigid, wood-framed wall in earthquake simulations because of its flexibility in earthquake type loading.\textsuperscript{12} Though this building method shows potential, a straw bale design would be extremely difficult to test and to model.

4.1.3 Cost and Availability
Straw bales are easy to obtain in Haiti because of the large amount of agricultural land available. The potentially low cost and availability of the straw bales makes this construction method appealing.

4.2 Earthbag Construction

4.2.1 Background
Earthbag use has been present for many decades, especially for flood control. In construction, a worker fills a bag with some sort of dense material including clay, gravel, soil, or even shells. Often, builders use polypropylene bags because of their cost and availability, though other types of bags are acceptable as well. The main issue in finding a material to use is to make sure the material does not compress after being stacked. Next, a worker stacks the bags in a staggered configuration. The bags can create arches or a dome roof, though some choose to attach their own roof. The arches made of earthbags are appealing to homebuilders because of the simplicity and their lack of need for additional materials. Many add a tin roof because many do not enjoy the aesthetics of the earthbag dome houses. Next, one covers the earthbag walls with some sort of plaster, ranging from earthen plaster, lime plaster, cement stucco, or papercrete.
Often builders place pieces of barbed wire between the bags to keep the bags from moving in the case of lateral loading.\textsuperscript{13}

4.2.2 Material Properties
Again, the material properties of earthbags are difficult to quantify, though it is possible. Bryce Callaghan Daigle wrote his thesis on the structural behavior of earthbag housing and put a quantity to some earthbag properties. He tested different size bags for their failure load, the load at which the bag rips, resulting in material loss. For smaller size bags, the resulting average failure load was 360 kN and for larger bags, the resulting average failure load was 700 kN, both of which are acceptable for the loading experienced.\textsuperscript{14}

4.2.3 Cost and Availability
Earthbag construction is appealing because of the low material costs. While the materials are inexpensive, there is a large amount of labor needed to create a modest-sized house. Typically, a polypropylene bag costs approximately $0.11, though one source states the cost in Haiti is closer to $0.13. Homebuilders ideally use a material that is free and readily available at the site of construction, either gravel or soil. Plaster is also a readily available material that workers easily and economically make or purchase before construction.\textsuperscript{15}

4.3 Bamboo Construction

4.3.1 Background
Bamboo is used as a construction material all over the world, though usually it is used as the actual walls of the building. The U.S. Navy researched the use of bamboo not as an actual wall component, but instead as the reinforcement in concrete. Bamboo has a high tensile stress, and can do an exceptional job at providing necessary tensile reinforcement for the ferrocement panels.\textsuperscript{16} There are more than 1,500 species of bamboo, which grow in an assortment of terrains all over the world. In addition, bamboo growth exceeds the growth of most wooden plants, with a top rate of 5cm per hour and some species growing up to a meter and a half in one day.\textsuperscript{17}

4.3.2 Material Properties
The structural properties of bamboo can be up to 11 times stronger than steel, though this is measuring the internode, the strongest segment of a piece of bamboo. The strength at the nodes, though, is much lower, and is where the material properties are measured.\textsuperscript{18} Figure 4 below shows the different components of bamboo.\textsuperscript{19}

![Node Internode Node](image)

**Figure 4: Bamboo Components**

The U.S. Naval Civil Engineering Laboratory tests resulted in the following table (Table 3) of bamboo’s material properties.
Table 3: Mechanical Properties of Bamboo

<table>
<thead>
<tr>
<th>Mechanical Property</th>
<th>Value (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate compressive strength</td>
<td>55.2</td>
</tr>
<tr>
<td>Allowable compressive stress</td>
<td>27.6</td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>124.1</td>
</tr>
<tr>
<td>Allowable tensile stress</td>
<td>27.6</td>
</tr>
<tr>
<td>Allowable bond stress</td>
<td>0.345</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>17200</td>
</tr>
</tbody>
</table>

As a comparison, for ASTM A36 structural steel, the ultimate tensile strength is 400 MPa and the allowable tensile strength is 250 MPa.\(^{20}\)

Though the source does not state what species of bamboo these properties are for, it does list the following general requirements to ensure the bamboo has these material properties: \(^{21}\)

1. Use only bamboo showing a pronounced brown color. This ensures that the plant is at least three years old.
2. Select the longest large diameter culms or stalks available.
3. Do not use whole culms of green, unseasoned bamboo.
4. Avoid bamboo cut in spring or early summer. These culms are generally weaker due to increased fiber moisture content.

Typical grade 40 steel rebar has a minimum yield strength of 275 MPa and a minimum tensile strength of 480 MPa. The modulus of elasticity of steel is 200 GPa. Although the steel material properties are much higher, the bamboo is less expensive and therefore can be added to the concrete in larger quantities.

4.3.3 Cost and Availability

Bamboo was not available in Haiti until recently. In 1999, Organization for the Rehabilitation of the Environment began bamboo plantations in Haiti. These plantations provide a way to fight against ravine and hillside soil erosion, as well as provide a viable substitute for wood. Other sources of bamboo are nearby countries, including Panama and Ecuador. One of our contacts, Bruce LeBel, the executive director of World Shelters, provided costs for imported bamboo from these countries, which ranged from $5.00 to $6.50 for a 3-meter length of bamboo. These costs were current for the first quarter of 2010, though LeBel stated the prices change often. Another contact, Wayne de Jong of the CRWRC, states the local price of bamboo in Haiti as $25 for a dozen 3-meter lengths, putting the price at about $2.10 per piece of bamboo. This is much less expensive than importing bamboo.

4.4 Bagasse Construction

4.4.1 Background

Bagasse is a natural byproduct of sugarcane production. Bagasse contains 50% fiber, 30% pith, while the remainder consists of moisture or other organic solids.\(^{22}\) According the Encyclopedia of the Nations, sugarcane is Haiti’s second largest cash crop, after coffee. This makes bagasse, a typically unused material, an especially wise choice as an alternative.\(^{23}\) Mixing the small natural fibers in the concrete panel has potential to decrease the amount of micro-cracking possible in the concrete. The fibers allow the concrete to hold together when the panel begins to bend.
4.4.2 Material Properties
The material properties of bagasse fiber depend largely on the type of sugar cane plant, its maturity, and the efficiency of the milling plant at removing the pith and other solids. Construction Competence and Consulting provides typical values for a bagasse fiber. A single fiber has an approximate diameter of 0.30 millimeters. The modulus of elasticity ranges from 15,000,000 to 19,000,000 MPa, while the ultimate strength ranges from 184,000 to 290,000 MPa.

We had doubts about the preservation of natural material in concrete. Professor Dornbos, a Calvin College Biology professor, responded to this doubt saying he did not think it would decompose if the concrete were sealed with concrete sealant and not exposed to moisture.

4.4.3 Cost and Availability
There is approximately 2,756,000 hectares in Haiti, and about 12% of this land is dedicated to permanent crops. While sugarcane is not the country’s most profitable crop, it claims by far the greatest volume of production. In 2009, Haiti produced 1,110,000 metric tonnes of sugarcane, while the next highest crop produced was sweet potatoes at 271,600 metric tonnes. Bagasse is a byproduct of sugar cane production, and therefore the source of bagasse in Haiti is exceptionally high. While there is plenty of sugarcane being produced, a definite amount of bagasse produced has not been found. A typical panel would only be 2% bagasse, and even if a few hundred homes were built using this material, the amount of bagasse used would still be an insignificant relative to their total bagasse. The approximate cost of bagasse is $10 per cubic meter, so the cost of bagasse per household would be less than $1. The cost and availability of bagasse make this an extremely appealing option for a concrete additive in Haiti.

5 Configuration of Material
Another consideration of the design is the configuration of the materials described above. Several options were considered.

Earthbags and straw bales can maintain only one configuration. Both materials have stacked walls, built similarly to a building with blocks. Bagasse and bamboo construction, meanwhile, are used within concrete, which can be configured in two main ways.

The first configuration for the concrete is as poured concrete walls. As Haitians do not trust concrete above their heads, poured walls higher than one meter would not be accepted. However, the remaining distance to the roof must be filled somehow. Potential designs included having woven bamboo as a wall and extending the roof overhangs.

The second configuration for the concrete is as ferrocement panels. Ferrocement is a form of concrete that foregoes the use of coarse aggregate and uses additional reinforcing material in the form of wire mesh for added shear strength. Traditional reinforcement still provides the majority of the tensile strength. In the panel configuration, the homes can be built incrementally, which follows the typical construction practices used in Haiti. Typically, Haitians build homes by purchasing one concrete block at a time. Though a ferrocement panel would represent a larger financial investment, each panel could still be purchased individually. The original idea for the project was to design a ferrocement panel home, similar to a design completed by a senior design team in 2003.

6 Design Criteria, Alternatives, and Decisions
Because of the nature of the project, the design stage involved two levels of decisions. First, we narrowed its focus to four design alternatives, as described in Material Research. The section Initial Design Decisions analyzes these alternatives. This initial design decision was completed within the first semester,
and therefore, certain properties of the design have changed since the decision. Based on the design criteria, we decided to pursue a ferrocement panel design. Ferrocement Focused Design Alternatives describes design alternatives related to the ferrocement panels themselves. The second major decision determined what type of reinforcing material to use in the ferrocement panel. This decision is highlighted in section the Conclusion section.

6.1 Initial Design Decisions
During the first semester, we developed four design options based on the research previously described. We chose and weighted design criteria and scored each of the design alternatives, from one to ten, according to the criteria in Table 4.

Table 4: Decision Matrix Between Building Materials

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Ranking</th>
<th>Straw Bales</th>
<th>Earth Bags</th>
<th>Bamboo Reinforced Solid Wall</th>
<th>Ferrocement Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feasibility</td>
<td>40%</td>
<td>1</td>
<td>2</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>Cultural Appropriateness</td>
<td>10%</td>
<td>4</td>
<td>1</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Trust</td>
<td>10%</td>
<td>2</td>
<td>6</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>Estimated Material/Equipment Cost</td>
<td>20%</td>
<td>6</td>
<td>10</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Ease of Construction</td>
<td>10%</td>
<td>7</td>
<td>10</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>Ease of Testing/Modeling</td>
<td>10%</td>
<td>1</td>
<td>0</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>Total Score</td>
<td>100%</td>
<td>3.0</td>
<td>4.5</td>
<td>4.8</td>
<td>6.6</td>
</tr>
</tbody>
</table>

We assigned weights to each design criteria according to importance. The score given to each alternative was based on comparison to all materials, not just the four considered. The following explains each criterion in detail, and tables justify the scores assigned to each design alternative.

6.1.1 Feasibility
This criterion stemmed from group discussions and opinions about the appeal and practicality of an alternative. This criterion included the difficulty of physical construction for testing purposes, the appeal of continuing work from the 2003 design team, the conduciveness to applying engineering principles, the difficulty level of the design, and the possibility of creating an innovative product. We weighted this criterion the highest because it is multi-faceted and directly affects the interest and drive of the team members and therefore the quality of work. Table 5 shows the justification for the scores of each alternative.
Table 5: Justification of Scores for Design Alternatives (Feasibility)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Have been designed already, not many engineering principles to apply</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Have been designed already, not many engineering principles to apply</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Have been designed already</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Most interesting to construct and test, able to optimize work completed by Positive Reinforcement, innovative combination of materials</td>
</tr>
</tbody>
</table>

6.1.2 Cultural Appropriateness

The culture of Haiti is much different than in the United States, and therefore special research and considerations were made to ensure that the design chosen is acceptable. Using helpful information from Willem’s parents, we rated the cultural appropriateness of each alternative as seen in Table 6 below.

Table 6: Justification of Scores for Design Alternatives (Cultural Appropriateness)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Not a status symbol like a concrete home, unknown whether this would be acceptable because it is not used in Haiti</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Living in an earthen home seen as a lower social status by the Haitian people</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Concrete homes are status symbols</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Concrete homes are status symbols</td>
</tr>
</tbody>
</table>

6.1.3 Trust

Since the 2010 earthquake, Haitians are hesitant to trust concrete above their heads. This fear restricted the home from being made entirely of panels as Positive Reinforcement had designed. Haitians still prefer concrete walls so if concrete is chosen it would be limited to walls. The fear of concrete falling down on them was the first consideration for this design criterion. It also covered climate considerations, as the people of Haiti must trust that the design can endure seasonal climates, especially the rainy season. Table 7 describes both considerations for each alternative.
Table 7: Justification of scores for design alternatives (Trust)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Not trustworthy during the rainy season due to mold issues</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Trusted for strength</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Design trusted because concrete does not extend above heads of humans</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Not as trustworthy because concrete at full height, but sectional nature gives impression of weak points</td>
</tr>
</tbody>
</table>

6.1.4 Estimated Material Cost
The estimated material costs considered for this criterion were from research and comparisons between Haiti and countries with a similar climate and level of development. The bamboo reinforced solid wall and the ferrocement panels received the same score because at the time the decision was made, the ferrocement panels were not going to contain rebar, and therefore, the materials for the bamboo reinforced solid wall and the ferrocement panels were the same. Larry Hulst gave general cost comparison estimates of the main components from each option. Table 8 describes the scores for each design alternative.

Table 8: Justification of Scores for Design Alternatives (Estimated Material Cost)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Material is inexpensive, but cost of labor higher</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Material is inexpensive and readily available</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Bamboo is less expensive than steel rebar, but requires cost of cement and bamboo</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Bamboo is less expensive than steel rebar, but requires cost of cement, bagasse, and bamboo</td>
</tr>
</tbody>
</table>

6.1.5 Ease of Construction
This included estimated time and skill required to construct an entire home. Varying levels of skill and time can produce varying performance for straw bales and earthbags. We estimated what amount of skill and time would be required to produce a home that would meet our design goals. We ranked design alternatives according to the justifications seen in Table 9.
Table 9: Justification of Scores for Design Alternatives (Ease of Construction)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Requires skill</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Skill level varies depending on design, but taken as low</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Extremely difficult to pour concrete, difficult to make large forms, concerns about concrete consistency</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Made on- or off-site, single batch of concrete makes one panel, repetitive nature would increase production, skill required to attach panels to each other, the foundation, and the roof elements</td>
</tr>
</tbody>
</table>

6.1.6 Ease of Testing

Materials Testing Consultants offered to allow testing of concrete samples. We also replicated the simple test that Positive Reinforcement did using sand bags. For straw and earth bags, no known testing procedures existed except creating a building full scale and testing on large shake tables. The use of shake tables was unlikely because they are in such high demand. In addition, being able to test the material on a smaller scale was desirable to reduce testing materials’ costs. The ability to model accurately how the material would respond to stresses was also advantageous. Table 10 explains the ratings of each design alternative in this category.

Table 10: Justification of Scores for Design Alternatives (Ease of Testing)

<table>
<thead>
<tr>
<th>Design Alternative</th>
<th>Justification of Score</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straw Bales</td>
<td>Difficult to test response of material to moisture, difficult to model in Algor</td>
</tr>
<tr>
<td>Earth Bags</td>
<td>Would require earthquake simulator for testing, difficult to model in Algor</td>
</tr>
<tr>
<td>Bamboo Reinforced Solid Wall</td>
<td>Difficult to test small samples, able to model in Algor</td>
</tr>
<tr>
<td>Ferrocement Panels</td>
<td>Easy to test small samples, able to model panels in Algor, difficult to model connections</td>
</tr>
</tbody>
</table>

6.2 Ferrocement Focused Design Alternatives

Based on the decision matrix in Because of the nature of the project, the design stage involved two levels of decisions. First, we narrowed its focus to four design alternatives, as described in Material Research. This initial design decision was completed within the first semester, and therefore, certain properties of the design have changed since the decision. Based on the design criteria, we decided to pursue a ferrocement panel design. Ferrocement Focused Design Alternatives describes design alternatives related to the ferrocement panel themselves. The second major decision determined what type of reinforcing material to use in the ferrocement panel. This decision is highlighted in section the Conclusion section.

Initial Design Decisions, we focused on a ferrocement design. There were several alternatives within the ferrocement panel design. The four design components considered were concrete, tensile reinforcement, wire mesh, and discontinuous fibers. The final design could have include any of several combinations of these components. We assessed each component according to its necessity for the strength of the overall design and its cost.
A major part of this project was testing and comparing different combinations of these design alternatives, we cannot make design decisions based solely on ranked criteria. Instead, design decisions were informed by computer modeling as well as physical construction and testing. The results of these tests are found below in the Material Testing section.

6.2.1 Concrete
In 2003, Positive Reinforcement maximized their strength of concrete with different ratios of sand, cement, and water. Initially, the mix design featured in their report was the mix chosen for our panel design. However, after conversations with Jim English, an experienced concrete worker, and discussions with our team advisor, several concrete mixes were poured and tested in compression. Results and decisions related to the mix design are covered in the Compression Testing section.

6.2.2 Tensile Reinforcement
Bamboo was one option to replace steel rebar. Bamboo would meet our objectives of having a low-cost design using local materials. Bamboo has high tensile strength and an increasing presence in Haiti. Ninety-eight percent of the country’s forests have been cut down so wood must be imported and is expensive. As bamboo has increased in popularity as an erosion control measure, it has increasingly become the economical choice for tensile reinforcement in concrete.

Steel rebar was the other major option for the tensile reinforcement design component. Bamboo was slightly less expensive than rebar. However, this decision also considered the fact that any steel must be imported, while bamboo is available from Haiti. As one goal for the design was sustainability and accessibility, the importation of a key design component could be a deciding factor.

The selection of tensile reinforcement was the primary design decision for the ferrocement panels. Much of the physical testing for the project was focused on comparing the strengths of these two materials.

6.2.3 Mesh
We compared square wire mesh to chicken (hexagonal) wire mesh. Positive Reinforcement did not test chicken wire mesh because it is less effective at handling tensile forces compared to the square wire mesh, despite being a quarter of the price. However, based on information from Ferrocement & Laminated Cementitious Composites, the additional strength from square mesh is relatively small (5% efficiency). The previous senior design team used four layers of square wire mesh in their panel, which was overdesigned, so decreasing the number of layers was also an option that could provide the required strength at a reduced cost.

Woven bamboo was also considered, but would require quite a lot of work and certainly only one mesh layer would be included in every panel.

