EMPOWERMENT MODEL FOR POST-QUEAKE RECONSTRUCTION OF URBAN HOUSING IN HAITI

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by

Dustin T. Mix

_________________________________
Tracy Kijewski-Correa, Co-Director

_________________________________
Alexandros Taflanidis, Co-Director

Graduate Program in Civil Engineering and Geological Sciences

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Abstract

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For the first time in human history, the majority of the world’s population lives in urban areas and of the 15 most populated cities, 11 are located in developing countries. This trend towards urbanization in the developing world is creating massive vulnerabilities to natural hazards, especially in housing, something that was tragically demonstrated in the 2010 Haiti Earthquake. This thesis offers a novel framework for post-quake residential construction in the developing world, which extends beyond engineering, and into financial, capacity, and cultural constraints. The ultimate goal is to establish sustainable solutions through the empowerment of local populations. This thesis also details a case study in the implementation of the framework in Haiti, which results in a paradigm shift from traditional masonry construction to a frame and panel system. The latter half of the thesis details the design of the new system, as well as constructability issues in urban Haiti.
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CHAPTER 1:
INTRODUCTION TO INFORMAL ECONOMIES, CONSTRUCTION, HOUSING, AND SETTLEMENTS

1.1 Construction, Housing, and Settlements in the Developing World

For the first time in human history, more than 50% of the global population lives in urban areas (Voss 2006). This shift in population distribution has taken place across the board; however, it has been particularly the case in the developing world. In 1950, 11 of the 15 most populated cities in the world were located in developed countries. By 2000, that ratio had inverted, and 11 of the 15 most populated cities in the world were located in developing countries (Voss 2006). As the global economy continues to expand and morph, and local sustenance economies are replaced with global supply chains, city centers are increasingly viewed as sources of opportunity and a higher quality of life. This is especially true in the developing world; however, more often than not, the opportunities desired do not exist, and the promise of formal employment, reliable income, and stability remains unfulfilled.

Without the formal opportunities expected, these newly urbanized populations must adapt and are left to create their own. Thus emerges an informal economy, defined by unregulated businesses, transactions, labor, and products. Operating
completely outside the formal systems established by governing bodies, this informal economy caters to the constraints of urban poverty. For those usually excluded from the systems that create a dignified standard of living, this type of economy serves as a surrogate, allowing the urban poor to create their own systems for delivering services and products.

However, while these informal economies serve the basic needs of those excluded from formal systems, they cannot always do so in a safe and dignified manner. A great example is the housing sector. Quality housing requires three elements of infrastructure: (1) financing mechanisms, (2) labor and material resources, and (3) engineering and design expertise. Deficiencies in any of these elements can lead to vulnerabilities that ultimately endanger human lives. When this housing sector infrastructure is placed into the context of informal economies, all three elements are impacted. Financing mechanisms, such as mortgages, become exclusionary, as persons are not able to validate their informal and irregular income and assets to be used as collateral are rare. Material and labor resources are scant, as cost becomes a suffocating constraint and formally trained labor is uncommon. Finally, engineering and design expertise are virtually nonexistent because of a lack of educational pipelines.
As the informal economy constrains the housing market, the result is an informal construction sector (Figure 1.1). A microcosm of the informal economy, the informal construction sector is also defined by a lack of government oversight and regulation. Building codes are typically absent, and even if adopted at a high regulatory level, have no inspection infrastructure to enforce them. Housing construction goes unchecked, creating the natural byproduct: informal housing. Often the product of do-it-yourself techniques, housing in the context of an informal economy carries vulnerabilities as a direct result of the resource constrained environment. As informal housing is replicated, larger informal settlements and neighborhoods are created. This creates local communities with common underlying vulnerabilities arising from localized constraints. Finally, as these neighborhoods expand around city centers, a direct result of the urban migration trend, slums are formed, which in many cases, are located on the periphery of the largest cities in the world.
1.2 The Haiti Case

The progression described is not simply an abstraction; there are countless examples of this progression across the world. An illustrative but devastating example is the country of Haiti. On January 12, 2010 the Republic of Haiti experienced an $M_w$ 7.0 earthquake that proved to be one of the most destructive earthquakes ever recorded. This however, was not due to unusually high ground motion intensity, but rather the vulnerability of the communities it impacted (Eberhard, et al. 2010). With death toll estimates above 200,000 and injury counts over 300,000, the impact of the earthquake on the country as a whole was staggering. It is estimated that 1.3 million people were displaced immediately after the earthquake, nearly 13% of the Haitian population, forced to reside in temporary shelters because of the damage and destruction caused to the urban residential housing sector. Even before the earthquake struck, approximately 80% of Haiti’s population was living below the poverty line, with 54% living in abject poverty (CIA 2010). These conditions fostered the informal construction sector, driving the level of destruction the earthquake caused. The earthquake exposed the deficiencies in the urban housing stock and also the dangers posed by informal, unstructured settlements throughout Haiti’s urban areas (CIA 2010).

The current absence of truly sustainable solutions to urban housing in Haiti is no surprise. Existing options used in other seismically active regions, and even other parts of the developing world, cannot be extrapolated to Haiti due to its unique requirements and constraints on housing. As the poorest Western nation, with import challenges, and severe deforestation (Farmer 2011), sustainable construction practices cannot rely on
the many engineered materials that are required in traditional code-compliant designs. The challenge is heightened with the need to ensure resilience to both hurricanes and earthquakes, while still supporting cultural preferences towards privacy and security (Mix, et al. 2011). Unfortunately, the ability to build Haitian capacity for sustainable and resilient infrastructure is further hampered by the preexisting lack of education, codification, and oversight to regulate its construction processes. The entire concept of hazard resistance in Haiti has been focused solely on hurricanes and flooding in the past few decades. Heavy rainfall and tropical storms occur on an annual basis, while the last seismic event that significantly affected this region dates to the 1800s. The structural systems therefore employed lack any aseismic considerations, which ultimately adds to the complexity of the problem. These factors provide an important context to understanding the causes for the widespread destruction after the earthquake, but sadly create what may be the most difficult reconstruction effort following any major disaster.

Earthquake-resistant construction does exist throughout the world, leading to the assumption that a similar level of resilience could be achieved in Haiti by simply importing and enforcing United States or international building codes and established structural systems such as confined masonry. It is first important to distinguish between critical public infrastructure and residential housing. Certainly, critical infrastructure like schools and hospitals can be designed to meet accepted codes and standards using such internationally conventional design and construction approaches, since historically their development in Haiti has been largely supervised and facilitated by the financial support
of NGOs. However, there is a fundamental flaw in assuming a similar level of support can or should be extended to residential housing, even in short-term rebuilding, as housing has never been financed to this degree in Haiti (Kidder 2003). The lack of locally available construction materials, including the wood necessary for formwork to cast earthquake-resilient concrete frames, the steel necessary to provide strength and robust ductile behavior, or the quality masonry for confined or load bearing masonry construction makes this style of construction too expensive for the majority of Haitians.

This lack of understanding surrounding the Haitian housing sector leads to the speculation that “...building code [adoption] and [strict enforcement]” is the one and only solution to the Haitian urban housing dilemma (Lindell 2010). This line of thinking leads to well-intentioned efforts to educate masons, architects, and engineers to facilitate Haitian-led masonry reconstruction. While this is certainly sustainable, as it uses local materials and native construction technologies, the lack of available resources to do it in an aseismic fashion, ensures the future reemployment of the same building systems that proved deadly in the 2010 earthquake (Kijewski-Correa and Taflanidis 2011). Solely encouraging the continued use of masonry-based structural systems by providing education and one-time access to high quality construction materials through relief funds, suggests to Haitian builders that such housing designs can be made truly resilient. If one considers the materials available to the typical Haitian family in the absence of foreign aid, that sentiment is simply not accurate. This is nothing short of false hope, particularly since updated assessments of the seismic hazard suggest that there is substantial earthquake risk throughout Haiti, with locations such as Port-au-
Prince and the Enriquillo Valley (the epicenter of recent seismic activity) demonstrating increased vulnerability due to potential site amplification phenomena (Frankel, et al. 2010). As these revised seismic hazard forecasts are significantly greater than previous estimates (Shedlock 1999), permanent housing introduced in the coming years must present solutions that span the resource spectrum available to Haitians. In other words, to avoid repeating this tragedy in the next earthquake, the poorest of Haitian families need to be presented with alternate affordable housing models with new structural systems and materials.

Although the focus has been on informal construction and its affect on housing in this discussion, commercial and public infrastructure also suffers from a lack of regulation and oversight. Haiti’s commercial and public infrastructure underwent catastrophic damage and destruction as well during the 2010 earthquake, much of which can be attributed to the same causes as the informal construction sector. Although the typologies are different, the core issues underpinning the destruction of commercial and public sectors and the residential sector are the same. This chapter will explore the building typologies used in each sector, as well as a description of how the residential sector faces particularly difficult challenges.

1.3 Commercial And Public Sector Typologies

The dominant mode of construction in the commercial and public sectors in Haiti is reinforced concrete moment resisting frames (RCMRFs) with masonry infill and concrete slab floors and roof. There are cases of steel-framed buildings, however, these
are usually constrained to warehouses and manufacturing facilities. Due to the funds available to commercial and public institutions, buildings have typically undergone an engineering design process, although the quality, depth, and thoroughness of that process may vary. Even if formal design procedures have been used, because of the lack of an adopted building code and any mechanism to inspect or enforce one, it is common for the design to fail to address all of its known vulnerabilities. Overall, the quality of these two sectors is much higher than the residential sector, although it is not always sufficient to ensure life safety. This section will explore both RCMRFs and steel-framed buildings in Haiti, how they are constructed, and the vulnerabilities that the informal construction sector introduces into them.

1.3.1 Reinforced Concrete Moment Resisting Frames

The most common structural system for commercial and public development is a reinforced concrete moment resisting frame, supporting concrete slab floors/roof, and completed with masonry infill walls. Many of these structures undergo a design process, typically by a local engineer/architect. Most of these engineers/architects are either trained in the capital Port-au-Prince, the United States, or other countries such as Canada and France (Kijewski-Correa, et al. 2012). Unlike many commercial buildings in the developed world, it is rare to see a commercial or public sector building reach over ten stories in Haiti. There are a few examples, such as the Digicel Headquarters in Port-au-Prince, that do surpass this height, but the majority lies in the 1 to 5-story range. As such, the design of these buildings is relatively standard, relying heavily on adequate
sizing and proper steel reinforcement detailing of beams and columns, as well as quality materials. As stated previously, there is no formal building code in Haiti, however, most construction projects, especially in the commercial and public sectors, must be approved by the local municipality. While this is an added layer of quality control, the technical knowledge base and diligence with which these plans are reviewed is lacking. There is no formal requirement for hurricane or seismic detailing, and even if there was, many engineers/architects do not have the training to sufficiently carry out such an analysis and design.

Figure 1.2: Example of commercial RCMRF construction in Haiti.

1.3.2 Steel-Framed Structures

Although not nearly as common as the RCMRFs, there are examples of pre-engineered metal buildings (PEMBs) in Haiti (Eberhard, et al. 2010). These structures are most common in highly urbanized areas, and are usually associated with manufacturing or storage facilities. Compared with the PEMBs common to the developed world, these structures differ slightly in Haiti. In the developed world, PEMBs consist of a steel portal
frame with metal cladding and roofing. In Haiti, while the roof is still clad in metal, the walls are typically unreinforced masonry infill (Eberhard, et al. 2010). There are also examples of reinforced concrete columns supporting a pre-engineered steel truss roofing system, as shown in Figure 1.3. Overall, pre-engineered metal structures do exist in Haiti, albeit uncommon, and usually employ a blend of the RCMRFs and masonry infill with the metal structure.

Figure 1.3: Example of hybrid PEMB construction in Haiti.

1.4 Residential Typologies

Haitian housing has a diverse history, varying across time periods and the rural to urban spectrum. From traditional wattle and daub construction, to timber “gingerbread” houses built in the early 1900s, to the current concrete and masonry models, the tradition of housing in Haiti has evolved as the constraints in the country have changed. A survey was conducted with 1,390 displaced individuals over the last
three years in the city of Léogâne, administered by a combination of the author, his advisors, and a team of Haitians. Each individual surveyed represented a household, and the surveys were conducted in eight different neighborhoods of Léogâne (the full survey can be found in Appendix B). Through these surveys, it was concluded that the typical urban household consisted of 4.4 persons, living in an average of 3.3 rooms. Most homes include an open-air porch, an outdoor kitchen, and an outdoor restroom. Haitian culture also drives homes to be internally segmented with many rooms, much like homes in the United States. As such, parents and children typically have separate bedrooms (Kijewski-Correa, et al. 2012). In the Port-au-Prince metropolitan area, one-story homes accounted for almost 63% of all homes as of 2010. In other urban areas, the percentage was nearly 72%. Multi-story homes or apartments only accounted for 30% and 8% of the sector in Port-au-Prince and other urban areas, respectively. It is clear that the majority of Haitians prefer single-family, single-story homes. Additionally, of the homes that were one-story, 76% had walls of concrete masonry units (CMUs), stone, or concrete walls, 82% had sheet metal roofs, and 64% had concrete floors (IHSI 2010). This data supports the claim that Haitian housing is highly dependent on availability and cost of building materials.

Security is also a large concern in Haiti. Based on data gathered by Global Facility for Disaster Reduction and Recovery (GFDRR), the population density of the urban areas in Haiti can range from 20,000 people per square kilometer up to 40,000 people per square kilometer (GFDRR 2010). This range puts urban Haiti among the top countries in urban population density in the world. Such close living quarters present problems for a
multitude of reasons, including security from intruders. Hence, exterior windows and doors typically have iron gates/bars attached to mitigate the threat of theft.

Below are summaries of the most common housing typologies historically and currently seen in Haiti.

1.4.1 Wattle and Daub

One of the oldest and most traditional forms of housing in Haiti is wattle and daub construction (Figure 1.4). Wattle is produced from weaving thin branches (usually palm) into a mat, which becomes the basis for wall construction. The wattle mats are placed upright between timber vertical members to create the skeleton of the wall. Daub is then created using a mixture of water, clay, mud, and straw (or other fibrous material), which provides reinforcement to the daub matrix. The daub is applied over the wattle in layers to finish the wall. This construction method is one of the oldest and widely used because of its relatively low cost and required skill level to construct. It can still be found throughout Haiti, both in the rural sections of the country and also in the poorer unstructured urban settlements.
1.4.2 Timber “Gingerbread” Homes

Built upon Victorian and French Colonial influences, “gingerbread” homes are wood structures scattered throughout urban areas in Haiti. Built primarily during the late 1800s and early 1900s, these structures rely on timber frame construction, aligning with the traditional European and American residential structural systems. They utilize a frame built of timber, with wood plank partitioning (Figure 1.5). This type of construction has become much more rare due to the fact that, over recent decades, the country of Haiti has become 97% deforested because of poor agricultural practices and the widespread use of charcoal for cooking purposes (Williams 2011). This typology has been most commonly used in 2-story applications, encompassing the larger homes on the residential spectrum in Haiti. They are most common in urban areas, especially town centers and moderately dense neighborhoods.
1.4.3 Reinforced Concrete and Masonry Construction

The most common type of construction today is a combination of reinforced concrete and masonry. With deforestation eliminating any opportunities to build with wood and imported structural steel far too costly, concrete and masonry are the most economical choices for building materials. Only requiring cement, water, sand, and aggregate, concrete can be mixed on site, often by hand. Water from adjacent shallow wells is used, and while crushed large aggregates are available, cost often drives the use of more affordable smooth river-rocks. Similarly, cheaper fine aggregates, such as beach sand containing salts that reduce concrete durability and strength, are commonly employed despite the availability of superior materials. The high cost of imported steel implies that the amount of reinforcing steel used in construction is often compromised. Walls are constructed using unreinforced concrete masonry units (CMUs), hand pressed
locally using sand and cement, with wide ranging quality, again proportional to cost.