6.2.4 Discontinuous Fibers
Bagasse has been proven to decrease micro-cracking within concrete, and was tested for strength. The effect on our dimensions was initially unknown, but it did not change the dimensions of the test panels.

7 Theoretical Calculations
We spent a lot of time during the second semester computing theoretical calculations in MathCAD to determine the properties and expected strength of the proposed ferrocement panels. We performed these calculations for both standard steel rebar as well as bamboo reinforcement. The physical properties for bamboo were found based on online sources. Though there were many important calculations, the key calculations were those used to determine the capacity of a single panel under wind loading. All calculations are in Appendicies VII - X.
7.1 ASTM Three Point Loading Flexural Test (Appendix VII)

The data in this section is from the tests described in the Material Testing Section and compared to the industry ‘rule of thumb’. The calculations take the observed applied force on the small beam and calculate the tensile strength of each panel. The beams tested had a tensile strength of 9.47 MPa. When converted to the 0.7 w/c, 350-lb of water mix that was used for the actual panel construction, a tensile strength of 7.4 MPa was achieved.

When using the industry rule of thumb, that the tensile strength of concrete is roughly 15% of the compressive strength, 19.3 MPa (2800-psi), a tensile strength of 2.9 MPa results. We chose to continue with a tensile strength of 2.9 MPa because it is more conservative and allows for a greater level of safety in our design.

7.2 Bamboo Reinforced Panel (Appendix VIII)

For the bamboo reinforced panel, the location of the neutral axis was determined based on the areas of material in the tension and compression zones of the panel. Concrete is omitted from the tensile zone because it provides little strength in tension. The transforming factors, \( n_{w} \) and \( n_{b} \), convert the material properties of the wire mesh and bamboo respectively to that of concrete which allows for proper combination in the neutral axis equation.

The uncracked transformed (modified for ‘all concrete’ material) moment of inertia (MOI) was calculated by subtracting an inner void MOI from that of the solid block, with the width and height of the panel and the uniform thickness of the flange. The cracked transformed MOI was calculated using concrete and the wire mesh in tension and the wire mesh in the compression side of the web and within the flange.

The cracked transformed MOI value allows the moment at which the concrete begins to crack in the tensile zone to be calculated, or cracking moments. This allows for the determination of whether or not the panel is to be modeled based on the cracked or uncracked MOI. Comparing the cracking moment of the concrete to the maximum allowable service moment, which is based on a modified yield strength of reinforcement, dictates that the panel needs to be modeled based on the cracked MOI. The yield strength is calculated using the area fraction of each reinforcement.

The maximum compression stress in the matrix (concrete) calculation determines that compression controls in terms of how and why the panel breaks under wind loading, assuming ideal bondage between the bamboo and concrete.

The panel in bending calculation specifies the maximum moment seen by the panel when the wind loading is applied based on modeling the panel as a simply supported beam and as a cantilever beam. As per the maximum allowable service moment calculation, which assumes ideal bondage and the reinforcing bamboo being located in the ideal location, the panel can handle up to a 20 kN-m worst case moment. The cantilever beam, being the larger of the two models, specifies a maximum moment applied of 3.1 kN-m. This highlights that each panel can handle the wind load exerted on it based on the calculations.

When located as part of the home design, each panel acts as something between what is represented by a simply supported beam and what is represented by a cantilever beam. This is due to inter-panel interaction through the connections.

The vibration test calculations were intended to provide an accurate modulus of elasticity of the composite panel, but the value produced is not accurate and the frequency value provided is more valuable for calibrating the Algor model as detailed in section Vibration Test section.
7.3 Rebar Reinforced Panel (Appendix IX)
The calculations performed for the rebar reinforced panel are identical to those performed for the bamboo reinforcement, outside of the material properties of the rebar. The transforming factor, $n_{Rb}$, acts the same as $n_{Bb}$ as highlighted in section 7.2 Bamboo Reinforced Panel (Appendix VIII) but with the appropriate rebar value.

From the MOI calculations, the maximum allowable service moment is much less than allowed with the bamboo panel, due to a much larger transforming factor. As with the bamboo panel, the compression strength of the concrete controls, assuming ideal bondage between the rebar and concrete and the ideal location of the rebar in the tension zone.

When comparing the exerted moment on a panel to the calculated capacity, 7.6 kN-m, the rebar panel is able to withstand the moments from the wind forces, 3.11 kN-m, with ease. The same is true with rebar as with bamboo that the panel acts as a beam somewhere between a simply supported and cantilever. Due to this unknown, the theoretical calculations performed do not provide a value that would conclusively specify whether the panels as a home can handle the wind forces, which is the reason for in depth Algor analysis (see the Computer Analysis section). The rebar panel also does not provide a reasonable composite modulus of elasticity from the vibrations testing.

7.4 Further Calculations (Appendix X)
Some of the additional calculations performed include the estimation of the strength requirement and capacity of the inter-panel connections. Each connection should expect 53 N-m of moment from the wind. This results in an expected stress of 966 MPa on each rod. This is roughly four times the estimated capacity of each rod. This value has only a small degree of meaning, however, as it is much more likely that the nut and rod strip or that the concrete fails before the rods do.

In the panel to foundation shear strength calculations, the behavior of the foundation connections in shear as a result of the wind loads on the ends were determined. The capacity of the connections under shear forces in both the horizontal and vertical axes are capable of withstanding the stress upon them.

The single panel web shear strength calculations determine the ability of the panel’s web to withstand the shear loading created by the wind. By calculation, the web sees 6.7 kN of shear force, and when compared to the capacity of a single panel, the concrete alone is able to handle this force at a 7 kN capacity. With the addition of the two layers of wire mesh, the capacity of the web in shear increases greatly (to 132 kN). But due diligence dictates that the panel capacity cannot exceed the nominal shear strength of the web, which it does. This means that the calculated capacity of the web is instead 39 kN, which is still more than enough to handle the shear loading.

The Panel in Compression calculations are simply to prove that the weight of the roof on an individual panel is negligible when compared to its capacity.

8 Material Testing
The two major tests were completed to determine mechanical properties for the concrete panel. The two tests were the compression strength test and the bending strength test. American Society for Testing of Materials standardizes these tests. We ran secondary tests as well such as the slump test to determine the workability and accuracy of the concrete mix created during each iteration of testing.
8.1 Compression Testing

8.1.1 Procedure
The purpose of this test was to compare different mixes and to find which concrete mix resulted in the desired strength. The compression test was performed by placing a standard concrete cylinder into a compression testing machine. The compression testing machine is owned by the Calvin College Engineering Department, which was donated by Material Testing Consultants, Inc. A typical cylinder is 30 cm high with a 7.5 cm radius. The testing procedures were carried out with the assistance of Mr. Bob DeKraakker. The compression testing machine applies a steady force to the surface of the cylinder until failure occurs. A picture of a cylinder in the test apparatus can be seen in figure below.

![Compression Testing Setup](image)

8.1.2 Mix Test Results
We used the compression testing to decide which concrete mix to use, targeting a 28-day compressive strength of approximately 17 MPa, or approximately 2,500 psi which was suggested by Larry Hulst, the team advisor. The results from the testing can be seen in Table 11 below. The calculations for the approximated 28-day compressive strength are included in Appendix XI. All sand and cement were received from the same lot which ensures our cylinders were consistent.

Table 11: Compression Testing Results

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.6</td>
<td>145</td>
<td>0.95</td>
<td>12.0</td>
<td>30.5</td>
<td>275900</td>
<td>15</td>
<td>22</td>
</tr>
<tr>
<td>B</td>
<td>0.6</td>
<td>159</td>
<td>3.81</td>
<td>12.1</td>
<td>30.5</td>
<td>320400</td>
<td>18</td>
<td>25</td>
</tr>
<tr>
<td>C</td>
<td>0.7</td>
<td>145</td>
<td>0.33</td>
<td>12.1</td>
<td>30.5</td>
<td>142400</td>
<td>8</td>
<td>11</td>
</tr>
<tr>
<td>D</td>
<td>0.7</td>
<td>159</td>
<td>7.62</td>
<td>11.8</td>
<td>29.2</td>
<td>249200</td>
<td>14</td>
<td>20</td>
</tr>
<tr>
<td>E</td>
<td>0.6</td>
<td>181</td>
<td>20.3</td>
<td>11.5</td>
<td>29.2</td>
<td>333750</td>
<td>18</td>
<td>26</td>
</tr>
</tbody>
</table>
After completing the testing, we created a decision matrix to decide which mix was most suitable for the project. The decision matrix can be seen in Table 12 below.

Table 12: Concrete Mix Decision Matrix

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Ranking</th>
<th>Mix A</th>
<th>Mix B</th>
<th>Mix C</th>
<th>Mix D</th>
<th>Mix E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Used</td>
<td>20%</td>
<td>5</td>
<td>2.5</td>
<td>10</td>
<td>7.5</td>
<td>1</td>
</tr>
<tr>
<td>Strength</td>
<td>50%</td>
<td>7</td>
<td>6</td>
<td>1</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>Workability</td>
<td>20%</td>
<td>2</td>
<td>4</td>
<td>1</td>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>Sand Used</td>
<td>10%</td>
<td>2.5</td>
<td>7.5</td>
<td>1</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>Total Score</td>
<td>100%</td>
<td>5.2</td>
<td>5.1</td>
<td>2.8</td>
<td>8.6</td>
<td>5.7</td>
</tr>
</tbody>
</table>

The total cement used was a design criteria because less cement results in a lower cost. The strength of the mixes was a very important design criteria. Mix D was the only mix which had a strength in the targeted range suggested by our industrial advisor. Workability was also an important criteria we used to choose our mix. The concrete had to be workable to ensure it would fill in between the mesh layers and not leave voids throughout the panel. Finally, the amount of sand used was also considered, though this was ranked the lowest of the design criteria because of the lower cost of sand. As seen in the decision matrix above, mix D was chosen, which consists of a 0.7 water/cement ratio and a base mass of water of 159 kg. The main reasons for the desirability of mix D was the approximate strength of the mixture as well as the workability.

8.1.3 Panel Compression Test Results

When mixing concrete for a panel, a cylinder was poured to test the approximate strength of each mixture used. When making the first panel, the chosen mixture behaved differently than it did in the original compression test. The mix was much dryer and would have resulted in much weaker concrete. To fix the problem we added cement and water in the appropriate 0.7 water/cement ratio. Our concrete consultant, Mr. Jim English, suggested the reason for this issue was because with time, the sand was drying out, and therefore becoming less dense, which resulted in too much sand being added to the mixture. Mr. English suggested that the water evaporating could result in a 4% weight change. From that point on, the appropriate amount of cement and water was added, and sand was added to create a mixture with a slump similar to the original concrete mix. The results of the compression tests for our panel mixtures can be seen in Table 13 below.
The Steel Rebar 1 mixture was when we found we had too much sand due to the evaporation problem. We overcompensated with our addition of water and cement, resulting in too strong a concrete. We decided to use this panel for our vibration analysis to determine the modulus of elasticity. The Steel Rebar 2 cylinder test results were uncertain because of how the test occurred. There was an initial break between 1557 and 2000 kN, though we reasoned this was because of an uneven top surface. The final break occurred at 2760 kN. The noted range is included to show there was an initial break somewhere within the range, though we still decided to use the 2760 kN as the final result. The remaining mixtures were near the target 17 MPa.

### Beam Moment Testing

#### Procedure

The purpose of this test is to quantitatively and qualitatively compare the different materials to be used in the panels. The beam moment tests were performed at Material Testing Consultants (MTC). MTC provided forms for a 15 cm by 15 cm by 50 cm beam, though we only filled the form up 5 to 8 cm in to determine how the materials would act in a situation more closely to that of ferrocement panels. The test was carried out according to the ASTM C78 - Standard Test Method for Flexural Strength of Concrete.

To begin, we calculated the loading rate to use with the specific samples using the following equation:

$$r = \frac{2Sbd^2}{3L}$$

where $r$ is the loading rate, $S$ is the rate of increase in the maximum stress on the tension face, $b$ is the average width of the specimen as oriented for testing, $d$ is the average depth of the specimen as oriented for testing, and $L$ is the span length. We calculated a load rate, $r$, of approximately 150 lbs/min, which was input into the digital display of the testing apparatus. This rate was important because it resulted in a loading which gave the most accurate readings. If the rate was too high, it could result in the machine breaking the panel without giving an accurate reading. Eventually during the test, we had to increase the loading rate because of the length of each test, though we still got good results for the panels except for two of the controls. We originally made two sets of three panels. The panels in each set contained the following materials:
• **Set 1**
  - Control
  - Bamboo
  - Rebar

• **Set 2**
  - Control
  - Wire Mesh
  - Bagasse

After the bamboo beam sample was created, we noticed the bamboo had caused the sample to crack. The bamboo had absorbed a lot of moisture, causing it to expand and therefore break the concrete beam. We decided to investigate further by testing whether sealing the bamboo would help prevent this problem. Two small samples were cut; one sample was kept untreated while the other was coated with sealant. Both samples were submerged in water for a set period of time and the samples were weighed periodically. A graph of this data can be seen in Figure 6 below. As seen in the figure, both samples seemed to be gaining water at approximately the same rate. To explore the possibility of treating bamboo, a third set of testing panels was created, which contained a control, an untreated bamboo sample, and a treated bamboo sample.

![Figure 6: Bamboo Sealing Results](image)

### 8.2.2 Results
The tests with various panels allowed us to compare the materials both quantitatively and qualitatively. There were a few conclusions made from the tests performed at MTC.

First, the use of bagasse is not beneficial to the concrete mix. In both the beam moment and compression tests, the samples containing bagasse performed worse than the controls. We concluded this was likely because the bagasse was often found in clumps, which weaken the sample along the plane where the clumps were found.
A second conclusion was that the wire mesh was acting as we suspected, preventing cracking in the sample as the force was slowly added. A picture of the sample taken looking at the side of the panel can be seen in Figure 7 below.

![Figure 7: Results of Mesh Sample Test](image)

As the machine pushed through the concrete, the concrete was never fully separated, and as can be seen in the figure, the chicken wire mesh held the concrete together through the sample failure. This is an important result, because the thin center section of the panel needs this type of reinforcement. Even if the concrete cracks, it still has the wire mesh support to keep the panel from completely separating and collapsing.

Finally, based on testing results, we concluded that the treatment of the bamboo did not help the beam sample in any way. The sample still cracked from expansion and also performed worse in the moment test, most likely because the sealant weakened the bond between the bamboo and the concrete.

The full results of the testing can be seen in Table 14 below. Some beam samples did not register because the force required to break them was not large enough to be recorded in the machine. The was likely because we were advised to raise the loading rate, though it should have remained lower for these beams.
Table 14: Beam Moment Test Results

<table>
<thead>
<tr>
<th>Beam Label</th>
<th>Cross Sectional Dimensions</th>
<th>Loading Rate [N/min]</th>
<th>Peak Load [N]</th>
<th>Modulus of Rupture [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set 1 Control</td>
<td>15.25cm x 5.6cm</td>
<td>676</td>
<td>6700</td>
<td>10.6</td>
</tr>
<tr>
<td>Rebar</td>
<td>15.25cm x 6.35cm</td>
<td>4448</td>
<td>21300</td>
<td>26.8</td>
</tr>
<tr>
<td>Bamboo</td>
<td>15.25cm x 6.9cm</td>
<td>2224</td>
<td>7000</td>
<td>7.4</td>
</tr>
<tr>
<td>Control</td>
<td>15.25cm x 5.5cm</td>
<td>2224</td>
<td>6200</td>
<td>10.4</td>
</tr>
<tr>
<td>Set 2 Bagasse</td>
<td>15.25cm x 6cm</td>
<td>2224</td>
<td>Did not register</td>
<td>-</td>
</tr>
<tr>
<td>Wire Mesh</td>
<td>15.25cm x 3.6cm</td>
<td>2224</td>
<td>2700</td>
<td>10.7</td>
</tr>
<tr>
<td>Set 3 Control</td>
<td>15.25cm x 5.5cm</td>
<td>2224</td>
<td>Did not register</td>
<td>-</td>
</tr>
<tr>
<td>Treated Bamboo</td>
<td>15.25cm x 6cm</td>
<td>2224</td>
<td>750</td>
<td>1.1</td>
</tr>
<tr>
<td>Untreated Bamboo</td>
<td>15.25cm x 6.35cm</td>
<td>2224</td>
<td>1100</td>
<td>1.4</td>
</tr>
</tbody>
</table>

8.3 Vibration Test

With the help of Professor Richard DeJong, we used vibration analysis to find the modulus of elasticity of a complete ferrocement panel. Due to time and material restraints, we decided to do the main test on the steel rebar panel, though we also tested a bamboo panel at one point as well to do a comparison with the steel rebar. The following equations were used to find an approximate value of the modulus of elasticity:

\[
\omega = 2 \cdot \pi \cdot f \quad \text{Equation 2}
\]

\[
\omega = 22.4 \sqrt[4]{\frac{E \cdot I}{\gamma \cdot A \cdot L^4}} \quad \text{Equation 3}
\]

where \( \omega \) is the natural frequency in rad/s, \( f \) is the natural frequency in Hertz that we measured with Professor DeJong, \( E \) is the modulus of elasticity, \( I \) is the moment of inertia, \( \gamma \) is the density of the composite metric, and \( L \) is the length of the panel. The tests gave a value of natural frequenc, \( f \), of the panel, which allowed the modulus value to be determined.

We took weekly tests because of the fact that the modulus changes as the concrete hardens. A small, steel plate was mounted on the end of the specimen. Next, a small accelerometer was attached to the plate by a magnet. The main purpose of the plate is to distribute the weight of the accelerometer. The accelerometer is only 3 grams in mass, but any outside weight could affect the results of the extremely sensitive test. The accelerometer and plate can be seen in figure below.
The entire panel was centered on two supports. The supports were located at a distance from the end of the panel using the following calculation:

$$0.224 \times \text{Total Panel Length}.$$

At this distance, only the first mode resonates, making it easy to spot the natural frequency on the graph from the accelerometer. Small rubber pads were placed between the wood supports and the panel. The experimental setup can be seen in figure below.

The accelerometer was then hooked up to a 1000x amplifier, which was then hooked up to a laptop. The software then monitored the vibrations through the panel after it was struck directly in the center of the web by a rubber hammer. The natural frequency readings for bamboo and steel can be seen in the figures below.
Figure 10: Acceleration Amplitude for the Steel Vibration Analysis

Figure 11: Resonant Frequency for the Steel Vibration Test
The results can be seen in Table 15 below for all tests. There is a maximum and minimum value of the modulus of because of some warping that occurs during testing.
Table 15: Modulus of Elasticity Results from Vibration Tests

<table>
<thead>
<tr>
<th>Week</th>
<th>Steel Reinforcement</th>
<th>Bamboo Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum Modulus of Elasticity $E_{\text{max}}$</td>
<td>Minimum Modulus of Elasticity</td>
</tr>
<tr>
<td></td>
<td>(ksi)</td>
<td>(MPa)</td>
</tr>
<tr>
<td>1</td>
<td>21,069.60</td>
<td>145.5</td>
</tr>
<tr>
<td>2</td>
<td>18,235.00</td>
<td>126</td>
</tr>
<tr>
<td>3</td>
<td>17,066.40</td>
<td>117.9</td>
</tr>
<tr>
<td>4</td>
<td>15,825.60</td>
<td>109.3</td>
</tr>
</tbody>
</table>

Through the weeks the natural frequency was becoming lower and lower, resulting in a lower modulus of elasticity. This is opposite of what we expected. We expected a higher modulus of elasticity as the concrete cured. Professor DeJong suggested this might be because the concrete is pulling away from the reinforcement, but that he was unsure of exactly what was occurring. There is a maximum and minimum range of the modulus values because of some warping that occurs. The warping calculations can be seen in the appendix. We also found the bamboo panel to have a weaker modulus of elasticity, which was expected.

8.4 Panel Bending Test at Calvin College

8.4.1 Procedure
The purpose of this test is to determine the bending strength of a complete ferrocement panel and to compare the bamboo and steel rebar as reinforcement. A three point loading test was used to determine the bending strength of the panels. Each panel was placed on two steel container supports, which were available in the Calvin College Engineering Building. A pallet was then used to load the panel with two line loads which were centered on the panels. Sand, gravel, and stones were loaded into bags and 5-gallon buckets and were used to load the panel until it failed. The total load in weight and the total deflection of the panel were recorded. The setup of the testing can be seen in figure below. After doing a few tests, we noticed the panels sometimes failed in compression and other times failed in tension. After talking to Professor DeRooy, we found out that a compressive failure is a sudden failure, that occurs on the top plane of the panel. In compressive failure there is little deflection. Tensile failure has much more deflection and is a much slower break. Many cracks occurred on the bottom surface of the panel which slowly grew as weight was added until ultimate failure. Compressive failure can occur when the concrete mixture is too weak. Tensile failure was expected throughout the tests, though we realized the importance of keeping the reinforcement near the lower surface, or the panel would fail at much smaller loads.
8.4.2 Results
The bamboo panel reached ultimate failure at a load of 550 kg, and an approximate deflection of 10cm. The end product of the panel bending test can be seen in the figures below. This panel failed in tension.
The steel rebar panel reached ultimate failure at a load of 660 kg and a deflection of approximate 3.8 cm. The results of this test were surprising. The rebar was expected to hold much more weight, though it only held slightly more than the bamboo panel. Also, the deflection was very low when the panel failed. After watching video of the failure we determined that this panel failed in compression. The panel barely showed any signs of cracking when it suddenly reached ultimate failure. This also made sense when looking at the compression test results from this specific concrete mixture. The 7-day strength of the concrete was about 15.2 MPa. The end results of the steel rebar testing can be seen in the figures below.
Though we did not get compression test results for the bamboo panel, the concrete mixture used for the bamboo panel was likely stronger than that of the steel rebar panel, with a strength of approximately 18 MPa, which allowed the bamboo panel to reach a larger deflection and fail in tension rather than compression. The difference in these two mixtures was another result of the evaporation of water from the sand. After the water evaporated from the sand, we initially struggled to recreate the desired strength of concrete. After two mixtures, we relied more on the the slump and feel of concrete to ensure we were
getting both the desired strength and the right workability. We also decided to create a stronger mix, so it would reach 19 MPa after just one week instead of a four-week strength.

We decided to create another bamboo and steel rebar panel to see how consistent the results were, and also aim for the 19 MPa one-week strength that we needed.

The next panel tested was the second steel rebar panel. The cylinder test showed that the concrete strength was near the ideal 19 MPa. The panel ended up failing at 550 kg and about a 2.5 cm deflection. We were very surprised by the results, but we assumed the panel once again failed in compression. The first steel rebar panel that failed in compression broke at 660 kg, though it had a weaker concrete strength. The rebar distance from the end was slightly more than the previous test. We made the conclusion that the reinforcement must be pushed closer to the tension surface to ensure we maximize the panels strength. The results of the test can be seen in the figures below. The concrete cylinders were used for the test to ensure the panel did not fall to the ground and crack so we could get a better idea of where the failure was actually occurring.

![Figure 20: Cracking from Failure of the Second Rebar Panel](image-url)
After vibration analysis was finished on the single steel rebar panel, we decided to test this as well. The concrete was at its four-week strength of approximately 28 MPa, though we still wanted to observe where the panel would break at this greater mixture strength. The panel failed at 800 kg and a deflection of about 5 cm. The panel failed in tension as can be seen from the failure photos, seen in the figures below.
Finally, we tested the second bamboo panel. The concrete strength of this panel was approximately 17 MPa. The panel failed at only 365 kg and a deflection of 7 cm. We found that the bamboo had raised up much farther than wanted, resulting in the concrete failing in tension on the bottom edge. We decided that these results should be noted and that the location of the reinforcement is very important, though the results do not show the true strength of our panels.

Video footage of the panel bending tests can be found on our website.\textsuperscript{43}

### 8.5 Connection Testing

#### 8.5.1 Procedure

The purpose of this test is to determine the strength of the bolt connections holding the panels together. Two half panels were constructed and connected using threaded rods. The ends of the panels were placed on stacked wood, which made sure the panels would not hit the ground even after deflecting. A modified pallet was placed on the centers of the connected pallet. Then we loaded the pallet similarly to the way we did in the full panel testing. The setup can be seen in figures below.
8.5.2 Results
The connections failed at a loading of 115 kg and a deflection of about 5 cm. We realized the test wouldn’t truly predict the strength of the connections, instead it showed us the load the concrete could
take in the connection areas. We decided to further explore the connection analysis using an Algor computer analysis.

9 Computer Analysis
The computer analysis portion of the project has been an integral part of every stage of the design process.

First, we created a base case design of a typical Haitian house in Autodesk Algor. We used the base case to ensure the calculated wind and earthquake forces are correct, and resulted in a typical pre-earthquake Haitian house collapsing, with stresses greater than the materials’ ultimate strength. The compressive strength of the concrete used in the Algor model is assumed to be approximately 9 MPa, so failure of the wall results with any stresses greater than this.

The second analysis was completed on the full sized ferrocement homes. The results of the vibration testing were used to calibrate the model.

Further description of the analysis completed can be found in the sections to follow.

9.1 Computer Analysis Loading

9.1.1 Hurricane Load Calculations
Since 1851, hurricanes in Haiti have been recorded and categorized. Since that time, Haiti has experienced four Category 3 hurricanes as recently as 1979, two Category 4 hurricanes as recently as 1964, and no Category 5 hurricanes. Due to the infrequency of Category 4 or 5 hurricanes, and upon the advice of Larry Hulst, we decided to design for Category 3 hurricanes. It would be a safer home if we were to design for category 4 or 5 hurricane but the increased cost would outweigh the safety benefits. At too high of a cost the design would simply not be built. The wind speeds of a Category 3 hurricane are between 179 and 208 km/hr so the wind speed chosen as a design parameter was 210 km/hr. An Excel spreadsheet was developed to compute the load seen by a structure with a basic wind gust speed of 210 km/hr.

For simplicity the gust was modeled not as a dynamic load but as a constant pressure on a windward wall and a leeward wall. The calculations for these pressures can be found in Appendix II. Modeling as a constant pressure is consistent with ASCE standards. The worst case scenario is when the pressures are on the longer 6.3 m walls so the results are from this configuration. These pressures were modeled on the 3D model within Algor. The wind load calculations assumed that the building was located in a relatively densely populated area, and that the building was not situated on a hill. These are parameters required for the calculation of pressures on a building due to a wind force.

9.1.2 Earthquake Load Calculations
Earthquake loads were calculated based on the short period acceleration of the ground from the 2010 earthquake based on data from the USGS website. The basic design principle uses the ground acceleration to create a coefficient of 0.83g, which when multiplied by the total mass of the building, yields the shear force seen at the base of the building.

9.2 Base Case Computer Analysis

9.2.1 Base Case Procedures
The base model building consists of concrete blocks 20.3cm by 20.3cm by 40.6 cm which creates a 6.1 m by 3.05 m building with 2.4 m tall walls. Larry Hulst, our industrial consultant, recounted how these concrete blocks were the dominant building material that he saw during his visit to Haiti. The roof was
modeled as if it had six trusses traversing the longer walls. The door and window were positioned identically to that of our design. This was not necessarily how all homes are built in Haiti but we decided to design a home of these specifications. Mortar is designed so the bonding strengths of the mortar to the block and the strength of the block exceed that of the concrete. Therefore, we modeled the building using continuous concrete walls. Another issue we faced is the fact that a concrete block has two holes through the center, which can be seen in the figure below.\(^47\)

![Figure 26: Typical Concrete Block](image)

To model the two holes in the concrete block in Algor would be very difficult. To ensure the accuracy of the model, we calculated an equivalent width of the wall to be 19.8 cm instead of the 20.8 cm wide wall. These calculations use the moment of inertias of the concrete block and found an equivalent solid wall width. These calculations can be seen in Appendix IV. The concrete used in Haiti does not meet the standard concrete properties that Algor offers. We calculated an approximate equivalent concrete modulus of elasticity using the following equation:

\[
E = 33 \cdot w_c^{1.5} \cdot \sqrt{f'_c}
\]  

where \(w_c\) is the weight of concrete in lb/ft\(^3\) and \(f'_c\) is the 28-day compressive strength of concrete in psi.\(^48\) Haitian-made concrete has an average compressive strength of 9 MPa, and the average weight of the concrete is approximately half of the weight used in the United States, which is approximately 1,200 kg/m\(^3\). The calculation resulted in a modulus of elasticity of 5 GPa. This is significantly smaller than the modulus of elasticity for quality concrete, which is approximately 20 GPa.\(^49\)\(^50\) The tensile strength of concrete is approximately 15% of the compressive strength which results in 1.4 MPa.

Algor internally calculated a new shear modulus for the concrete, using the following equation:

\[
G = \frac{E}{2(1 + \nu)}
\]  

where \(E\) is the modulus of elasticity and \(\nu\) is Poisson’s ratio for concrete, which is approximately 0.3.\(^51\)
The model also consisted of a pinned boundary condition along the bottom of the wall, rather than a fixed boundary condition, which most closely resembles the interaction between the concrete block and concrete foundation. It is difficult to determine what connection type is actually occurring and would likely be a combination of both. Using pinned as opposed to fixed would result in lower stresses at the connections. The final design uses pinned connections to properly compare the base case to the final design pinned connections were chosen.

Wood was modeled within Algor as having a density of 630 kg/m$^3$, a compressive and tensile strength of 20 MPa, and a modulus of elasticity of 9 GPa.

### 9.2.1.1 Hurricane Loading

The hurricane loading resulted in a stress of 0.14 MPa within the panels and 1.9 MPa at the connection point between the wall and the truss. These values do not exceed either the tensile strength of Haitian concrete or the compressive strength of wood. Therefore, the resulting loading from Category 3 hurricane winds would not cause failure for a concrete home. This result is exactly what we expected, since the larger concrete walls should hold up against a wind loading. The Algor modeling results can be seen in Figure 27 below.

*Figure 27: Base Case Results for Hurricane Loading*
9.2.1.2 Earthquake Loading
The earthquake forces were calculated by multiplying the mass of the building (Appendix III) by the 0.83g acceleration recorded during the 2010 earthquake in Haiti. The earthquake loading resulted in a stress of 2 MPa within the concrete and 20 MPa within the wood truss above the doorway. The home could have been constructed with a stronger wooden sill but the concrete failed in tension regardless. We expected this result, since earthquake loading depends heavily on the mass of the building. A building made completely of concrete is heavy and would require substantial reinforcement. As discussed previously reinforcement was often poorly used or non-existent. The results can be seen in Figure 28 below.

![Figure 28: Base Case Results for Earthquake Loading](image)

9.2.1.3 Confidence from Base Case
The correlation between the computer modeling results and reality gave us confidence that the results from applying the same methods to our ferrocement home would be reliable.
9.3 Ferrocement Panel Computer Analysis

We cannot be certain that extrapolating results from a panel to a full sized home is accurate. However, as building and testing an entire home was not feasible, another method of extrapolating from a panel to a home was required. The modulus of elasticity of our panel was unknown and is the critical material property used in Algor. Professor De Jong advised us to use vibration testing to calibrate our Algor panel’s and home’s modulus of elasticity. To gain confidence that our full sized home could withstand both hurricane and earthquake forces this process was followed:

1. Obtain modulus of elasticity from calibrating the Algor panel to physical vibration tests
2. Simulate the physical bending moment test within Algor and record stress
3. Simulate the hurricane and earthquake forces on the designed home and record stress
4. Compare the resulting stresses

9.3.1 Vibration Calibration

Initially, we attempted to calibrate the Algor model by calculating a modulus of elasticity from the vibration test and the equations listed previously in section 8.3. However, after further conversations with Professor De Jong, we decided to calibrate the Algor model by simulating a single panel vibration test using the Natural Frequency (Modal) Response Analysis. The Algor model calculated the first natural frequency of the single panel, which was matched to the measured natural frequency taken during the vibration testing.

9.3.1.1 Ferrocement Panel Calibration

A single panel of equivalent thickness calculated from the moment of inertia of the uncracked panel was drawn in Autodesk Inventor. This was done for both bamboo, seen in Appendix VIII, and steel, seen in Appendix IX. The uncracked moment of inertia was chosen because during the physical vibration tests the panels were uncracked. The bamboo and steel have an equivalent uncracked thickness of 7.7 cm and 7.9 cm, respectively. The panels were then imported into Algor and modeled as plates. The modulus of elasticity was varied within the Modal Response Analysis until a resulting frequency matched that of physical testing. This process was completed for both the steel and bamboo reinforced panels. The resulting modulus of elasticity was 20 GPa for the steel panel and 12 GPa for the bamboo panel. Poisson’s ratio was specified as 0.3 upon direction of Professor De Jong. The two calibrations can be seen below.
Figure 29: Bamboo Panel Frequency Calibration

Figure 30: Steel Panel Frequency Calibration
9.3.2 Simulating Full-Sized Bending Moment Test
A single panel of equivalent thickness calculated from the moment of inertia of the cracked panel was drawn in Autodesk Inventor. This was done for both bamboo, seen in Appendix VIII, and steel, seen in Appendix IX. During the full-sized bending moment tests the panels did bend and crack so the cracked moment of inertia was used. The bamboo and steel have an equivalent cracked thickness of 4.6 cm and 4.9 cm, respectively. The panels were then imported into Algor and modeled as plates. The full-sized bending moment tests were modeled within Algor and the stresses were recorded.

Pinned supports were used because the panel did not move, due to friction, during any of the physical tests. The forces within Algor were placed as line loads at the same point that was done during physical testing. The bamboo and steel panel simulated tests resulted in a maximum stresses of 8.4 MPa and 12 MPa, respectively. The Algor outputs can be seen in the figures below.

Figure 31: Bamboo Panel Simulated
9.3.3 Modeling of Full Sized Home

The full sized home, complete with two windows, a door, the wooden sill connecting the top of the panels, and wooden beams in place of the trusses, were drawn in Inventor. During extreme forces the home may deflect and crack so the equivalent thickness of the walls was determined using the cracked moment of inertia.

The calculated hurricane pressures were added to both the leeward and the windward side of the home. For hurricane modeling the Static Stress with Linear Material Models was used.

The earthquake forces were calculated by multiplying the mass of the building (Appendix III) by the 0.83g acceleration recorded during the 2010 earthquake in Haiti. This resulted in 31 kN distributed over twelve points at the base of the home. For earthquake modeling, Transient Stress was used, and this required a load curve. Upon the advice of Professor De Jong and research verification, a load curve of 0.5, -1, 0.5 over a period of 0.12 seconds was used.⁵³

Both analyses was performed on the bamboo and steel reinforced home. Walls were modeled as pinned to the ground which is most similar to our bolt connections.
9.3.3.1 Bamboo Reinforced Home

The hurricane analysis of the bamboo-reinforced home resulted in a maximum stress of 4 MPa and the earthquake maximum stress max was 2 MPa. The higher stresses at the windows can be handled by the wood lining these openings. The stress of the bamboo reinforced home under hurricane and earthquake conditions can be seen in Figure 33 and 34 below.

![Figure 33: Bamboo-Reinforced Home Hurricane Forces](image-url)
9.3.3.2 Steel Reinforced Home

The hurricane analysis of the steel-reinforced home resulted in a maximum stress of 4 MPa and the earthquake maximum stress max was 3 MPa. The stress of the bamboo reinforced home under hurricane and earthquake conditions can be seen in Figure 35 and 36 respectively below.
Figure 35: Steel-Reinforced Home Hurricane Forces
Figure 36: Steel-Reinforced Home Earthquake Forces

9.4 Results

The results of all the Algor simulations can be seen in Table 16.

Table 16: Algor Simulation Results

<table>
<thead>
<tr>
<th></th>
<th>Simulated Stress (MPa)</th>
<th>Hurricane Stress (MPa)</th>
<th>Earthquake Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bamboo</td>
<td>8.4</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>Steel</td>
<td>12</td>
<td>4</td>
<td>3</td>
</tr>
</tbody>
</table>

The results show that the simulated panel test resulted in much larger stresses than would be felt by a home in either a Category 3 hurricane or an earthquake similar to what hit Haiti in Jan 2010. We are confident our ferrocement panels would be able to stand up to hurricane and earthquake loading when placed in the full home configuration.

10 Structural Details and Specifications

All structural details and specifications are included as design drawings. These drawings are placed as PDFs on our website. The following sections explain some key design decisions related to the structural details.
10.1 Foundation Design
Though much of the foundation design was outside the scope of this project, the connection of the panel to the foundation was designed. The panels are designed to connect to the foundation using L-brackets connected with a threaded rod, and bolted to the foundation with anchor bolts.