Figure 1.6 shows a few examples of construction materials used widely in Léogâne.

Figure 1.6: Construction materials in Haiti: (a) CMUs (b) smooth river rock used as concrete aggregate, (c) collection of materials outside of residential construction site, and (d) hand press used to make CMUs. (Photo Credits: B. Grissinger, B. Dolan)
The most common structural system created from the concrete and masonry is an informal hybrid of a reinforced concrete frame and load-bearing masonry walls. Walls are created by stacking unreinforced CMUs in sections, separated by small reinforced concrete columns (Figure 1.7). Once the desired height is reached, the walls are capped with either a sheet metal or concrete slab roof (Figure 1.8). Unlike other systems, such as confined masonry, there is typically no ring beam, and steel reinforcement in walls is rare (Mix, et al. 2011). In light of the extreme poverty and the lack of any lending and financing system, the construction of a home progresses as funds become available, designed in the absence of a municipal building code. This leads to an incremental construction model (Figure 1.7), not uncommon throughout the world (Youd, et al. 2000), and implies that homes can take a significant amount of time to complete (on the order of 8-9 years for a typical urban home). As a result, this mode of
construction often affords the provision to add floors later, as funds become available. Flat concrete slabs therefore often serve not only as floors, but also as roof systems in many multi-story homes (Figure 1.8). In many cases, the slabs are not strictly formed from reinforced concrete, but rather CMU may be placed as fillers with concrete is then cast around them to form the slab. Longitudinal reinforcement commonly protrudes from these roof slabs to facilitate column splices for an additional floor if funds become available. This typology revolves around the ability to construct and expand incrementally, which often requires a sacrifice in material quality and consistency.

![Figure 1.8: Block and concrete construction.](image)

CMU also serves an important role in construction. Due to the deforestation issues, the availability of wood for formwork is quite limited. Therefore, building CMU walls first adjacent to rebar cages anchored in the slab or foundation has an important practical implication: it effectively provides “formwork” for up to two sides of a column. The formwork for the remaining sides of the column is planked with available scrap
wood (Figure 1.9). Due to this construction practice, it is fairly common for the size of
the CMU blocks to dictate the size of the concrete columns used, regardless of the
lateral or even gravity demands on the structure. The most common practices witnessed
in Haiti were poorly confined, undersized columns measuring 16x16 cm (6.3x6.3 in) with
4 longitudinal bars (typically no. 3 and in some cases smooth) and transverse
reinforcement with spacing at 20 cm (8 inches) and a 90° hook. As a result of this
practice, the columns in Haitian construction are considerably smaller [typical cross
sectional area $A=256 \text{ cm}^2 (41 \text{ in}^2)$] with less longitudinal reinforcement than those
observed in reconnaissance of similar urban structures in Turkey [$A=1500 \text{ cm}^2 (240 \text{ in}^2)$]
and India [$A=1035 \text{ cm}^2 (165.6 \text{ in}^2)$] (Youd, et al. 2000; Jain 2002).

Figure 1.9: Utilizing CMU as partial formwork.

In multi-story homes, once all the CMU walls are constructed and concrete
columns cast, plywood sheets are shored up to create a formwork for the concrete slab
Thus the floors can be considered flat plates, devoid of beams to tie the lateral system and without the presence of column capitals or drop panels that would increase the shear and moment resistance of the system (Taranath 1998). In the case of low-income, single-story houses, the CMU walls and columns are often topped with a wood-framed, corrugated metal roofing system, often without the presence of a ring beam (Figure 1.10).

Although all of these systems exist in Haiti as presented, there are also cases where multiple typologies are combined to create hybrids of the three basic systems explored here. Therefore, it is also advantageous to reduce each typology into its individual elements, and view housing in terms of structural systems, as opposed to specific typologies. Table 1.1 canvases the structural systems arising from the typologies explored above as well as their individual elements. This allows one to visualize the range of options that are available for residential housing in Haiti under various
structural systems. The table illustrates the range of options available to Haitians and will prove helpful in later chapters as both current vulnerabilities and future options are investigated.

TABLE 1.1:
CURRENT STRUCTURAL SYSTEMS IN HAITIAN HOUSING

<table>
<thead>
<tr>
<th>STRUCTURAL SYSTEM</th>
<th>SYSTEM MATERIAL CHOICES</th>
<th>PARTITIONING CHOICES</th>
<th>ROOF SYSTEMS</th>
<th>FLOORS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Bearing Walls</td>
<td>• CMU • Earthen • Stone • Wood</td>
<td>• CMU • Earthen • Stone • Wood</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Confined Masonry</td>
<td>• CMU</td>
<td>• CMU</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frames</td>
<td>• Wood • Concrete</td>
<td>• Woven Thatch • CMU • Metal Sheeting • Wattle &amp; Daub</td>
<td>• Metal Sheeting • Concrete Slab</td>
<td>• Concrete Slab • Wood Plank • Stone • Compacted Earth</td>
</tr>
<tr>
<td>Hybrid</td>
<td><em>Ground floor consisting of a standard heavier system (frame or load bearing wall type) topped by light framed system</em></td>
<td><em>Dependent on the light framed system choice</em></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
1.5 Research Question and Objectives

This master’s degree thesis will conceptualize and analyze possible models for affordable housing solutions, and ultimately recommend an alternative urban housing paradigm in Haiti. The challenges to doing so are significant and intertwined with many other areas of development related to education, health, governance and security. Creative, holistic solutions that encapsulate the problem and approach it in new and innovative ways are key in tackling the housing problem. That being said, any solution must be mindful of historical and cultural context and preferences with the flexibility to match specific community and familial needs.

The solution to the task of creating an affordable housing paradigm in Haiti has been reduced to four key areas of research. This thesis will not directly address all four areas, but those that are not addressed here can be found in another thesis written by the author (Mix 2013), focused specifically on how a viable local business could be built around the housing model proposed. All areas are listed here, however, to show the interconnected nature of the research with subjects outside the realm of engineering and the influence they have on those that are covered in this document. The four key areas that must be addressed are defined and explained in the following sections.

1.5.1 Resiliency

The first core area of research is to understand the physical housing structure and the environment in which it resides. This area of research will focus on understanding the hazards that affect Haiti and the vulnerabilities in current housing
paradigms. This will result in the selection of a structural system that can ensure life safety and protection against the range of hazards that threaten Haiti. The tasks and objectives associated with this area of research are as follows:

Tasks:

V1. Field reconnaissance and inventory of current urban housing models and their resilience and vulnerabilities to hazards, with a particular focus on hurricanes and earthquakes.

V2. Testing of pre-existing and proposed construction materials.

V3. Modeling of proposed structural systems and analysis under gravity, wind, and earthquake loadings.

Objectives:

F1. Understanding of the current urban housing stock, their properties, and vulnerabilities to natural hazards.

F2. Spectrum of viable materials that could be used for different elements of the structure.

F3. Selection of possible structural systems, including materials, dimensions, and reinforcement details based upon modeling and capacity evaluation.

1.5.2 Sustainability

The second core area of research is to understand the economic constraints that influence the selection of a housing system. This area of research will focus on the intersection of a household’s financial capacity and the market reality of labor, resources, and engineering expertise. This will result in a housing model that can be supported indefinitely using local resources (economic and natural), technologies, and skill sets and can adapt to the evolving needs of the community. The tasks and objectives associated with this area of research are as follows:
Tasks:

V1. Identify current raw material and labor costs of existing urban housing models.

V2. Research viability of seeding alternative materials and technologies from a production, distribution, and retail basis.

V3. Discuss opportunities through new industries and supply chains for those currently making their livelihood from materials not used in the proposed urban housing paradigm(s).

Objectives:

R1. Discuss a potential plan for creating and seeding new industries seeded by alternative building material production.

R2. Cost breakdown of proposed urban housing model and research proving its availability to the informal housing market demographic.

R3. List of possible funding mechanisms for the procurement of materials and labor required to build a house conforming to the proposed paradigm.

1.5.3 Feasibility

The third core area of research is to understand the capacity constraints, including materials and labor, which influence the ability to seed a new housing model. This area of research will focus on understanding the local availability of material resources and labor skill sets that influence which housing systems can be implemented. This will result in a housing model that can be practically implemented using locally available technologies, capabilities, and resources. The tasks and objectives associated with this area of research are as follows:

Tasks:

R1. Research and classify current and historical technical skill sets used in the construction of urban housing in Haiti.
R2. Understand construction knowledge dissemination modalities and the different types of construction crew models.

R3. Identify skill sets required for proposed urban housing paradigm(s) and evaluate them against existing skill sets in Haiti.

R4. Design an education and training program that builds upon pre-existing technical skill sets to teach the labor force the skills required for the new housing model.

Objectives:

R1. Inventory of technical skill set capacity of Haiti, specifically of the city of Léogâne.

R2. Urban housing paradigm that utilizes and expands pre-existing skills and local capacity.

R3. Set the foundation for training and education program to disseminate knowledge that enhances technical skill sets, including quality control, inspection, and basic seismic and hurricane resistant design/construction practices.

1.5.4 Viability

The fourth core area of research is to understand the historical, cultural, and societal traditions and preferences that determine the willingness of the people to call a structure a home. This area of research will focus on understanding the cultural expectations of housing (e.g., appearance, functionality, materials, and layout) that determine whether a housing system will be accepted in the community. This will result in a housing model that can earn the support of all stakeholders so that its associated practices, policies, and technologies are not only accepted, but also embraced and promoted throughout Haiti. The tasks and objectives associated with this area of research are as follows:

Tasks:
R1. Understand the history and culture of the nation of Haiti, including (but not limited to) family structure, religious beliefs, community dynamics, language, local economy and industry, and demographics.

R2. Engage community in participatory surveys and workshops to understand cultural preferences, specifically as it applies to urban housing, including issues pertaining to owning versus renting, house layout, desired amenities, and opinions about alternative materials.

R3. Propose a feedback loop that continues to engage the community throughout the development of a paradigm and into its demonstration, implementation, and assessment/monitoring stages.

Objectives:

R1. Structure for community engagement and feedback that values the opinions and desires of the Léogâne community, while also providing an avenue through which the proposed paradigm can be explained and understood by the Haitians from a technical perspective.

R2. An urban housing solution that fits into the Haitian history and culture and becomes utilized, embraced, and promoted in the future.

1.6 Thesis Outline

This introduction section has demonstrated the vulnerabilities created by informal construction, with a particular focus on the residential sector in Haiti, post-2010 earthquake. However, the structural vulnerabilities inherent to the current housing paradigms are not strictly defined by engineering and construction issues. The challenge of creating a structural system for residential housing in Haiti that can ensure life safety must be cognizant of the ecosystem within which housing exists, and thus, any solution must address these non-engineering constraints as well. The remaining sections of this thesis will define the environment within which housing exists in Haiti,
and place the selection of a new structural system for housing in that context. The thesis will culminate in the selection of a new housing model and a proposed implementation strategy. More specifically, Chapter 2 will address the 2010 Haiti earthquake with emphasis on reconnaissance data, structural system case studies, and conclusions about Haitian housing vulnerabilities. Chapter 3 will define the Haitian housing sector, in terms of four classes of constraints, directly correlated to the four core areas of research; namely, (1) Hazards and Vulnerabilities, (2) Economic Constraints, (3) Capacity Constraints, and (4) Cultural Constraints. Chapter 4 will focus on defining the solution space, the creation of an evaluation tool for proposed solutions, and the selection of viable options and their associated inputs to this tool. Chapter 5 will focus on one solution in particular, namely a frame and panel structural system, presenting the frame design process as well as constructability issues. Issues addressed will include frame design, behavior, the influence of the panels on the overall system behavior, concrete formwork, and prefabrication of structural elements. Chapter 6 will address the design of the panels, as well as its practical issues of implementation. Specific focus will be given to concrete mix design, connection detailing, formwork design, and constructability. The thesis will conclude with Chapter 7, in which further research will be discussed, such as the construction of a full-scale prototype and future foundation and system level topics, followed by a section devoted to conclusions.
CHAPTER 2:
THE JANUARY 2010 HAITI EARTHQUAKE

2.1 January 12, 2010 Earthquake

The nation of Haiti historically has a long history of seismic activity; however, excluding the 1946 earthquake in the Dominican Republic, the faults on this shared island laid dormant for over 100 years until January 12, 2010. The tremendous devastation of this event can be largely attributed to the many political and economic issues that have faced this nation. Struggles with education, government oversight of civil works, and a general lack of resources have historically prohibited the establishment of reliable civil infrastructure. With no seismic activity in recent decades and regular seasonal rains, tropical storms, and hurricanes, any ad hoc considerations in design and construction were directed toward these annually reoccurring events. Furthermore, as the poorest nation in the Western Hemisphere, Haiti struggled before the earthquake to provide its people with basic necessities, so it is not surprising that hazard resilience of civil infrastructure was not prioritized.

On January 12, 2010 a moment magnitude 7.0 earthquake struck the island of Hispaniola at 4:53 pm local time. The epicenter of the earthquake was in the country of Haiti, approximately 25 km southwest of the capital, Port-au-Prince. The earthquake is
believed to have killed as many as 300,000 people, and displaced approximately 1.5 million people from their homes (Eberhard, et al. 2010). The devastation of this earthquake was particularly apparent in the city of Léogâne, located 29 km west of Port-au-Prince and approximately 10 km northwest of the epicenter of the quake, with a population close to 130,000 people (GeoNames, 2010). Damages to 93% of the buildings in Léogâne were estimated, the majority corresponding to collapses (Eberhard et al., 2010). These reports of extensive structural failures in Léogâne, where the author’s institution maintains a program to combat the spread of lymphatic filariasis, was motivation for the author and his advisors to form a research team to study the vulnerabilities presented by the earthquake, as well as to explore possible future solutions. This process began with two trips to Léogâne. The first occurred in March 2010, with the goal of performing reconnaissance to understand the reasons for the extensive structural failures. The second, in August 2010, was used to observe the progress of reconstruction, to speak with local architects and engineers to understand the best mechanisms to disseminate new knowledge and practices, and to engage local families to understand the functional requirements of a home and the economic resources available to them for reconstruction. This section first presents the major findings of the research team, with particular emphasis placed on the reconnaissance work from the March 2010 visit to Léogâne. Although the focus is on residential housing in Léogâne, case studies include a school in Port-au-Prince to underscore the vulnerabilities created by non-structural elements, even in aseismically-designed structures.
2.2 Reconnaissance