10.2 Roof Design
The roof design is that of corrugated tin roofing on wooden trusses. The truss is a King truss because this type of truss uses the least amount of wood.

On the top of each panel, two cast in anchor bolts hold a 2x4 segment onto the top of the panel. A second 2x4 are screwed to the top 2x4 segments, spanning the length of the home. The wooden trusses can be screwed or bolted into the 2x4 plank.

10.3 Panel Design
The panels are nominally 2.5 ft by 7.5 ft by 2.5 in. The “shoebox” design is modeled after the design ideas found in the ferrocement textbook. Exact dimensions for the panel, form, and connections can be found in the design drawings.

11 Construction Procedure
We chose to create a construction manual which would describe the creation of the panel in detail, yet make the manual easy enough for an uneducated worker. To make the process as easy as possible, we took many pictures while constructing testing panels. The construction manual can be seen in Appendix XII. The manual gives a good description of how to build each item, and also supplies alternatives to use if one method does not work properly. Though the main aim for the manual is for a supervisor to use to train employees, if a worker ever had a question they could use the manual as a quick reference to keep work moving. The manual could also be used by individuals trying to use this method to create their own house. The manual is broken down into the following sections:

- Form Construction
- Concrete Mixing
- Panel Pouring
- Foundation
- Connections
- Panel-to-Panel
- Panel-to-Foundation
- Panel-to-Roof
- Roof Attachment

12 Abbreviated Business Plan
The following plan outlines a potential nonprofit organization structured around the product of ferrocement homes. The nonprofit would pinpoint the market of Haiti as its target. Included in the following sections are the development costs of the current project, projected costs for a home in Haiti, and implementation of the design in Haiti.

12.1 Development
Development costs for this project included the cost of supplies: bamboo, concrete, low-grade wood for the forms, wire mesh, rebar.
Funding for the development of the prototype came from the remainder of our budget from Calvin College. We formally requested a budget of $300 from Calvin to cover the expenses incurred, and the college granted a budget of $400 to allow for contingencies. The bagasse was free of charge because the Calvin College Biology department had an excess that we were able to use. The materials to make the concrete for testing purposes were donated by Grand Rapids Gravel.

Table 17: Team Budget

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost Per Unit</th>
<th>Amount</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bamboo (+ Shipping &amp; Handling)</td>
<td>$39.77</td>
<td>6</td>
<td>$239</td>
</tr>
<tr>
<td>Bagasse</td>
<td>$0.00</td>
<td>-</td>
<td>$0</td>
</tr>
<tr>
<td>Rolls of Poultry Fencing</td>
<td>$14.88</td>
<td>4</td>
<td>$60</td>
</tr>
<tr>
<td>2 x 4 Stud - 9' piece</td>
<td>$2.07</td>
<td>6</td>
<td>$12</td>
</tr>
<tr>
<td>1/2&quot; - 4' x 8' Pine Plywood</td>
<td>$20.47</td>
<td>1</td>
<td>$20</td>
</tr>
<tr>
<td>Cement</td>
<td>$0.00</td>
<td>180 kg</td>
<td>$0</td>
</tr>
<tr>
<td>Sand</td>
<td>$0.00</td>
<td>900 kg</td>
<td>$0</td>
</tr>
<tr>
<td>3/8&quot; Rebar - 10' piece</td>
<td>$5.55</td>
<td>8</td>
<td>$44</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>$375</strong></td>
<td></td>
<td></td>
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</tbody>
</table>

Above in Table 17, the local cost of bamboo in the United States is $250, including shipping and handling, and the cost for wood for forms was estimated at $40, though we ended up under budget for this material. Within our testing, a comparison of small panels with and without the mesh, steel rebar versus bamboo, and bagasse versus no bagasse required $60 worth of the wire mesh. We looked at travel to Haiti, but upon not receiving any external funding, such as through the Innotec Grant, we decided not to fly down to Haiti.

12.2 Production

For the production of a house made from the panels that result from our design, we calculated costs based upon material prices provided by Wayne de Jong. The direct production costs are in Table 18.
The bamboo cost is from a dozen dry pieces of about 3 meters in length and for each building, made of 30 panels, requires 183 meters or 61 pieces of bamboo. The cement cost comes from using 42.5-kilogram bags of cement. Each bag costs $7.50 and produces 1.164 panels. The sand cost comes from a desired 1:2:2 concrete mixture (water : cement : sand), based upon the same volumetric requirement as cement per panel. The cost of sand in Haiti is $16.25 per cubic meter. The costs for cement and sand are from a ‘Cost of Construction Materials’ pdf that provides ‘typical CMU (Concrete Masonry Unit) construction estimates in St. Marc, Haiti.’ The cost of labor is for four unskilled workers at a rate of $2 (compared to the November estimate of $6.25) per day and an unskilled labor supervisor working at a rate of $8 per day (compared to the November estimate of $11.25). The cost of labor is from the conversation that took place between us and Larry Hulst. We met the $2,500 shelters that CRWRC produces, and estimate our homes to be approximately $2,000 with a contingency. The homes are an even less expensive option than current organizations are producing, and therefore we believe our homes could be implemented in Haiti.

12.3 Production and Implementation

The end goal for this design is to create a repeatable model for Haitians to build themselves. Part of the marketing process is embedded in the implementation of our design, as we created a basic enough manual and construction process that unskilled laborers can build a functioning and structurally sound home under the supervision of a single knowledgeable worker. This would create accountability for doing it correctly and would create a process that Haitians can take pride in, having used their own materials to create homes for themselves. The product that we will send to Haiti will be the construction manual, which will possibly be translated in the near future into Creole, Haiti’s native language. The specifics within the manual, the actual procedure recommended, were practiced during the second semester when we created sample panels to maximize simplicity. This manual is included in Appendix XII.
13 Future Work
Though we consider the project a success as a whole, there are a few areas that we would explore further if there were more time to complete them. The connections of the panel passed the stress requirements in our Algor model, though we never had a chance to complete a second connections test to verify that the connections would truly hold in a real life situation. Though we performed a connection test, we also had issues with the location of the reinforcement, and we would like to create a better experimental setup to ensure our test is as close as possible to the forces it would see in a real life situation.

A second revision would be specifying the exact sizes of L-brackets and washers that are needed to ensure a safe setup. We addressed this for our bolt size, though we did not have a chance to specify the L-brackets which connect the panels to the foundation.

A final exploration that would be needed, though it would likely not be possible unless we built a complete home, is to look more closely at the total time and total workers needed for construction of a full home and the order of construction. Our cost estimate has a $300 dollar contingency, so we are confident our error would lie within this contingency, though figuring out the most efficient way to lay the home out on the foundation, to set the epoxy, and to finish the panel installation would be important.

14 Conclusion
The design of a low-cost, safe home for Haiti was the basic goal of the project. We are confident that the ferrocement panel design with two layers of chicken wire mesh and steel rebar fits these criterion. The price of a panel home with this configuration would be approximately $2,000, makes it a feasible option for home construction in Haiti. Through the application of physical testing, theoretical calculations, and computer modeling, the structural security of the design has been verified.

If the project were to be expanded on in the future, the focus of the design could be on differing configurations of the panels, connections, and specific construction techniques. Overall, the project has been successful at accomplishing its goal of designing a low-cost, structurally secure home for deployment in Haiti.


8 Hulst, Larry. Personal interview. 16 Nov. 2011.

9 Both, Mary M. Personal interview. 10 Oct. 2011.


27 Hulst, Larry. Personal interview. 16 Nov. 2011.


29 Both, 10 Oct 2011


Hulst, 17 Nov 2011

Hulst, 17 Nov 2011


American Concrete Institute. ACI Committee 318 (2008). *ACI 318-08: Building Code Requirements for Structural Concrete and Commentary*. 


57 Hulst, 17 Nov 2011
Appendix I: Work Breakdown Schedule

The work breakdown schedule for the entire design project is included on the following pages.
<table>
<thead>
<tr>
<th></th>
<th></th>
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<tr>
<td>Contact MTC</td>
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<td>Constructed Bamboo &amp; Rebar Panels</td>
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<td>Test at MTC</td>
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<td>Talked to Jim English about Mix Issues (Sand Density)</td>
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<td>Tested Bamboo &amp; Rebar Panels</td>
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<td>Design Report Executive Summary</td>
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<td>Vibration Analysis on Rebar Panel</td>
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<td>Increased Biaxial Tension Stiffness</td>
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</tbody>
</table>
16 Appendix II: Wind Load Calculations

## WIND LOAD CALCULATOR: ASCE 7-10

<table>
<thead>
<tr>
<th>INPUT</th>
<th>WHERE TO GET DATA</th>
</tr>
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<tbody>
<tr>
<td>V, basic wind speed</td>
<td>130 mph</td>
</tr>
<tr>
<td>Kd, wind directionality factor</td>
<td>0.85</td>
</tr>
<tr>
<td>Building category</td>
<td>II</td>
</tr>
<tr>
<td>I, importance factor</td>
<td>1</td>
</tr>
<tr>
<td>Exposure Factor</td>
<td>C</td>
</tr>
<tr>
<td>Is the building situated on a hill?</td>
<td>NO</td>
</tr>
<tr>
<td>Kzt, topographic factor</td>
<td>1</td>
</tr>
<tr>
<td>G, gust effect factor</td>
<td>0.85</td>
</tr>
<tr>
<td>Enclosure classification</td>
<td>Partially Enclosed</td>
</tr>
<tr>
<td>Gcpi, internal pressure coefficient</td>
<td>0.55</td>
</tr>
<tr>
<td>z, height above ground level***</td>
<td>8 ft</td>
</tr>
<tr>
<td>z_g, nominal layer of atmospheric boundary layer</td>
<td>900 ft</td>
</tr>
<tr>
<td>alpha, 3 sec gust speed power law exponent</td>
<td>9.5</td>
</tr>
<tr>
<td>Kz, velocity pressure exposure coefficient</td>
<td>0.849</td>
</tr>
<tr>
<td>qz or qh, velocity pressure</td>
<td>31.22 lb/ft^2</td>
</tr>
<tr>
<td>GCpf, external pressure coefficient</td>
<td>0.80</td>
</tr>
<tr>
<td>p, design wind load</td>
<td>38.40 lb/ft^2</td>
</tr>
</tbody>
</table>

### Calculations
- **Assuming the height of the building is less than 60 ft**
- **Determined by enclosure classification choice**
- **Determined by exposure factor choice**

**Output:**

- **p, design wind load** = 38.3971626 lb/ft^2
### Appendix III: Mass of Base Case and Panel Home

#### SEISMIC LOAD CALCULATIONS: Base Case

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Variable</th>
<th>Equation or Table Reference</th>
<th>Assumptions</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seismic Base Shear</strong></td>
<td>V</td>
<td>$V = C_s \cdot W$ (Eqn. 12.8.1)</td>
<td></td>
<td>103769.1</td>
<td>lb</td>
</tr>
<tr>
<td><strong>Seismic Response Coefficient</strong></td>
<td>$C_s$</td>
<td>$C_s = \frac{S_{DS}}{R/I_e}$</td>
<td></td>
<td>13.1214</td>
<td>ft/s²</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration</td>
<td>$S_{DS}$</td>
<td>$S_{DS} = \frac{2}{3} S_{MS}$</td>
<td></td>
<td>19.6817</td>
<td>ft/s²</td>
</tr>
<tr>
<td>Spectral Response Acceleration</td>
<td>$S_{MS}$</td>
<td>$S_{MS} = F_a S_S$</td>
<td></td>
<td>29.5225</td>
<td>ft/s²</td>
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<tr>
<td>Mapped Short Period Spectral Response Acceleration</td>
<td>$S_S$</td>
<td>$S_S = \text{USGS Website}$</td>
<td>$=83.35% \times$ gravitational constant</td>
<td>26.8387</td>
<td>ft/s²</td>
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<tr>
<td>Site Coefficients</td>
<td>$F_a$</td>
<td>Table 11.4-1</td>
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<td>Response Modification Factor</td>
<td>$R$</td>
<td>Table 12.2-1</td>
<td>A4. Ordinary Plain Concrete Shear Walls</td>
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<td>Importance Factor</td>
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<td>Risk Category II Building</td>
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<tr>
<td><strong>Seismic Weight of Building</strong></td>
<td>$W$</td>
<td>$W = 2W_{w1} + 2W_{w2} + W_{roof}$</td>
<td></td>
<td>7908.542</td>
<td>lb</td>
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<tr>
<td>Type 1 Wall Weight</td>
<td>$W_{w1}$</td>
<td>$W_{w1} = W_c \cdot L \cdot w_{bl}/12 \cdot H$</td>
<td></td>
<td>1860.833</td>
<td>lb</td>
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<tr>
<td>Type 2 Wall Weight</td>
<td>$W_{w2}$</td>
<td>$W_{w2} = W_c \cdot w \cdot w_{bl}/12 \cdot H$</td>
<td></td>
<td>930.4167</td>
<td>lb</td>
</tr>
<tr>
<td>Weight of Roof</td>
<td>$W_{roof}$</td>
<td>$W_{roof} = W_c \cdot L \cdot w_{bl}/12$</td>
<td></td>
<td>2326.042</td>
<td>lb</td>
</tr>
<tr>
<td>Weight of Concrete</td>
<td>$W_c$</td>
<td>$W_c = \text{Assuming 1/8 Weight of Normal Concrete}$</td>
<td>18.125 lb/ft³</td>
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<td></td>
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<tr>
<td>Length of Building</td>
<td>$L$</td>
<td></td>
<td></td>
<td>20 ft</td>
<td></td>
</tr>
<tr>
<td>Width of Building</td>
<td>$w$</td>
<td></td>
<td></td>
<td>10 ft</td>
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<tr>
<td>Block width</td>
<td>$w_{bl}$</td>
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<td>7.7 in</td>
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<tr>
<td>Height of Building</td>
<td>$H$</td>
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## SEISMIC LOAD CALCULATIONS: Equivalent Lateral Force Procedure

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<thead>
<tr>
<th>Parameter</th>
<th>Variable</th>
<th>Equation or Table Reference</th>
<th>Assumptions</th>
<th>Value</th>
<th>Units</th>
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</thead>
<tbody>
<tr>
<td>Seismic Base Shear</td>
<td>V</td>
<td>( V = C_s \times W ) (Eqn. 12.8.1)</td>
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<td>54827.48 lb</td>
<td>lb</td>
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<td>Seismic Response Coefficient</td>
<td>( C_s )</td>
<td>( C_s = \frac{S_D}{(R/I_e)} )</td>
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<td>6.560571 ft/s^2</td>
<td>ft/s^2</td>
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<td>Design Spectral Response Acceleration</td>
<td>( S_{DS} )</td>
<td>( S_{DS} = \frac{2}{3} \times S_{MS} )</td>
<td></td>
<td>19.68171 ft/s^2</td>
<td>ft/s^2</td>
</tr>
<tr>
<td>Spectral Response Acceleration</td>
<td>( S_{MS} )</td>
<td>( S_{MS} = F_A \times S_S )</td>
<td></td>
<td>29.52257 ft/s^2</td>
<td>ft/s^2</td>
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<tr>
<td>Mapped Short Period Spectral Response Acceleration</td>
<td>( S_S )</td>
<td>USGS Website</td>
<td>=83.35% * gravitational constant</td>
<td>26.8387 ft/s^2</td>
<td>ft/s^2</td>
</tr>
<tr>
<td>Site Coefficients</td>
<td>( F_a )</td>
<td>Table 11.4-1</td>
<td>Site Class D</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Response Modification Factor</td>
<td>( R )</td>
<td>Table 12.2-1</td>
<td>A6. Ordinary Precast Shear Walls</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Importance Factor</td>
<td>( I_e )</td>
<td>Table 1.5-2</td>
<td>Risk Category II Building</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Seismic Weight of Building</td>
<td>( W )</td>
<td>( W = 2 \times W_{w1} + 2 \times W_{w2} + W_{roof} )</td>
<td></td>
<td>8357.12 lb</td>
<td>lb</td>
</tr>
<tr>
<td>Type 1 Wall Weight</td>
<td>( W_{w1} )</td>
<td>( W_{w1} = W_{c} \times L \times \text{vol} )</td>
<td></td>
<td>2739.04 lb</td>
<td>lb</td>
</tr>
<tr>
<td>Type 2 Wall Weight</td>
<td>( W_{w2} )</td>
<td>( W_{w2} = W_{c} \times w \times \text{vol} )</td>
<td></td>
<td>1369.52 lb</td>
<td>lb</td>
</tr>
<tr>
<td>Weight of Roof</td>
<td>( W_{roof} )</td>
<td>( W_{roof} = W_{roofp} \times w_{P} \times L \times w_{P} \times w )</td>
<td></td>
<td>140 lb</td>
<td></td>
</tr>
<tr>
<td>Weight of Concrete</td>
<td>( W_{c} )</td>
<td></td>
<td></td>
<td>136 lb/ft^3</td>
<td></td>
</tr>
<tr>
<td>Length of Building (in panels)</td>
<td>( L )</td>
<td></td>
<td></td>
<td>10 panels</td>
<td></td>
</tr>
<tr>
<td>Width of Building (in panels)</td>
<td>( w )</td>
<td></td>
<td></td>
<td>5 panels</td>
<td></td>
</tr>
<tr>
<td>Panel Volume</td>
<td>( \text{vol} )</td>
<td></td>
<td></td>
<td>2.014 ft^3</td>
<td></td>
</tr>
<tr>
<td>Panel Width</td>
<td>( w_{P} )</td>
<td></td>
<td></td>
<td>2.07 ft</td>
<td></td>
</tr>
<tr>
<td>Panel Height</td>
<td>( H )</td>
<td></td>
<td></td>
<td>7.598 ft</td>
<td></td>
</tr>
</tbody>
</table>
Tables from the ASCE 7-10 code for calculating wind loads