Reconnaissance activities were conducted over the course of two trips: the first from March 8-10, 2010 and the second from August 19-22, 2010. A total of 76 buildings were databased, 57 on the first trip and 19 on the second trip, in the cities of Léogâne and Port-au-Prince (Figure 2.1). The first trip was strictly focused on buildings that existed pre-earthquake, while the second trip also included post-earthquake construction. All structures were physically visited and examined by the research team. Data was recorded through a variety of mediums, including photographs, videos, voice recordings, written assessments, physical measurements, and material sampling. Each structure first went through a damage assessment (see Appendix A for full assessment form), where data was collected concerning the location, building function, structural system typology, layout, dimensions, and an assessment of damage to structural elements.
Figure 2.1: Reconnaissance trips site locations for Léogâne (top) and Port-au-Prince (bottom).
2.2.1 Damage Classification and Assessment

The elements evaluated for damage were columns, beams, and walls. Columns were rated on a damage scale of 0-5, with 0 representing no damage and 5 representing complete failure. Beams were evaluated under four categories: no damage, cracking, spalling, and collapse. Walls were evaluated on the following scale: no damage, in-plane damage, out-of-plane damage, and complete collapse. These element damage classifications were combined with a system-wide analysis to arrive at an overall damage rating. For consistency with existing damage assessment frameworks, rating scales from both the Unitar’s Operational Satellite Applications Programme (UNOSAT) and the Earthquake Engineering Research Institute were included on the Damage Assessment Form in Appendix A.

In addition to qualitative damage assessments, measurements of crack widths in columns, beams, and walls were acquired and then classified using a system developed by the Architectural Institute of Japan (AIJ). Additionally, steel reinforcement type, sizing, and spacing were also recorded (see Appendix A).

2.2.2 Material Samples

In addition to the data discussed in the damage assessment, material samples were also collected throughout both reconnaissance trips. This included steel reinforcement specimens, concrete samples, as well as CMU samples. Both the steel specimens and the concrete specimens were tested upon returning to Notre Dame according to ASTM standards for ultimate tensile strength (ASTM International 2012).
and ultimate compressive strength (ASTM International 2012), respectively. Table 2.1 itemizes the samples collected for each material, the average strengths and standard deviations in the respective tests. Although only limited samples could be tested due to the difficulties of transportation from Haiti to the United States, a few trends can still be drawn from the data. Firstly, it is apparent that the #3 deformed steel is Grade 60, based on the ultimate strength results and the #2 smooth steel is Grade 40. Secondly, and possibly the most important conclusion that can be drawn from the results shown, is the variability in concrete strengths. Because most concrete is mixed on site, by hand, with shovels, Haitian concrete lacks both strength and consistency from batch to batch. This points to larger, systematic problems that arise from a lack of proper equipment in the informal construction sector. The results here, although from a very small sample size, are consistent with more rigorous Haitian concrete testing done at Georgia Tech, in which constitutive ingredients were gathered in Haiti and then mixed and tested in a lab at the university (DesRoches, et al. 2011).
TABLE 2.1:
HAITI POST-EARTHQUAKE MATERIAL SAMPLE TESTING RESULTS

<table>
<thead>
<tr>
<th>Material</th>
<th>Sample Type</th>
<th># of Samples</th>
<th>Average Yield Strength (psi)</th>
<th>Standard Deviation (psi)</th>
<th>Average Ultimate Strength (psi)</th>
<th>Standard Deviation (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>#3 Deformed Bars</td>
<td>2</td>
<td>62,750</td>
<td>3,875</td>
<td>90,318</td>
<td>2,802</td>
</tr>
<tr>
<td>Steel</td>
<td>#2 Smooth Bars</td>
<td>2</td>
<td>36,357</td>
<td>1,112</td>
<td>53,897</td>
<td>2,424</td>
</tr>
<tr>
<td>Concrete</td>
<td>Concrete Cylinder (mixed on site in Haiti)</td>
<td>4</td>
<td>N/A</td>
<td>N/A</td>
<td>1,978</td>
<td>1,480</td>
</tr>
</tbody>
</table>

2.3 Reconnaissance Case Studies

Since the comprehensive database of photos, measured data and annotated metadata from the reconnaissance would be too great to present here, a few case studies are included to illustrate the nature of the reconnaissance work and classes of failure modes and vulnerabilities it revealed in urban construction. Some of the case studies focus on specific buildings and others offer more general trends. With the exception of the first case study, presented to reiterate the vulnerabilities introduced by CMU walls even in well-engineered concrete frames, all other case studies refer to construction in Léogâne. The latitudinal and longitudinal coordinates are provided in parenthesis (N°, W°) when appropriate.
2.3.1 Basil Moreau School, Port-Au-Prince (N18.52841°, W72.38075°)

The Basil Moreau School was built in Port-au-Prince by the Congregation of the Holy Cross and from all external observations, is a well engineered, aseismically designed structure. The school is a three-story building with a structural system consisting of a reinforced concrete moment resisting frame supporting a reinforced concrete slab, with infill CMU walls. The structure is built on a plot with a variable grade, so the ground floor columns are different heights on opposing sides of the building. It is clear that this fact was taken into account in the seismic design however, as the shorter columns have considerably larger dimensions and appear to be designed for the increased shear demands (Figure 2.2). Column dimensions ranged from 35 x 35 cm (14 x 14 in.) [top floor] to 70 x 70 cm (28 x 28 in.) [shorter columns in ground floor]. On the second and third floors, the frames in the longitudinal direction are infilled with CMU walls to either the mid or three quarters height of the column height (Figure 2.3). The remaining space is filled with louvered aluminum windows. Unfortunately, these walls created a “short column” effect evidenced by large shear cracks that failed all the columns on the second floor. The shear failures exposed rows of three #4 longitudinal reinforcement bars with transverse reinforcement near the beam-column joints on the order of 15-20 cm (6-8 in.) hooked only 90° (Figure 2.3). The restraint offered by the partial height CMU infill walls, leading to a reduction in the effective length of the columns, was clearly not considered, as it would have necessitated greater provision for shear effects in their design. Sliding shear failure was also prevalent in the walls in the
transverse direction, particularly adjacent to the staircases, though no walls experienced out of plane failure.

Figure 2.2: Basil Moreau School in Port-au-Prince. Note the size difference in the ground floor columns with those in the upper stories, the failure in the second floor columns, and the extensive shoring used to support the structure post-earthquake.
Figure 2.3: Basil Moreau School. Stiff partitioning creating "short" columns and shear damages in those columns (left). Closer view of failed column. Note the large spacing in transverse reinforcement and the PVC pipe running through the column (right).

2.3.2 Three-Story House In Central Léogâne (N18.51457°, W72.6335°)

The second case study is a three-story residence located in the central district of Léogâne adopting the flat plate system defined in a previous section. The house has a rather elongated floor plan that was short in the direction parallel to the street but ran deep into the lot. On all three floors, the structure showed evidence of sliding shear cracks in walls running into the plot and diagonal shear cracks in walls running in the opposing direction, parallel with the street. The house was heavily partitioned with unreinforced CMU walls, particularly toward the back of the home, which housed the kitchen and bathrooms ventilated by relatively small windows. The front of the home had considerably larger rooms with wider, barred windows, particularly on the street face. In comparison to other houses in Léogâne, this is considered a high-end home built
with funds unavailable to most of the population and designed by a local architect also responsible for many commercial and medical facilities in the area. This particular house showcased many of the recurring failures observed: damage to the columns initiated by shear failure of CMU walls that attract significant shear forces, but have insufficient strength for these seismic demands and exhibit brittle failures expressed by diagonal in-plane shear cracks. As shown in the second floor of this structure, the CMU ultimately failed out of plane, falling into the street (Figure 2.4)\(^1\). As witnessed time and again in other earthquakes around the world, this is all a consequence of columns designed for frame action but ultimately failing by shear action (Jain, 2002). Damage to the ground floor columns (Figure 2.4) led to a permanent drift sufficient to warrant the demolition of the structure. The shear failures induced by stiff masonry walls were prominent throughout Léogâne and often led to even more dramatic column failures, as shown in Figure 2.5. Another classic failure pattern observed in this case study and throughout downtown Léogâne was pounding between adjacent structures (Figure 2.6), which cased the masonry wall at (N18.51457°, W72.6335°) to deflect more than 7 cm (Figure 2.6). This scenario unfortunately played out far differently in other collapsed structures in Léogâne, where pounding led to the direct failure of undersized columns.

\(^1\) Similar vulnerabilities attributed to metal gates and barred windows were also observed at other locations in Léogâne, including a two-story structure located at (N18.51430°, W72.6339°).
Figure 2.4: Barred window in the second floor with shear cracking and out of plane failure of the surrounding CMU wall (left). A ground floor column at the front of a house in central Léogâne. The column has hinged and failed due to the transfer of shear forces from failed CMU walls.

Figure 2.5: Typical shear cracking in a CMU wall (left). The effects of wall shear failure on surrounding columns (right).
2.3.3 Houses In Wealthier Area Of Léogâne

The houses discussed in this subsection are located on the northeast side of Léogâne, the wealthier neighborhood of the city. Most houses in this area have relatively larger room dimensions, with column spacing of typically 4-5 m (13-16.5 ft.), ultimately larger floor plans [160-220 m² (1720-2365 ft²)], and many go as high as three stories. Despite the comparatively larger floor plans, the same structural system and construction practices used for smaller homes in the denser downtown area were extended to these homes. A collapsed three-story home in this area (N18.51495°, W72.62756°) provides an interesting example of this phenomenon. While some
neighboring homes experienced the same collapse sequence, often the concrete slabs would be largely intact, but in this instance, even the slab displayed a brittle failure (Figure 2.7), which illustrates the poor quality of materials used. The typical slab thickness of 15-17 cm (6-6.8 in.) for these homes was found to be reasonably sized for gravity demands, meeting the minimum thickness requirement as stated in ACI 318-08 (ACI, 2008). However, when placed 3.5-4 m (11.5-13.2 ft.) apart, the columns connecting to these slabs should have cross sections on the order of 25-30 cm (10-12 in.) square, just to satisfy minimum design requirements and does not consider additional lateral resistance requirements for earthquakes. Unfortunately, the columns observed were 16 x 16 cm (6.4 x 6.4 in) with four #4 longitudinal reinforcement bars and transverse reinforcement with spacing approximately 20 cm (8 in.), implying they were a quarter of these minimum cross-sectional area recommendations. In fact, regardless of the size of the home or the number of stories, column sizing is largely uniform and dictated, as discussed earlier, by the size of the CMU walls used as part of the formwork. As a result, virtually every home in this area exhibited catastrophic failure. This practice creates a new vulnerability since the diaphragmatic function of the slab is inadequate in the absence of beams, and the columns are thus not sufficiently engaged to provide a unified lateral system. This vulnerability is exacerbated by the fact that most of these houses have multiple floors, leading to increased shear forces on the ground floor columns. Since column size is not selected based on any earthquake design criteria, or even the number of stories, but rather chosen based on dimension of CMU blocks, columns are grossly undersized for the seismic demands. The result -- a highly
vulnerable system that resulted in repeated pancake collapses throughout this wealthy district in Léogâne. In many cases, the largely intact slabs showed evidence of punching shear (N18.51627°, W72.62913°) (Figure 2.7).

Within this district of Léogâne, there were older, even larger concrete homes that still exhibited soft story failures, often due to the vulnerabilities created by incremental construction (N18.51452°, W72.62413°) (Figure 2.8). Though these homes still lacked beams, they appeared to have larger columns and deeper slabs. In fact, a rather impressive home at (N18.51471°, W72.62046°), whose slab was perforated by an open entrance hall that extended the full height of the structure, sustained significant damage (Figure 2.9). The structure exhibited not only classic failures of CMU walls (both in-plane and out-of-plane shear failures) but also hinging in the first floor interior columns (Figure 2.9), leading to eventual partial collapse of the second story.

Figure 2.7: Pancake collapses of multi-story homes in wealthy area of Léogâne due to undersized columns. Notice the poor quality of concrete that led to the brittle failure of all elements, including the slab (left). Commonly, slabs would remain intact, though often displaying punching shear failures (right).
2.3.4 One-Story House In Poor Area Of Léogâne (N18.5077°, W72.6350°)

The next case study is a one-story house on the poorer western side of Léogâne. This area is densely populated and consists of a variety of different housing types, assembled based on the availability of materials at the time of construction. The home
presented in this case study represents a typical lower-income home and employs mildly reinforced concrete columns and CMU walls without evidence of a ring beam tying the system at the top of the walls. The roof system consists of a timber frame, covered in corrugated tin roofing, similar to that shown previously in Figure 1.10. The house’s layout is a square, with two interior walls bisecting each direction to split the house into four equal rooms, each 3-4 m (9.8-13.1 ft.) square. One of the interior walls is taller than the two exterior walls running parallel with it, in order to accomplish a gable style roof. There was evidence of columns at each of the four corners of the structure, but it is unclear if there were any interior columns.

It is evident given the smaller scale of the house, its single story design and comparatively lighter roof system, diminished the seismic demands so that vulnerabilities in the lateral system and the sizing and reinforcement of the columns were not exposed; however, the home still experienced in-plane shear cracking in the CMU walls, some of which failed out of plane. Although these damages were non-structural in nature, they posed considerable safety risks to the family due to the weight of the collapsing CMU walls that were yet to be replaced at the time of the authors’ August 2010 visit. At that point the family was occupying the remaining rooms of the home, repairing cracks to these CMU walls with mortar. This family was fortunate; however many others in Léogâne experienced total collapse of similar homes and as of August 2010 were still awaiting temporary shelters.
2.3.5 Surviving Systems Throughout Léogâne

Even though a significant percentage of the buildings in Léogâne were either collapsed or severely damaged, there were some instances of buildings that performed well during the seismic event. It is important for future rebuilding efforts to understand the important characteristics of these structures that contributed to their survival. It should be pointed out that in many of the houses that remained structurally intact, significant shear cracking and out-of-plane collapse were identified in the walls. Such non-structural failures should not be discounted as they could lead to significant injuries.

One type of structure that performed relatively well in Léogâne is the wood-framed homes on the main street through Léogâne, Grand Rue (Figure 2.10). These historic structures date as far back as the 1800s, as European influences mixed with Haitian architecture. Although many of them show significant signs of aging and deterioration, these comparatively lighter structures, framed and partitioned completely in wood, survived the earthquake with little damage. By comparison neighboring reinforced concrete structures had severe damage and most of them suffered complete collapse. It should be noted that this model of construction, though proven effective in this seismic event, is an unrealistic solution for sustainable future rebuilding, due of lack of local construction grade wood.
Figure 2.10: Timber-framed houses that survived during the earthquake. Notice the completely collapsed reinforced concrete structure in the foreground (right).

Another noteworthy success is the use of reinforced concrete moment-resisting frames. The isolated examples of these structures demonstrated how a properly engaged lateral system could respond favorably. In some cases, part of the structural frame was exposed, with the CMU wall recessed away from the perimeter of the structure. This provides what is commonly referred to as a “wrap-around porch.” Consequently, since there was no interaction between the structural frame and the masonry walls at these locations, there was also no mechanism to transfer concentrated shear forces and columns suffered no visible signs of damage. The two-story structure shown in Figure 2.11 and located at (N18.51048°, W72.63190°) is on the same street as the aforementioned historic wooden houses. Again this is an area where extensive damage was observed to all other surrounding reinforced concrete structures, which appeared to be a mixture of both commercial and residential. Significant shear cracking is evident in the masonry walls, and these damages will need to be repaired, but these
cracks did not lead to compromise of the structural integrity due to the effective isolation from the primary frame system. It should be pointed out that the columns themselves are also considerably larger than those seen in other areas, which also contributed to the better performance. Again, this was made possible due to sheer economics – the ability to afford a better engineered structure and the formwork necessary to create free-standing moment resisting frames whose column sizes were then not dictated by the dimension of CMU traditionally used as partial formwork. CMU was then used as originally intended – as a non-structural infill secondary in the construction sequence, as opposed to a primary element.