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Directionality Factor, $K_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings</td>
<td></td>
</tr>
<tr>
<td>Main Wind Force Resisting System</td>
<td>0.85</td>
</tr>
<tr>
<td>Components and Cladding</td>
<td>0.85</td>
</tr>
<tr>
<td>Arched Roofs</td>
<td>0.85</td>
</tr>
<tr>
<td>Chimneys, Tanks, and Similar Structures</td>
<td></td>
</tr>
<tr>
<td>Square</td>
<td>0.9</td>
</tr>
<tr>
<td>Hexagonal</td>
<td>0.95</td>
</tr>
<tr>
<td>Round</td>
<td>0.95</td>
</tr>
<tr>
<td>Solid Signs</td>
<td>0.85</td>
</tr>
<tr>
<td>Open Signs and Lattice Framework</td>
<td>0.85</td>
</tr>
<tr>
<td>Trussed Towers</td>
<td></td>
</tr>
<tr>
<td>Triangular, square, rectangular</td>
<td>0.85</td>
</tr>
<tr>
<td>All other cross sections</td>
<td>0.95</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Category</th>
<th>Non-Hurricane Prone Regions</th>
<th>Hurricane Prone Regions</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.87</td>
<td>0.77</td>
</tr>
<tr>
<td>II</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>III</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td>IV</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td>Surface Roughness Factor</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or smaller</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>Open terrain with scattered obstructions having heights generally less than 30 ft. This category includes flat open country and grasslands.</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salf flats, and unbroken ice.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Exposure Factor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>For buildings with a mean roof height of less than or equal to 30 ft, Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 1500 ft. For buildings with a mean roof height greater than 30 ft, Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2600 ft or 20 times the length of the building, whichever is greater.</td>
</tr>
<tr>
<td>C</td>
<td>Exposure C shall apply for all cases where Exposure B or D do not apply.</td>
</tr>
<tr>
<td>D</td>
<td>Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5000 ft or 20 times the building height, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 ft or 20 times the building height, whichever is greater, from an Exposure D condition, as defined in the previous sentence.</td>
</tr>
<tr>
<td>Classification</td>
<td>Enclosure Classification Definitions</td>
</tr>
<tr>
<td>-------------------</td>
<td>--------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Enclosed</td>
<td>A building that does not comply with the requirements for open or partially enclosed buildings</td>
</tr>
<tr>
<td>Partially Enclosed</td>
<td>A building that complies with both of the following conditions: 1) The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope by more than 10%. 2) The total area of openings in a wall that receives positive external pressure exceeds 4 ft² or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20%.</td>
</tr>
<tr>
<td>Open</td>
<td>A building having each wall at least 80% open.</td>
</tr>
</tbody>
</table>
18 Appendix IV: Base Case Material Properties for Algor

**Base Case**

**Moment of Inertia of a Concrete Block**

\[ I := \frac{lo \cdot wo^3}{12} - 2 \cdot \frac{li \cdot wi^3}{12} = 640 \cdot \text{in}^4 \]

Moment of inertia of concrete block with two holes

\[ t := \left( \frac{12 \cdot I}{lo} \right)^{\frac{1}{3}} = 7.83 \cdot \text{in} \]

Equivalent thickness of solid block with same moment of inertia

**Modulus of Elasticity**

\[ wc := 75 \]

Weight of concrete in lb/ft^3

\[ 9 \text{MPa} = 1.305 \times 10^3 \cdot \frac{\text{lbf}}{\text{in}^2} \]

From research

\[ fc := 1300 \]

Compression strength in psi

\[ Ec1 := 33 \cdot wc^{1.5} \cdot fc^{0.5} = 7.728 \times 10^5 \]

Modulus of elasticity of base case

\[ Ec2 := Ec1 \cdot \frac{\text{lbf}}{\text{in}^2} = 5.328 \times 10^9 \text{Pa} \]
Appendix V: Cost Estimate Calculations

Concrete Mix

\[
\rho_{\text{cement per bag}} = 94 \text{ lb/ft}^3
\]

\[
V_{\text{panel}} := \left[ (1.5\cdot 30 - 98.4) \text{ in}^3 - (1.5\cdot 26 - 94.4) \text{ in}^3 \right] + (1.984\cdot 30) \text{ in}^3 = 0.061 \text{ m}^3 \quad \text{Panel Volume}
\]

\[
V_{\text{house}} := V_{\text{panel}}^{30} = 1.818 \text{ m}^3
\]

\[
V_{\text{cement}} := V_{\text{house}} \cdot 0.4 = 0.727 \text{ m}^3
\]

\[
m_{\text{cement}} := V_{\text{cement}} \rho_{\text{cement per bag}}
\]

\[
m_{\text{cement}} = 1.095 \times 10^3 \text{ kg}
\]

\[
bags := \frac{m_{\text{cement}}}{42.5 \text{ kg}} = 25.767 \quad \text{Bags Required}
\]

\[
cost := 7.50 \frac{\text{Dollars}}{\text{Bag}}
\]

\[
bags \cdot \text{cost} = 193 \quad \text{Dollars}
\]

\[
\frac{42.5 \text{ kg}}{V_{\text{panel}}^{0.4} \rho_{\text{cement per bag}}} = 1.164 \frac{\text{Panels}}{\text{Bag}}
\]

\[
V_{\text{sand}} = V_{\text{cement}}
\]

\[
sand := \frac{16.25}{\text{m}^3} \quad \text{Dollars}
\]

\[
V_{\text{sand}} \cdot \text{sand} = 12 \quad \text{Dollars}
\]

**Labor Costs:**

Unskilled Labor: Unskilled Labor Supervisor:

\[
wage_{\text{unsk}} := \frac{2}{\text{day}} \quad \text{Dollars}
\]

\[
wage_{\text{ULS}} := \frac{8}{\text{day}} \quad \text{Dollars}
\]

\[
time_{\text{const}} := 300 \text{ hr}
\]

\[
wage := (wage_{\text{unsk}}^4 + wage_{\text{ULS}}) \cdot time_{\text{const}} \quad \text{wage} = 200 \quad \text{Dollars}
\]
20 Appendix VI: Specification Drawings
HOMES FOR HAITI DESIGN PROJECT
NORTH SIDE ELEVATION

- 0.63m
- 0.55m
- 0.68m
- 6.30m
- 2.32m
- 1.69m

- Window
- Eaves
- Parapet Wall
21 Appendix VII: ASTM Concrete Strength Calculations
ASTM Three Point Loading Flexural Test

Concrete Specimen: Mix Used is w/c = 0.6 and 350lb of Water

\[ L_{\text{ASTM}} := 45.7\text{cm} \]  
(Span between supports)

\[ b_{\text{ASTM}} := 15.2\text{cm} \]  
(Specimen Dimensions)

**Test Results**

\[ P_1 := 6717\text{N} \quad t_1 := 5.6\text{cm} \]  
(Load and Beam Thickness Values for First Sample)

\[ P_2 := 6228\text{N} \quad t_2 := 5.5\text{cm} \]  
(Load and Beam Thickness Values for Second Sample)

\[ R_{\text{ASTM}} := \frac{3P_{\text{ASTM}}\cdot L_{\text{ASTM}}}{2b_{\text{ASTM}}\cdot t_{\text{ASTM}}^2} \]  
(Modulus of Rupture)

\[ R_1 = 9.66\text{MPa} \]  
(Modulus of Rupture of First Sample)

\[ R_2 = 9.29\text{MPa} \]  
(Modulus of Rupture of Second Sample)

**Average Flexural Strength in the Concrete**

\[ \sigma_{\text{avg}} := \frac{\sum_{i=1}^{2} R_i}{2} \]  
(Ultimate Concrete Flexural Strength under Bending)

\[ \sigma_{\text{avg}} = 9.5\text{MPa} \]

\[ \sigma_{\mu B} := \sigma_{\text{avg}} \]  
(Ultimate Concrete Tensile Stress under Bending - Mix B)

\[ \sigma_{\text{MB}} := 17.5\text{MPa} \quad \sigma_{\text{MD}} := 13.7\text{MPa} \]  
(Compressive Strengths of Mix B and D Respectively)

\[ \sigma_{\mu D} := \frac{\sigma_{\text{MD}}}{\sigma_{\text{MB}}} \cdot \sigma_{\mu B} = 7.4\text{MPa} \]  
(Ultimate Concrete Tensile Stress under Bending - Mix D)

\[ f_c := 19.3\text{MPa} \]  
(Compressive Strength of Concrete - 2800 psi)

\[ 15\% f_c \text{ industry rule-of-thumb} \]

\[ \sigma_{\text{MTS}} := f_c \cdot 0.15 = 2.9\text{MPa} \]  
(Maximum Tensile Stress for Concrete)
22 Appendix VIII: Bamboo Reinforced Panel Calculations
Variable Definition - Bamboo

\( \text{b}_P := 63 \text{cm} \) (Panel Width) \hspace{1cm} \( \text{b}_\text{Fl} := 6.7 \text{cm} \) (Flange Width)

\( \text{t}_\text{Fl} := 62 \text{mm} \) (Flange Thickness) \hspace{1cm} \( \text{t}_w := \text{t}_\text{Fl} - 4.1 \text{cm} \) (Web Thickness)

\( E_m := 0.043 - \text{MPa} \cdot 2200 \cdot \frac{1.5}{\sqrt{1.5}} = 19.493 - \text{GPa} \) (Modulus of Elasticity of Matrix)

(For Bamboo)

\( E_{rBb} := 19 \text{GPa} \) (Modulus of Elasticity) \hspace{1cm} \( E_{rWi} := 104 \text{GPa} \)

\( n_{Bb} := \frac{E_{rBb}}{E_m} = 0.975 \) (Transforming Factor) \hspace{1cm} \( n_{Wi} := \frac{E_{rWi}}{E_m} = 5.335 \)

\( A_{Bb} := 5 \text{mm} \cdot 27 \text{mm} = 1.35 \cdot \text{cm}^2 \) (Cross-Sectional Area of Bamboo)

\( d_{Wi} := 0.812 \text{mm} \) (Wire Mesh Diameter) \hspace{1cm} \( N_{Bb} := 2 \) (Number of Bamboo Strips per Width)

\( A_{Wi} := \left( \frac{\pi}{4} \right) d_{Wi}^2 = 0.518 \text{mm}^2 \) (Area of One Reinforcing Wire)

\( W_{i\text{sp}} := 1.125 \text{in} \) (Wire Spacing) \hspace{1cm} \( W_{i\text{w}} := \frac{b_P}{W_{i\text{sp}}} \) (Mesh Squares per Panel Width)

\( W_{i\text{ Lay}} := 2 \) (Layers of Mesh in Web) \hspace{1cm} \( W_{i\text{Fl}} := \frac{b_{Fl}}{W_{i\text{sp}}} \) (Mesh Squares within Side Flange)

\( A_{rW} := W_{i\text{w}} \cdot A_{Wi} = 0.114 \cdot \text{cm}^2 \) (Area of Wire Reinforcement per Layer in Web)

\( A_{rFl} := W_{i\text{Fl}} \cdot A_{Wi} = 0.012 \cdot \text{cm}^2 \) (Area of Wire Reinforcement per Layer within Side Flanges)

\( d_{Bb} := 2 \sqrt{\frac{A_{Bb}}{\pi}} = 1.311 \cdot \text{cm} \) (Equivalent Bamboo Diameter)

\( D_{Bb} := t_{\text{Fl}} - 1.75 \cdot \text{cm} = 4.45 \cdot \text{cm} \) (Distance from Center of Bamboo to Web Surface)

\( \%Wi := \frac{(A_{rW} + A_{rFl})}{A_{Bb} \cdot N_{Bb} + A_{rW} + A_{rFl}} = 0.045 \) (Percent of Total Reinforcement by Area - Wire Mesh)

\( n := n_{Bb} (1 - \%Wi) + n_{Wi} \%Wi = 1.17 \) (Modified Transforming Factor)

\( V_{\text{cyl}} := \pi (7.6 \text{cm})^2 \cdot 30.5 \text{cm} = 5.534 \times 10^3 \cdot \text{cm}^3 \) (Volume of Test Cylinder)
Cross Sectional Properties

\[ A_G := b_P \cdot t_w + 2 \left( t_{f_l} - t_w \right) \cdot b_{f_l} = 187 \cdot \text{cm}^2 \]  
\((\text{Gross Cross-Sectional Area})\)

\[ b_{\text{Mesh}} := b_P + 2.54 \cdot \text{cm} = 65.54 \cdot \text{cm} \]  
\((\text{Width of Mesh})\)

\[ W_{l_w} := \frac{b_{\text{Mesh}}}{W_{l_s}} \]  
\((\text{Mesh Wires per Mesh Layer Width})\)

\[ \text{GC}_{Wi} := W_{l_w} \cdot W_{l_w} \]  
\((\text{Number of Wires in Cross-Section})\)

\[ A_{rGC} := \text{GC}_{Wi} \cdot A_{Wi} + N_{Bb} \cdot A_{Bb} = 2.94 \cdot \text{cm}^2 \]  
\((\text{Cross-Sectional Area of Reinforcement})\)

\[ A_M := A_{rGC} - A_{GC} \]  
\((\text{Cross-Sectional Area of Matrix})\)

\[ VF_R := \frac{A_{rGC}}{A_{GC}} = 0.016 \]  
\((\text{Volume Fraction of Reinforcement})\)

\[ VF_M := \frac{A_M}{A_{GC}} = 0.984 \]  
\((\text{Volume Fraction of Matrix})\)

\[ \gamma_{\text{Conc}} := \left( \frac{26.55 \text{lb}}{V_{cyl}} \right) = 2176 \cdot \frac{\text{kg}}{\text{m}^3} \]  
\((\text{Density of Concrete})\)

\[ \gamma_{\text{Steel}} = 9000 \cdot \frac{\text{kg}}{\text{m}^3} \]  
\((\text{Density of Steel})\)

\[ \gamma_{Bb} = 350 \cdot \frac{\text{kg}}{\text{m}^3} \]  
\((\text{Density of Bamboo})\)

\[ \gamma_r := \gamma_{\text{Steel}} \cdot %Wi + \gamma_{Bb} \cdot (1 - %Wi) \]  
\((\text{Density of Reinforcement})\)

\[ \gamma_{\text{Comp}} := \gamma_{\text{Conc}} \cdot VF_M + \gamma_r \cdot VF_R \]  
\((\text{Density of Composite})\)

\[ E_r := E_{rBb} \cdot (1 - %Wi) + E_{rWi} \cdot %Wi \]  
\((\text{Modified Elastic Modulus of Reinforcement})\)

\[ E_r = 22.8 \cdot \text{GPa} \]

\[ E_{\text{Comp}} := \gamma_{\text{Comp}} \cdot \text{VF}_M + E_r \cdot \text{VF}_R \]  
\((\text{Elastic Modulus of Composite})\)

\[ E_{\text{Comp}} = 19.5 \cdot \text{GPa} \]
Solving for Maximum Possible Number of Layers of Mesh in Flange Thickness

Given

\[ 2 \cdot \text{Cover} + X(d_{Wi}) = t_{Fl} \]

\[ Y := \text{Find}(X) \]

\[ P_{\text{Lay}} := \text{floor}(Y) = 52 \]

\[ i := 1 \ldots P_{\text{Lay}} \]

\[ D_i := \text{Cover} + i \cdot d_{Wi} \]

\[ D_1 = 1.05 \text{ cm} \]

\[ c := \text{Find}(c) \]

\[ c = 2.119 \text{ cm} \]

\[ D_{14} = 2.106 \text{ cm} \]

\[ D_{15} = 2.187 \text{ cm} \]

\[ X := 25 \quad c := 1 \text{ cm} \quad \text{(Solve Block Guesses)} \]

(floor\(Y\) returns the greatest integer \(<= Y)\)

\[ N_{\text{Ap}} := 15 \quad \text{(Nearest Layer to Neutral Axis on Tension Side)} \]

\[ N_{\text{Am}} := N_{\text{Ap}} - 1 \quad \text{(Nearest Layer to Neutral Axis on Compression Side)} \]

\[ N_{\text{Am}} := N_{\text{Ap}} - 1 \]

\[ \sum_{i = 1}^{P_{\text{Lay}}} \left( (P_{\text{Lay}} - N_{\text{Am}}) \cdot A_{\text{Fl}} \left( D_i - c \right) \right) + \sum_{i = N_{\text{Ap}}}^{P_{\text{Lay}}} \left( (P_{\text{Lay}} - N_{\text{Am}}) \cdot A_{\text{Fl}} \left( D_i - c \right) \right) + \sum_{i = 3}^{N_{\text{Am}}} \left( (N_{\text{Am}} - 2) \cdot A_{\text{Fl}} \left( c - D_i \right) \right) \]

\[ c \] is the distance from the outside web face to the neutral axis

\[ D_{14} < c < D_{15} \]

\[ c = 2.119 \text{ cm} \]

\[ D_{14} = 2.106 \text{ cm} \]

\[ D_{15} = 2.187 \text{ cm} \]

\[ \text{Neutral Axis between Layers 14 and 15} \]

\[ \text{Distance from the Outside Web Face to the Neutral Axis} \]
Panel Cross-Section - Uncracked Transformed Moment of Inertia

Method: 1. Find Moment of Inertia of 6.2 cm by 62 cm Slab - "Solid Block"
2. Find Moment of Inertia of 4.1 cm by 50 cm Slab - "Inner Void"
3. Subtract Inner Void from Solid Block for Panel's Moment of Inertia

Solid Block: Step 1

\[
\text{Cover}_{SB} := \frac{t_{Fl} - p_{Lay} \cdot d_{Wi}}{2} = 0.99 \text{ cm}
\]

(Distance between Extreme Reinforcing Layer and Surface)

\[
D_{SB_i} := \text{Cover}_{SB} + i \cdot d_{Wi}
\]

(Distance from Surface of Solid Block to Layer i)

\[
I_{SB} := b_{p} \cdot t_{Fl}^3 \left/ 12 \right. + \left( n_{Bb} - 1 \right) \cdot n_{Bb} \cdot A_{Bb} \left( \frac{t_{Fl}}{2} - D_{Bb} \right)^2 + \left( n_{Wi} - 1 \right) \cdot \sum_{i = 1}^{P_{Lay}} \left[ P_{Lay} \cdot A_{IV} \left( \frac{t_{Fl}}{2} - D_{SB_i} \right)^2 \right]
\]

(Uncracked Transformed Moment of Inertia - Solid Block)

\[
A_{SB} := b_{p} \cdot t_{Fl}
\]

(Cross Sectional Area - Solid Block)

\[
d_{ctoSB} := \frac{t_{Fl}}{2} - c
\]

(Distance from Neutral Axis to Centroid - Solid Block)

Inner Void: Step 2

\[
b_{IV} := 50 \text{ cm}
\]

(Width of Inner Void)

\[
t_{IV} := t_{Fl} - t_{w} = 4.1 \text{ cm}
\]

(Thickness of Inner Void)

\[
W_{i,IV} := \frac{b_{IV}}{W_{sp}} = 17.5
\]

(Mesh Squares along Inner Void Width)

\[
A_{r,IV} := W_{i,IV} \cdot A_{Wi}
\]

(Area of Reinforcement per Layer Width)

Given

\[
t_{IV}^2 - X \cdot d_{Wi} + X \left( d_{Wi} \right) = t_{IV}
\]

(Number of Wire Layers in Inner Void)

\[
Y_{IV} := \text{Find}(X)
\]

\[
Y_{IV} := \text{floor}(Y_{IV}) = 25
\]

\[
\text{Cover}_{IV} := \frac{t_{IV} - Y_{IV} \cdot d_{Wi}}{2} = 1.03 \text{ cm}
\]

(Distance between Extreme Reinforcing Layer and Surface)

\[
D_{IV_i} := \text{Cover}_{IV} + i \cdot d_{Wi}
\]

(Distance from Surface of Inner Void to Layer i)

\[
I_{IV} := b_{IV} \cdot t_{IV}^3 \left/ 12 \right. + \left( n_{Wi} - 1 \right) \cdot \sum_{i = 1}^{Y_{IV}} \left[ Y_{IV} \cdot A_{IV} \left( \frac{t_{IV}}{2} - D_{IV_i} \right)^2 \right]
\]

(Uncracked Transformed Moment of Inertia - Inner Void)

\[
A_{IV} := b_{IV} \cdot t_{IV}
\]

(Cross Sectional Area - Inner Void)

\[
d_{ctoIV} := \frac{t_{IV}}{2} - \left( c - t_{w} \right)
\]

(Distance from Neutral Axis to Centroid - Inner Void)

\[
I_{\text{Tuncracked}} := I_{SB} + A_{SB} \cdot d_{ctoSB}^2 - I_{IV} - A_{IV} \cdot d_{ctoIV}^2
\]

(Transformed Moment of Inertia - Uncracked Section)
\[ t_{eqUn} := \sqrt{\frac{3}{b_p} \frac{I_{Tuncracked}}{12}} = 7.7 \text{ cm} \] (Equivalent Uncracked Panel Thickness - For Algor Plate Analysis)

**Panel Cross-Section - Cracked Transformed Moment of Inertia**

\[ I_{TCR1} := \frac{b_p c^3}{3} + n_{Bb} N_{Bb} A_{Bb} (4.73 \text{ cm} - c)^2 + n_{Wi} \sum_{i=NAp}^{P_{Lay}} \left[ (P_{Lay} - N_{Am}) A_{Fi} (D_i - c)^2 \right] \]

\[ I_{TCR2} := (n_{Wi} - 1) \sum_{i=1}^{2} \left[ A_{Wc} (c - D_i)^2 \right] + (n_{Wi} - 1) \sum_{i=3}^{NAm} \left[ (N_{Am} - 2) A_{Fi} (c - D_i)^2 \right] \]

\[ I_{Tcracked} := I_{TCR1} + I_{TCR2} = 5.257 \times 10^6 \cdot \text{mm}^4 \] (Transformed Moment of Inertia - Cracked Section)

\[ t_{eqCr} := \sqrt{\frac{3}{b_p} \frac{I_{Tcracked}}{12}} = 4.6 \text{ cm} \] (Equivalent Cracked Panel Thickness - For Algor Plate Analysis)

**Determine Cracking Moment**

\[ FS := 0.8 \] (Factor of Safety)

\[ \sigma_{mts} = 2.9 \cdot \text{MPa} \] (Maximum Tensile Stress for Concrete)

\[ M_{Cr} := \frac{2 \cdot \sigma_{mts} \cdot I_{Tuncracked}}{t_{Fl}} = 2.2 \cdot \text{kN} \cdot \text{m} \] (Cracking Moment of Concrete)

**Maximum Allowable Service Moment for Extreme Reinforcing Layer**

\[ \sigma_{rBb} := 135 \text{MPa} \] (Yield Strength of Bamboo)

\[ \sigma_{rWi} := 310 \text{MPa} \] (Yield Strength of Wire Mesh)

\[ \sigma_{ry} := \sigma_{rWi} \cdot \%Wi + \sigma_{rBb} (1 - \%Wi) = 143 \cdot \text{MPa} \] (Modified Yield Strength of Reinforcement)

\[ \sigma_{rMax} := \sigma_{ry} \cdot FS = 114.3 \text{MPa} \] (Design Yield Strength)

\[ M_{ServMax} := \frac{\sigma_{rMax} \cdot I_{Tcracked}}{n} \left[ \frac{D_{Bb} + \frac{1}{2} \cdot 5 \text{mm}}{2} - c \right] = 20 \cdot \text{kN} \cdot \text{m} \] (Maximum Allowable Service Moment)

**Maximum Compression Stress in Matrix at Maximum Service Moment**

\[ \sigma_{CompC} := \frac{M_{ServMax} \cdot c}{I_{Tcracked}} = 80 \cdot \text{MPa} \] (Compressive Stress Seen by Concrete)

\[ \sigma_{MaxAll} := FS \cdot f_c = 15 \cdot \text{MPa} \] (Maximum Allowable Compressive Stress in Concrete)

Note: Compressive stress seen by the concrete exceeds the maximum allowable compressive stress in the concrete and assuming ideal bondage between bamboo and concrete - Compression Controls
**Panel in Bending - Wind Loading**

Maximum Moment Seen by a Panel Under Wind Loading

\[
L_P := 2.316 \text{m} \quad \text{(Panel Length)} \quad P_{\text{wind}} := 1.84 \text{kPa} \quad \text{(Wind Pressure)}
\]

\[
A_{\text{GC}} = 187.24 \text{cm}^2 \quad \text{(Gross Cross-Sectional Area)}
\]

\[
A_{\text{Panel}} := g_{\text{Comp}} \cdot 1.1 \cdot A_{\text{GC}} = \frac{435 \text{N}}{\text{m}} \quad \text{(Self Weight)}
\]

\[
\omega_{\text{wind}} := P_{\text{wind}} \cdot b_P = 1159 \frac{\text{N}}{\text{m}} \quad \text{(Wind Load per Foot of Length)}
\]

Wind Acting on a Simply Supported Beam

\[
M_{\text{WindMaxSimple}} := \frac{(\omega_{\text{wind}} - SW_{\text{Panel}}) \cdot L_P^2}{8} \quad \text{(Maximum Moment from Wind Equivalent on Simply Supported Beam)}
\]

\[
M_{\text{WindMaxSimple}} = 0.49 \text{kN} \cdot \text{m}
\]

Wind Acting on Cantilever Beam

\[
M_{\text{WindMaxCant}} := \frac{\omega_{\text{wind}} \cdot L_P^2}{2} \quad \text{(Maximum Moment from Wind Equivalent on Cantilever Beam)}
\]

\[
M_{\text{WindMaxCant}} = 3.11 \text{kN} \cdot \text{m}
\]
Panel in Bending - 4 Point Load Test

Maximum Moment Seen During Bending Test - Web in Compression

\[ F_{\text{MaxC}} := 5338 \text{N} \]  
(Maximum Load - Compression)

\[ D_C := 74.9 \text{cm} \]  
(Horizontal Distance from Center of Channel to Support)

\[ R_C := \frac{F_{\text{MaxC}}}{2} \]  
(Each Support Reaction)

\[ M_{\text{FailC}} := R_C \cdot D_C = 1999 \text{N.m} \]  
(Moment at Failure for Web in Compression)

Vibration Test Calculations

\[ I_{\text{Tunracked}} = 2.4 \times 10^7 \cdot \text{mm}^4 \]  
(Moment of Inertia)

\[ \gamma_{\text{Comp}} = 2.153 \times 10^3 \frac{\text{kg}}{\text{m}^3} \]  
(Density of Composite Metric)

\[ A_{\text{GC}} = 1.872 \times 10^4 \cdot \text{mm}^2 \]  
(Gross Cross-Sectional Area)

\[ L_P = 2.316 \text{m} \]  
(Length of the Panel)

\[ f_n := 20.2 \text{Hz} \]  
(Measured Natural Frequency)

\[ \omega_n := 2 \cdot \pi \cdot f_n = 126.92 \frac{\text{rad}}{\text{s}} \]  
(Natural Frequency in rad/s)

\[ E_{\text{calc}} := 0.5 \cdot \text{GPa} \]  
(Guess value)

Given

\[ \omega_n = 22.4 \cdot \sqrt[4]{\frac{E_{\text{calc}} \cdot I_{\text{Tunracked}}}{\gamma_{\text{Comp}} \cdot A_{\text{GC}} \cdot L_P^4}} \]

\[ E_{\text{new}} := \text{Find}(E_{\text{calc}}) \quad E_{\text{new}} = 1.552 \cdot \text{GPa} \]  
(New Modulus of Elasticity - Panel)
23 Appendix IX: Rebar Reinforced Panel Calculations
Variable Definition - Rebar

\( b_P := 63 \text{cm} \) (Panel Width)  \( b_{Fl} := 6.7 \text{cm} \) (Flange Width)

\( t_{Fl} := 62 \text{mm} \) (Flange Thickness) \( t_w := t_{Fl} - 4.1 \text{cm} \) (Web Thickness)

\( E_m := 0.043 \text{MPa} \cdot 2200 \sqrt{\frac{f_c}{\text{MPa}}} = 19.493 \text{GPa} \) (Modulus of Elasticity of Matrix)

\( E_{rRb} := 200 \text{GPa} \) (Modulus of Elasticity) \( E_{rWi} := 104 \text{GPa} \)

\( n_{Rb} := \frac{E_{rRb}}{E_m} = 10.26 \) (Transforming Factor) \( n_{Wi} := \frac{E_{rWi}}{E_m} = 5.335 \)

\( d_{Rb} := \frac{3 \text{in}}{8} \) (Diameter of Rebar)

\( A_{Rb} := \left( \frac{\pi}{4} \right) d_{Rb}^2 \) (Cross-Sectional Area of Rebar)

\( d_{Wi} := 0.812 \text{mm} \) (Wire Mesh Diameter) \( N_{Rb} := 2 \) (Number of Rebar Strips per Width)

\( A_{Wi} := \left( \frac{\pi}{4} \right) d_{Wi}^2 = 0.518 \text{mm}^2 \) (Area of One Reinforcing Wire)

\( W_{i_{sp}} := 1.125 \text{in} \) (Wire Spacing) \( W_{i_{W}} := \frac{b_P}{W_{i_{sp}}} \) (Mesh Squares per Panel Width)

\( W_{i_{Lay}} := 2 \) (Layers of Mesh in Web) \( W_{i_{Fl}} := \frac{b_{Fl}}{W_{i_{sp}}} \) (Mesh Squares within Side Flange)

\( A_{rW} := W_{i_{W}} \cdot A_{Wi} = 0.114 \text{cm}^2 \) (Area of Wire Reinforcement per Layer in Web)

\( A_{rFl} := W_{i_{Fl}} \cdot A_{Wi} = 0.012 \text{cm}^2 \) (Area of Wire Reinforcement per Layer within Side Flanges)

\( D_{Rb} := t_{Fl} - 1.75 \text{cm} = 4.45 \text{cm} \) (Distance from Center of Rebar to Web Surface)

\( %_{Wi} := \frac{A_{rW} + A_{rFl}}{A_{Rb} N_{Rb} + A_{rW} + A_{rFl}} = 0.081 \) (Percent of Total Reinforcement by Area - Wire Mesh)

\( n := n_{Rb} \left( 1 - %_{Wi} \right) + n_{Wi} %_{Wi} = 9.859 \) (Modified Transforming Factor)

\( V_{cyl} := \pi (7.6 \text{cm})^2 \cdot 30.5 \text{cm} = 5.534 \times 10^3 \text{cm}^3 \) (Volume of Test Cylinder)
Cross Sectional Properties

\[ A_{GC} := b_p t_w + 2 \left( t_{fi} - t_w \right) b_{fi} = 187 \cdot \text{cm}^2 \]  
\( \text{(Gross Cross-Sectional Area)} \)

\[ b_{\text{Mesh}} := b_p + 2.54 \text{cm} = 65.54 \text{cm} \]  
\( \text{(Width of Mesh)} \)

\[ W_{i_{LW}} := \frac{b_{\text{Mesh}}}{W_{i_{sp}}} \]  
\( \text{(Mesh Wires per Mesh Layer Width)} \)

\[ G_{C Wi} := W_{i_{Lay}} W_{i_{LW}} \]  
\( \text{(Number of Wires in Cross-Section)} \)

\[ A_{rGC} := G_{C Wi} A_{Wi} + N_{Rb} A_{Rb} = 1.66 \cdot \text{cm}^2 \]  
\( \text{(Cross-Sectional Area of Reinforcement)} \)

\[ A_M := A_{GC} - A_{rGC} \]  
\( \text{(Cross-Sectional Area of Matrix)} \)

\[ V_{F_R} := \frac{A_{rGC}}{A_{GC}} = 8.88 \times 10^{-3} \]  
\( \text{(Volume Fraction of Reinforcement)} \)

\[ V_{F_M} := \frac{A_M}{A_{GC}} = 0.991 \]  
\( \text{(Volume Fraction of Matrix)} \)

\[ \gamma_{\text{Conc}} := \frac{26.6 \text{lb}}{2180 \text{ kg} / \text{m}^3} = 12.9 \text{ kg} / \text{m}^3 \]  
\( \text{(Density of Concrete)} \)

\[ \gamma_{\text{Steel}} := 9000 \text{ kg} / \text{m}^3 \]  
\( \text{(Density of Steel)} \)

\[ \gamma_{\text{Comp}} := \gamma_{\text{Conc}} V_{F_M} + \gamma_{\text{Steel}} V_{F_R} = 2241 \text{ kg} / \text{m}^3 \]  
\( \text{(Density of Composite)} \)

\[ E_r := E_{rRb} \left( 1 - \%Wi \right) + E_{rWi} \%Wi \]  
\( \text{(Modified Elastic Modulus of Reinforcement)} \)

\[ E_r = 192.2 \text{ GPa} \]

\[ E_{\text{Comp}} := E_m V_{F_M} + E_r V_{F_R} \]  
\( \text{(Elastic Modulus of Composite)} \)

\[ E_{\text{Comp}} = 21 \text{ GPa} \]
Neutral Axis - Panel Cross Section

Cover := \left(\frac{t_w - W_{	ext{lay}} \cdot d_{\text{wi}}}{2}\right) = 0.969 \text{ cm} \quad \text{(Distance Between Wire and Edge of Concrete - Web)}

Solving for Maximum Possible Number of Layers of Mesh in Flange Thickness

Given

\[2 \cdot \text{Cover} + X (d_{\text{wi}}) = t_{\text{fl}}\]

\[Y := \text{Find}(X)\]

\[P_{\text{lay}} := \text{floor}(Y) = 52\]

\[X := 25 \quad c := 1 \text{ cm} \quad \text{(Solve Block Guesses)}\]

\[2 \cdot \text{Cover} + X (d_{\text{wi}}) = t_{\text{fl}}\]

\[\text{PLay} := \text{floor}(Y) = 52\]

\[\text{NAp} := 15\]

\[\text{NAm} := \text{NAp} - 1\]

\(i := 1 \ldots P_{\text{lay}}\)

\[D_i := \text{Cover} + i \cdot d_{\text{wi}}\]

\[D_1 = 1.05 \text{ cm} \quad \text{(Distance to Layer 1)}\]

\[\text{To Layer 1}\]

\[\text{Given}\]

\[\text{Compression}\]

\[\text{Tension}\]

\[\left[b \cdot t_w + 2 \cdot (t_{\text{fl}} - t_w) \left(\begin{array}{c} c \end{array}\right) + (n_{\text{wi}} - 1) \right] \sum_{i=1}^{2} \left[2 \cdot A_{\text{fwi}} \left(\begin{array}{c} c \end{array}\right) - D_i\right] \ldots = n_{\text{wi}} \cdot \sum_{i=\text{NAm}}^{P_{\text{lay}}} \left[(P_{\text{lay}} - \text{NAm}) \cdot A_{\text{fl}} \left(D_i - c\right)\right] + n_{\text{rb}} \cdot (t_{\text{fl}} - D_{\text{rb}} - c) \cdot N_{\text{rb}} \cdot A_{\text{rb}}\]

\[+ (n_{\text{wi}} - 1) \sum_{i=3}^{\text{NAm}} \left[(\text{NAm} - 2) \cdot A_{\text{fwi}} \left(c - D_i\right)\right]\]

\[c := \text{Find}(c)\]

\[c = 2.1 \text{ cm} \quad \text{(Distance from the Outside Web Face to the Neutral Axis)}\]

\[D_{14} = 2.1 \text{ cm}\]

\[D_{15} = 2.2 \text{ cm}\]

Neutral Axis between Layers 14 and 15 \[D_{14} < c < D_{15}\]
Panel Cross-Section - Uncracked Transformed Moment of Inertia

Method: 1. Find Moment of Inertia of 6.2 cm by 62 cm Slab - "Solid Block"
2. Find Moment of Inertia of 4.1 cm by 50 cm Slab - "Inner Void"
3. Subtract Inner Void from Solid Block for Panel's Moment of Inertia

Solid Block: Step 1

\[ \text{Cover}_{SB} := \frac{t_{FL} - P_{Lay} d_{Wi}}{2} = 0.99 \text{ cm} \]  
(Distance between Extreme Reinforcing Layer and Surface)

\[ D_{SB} := \text{Cover}_{SB} + i \cdot d_{Wi} \]  
(Distance from Surface of Solid Block to Layer \( i \))

\[ I_{SB} := b_p t_{FL}^3 \frac{1}{12} + \left(n_{Rb} - 1\right) N_{Rb} A_{Rb} \left(\frac{t_{FL}}{2} - D_{Rb}\right)^2 + \left(n_{Wi} - 1\right) \sum_{i=1}^{P_{Lay}} \left[P_{Lay} A_{R\!\!W} \left(\frac{t_{FL}}{2} - D_{SB, i}\right)^2\right] \]  
(Uncracked Transformed Moment of Inertia - Solid Block)

\[ A_{SB} := b_p t_{FL} \]  
(Cross Sectional Area - Solid Block)

\[ d_{ctoSB} := \frac{t_{FL}}{2} - c \]  
(Distance from Neutral Axis to Centroid - Solid Block)