Figure 2.11: Two-story structure with columns isolated from the CMU walls. While CMU walls did sustain some shear cracking, they did not create vulnerabilities for the primary structural system.
2.4 Vulnerabilities

Based on the field reconnaissance in Léogâne, the authors identified several recurring failure mechanisms:

- **Shear Failures**: Rigid CMU walls, or even ironwork over openings, are sufficiently stiff to attract seismic forces but have insufficient strength to resist them, transferring significant shear forces to adjacent columns upon failure.

- **Short Column Effect**: Partial height infill CMU walls in concrete frames significantly reduce the effective length of the columns and increases shear demands, a factor not accounted for in their design.

- **Flexible Diaphragms**: Flat plates lack sufficient diaphragm action to engage columns and form an effective lateral system in homes with elongated floor plans.

Contributing factors to the above failure mechanisms:

- **Inadequate Columns**: Columns are not sized for strength, but for constructability (to match the depth of CMU), and are inadequately detailed and confined.

- **Excessive Mass**: The typical home has a reinforced concrete floor/roof system, with CMU used for all partitions, creating a structure with a large mass.

- **Poor Materials**: Concrete is commonly mixed from smooth river rock, beach sands, and well water – not ideal for reliable strength. Additionally there is also often separation of large aggregates from the mix in casting.
• **Incremental Construction**: Due to economic constraints, homes are built over long periods of time, causing significant variation in material quality and workmanship.

These mechanisms match closely other independent reconnaissance studies completed, and in particular on non-engineered construction (Paultre, et al. 2013). From these failure mechanisms and associated factors, four major contributors were identified as the vulnerabilities in typical unreinforced masonry residential construction. These four characteristics are explored in further detail in the following sections.

2.4.1 Unreinforced CMU Walls

Although CMUs address and alleviate many problems that plague the housing sector in Haiti, they pose problems in seismic events. Firstly, walls created from CMUs in Haiti are rarely reinforced in either the vertical or horizontal directions. This severely limits the ductility of the walls and, in a system where the walls are gravity load-bearing elements, any damage due to lateral loading creates vulnerability to total collapse.

There are also three characteristics of the CMUs in Haiti that make them susceptible to failure during earthquakes. The first is that the CMUs are very stiff elements. In the case of Haitian construction, the walls created from CMUs are relatively stiff compared to the other elements in the structure and therefore attract substantial forces. This is especially true considering the poor design, detailing, and construction of the pseudo-columns observed in masonry construction. Secondly, the CMUs are also very brittle and therefore undergo little deformation under loading before they fail. During the
earthquake, the CMU walls have little to no capacity to dissipate energy through deformation. Finally, the type of CMU produced in Haiti has extremely low compressive strengths. CMU samples that have been collected in Haiti and undergone standard ASTM testing have been shown to have an average compressive strength of 1000 psi or less (Lamothe and Filiatrault 2012). Comparatively, ASTM standards in the United States require a minimum of 1900 psi, nearly double the actual strengths observed in Haiti (ASTM 2012). The combination of these three properties put systems utilizing CMUs as primary gravity and lateral structural elements in a dangerous position: due to their stiffness they attract significant seismic forces, but because of their brittleness and low strength, they are incapable of resisting them. This leads to early and explosive failures, often resulting in the progressive, pancake collapse of entire structures, as shown in the reconnaissance.

2.4.2 Inadequate Columns

In most structural systems, columns serve a primary purpose in the structural system, resisting gravity loads, lateral loads, or both. However, in the Haitian housing system columns are a secondary element. They are constructed by leaving vertical gaps between wall sections, placing rebar cages, and then infilling concrete (Figure 1.7). The portion of gravity and lateral loads that they resist is actually minimal, as the unreinforced CMU walls bear most of the demand. Construction process and a lack of engineering expertise in the residential sector have led to severely undersized and underreinforced columns. During the vast majority of the structure’s lifespan, this is not
consequential because the walls are capable of handing the day-to-day gravity loads. In the event of an earthquake however, it has already been established that the CMU walls fail in a brittle and dramatic fashion. Once the walls fail, the forces they were once carrying (gravity and lateral) must be transferred to other elements; the columns in this case. The undersized and underreinforced columns are not capable of carrying this additional load demand and yield and potentially fail soon after the walls. This leads to a progressive collapse of the structure. Even if the structure does not completely collapse, the damaged columns are still inadequate to ensure habitability of the structure. Based on their size and detailing, the columns in isolation are deficient, but when combined with the CMU in this structural system, the deficiency is hidden until the walls fail.

2.4.3 Insufficient Lateral Structural System

The shortcomings of the CMU and columns point to a larger, system wide flaw: a lack of redundancy in the lateral structural system. As soon as the CMU walls fail, there is no alternative load path to transfer the forces down to the foundation. In effect, the structure is doomed to collapse if the CMU walls fail or are significantly damaged. No alternate mechanisms, such as beams exist to provide redundancy. Given the incremental nature of construction in Haiti, it is very difficult for this type of concrete and masonry system to be integrated to form these redundancies. Given that the elements are not precast elements, where this may be possible, the staged construction processes employed in Haiti in the residential sector greatly reduce the ability to build a redundant lateral system.
2.4.4 Excessive Mass

Construction materials in Haiti are limited to concrete, CMU, and stone in the majority of the residential sector. Structures tend to be very heavy, and thereby experience increased seismic load demands. Additionally, the heaviest elements in the home are also the most brittle and typically have low strength. The increase in forces due to the relatively large masses, paired with the vulnerabilities in the CMU, columns, and lateral system are a recipe for severe damage and collapse.

2.5 An Urban Housing Crisis

In the months after the earthquake, housing the displaced population became the country’s most pressing problem (Mix, et al. 2011). The corridor between Port-au-Prince and Léogâne was the hardest hit area, but also the most urban. The Port-au-Prince metropolitan area has a population of over 2 million people, nearly 20% of the entire country. With such a high population and population density, finding adequate space to house those displaced, while still maintaining international minimum standards on clean drinking water, sanitation, and shelter became nearly impossible. In the six months after the earthquake, there were 1,555 internally displaced persons (IDP) camps throughout the affected area housing the 1.5 million displaced persons (IASC 2011). Léogâne was home to 253 of these camps, the second highest number for any neighborhood in the Port-au-Prince metro area. The conditions of these camps were deplorable, often a collection of tents and makeshift shelters, with little or no access to clean drinking water and latrines. As late as February 2012, over two years post-
earthquake, 92% of the 660 camps that remained had yet to receive official transitional shelters (IASC 2011).

Housing is almost always a private development and therefore, many of the funds that flowed into Haiti by the international community after the earthquake were dedicated to causes other than permanent housing. Immediate shelter accommodations, clean drinking water, sanitation, infrastructure, schools, healthcare facilities, and rubble removal were the primary programs to which funds were directed. Although substantial funds were put toward tents and transitional shelters (T-shelters), there was no concerted effort toward integrating a permanent housing strategy into recovery and rebuilding plans. Most families, to this day, are still left to their own devices to secure adequate shelter. Unfortunately, the only available option for permanent housing is to go back to the practices and models that led to such widespread destruction and fatalities during the 2010 earthquake. This has led to a standstill in the housing sector because most people are terrified of reverting back to the practices that performed so poorly and they lack access to capital for reconstruction.