Inner Void: Step 2

\[ b_{IV} := 50 \text{ cm} \]  
(Width of Inner Void)

\[ W_{i, IV} := \frac{b_{IV}}{W_{isp}} = 17.5 \]  
(Mesh Squares along Inner Void Width)

\[ A_{i, IV} := W_{i, IV} \cdot A_{Wi} \]  
(Area of Reinforcement per Layer Width)

Given

\[ t_{IV} - X \cdot d_{Wi} + X(d_{W_i}) = t_{IV} \]  
(Number of Wire Layers in Inner Void)

\[ Y_{IV} := \text{Find}(X) \]  
\( IV_{Lay} := \text{floor}(Y_{IV}) = 25 \)

\[ \text{Cover}_{IV} := \frac{t_{IV} - IV_{Lay} \cdot d_{Wi}}{2} = 1.03 \text{ cm} \]  
(Distance between Extreme Reinforcing Layer and Surface)

\[ D_{IV} := \text{Cover}_{IV} + i \cdot d_{Wi} \]  
(Distance from Surface of Inner Void to Layer \( i \))

\[ I_{IV} := b_{IV} t_{IV}^3 \frac{1}{12} + \left(n_{Wi} - 1\right) \sum_{i=1}^{IV_{Lay}} IV_{Lay} A_{R\!\!W} \left(\frac{t_{IV}}{2} - D_{IV, i}\right)^2 \]  
(Uncracked Transformed Moment of Inertia - Inner Void)

\[ A_{IV} := b_{IV} t_{IV} \]  
(Cross Sectional Area - Inner Void)

\[ d_{ctoIV} := \frac{t_{IV}}{2} - \left(c - t_w\right) \]  
(Distance from Neutral Axis to Centroid - Inner Void)

\[ I_{\text{Tuncracked}} := I_{SB} + A_{SB} d_{ctoSB}^2 - I_{IV} - A_{IV} d_{ctoIV}^2 \]
\[ I_{\text{uncracked}} = 2.422 \times 10^7 \cdot \text{mm}^4 \]  
(Transposed Moment of Inertia - Uncracked Section)

\[ t_{\text{eqUn}} := \sqrt{\frac{I_{\text{uncracked}} \cdot 12}{b_p}} = 7.7 \cdot \text{cm} \]  
(Equivalent Uncracked Panel Thickness - For Algor Plate Analysis)

**Panel Cross-Section - Cracked Transformed Moment of Inertia**

\[ I_{\text{TCR1}} := b_p \cdot \frac{c^3}{3} + R_b \cdot A_{Rb} \cdot (4.73 \cdot c - b)^2 + n_{Wi} \cdot \sum_{i=N_{Ap}}^{P_{Lay}} \left[ (P_{Lay} - N_{Am}) \cdot A_{Fl} \cdot (D_i - c)^2 \right] \]

\[ I_{\text{TCR2}} := (n_{Wi} - 1) \sum_{i=1}^{2} A_{W} \cdot (c - D_i)^2 + (n_{Wi} - 1) \cdot \sum_{i=3}^{N_{Am}} (N_{Am} - 2) \cdot A_{Fl} \cdot (c - D_i)^2 \]

\[ I_{\text{cracked}} := I_{\text{TCR1}} + I_{\text{TCR2}} = 6.094 \times 10^6 \cdot \text{mm}^4 \]  
(Transposed Moment of Inertia - Cracked Section)

\[ t_{\text{eqCr}} := \sqrt{\frac{I_{\text{cracked}} \cdot 12}{b_p}} = 4.9 \cdot \text{cm} \]  
(Equivalent Cracked Panel Thickness - For Algor Plate Analysis)

Determine Cracking Moment

\[ FS := 0.8 \]  
(Factor of Safety)

\[ \sigma_{mts} = 2.895 \cdot \text{MPa} \]  
(Maximum Tensile Stress for Concrete)

\[ M_{Cr} := \frac{2 \cdot \sigma_{mts} \cdot I_{\text{uncracked}}}{t_{\text{Fl}}} = 2.3 \cdot \text{kN} \cdot \text{m} \]  
(Cracking Moment of Concrete)

Maximum Allowable Service Moment for Extreme Reinforcing Layer

\[ \sigma_{rRb} := 410 \cdot \text{MPa} \]  
(Yield Strength of Rebar)

\[ \sigma_{rWi} := 310 \cdot \text{MPa} \]  
(Yield Strength of Wire Mesh)

\[ \sigma_{r} := \sigma_{rWi} \cdot \%Wi + \sigma_{rRb} \cdot (1 - \%Wi) = 402 \cdot \text{MPa} \]  
(Modified Yield Strength of Reinforcement)

\[ \sigma_{rMax} := \sigma_{r} \cdot FS = 321.5 \cdot \text{MPa} \]  
(Design Yield Strength)

\[ M_{ServMax} := \frac{\sigma_{rMax} \cdot I_{\text{cracked}}}{n \left[ (D_{Rb} + \frac{1}{2} \cdot 5 \cdot \text{mm}) - c \right]} = 7.6 \cdot \text{kN} \cdot \text{m} \]  
(Maximum Allowable Service Moment)

Maximum Compression Stress in Matrix at Maximum Service Moment

\[ \sigma_{\text{CompC}} := \frac{M_{ServMax} \cdot c}{I_{\text{cracked}}} = 26 \cdot \text{MPa} \]  
(Compressive Stress Seen by Concrete)

\[ \sigma_{\text{MaxAll}} := FS \cdot \frac{f_c}{t_c} = 15 \cdot \text{MPa} \]  
(Maximum Allowable Compressive Stress in Concrete)

Note: Compressive stress seen by the concrete exceeds the maximum allowable compressive stress in the concrete and assuming ideal bondage between the rebar and concrete - Compression Controls
Panel in Bending - Wind Loading

Maximum Moment Seen by a Panel Under Wind Loading

\[ L_P := 2.316 \text{m} \quad \text{(Panel Length)} \]
\[ P_{\text{wind}} := 1.84 \text{kPa} \quad \text{(Wind Pressure)} \]
\[ A_{\text{GC}} = 187.24 \text{cm}^2 \]
\[ S_{\text{W Panel}} := \gamma_{\text{Comp}} g 1.1 A_{\text{GC}} = 453 \frac{\text{N}}{\text{m}} \]
\[ \omega_{\text{wind}} := P_{\text{wind}} b_P = 1159 \frac{\text{N}}{\text{m}} \]

Wind Acting on a Simply Supported Beam

\[ M_{\text{WindMaxSimple}} := \frac{\omega_{\text{wind}} - S_{\text{W Panel}}}{8} L_P^2 \]

\[ M_{\text{WindMaxSimple}} = 0.47 \text{kN} \cdot \text{m} \]

Wind Acting on Cantilever Beam

\[ M_{\text{WindMaxCant}} := \frac{\omega_{\text{wind}} L_P^2}{2} \]

\[ M_{\text{WindMaxCant}} = 3.11 \text{kN} \cdot \text{m} \]
Panel in Bending - 4 Point Load Test

Maximum Moment Seen During Bending Test - Web in Compression - Test 1 (April 11, 2012)

\[ F_{\text{Max}C1} := 6450 \text{N} \]  
\[ D_{C1} := 66 \text{cm} \]  
\[ R_{C1} := \frac{F_{\text{Max}C1}}{2} \]  

\[ M_{\text{Fail}C1} := R_{C1} D_{C1} = 2129 \text{N}\cdot\text{m} \]  

\[ \sigma_{\text{Fail1}} := \frac{M_{\text{Fail}C1} c}{I_{\text{cracked}}} = 7.3 \text{MPa} \]

Maximum Moment Seen During Bending Test - Web in Compression - Test 2 (April 25, 2012)

\[ F_{\text{Max}C2} := 5151 \text{N} \]  
\[ D_{C2} := 66 \text{cm} \]  
\[ R_{C2} := \frac{F_{\text{Max}C2}}{2} \]  

\[ M_{\text{Fail}C2} := R_{C2} D_{C2} = 1700 \text{N}\cdot\text{m} \]  

\[ \sigma_{\text{Fail2}} := \frac{M_{\text{Fail}C2} c}{I_{\text{cracked}}} = 5.8 \text{MPa} \]

Maximum Moment Seen During Bending Test - Web in Compression - Test 3 (April 30, 2012)

\[ F_{\text{Max}C3} := 7800 \text{N} \]  
\[ D_{C3} := 66 \text{cm} \]  
\[ R_{C3} := \frac{F_{\text{Max}C3}}{2} \]  

\[ M_{\text{Fail}C3} := R_{C3} D_{C3} = 2574 \text{N}\cdot\text{m} \]  

\[ \sigma_{\text{Fail3}} := \frac{M_{\text{Fail}C3} c}{I_{\text{cracked}}} = 8.9 \text{MPa} \]

Note: Tests 1 and 2 were concrete panels that cured for 1 week each, while test 3 was a concrete panel that was allowed to cure for 4 weeks. This provided 100% concrete strength for test 3, compared to 70% for tests 1 and 2.
Vibration Test Calculations

\[ I_{\text{Tuncracked}} = 2.422 \times 10^7 \cdot \text{mm}^4 \]  
(Moment of Inertia)

\[ \gamma_{\text{Comp}} = 2.241 \times 10^3 \frac{\text{kg}}{\text{m}^3} \]  
(Density of Composite Metric)

\[ A_{\text{GC}} = 1.872 \times 10^4 \cdot \text{mm}^2 \]  
(Gross Cross-Sectional Area)

\[ L_p = 2.316 \text{ m} \]  
(Length of the Panel)

\[ f_n := 28.6\text{Hz} \]  
(Measured Natural Frequency)

\[ \omega_n := 2 \cdot \pi \cdot f_n = 179.699 \frac{\text{rad}}{\text{s}} \]  
(Natural Frequency in rad/s)

\[ E_{\text{calc}} := 0.5 \text{ GPa} \]  
(Guess value)

\[ \frac{E_{\text{calc}} \cdot I_{\text{Tuncracked}}}{\gamma_{\text{Comp}} \cdot A_{\text{GC}} \cdot L_p^4} \]

Given

\[ \omega_n = 22.4 \cdot \frac{E_{\text{calc}} \cdot I_{\text{Tuncracked}}}{\gamma_{\text{Comp}} \cdot A_{\text{GC}} \cdot L_p^4} \]

\[ E_{\text{new}} := \text{Find}(E_{\text{calc}}) \]

\[ E_{\text{new}} = 3.2 \text{ GPa} \]  
(New Modulus of Elasticity - Panel)
24 Appendix X: Wind Loading and Other Calculations
Wind Loading - Panel to Panel Connection Strength

Method: Analyze 1.2 meter tall strip of end wall in horizontal bending simply supported on two ends with evenly distributed wind of 1840 Newtons per square meter. This conservatively approximates the peak moment seen by the connection at the top of the wall.

\[ P_{\text{wind}} = 1.84 \text{ kPa} \quad \text{(Wind Load)} \]

\[ H_{\text{EW}} := \frac{L_P}{4} = 0.579 \text{ m} \quad \text{(Height Analyzed in Bending - 4 Connections)} \]

\[ \omega := P_{\text{wind}} H_{\text{EW}} \quad \text{(Evenly Distributed Wind Load)} \]

\[ M_{\text{EWC}} := \frac{\omega L_{\text{EW}}^2}{8} = 53 \text{ N \cdot m} \quad \text{(Greatest Moment Seen by Connection)} \]

**Threaded Rod Strength Calculations**

\[ D_B := t_{fI} - 2 \text{ cm} \quad \text{(Distance from Panel Surface to Center of Rods)} \]

\[ d_B := 0.823 \text{ cm} \quad \text{(Diameter of Rod)} \]

\[ I_{\text{rod}} := \frac{\pi d_B^4}{64} \quad \text{(Moment of Inertia of the Rod)} \]

\[ \sigma_B := \frac{M_{\text{EWC}} d_B}{I_{\text{rod}}^2} = 966 \text{ MPa} \quad \text{(Expected Stress on Rod)} \]

Note: These calculations assume all of the wind load is sent to the connection and handled by rod, with no failure of the concrete.

**Load Test Result**

\[ P_A := 1380 \text{ N} \quad \text{(Applied Weight)} \]

\[ S_{\text{Panel}} = 453 \frac{\text{N}}{\text{m}} \quad \text{(Self Weight of Panels Between Supports)} \]

\[ M_L := \frac{P_A L_S}{16} + \frac{S_{\text{Panel}} L_S^2}{8} \quad \text{(Moment on Panels from Load)} \]

\[ F_B := \frac{M_L}{D_B} \quad \text{(Force on Each of the Two Moment Resisting Bolts)} \]

\[ M_{\text{TF}} := F_B D_B \quad \text{(Moment Causing a Single Torsional Failure at Panel End)} \]

\[ M_{\text{TF}} = 38 \text{ N \cdot m} \]
Conclusion: Panel connections failed load testing ($M > M_{TF}$)

Wind Loading - Panel to Foundation Shear Strength

Shear Loading created by Wind Translated to Side Lengths

$$\upsilon_{\text{side}} := P_{\text{wind}} \cdot \frac{1}{2} \cdot L_p \cdot \frac{10}{2} \cdot b_p$$

$$\upsilon_{\text{side}} = 6.7 \text{ kN}$$

Horizontal and Vertical Shear on Bolts Connecting Panel to Foundation Brackets

$$A_B := \frac{\pi \cdot d_B^2}{4} \quad \text{(Cross Sectional Area of Rod)}$$

$$F_{Bx} := \frac{\upsilon_{\text{side}} \cdot L_p}{2} = 3.4 \text{ kN}$$

$$F_{By} := \frac{\upsilon_{\text{side}} \cdot b_p}{b_p} = 25 \text{ kN}$$

$$\sigma_{\text{ShearX}} := \frac{2 \cdot F_{Bx}}{\pi \cdot d_B^2} = 32 \text{ MPa}$$

$$\sigma_{\text{CapX}} := \frac{70 \text{ kN}}{A_B} = 1316 \text{ MPa} \quad \text{Both are well under material limits}$$

$$\sigma_{\text{ShearY}} := \frac{F_{By}}{d_B \cdot t_{fI}} = 48 \text{ MPa}$$

$$\sigma_{\text{CapY}} := \frac{45 \text{ kN}}{A_B} = 846 \text{ MPa}$$

Wind Loading - Single Panel Web Shear Strength

Shear Force Applied on Web

$$\upsilon_{\text{side}} = 6.7 \text{ kN}$$

(Shear Force within Web)

Shear Force Capacity of Web

$$d := 0.8 \cdot b_p \quad \text{(Modified Panel Width)}$$

$$\lambda := 0.85 \quad \text{(Lightweight Concrete Modification Factor)}$$

$$f_y := 310 \text{ MPa \quad (Yield Strength of Shear Reinforcement - 46 ksi)}$$

$$V_{n\text{Max}} := 0.83 \cdot MPa \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot t_w \cdot d \quad \text{(Nominal Shear Strength - Capacity of Panel Shall Not Exceed)}$$

$$V_{n\text{Max}} = 39 \text{ kN}$$

$$V_c := 0.17 \text{ MPa} \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{MPa}}} \cdot t_w \cdot d \quad \text{(Shear Strength of Concrete in a Panel)}$$

$$V_c = 7 \text{ kN}$$

$$V_s := \frac{W_{I\text{Lay} \cdot A_r \cdot W} \cdot f_y \cdot d}{W_{I\text{sp}}} \quad \text{(Shear Strength of Chicken Wire Mesh in a Panel)}$$

$$V_s = 125 \text{ kN}$$

$$V_n := V_s + V_c = 132 \text{ kN} \quad V_n > V_{n\text{Max}} \text{ Therefore } V_{n\text{Max}} \text{ value shall be used}$$

The shear strength of the concrete web in one panel is enough to handle the experienced wind.
loads. In combination with the shear strength of the reinforcement, the panel is much more than able to handle these shear forces. (Comparing $V_{nMax} - 39 \text{ kN}$ to $u_{side} - 6.7 \text{ kN}$)

**Tensile Response in Cracked Section of Panel**

**Theoretical Tensile Stress in Composite Due to Roof Uplift**

\[
\begin{align*}
L_{\text{build}} &:= 10 \cdot b_p = 6.3 \text{ m} \\
W_{\text{build}} &:= 5 \cdot b_p = 3.15 \text{ m} \\
L_p &:= 2.316 \text{ m} \\
V &:= 130 \frac{\text{mi}}{\text{hr}} = 209.215 \frac{\text{km}}{\text{hr}} \\
I &:= 1 \\
K_z &:= 0.85 \\
K_{zt} &:= 1 \\
K_d &:= 0.85 \\
q_z &:= 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \left( \frac{V}{\text{mi}} \right)^2 \frac{\text{lb}f}{\text{ft}^2} \cdot I = 1.497 \times 10^3 \cdot \text{Pa} \\
h_{\text{roof}} &:= 0.305 \text{ m} \\
h &:= L_p + h_{\text{roof}} \\
q_h &:= q_z \\
G &:= 0.85 \\
G_{Cpi} &:= 0.55
\end{align*}
\]
Wind Pressure Acting on Building

Windward Wall

\[ C_{\text{windwall}} := 0.8 \]

\[ p_{1\text{windwall}} := q_h \cdot G \cdot C_{\text{windwall}} - q_h \cdot G \cdot C_p = 195 \text{ Pa} \]

\[ p_{2\text{windwall}} := q_h \cdot G \cdot C_{\text{windwall}} + q_h \cdot G \cdot C_p = 1841 \text{ Pa} \]

Leeward Wall

\[ C_{\text{leewall}} := -0.5 \]

\[ p_{1\text{leewall}} := q_h \cdot G \cdot C_{\text{leewall}} - q_h \cdot G \cdot C_p = -1459 \text{ Pa} \]

\[ p_{2\text{leewall}} := q_h \cdot G \cdot C_{\text{leewall}} + q_h \cdot G \cdot C_p = 187 \text{ Pa} \]

Side Walls

\[ C_{\text{sidewall}} := -0.7 \]

\[ p_{1\text{sidewall}} := q_h \cdot G \cdot C_{\text{sidewall}} - q_h \cdot G \cdot C_p = -1714 \text{ Pa} \]

\[ p_{2\text{sidewall}} := q_h \cdot G \cdot C_{\text{sidewall}} + q_h \cdot G \cdot C_p = -67 \text{ Pa} \]

Windward Roof

\[ C_{\text{windroof}} := -1.044 \]

\[ p_{1\text{windroof}} := q_h \cdot G \cdot C_{\text{windroof}} - q_h \cdot G \cdot C_p = -2151 \text{ Pa} \]

\[ p_{2\text{windroof}} := q_h \cdot G \cdot C_{\text{windroof}} + q_h \cdot G \cdot C_p = -505 \text{ Pa} \]

Leeward Roof

\[ C_{\text{leeroof}} := -0.572 \]

\[ p_{1\text{leeroof}} := q_h \cdot G \cdot C_{\text{leeroof}} - q_h \cdot G \cdot C_p = -1551 \text{ Pa} \]

\[ p_{2\text{leeroof}} := q_h \cdot G \cdot C_{\text{leeroof}} + q_h \cdot G \cdot C_p = 95 \text{ Pa} \]
**Panel in Compression**

**Tin Roof Dimensions**

\[ L_{\text{build}} = 6.3 \text{ m} \]
\[ W_{\text{build}} = 3.15 \text{ m} \]
\[ L_{\text{tin}} := L_{\text{build}} + 1\text{ft} = 6.605 \text{ m} \]
\[ W_{\text{tin}} := (W_{\text{build}} + 0.5\text{ft}) = 3.302 \text{ m} \]
\[ L_{\text{tin}} \cdot W_{\text{tin}} = 21.812 \text{ m}^2 \]
\( t_{\text{tin}} := 0.014\text{in} = 0.356 \text{ mm} \)

(Thickness of 29 guage roofing)

\[ V_{\text{tin}} := L_{\text{build}} \cdot W_{\text{build}} \cdot t_{\text{tin}} = 7.057 \times 10^3 \text{ cm}^3 \]
\[ \text{Tin density} := \frac{7.365 \text{ gm}}{\text{cm}^3} \]
\[ \text{Weight}_{\text{tin}} := \text{Tin density} \cdot V_{\text{tin}} = 51.974 \text{ kg} \]

(Weight of tin on half of the roof)

**Panel Weight**

\[ V_{\text{cyl}} := \pi (3\text{in})^2 \cdot 12\text{in} = 5.56 \times 10^3 \text{ cm}^3 \]
\[ \text{Weight}_{\text{cyl}} := 26.25 \text{ lb} = 11.907 \text{ kg} \]
\[ \rho_{\text{mix}} := \frac{\text{Weight}_{\text{cyl}}}{V_{\text{cyl}}} = 133.69 \frac{\text{lb}}{\text{ft}^3} \]
\[ V_{\text{panel}} := 2.014 \text{ ft}^3 = 5.703 \times 10^4 \text{ cm}^3 \]
\[ m_p := \rho_{\text{mix}} \cdot V_{\text{panel}} = 122.131 \text{ kg} \]

**Truss Weight**

\[ \text{length}_{\text{lumber}} := 3.28\text{ft} + 3.28\text{ft} + 6.25\text{ft} + 1\text{ft} = 4.209 \text{ m} \]
\[ \rho_{\text{lumber}} := 400.5 \frac{\text{kg}}{\text{m}^3} \]

(Engineering Toolbox)

\[ V_{\text{lumber}} := \text{length}_{\text{lumber}} \cdot 1.5\text{in} \cdot 3.5\text{in} = 1.426 \times 10^4 \text{ cm}^3 \]

(Dimensions of 2 X 4)

\[ \text{Weight}_{\text{lumber}} := \rho_{\text{lumber}} \cdot V_{\text{lumber}} = 5.71 \text{ kg} \]
Weight\textsubscript{truss} := 4 \cdot \frac{\text{Weight}\textsubscript{lumber}}{2} = 11.42 \text{ kg} \quad \text{ (4 trusses along each 10 panel length - divide by two for half of the roof's lumber)}

\textbf{Compressive Strength Seen by Panel Under Worst Conditions}

\text{Weight}\textsubscript{truss} = 11.42 \text{ kg} \\
\text{Weight}\textsubscript{tin} = 51.974 \text{ kg} \\
\text{Weight}\textsubscript{roof} := \text{Weight}\textsubscript{truss} + \text{Weight}\textsubscript{tin} \quad \text{ (Weight of Roof on Half of Building)}

\text{Area}\textsubscript{roof} := L\textsubscript{tin} \cdot W\textsubscript{tin} \\
\text{Max}\textsubscript{RP} := 94.486 \text{ Pa} \quad \text{ (Max Downward Wind Pressure Seen by One Side of Roof)}

\text{Load}\textsubscript{roof} := \text{Max}\textsubscript{RP} \cdot \text{Area}\textsubscript{roof} \quad \text{ (Live Load on Half of Total Roof Area)}

\text{VerticalLoad}\textsubscript{roof} := \text{Load}\textsubscript{roof} \cdot \cos(17.75) \quad \text{ (Downward Force on Half the Roof From Wind Pressure)}

\text{VerticalLoad}\textsubscript{roof} = 935.632 \text{ N}

\text{Load}\textsubscript{comp} := \frac{\left(\text{VerticalLoad}\textsubscript{roof} + \text{Weight}\textsubscript{roof} \cdot g\right)}{10} \quad \text{ (Total Compressive Load in Each Panel)}

\text{Load}\textsubscript{comp} = 155.731 \text{ N}

\sigma\textsubscript{comp} := \frac{\text{Load}\textsubscript{comp}}{A\textsubscript{GC}} \quad \text{ (Compressive Stress in Each Panel)}

\sigma\textsubscript{comp} = 8.3 \text{ kPa}

\textbf{Allowable Compressive Stress}

f\textsubscript{c} = 19.3 \text{ MPa} \\
\sigma\textsubscript{compallow} := \text{FS} \cdot f\textsubscript{c} \quad \text{ (Compressive Capacity in Each Panel)}

\sigma\textsubscript{compallow} = 15440 \text{ kPa}
Appendix XI: 28-Day Concrete Strength Calculations
Concrete Compression Test Calculations

Density

\[ \gamma_{\text{Conc}} = 2.18 \times 10^3 \frac{\text{kg}}{\text{m}^3} \]

From correlation graph to find out approximate ratio of 21 day to 28 day strength

\[ \text{Strength}_{28\text{day}} = 41.719 \frac{\text{N}}{\text{mm}^2} \]

\[ \text{Strength}_{21\text{day}} = 39.14 \frac{\text{N}}{\text{mm}^2} \]

\[ \text{Ratio}_{21\text{day}} = \frac{\text{Strength}_{21\text{day}}}{\text{Strength}_{28\text{day}}} = 0.938 \]

\[ \text{Area}_{\text{Cylinder}} = 182 \text{cm}^2 \quad \text{(Cross-Sectional Area of Cylinder)} \]

\[ \text{Ratio}_{21\text{day}} = 0.938 \quad \text{(Seven Day Strength is 70% of 28-Day Strength)} \]

Repeating 2003’s Positive Reinforcement’s Mix

**Compressive Strength of Cylinder 1 (1:1:5 Water:Cement:Water in Mixer)**

\[ \text{Load}_{\text{MaxComp1}} = 160000 \text{ lbf} \quad \text{(Observed Maximum Load Applied by Compression Tester)} \]

\[ \sigma_{\text{Test}} = \frac{\text{Load}_{\text{MaxComp1}}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 39 \text{ MPa} \]

\[ \sigma_{28\text{day}} = \frac{\sigma_{\text{Test}}}{\text{Ratio}_{21\text{day}}} \quad \sigma_{28\text{day}} = 42 \text{ MPa} \]

**Compressive Strength of Cylinder 2 (1:1:5 Water:Cement:Water Bucket)**

\[ \text{Load}_{\text{MaxComp2}} = 157500 \text{ lbf} \quad \text{(Observed Maximum Load Applied by Compression Tester)} \]

\[ \sigma_{\text{Test}} = \frac{\text{Load}_{\text{MaxComp2}}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 38 \text{ MPa} \]
\[
\sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{21\text{day}}} \quad \sigma_{28\text{day}} = 41\,\text{MPa}
\]

**Compressive Strength of Cylinder 3 (W/C = 0.6, Water = 350 lb)**

Load_{\text{MaxComp}3} := 185000\,\text{lbf} \quad (\text{Observed Maximum Load Applied by Compression Tester})

\[
\sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp}3}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 45\,\text{MPa}
\]

\[
\sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{21\text{day}}} \quad \sigma_{28\text{day}} = 48\,\text{MPa}
\]

**Mix Optimization Tests**

\[
\text{Ratio}_{7\text{day}} := .7 \quad (\text{Ratio of 7-day strength to 28-day strength})
\]

**Compressive Strength of Mix A (W/C = 0.6, Water = 320 lb)**

Load_{\text{MaxComp}7} := 62000\,\text{lbf} \quad (\text{Observed Maximum Load Applied by Compression Tester})

\[
\sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp}7}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 15\,\text{MPa}
\]

\[
\sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{7\text{day}}} \quad \sigma_{28\text{day}} = 22\,\text{MPa}
\]

**Compressive Strength of Mix B (W/C = 0.6, Water = 350 lb)**

Load_{\text{MaxComp}8} := 72000\,\text{lbf} \quad (\text{Observed Maximum Load Applied by Compression Tester})

\[
\sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp}8}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 18\,\text{MPa}
\]

\[
\sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{7\text{day}}} \quad \sigma_{28\text{day}} = 25\,\text{MPa}
\]

**Compressive Strength of Mix C (W/C = 0.7, Water = 320 lb)**

Load_{\text{MaxComp}4} := 32000\,\text{lbf} \quad (\text{Observed Maximum Load Applied by Compression Tester})

\[
\sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp}4}}{\text{Area}_{\text{Cylinder}}} \quad \sigma_{\text{Test}} = 7.8\,\text{MPa}
\]
\( \sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{7\text{day}}} \)

\( \sigma_{28\text{day}} = 11 \text{-MPa} \)

**Compressive Strength of Mix D (W/C = 0.7, Water = 350 lb)**

\( \text{Load}_{\text{MaxComp5}} := 56000 \text{-lbf} \)  
(Observed Maximum Load Applied by Compression Tester)

\( \sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp5}}}{\text{Area}_{\text{Cylinder}}} \)

\( \sigma_{\text{Test}} = 14 \text{-MPa} \)

\( \sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{7\text{day}}} \)

\( \sigma_{28\text{day}} = 20 \text{-MPa} \)

**Compressive Strength of Mix E (W/C = 0.6, Water = 400 lb)**

\( \text{Load}_{\text{MaxComp6}} := 75000 \text{-lbf} \)  
(Observed Maximum Load Applied by Compression Tester)

\( \sigma_{\text{Test}} := \frac{\text{Load}_{\text{MaxComp6}}}{\text{Area}_{\text{Cylinder}}} \)

\( \sigma_{\text{Test}} = 18 \text{-MPa} \)

\( \sigma_{28\text{day}} := \frac{\sigma_{\text{Test}}}{\text{Ratio}_{7\text{day}}} \)

\( \sigma_{28\text{day}} = 26 \text{-MPa} \)
26 Appendix XII: Construction Manual
Homes for Haiti

Team 13

Ferrocement Panel Construction Manual

May 5, 2012

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Getting Started

So you want to build a ferrocement panel house? This manual will help guide you through the construction process. Before you begin, let’s review what materials you will use:

**Materials for Form**

- 1 - 122cm x 244cm (4’ x 8’) Plywood Piece
- 3 - 5.08cm x 10.16cm (2”x4”) Wood Pieces
- Drill
- Adjustable Wrench
- 10 - 0.952 cm (3/8”) Bolts and nuts
- Plastic Covering (Visqueen)
- Lubricant (oil works well)

**Materials for Internal Support System**

- Chicken wire
- 0.952 cm (3/8”) Ribbed Rebar
- Bamboo

**WARNING:** Using smooth rebar will result in less strength and can result in serious injury to the inhabitants

**Materials for Concrete**

- Sand
- Cement
- Water
- **The mass ratio for Sand:Cement:Water = 6:1:0.7**
- **The volumetric ratio for Sand:Cement:Water = 12:3:2**
- See attached sheet for amount of materials needed for a specific size house

**WARNING:** Changing the concrete mixture ratios will weaken the concrete and can result in serious injury to the inhabitants

**Materials for Connections for Each Panel**

- 6 – 0.952 cm (3/8”) Bolts
- 6 - 0.952 cm (3/8”) Anchor Bolts
- 4 - L-shaped Brackets for each panel
Step-by-Step Instructions

The following manual will take you step by step through the construction process. We strongly recommend that you follow this guide as closely as you can, as changing materials or dimensions could change the overall strength and stability of the building.

Form
The form will be constructed using plywood and 2x4s.

<table>
<thead>
<tr>
<th>Part</th>
<th>Dimensions (cm)</th>
<th>Quantity Required</th>
<th>Label</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside Plywood</td>
<td>239 X 71 X 1.3</td>
<td>1</td>
<td>A</td>
</tr>
<tr>
<td>Outside Long</td>
<td>232 X 3.8 X 6.4</td>
<td>2</td>
<td>B</td>
</tr>
<tr>
<td>Outside Short</td>
<td>71 X 3.8 X 6.4</td>
<td>2</td>
<td>C</td>
</tr>
<tr>
<td>Inside Plywood</td>
<td>220 X 51 X 1.3</td>
<td>1</td>
<td>D</td>
</tr>
<tr>
<td>Inside Long</td>
<td>220 X 3.8 X 2.5</td>
<td>2</td>
<td>E</td>
</tr>
<tr>
<td>Inside Short</td>
<td>43.4 X 3.8 X 2.5</td>
<td>5</td>
<td>F</td>
</tr>
<tr>
<td>Bolts</td>
<td>1.3 X 8.0</td>
<td>10</td>
<td>G</td>
</tr>
<tr>
<td>Washers</td>
<td>1.3 X 4.0</td>
<td>20</td>
<td>H</td>
</tr>
<tr>
<td>Nuts</td>
<td>1.3</td>
<td>10</td>
<td>I</td>
</tr>
<tr>
<td>Wood Screws</td>
<td>5.0</td>
<td>40+</td>
<td>J</td>
</tr>
</tbody>
</table>

1. Cut the plywood sheet into the two specified pieces (Parts A and D).
2. Cut 2x4s lengthwise, cutting the 89mm width into the two specified pieces (64mm and 25mm wide pieces)
3. Cut the 64mm pieces into the proper length for the outside walls of the form (Parts B and C)
4. Cut the 25mm pieces into the proper length for bracing the raised middle (Parts E and F)
5. Drill holes for bolts into plywood and walls in specified locations.

6. Screw the inside short and long pieces to the inside plywood. (Screw parts E and F to part D)
7. Use circular saw or table saw to cut a 5-10 degree draft angle for inside piece of form.

8. Screw the inside plywood down into the outside plywood. Be sure to be generous with the use of screws.
9. Staple or nail the plastic on the inside of the form. Either fastener is acceptable. Be ensure concrete will not be able to get underneath the plastic at any location

10. Coat the plastic with a layer of oil

    The form is now ready for concrete pouring
Exploded View of Form
Assembled Form
**Internal Support System**
The internal support system uses rebar and layers of chicken wire to strengthen the concrete. To put together the internal support system, follow these instructions.

1. Cut two mesh pieces to 226 cm by 63.5 cm (89” by 25”). Flatten the mesh as much as possible by walking on it and/or rolling a heavy object over the surface.
2. Cut rebar to be 2.5 cm (1”) shorter than the inner walls of the form for both long and short ends.
3. Roll the rebar into the mesh 1 to 2 times so the internal support system fits in the form.

**Mixing and Pouring Concrete**
The specified concrete has the following ratios

- The mass ratio for Sand:Cement:Water = 6:1:0.7
- The volumetric ratio for Sand:Cement:Water = 12:3:2

Use the following amounts of each material for one panel:

- **Sand**: 12.5 shovel-fulls
- **Cement**: 3.5 shovel-fulls
- **Water**: 0.5 5-gallon buckets

As specified earlier, use these amounts as a starting point, but also pay close attention to the overall slump and workability of the mixture.
We strongly recommend that you follow this formula. If other substitutions are made for the materials, the overall strength of the concrete could be jeopardized.

1. Mix the concrete using an electric mixer if at all possible. Because of the amount of material, the material should be split in half. Mix half of the total panel at a time in a mixer.

2. Ensure the concrete has been thoroughly mixed before beginning to pour concrete

3. Perform a standard slump test on the mix. The slump of the mix should be approximately 5 cm to 7.5 cm (2” to 3”). If it is not, add sand to get a smaller slump or water & cement (in a ratio of 0.7) to get a larger slump. If a slump test cone is not available, find a material with similar dimensions, and perform the same test.

4. Pour a 1 to 1.5 cm (0.5 in) layer of concrete into the form

5. Install the mesh and rebar internal support system over the layer of concrete. Ensure the rebar is properly centered in the outside lip. Push down internal support system until ideal cover (0.5” or 1.27 cm) is reached.
6. Pour the rest of the concrete into the panel until it is filled.
7. Use a pneumatic hammer drill to vibrate the side of the panel. This fills any voids that might be present in the panel.

8. Use a trowel or piece of lumber to level concrete
9. Lightly spray the surface of the panel and keep covered with plastic or cloth to keep moisture inside. The panel is ready for removal one to two days after completion.
Removal of Form from Panel
1. After one to two days, detach larger wood pieces from form by removing bolts.
2. Rotate complete form and panel on side

3. Use rubber hammer to tap on wooden form to separate from concrete panel
4. Use hand or crowbar to pry between form and panel
Connection Drawings

[Diagram showing various measurements and components, including "6.35 cm", "6.50 cm", "7.75 cm", "Bolt", "Ferro-cement panel", and "Roof connection detail."
PANEL TO PANEL CONNECTION