Despite the billions of dollars that have been dedicated to the recovery and rebuilding of Haiti, it seems that progress has been slow. The failure to address the permanent housing problem is one of many contributing factors. Without a mechanism to move people into adequate, safe, and permanent shelter, the other issues will continue to progress at an unsatisfactory pace. The provision of clean drinking water and robust sanitation systems along with ensuring law and order and limiting disease
outbreaks could all be substantially aided by first ensuring that all those displaced had a viable path toward adequate permanent shelter. The home unit provides a backbone on which to build out the rest of the systems critical to the recovery and development of the country.

Much has been written on the fact that the 2010 Haiti earthquake was not a natural disaster, but rather a man-made one (Noriega 2010). Many similar, and even larger, earthquakes have occurred in other locations without the massive destruction seen in Haiti. In fact, less than two months after the Haiti earthquake, a moment magnitude 8.8 earthquake struck off the coast of Chile, near the capital city of Santiago. The Chile earthquake was over 500 times stronger, in terms of energy released, than the Haiti earthquake (Wald, et al. 2010). However, the life and property loss paled in comparison to that of Haiti. Approximately 525 people lost their lives and only 8.8% of the population in the affected area suffered damaged or destroyed homes (Hinrichs, et al. 2011). The incredible difference can be attributed to the increased quality of construction, design, and urban planning in Chile. These factors can, in turn, be highly correlated to the overall development of Chile in the last 25 years, especially to its economic development and the ability of its government to enforce and dictate design and construction standards. In contrast, Haiti is a perfect example of a country where susceptibility to major natural events, large urban populations, and substandard housing practices have led to a humanitarian disaster whose impact has not been matched in modern day history. Unfortunately, as discussed in the introduction of this thesis, Haiti is not a unique case. Across the developing world, urban slums sit at the precipice of
massive devastation. It will only take one hurricane or earthquake to strike to expose the same vulnerabilities that plague Haiti. The problem threatens to only get worse, as developing world populations continue to trade rural living for urban centers in hopes of economic advancement.
CHAPTER 3:
THE HAITIAN HOUSING SECTOR

3.1 A Holistic View

For many in the civil and earthquake engineering community, the field reconnaissance in Haiti following the January 2010 earthquake was a sobering reminder that while the causes of structural failures were easy to identify, practical remedies prove far more elusive (Kijewski-Correa and Taflanidis 2011). In a setting where basic human needs were often unmet before the disaster, the understandable response was to import immediate solutions. Certainly the importation of transitory solutions by NGOs, including temporary shelters, was a necessary and important measure in post-quake Haiti. However, these solutions were transitory at best. Without the equally necessary and important step of introducing options for affordable permanent housing, many Haitians would be left indefinitely in these transitory shelters (Figure 3.1) or would revert back to pre-quake, failed construction practices (Figure 3.2). Ultimately, if any solution seeks to be a real option for the people of Haiti, it must avoid importation, imposition by foreign entities, or heavy subsidization, and instead advocate for self-reliance. However, in order to do this, the entire ecosystem surrounding housing in Haiti must be understood. Unfortunately, this is a process that takes considerable time and,
at times, requires one to look past immediate needs and think about long-term sustainability. This understanding extends far beyond the faulty engineering and construction practices that contributed to the earthquake disaster and into the underlying forces that influence the housing sector. This type of holistic view is necessary to understand why local engineering and construction practices exist, and ultimately, how systems, not just projects, can be implemented to mitigate the issues created by informal construction.

Figure 3.1: Transitory sheltering has reached some families in Léogâne (left) whose hand painted addresses on customized shelters (center) suggests an understanding of the long path to a permanent home. Others remain in informal shelters three years after the earthquake (right).
3.2 Elements of the Housing Sector

There are four dominant forces that influence housing in the developing world. These forces are interwoven, but diverse in their origins and impact. Understanding these four influencers lays the groundwork for treating the chronic problems that face the Haitian housing sector. The four forces are listed here and also presented in Figure 3.3:

- **Hazards and Vulnerabilities:** The natural hazards and other environmental factors that threaten to expose housing vulnerabilities.

- **Economic Constraints:** The intersection of a household’s financial capacity and the market reality of labor, resources, and engineering expertise.
• **Capacity Constraints**: The local availability of material resources and labor skill sets that determine which housing systems can be implemented.

• **Cultural Constraints**: Cultural expectations of housing that determine whether a housing system will be accepted in the community.

![Diagram of Housing Sector with circles labeled Hazards & Vulnerabilities, Economic Constraints, Capacity Constraints, and Cultural Constraints]

Figure 3.3: The forces underlying the Haitian housing sector.

The following sections will explore the dynamics of each influencing force in the context of Haiti. The analysis will not only define the forces, but also explain how their effects contributed to the structural failures that occurred during the 2010 earthquake.

3.2.1 Hazards and Vulnerabilities

There are three main classes of hazards that could expose housing vulnerabilities in Haiti. The first two are naturally occurring: climatological and geophysical. The third class, urban security, is man-made. All three impose different demands on the design,
construction, and location of housing in Haiti. Depending on the location, some hazards are more prominent than others, but all urban housing in Haiti is subject to these three classes.

3.2.1.1 Climatological Hazards

There are multiple hazards within the climatological category. Table 3.1 describes each one, including the locations most susceptible to that particular threat.

**TABLE 3.1:**

CLIMATOLOGICAL HAZARDS

<table>
<thead>
<tr>
<th>Hazard</th>
<th>Description</th>
<th>Effect on Housing</th>
<th>Susceptible Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind</td>
<td>High winds are common in Haiti, mainly due to frequent tropical storms and hurricanes.</td>
<td>Damage to sheet metal roofing, windows, and doors</td>
<td>Coastal areas, high elevations</td>
</tr>
<tr>
<td>Flooding</td>
<td>Flash flooding is rampant due to deforestation and topography</td>
<td>Can wash out foundations, damage home interior, and cause mold and mildew</td>
<td>Flood plains, river valleys, dense urban areas</td>
</tr>
<tr>
<td>Rain</td>
<td>The tropical climate brings significant rainfall during rainy season and hurricane season</td>
<td>Can cause flooding (see above), mold, mildew, mosquito breeding grounds</td>
<td>All areas</td>
</tr>
<tr>
<td>Heat</td>
<td>The tropical climate keeps average temperatures near 90° year round</td>
<td>Can cause health problems (heat stroke, fainting) if home is not well ventilated</td>
<td>All areas</td>
</tr>
<tr>
<td>Sun</td>
<td>Haiti is relatively close to the equator and non-rainy season months can average 20+ days of sunshine</td>
<td>Can cause degradation and warping of materials</td>
<td>All areas</td>
</tr>
</tbody>
</table>
The wind hazard for the island of Hispaniola is illustrated in Figure 3.4, with the peak wind speeds shown for ASCE 7 Exposure C for various return periods. The 50-year return period places most of the country of Haiti at risk for Category I or II hurricanes, and at risk for a Category III hurricane for a 100-year return period, based on the Saffir-Simpson Scale.

3.2.1.2 Geophysical Hazards

There are considerably fewer geophysical Hazards than climatological; however, they are equally as important and dangerous. The first is earthquakes. Haiti is located over two fault lines that pass through the Caribbean. The Enriquillo Fault runs east-west...
through the southern portion of Haiti near the cities of Port-au-Prince and Léogâne, and is the fault responsible for the 2010 earthquake (Figure 3.5). The Septentrional Fault passes through the most northern part of the country, near the former capital city of Cap Haitian (Figure 3.5). Historically, both faults have been active and have produced major earthquakes, although no major earthquakes have struck in the last century (excluding the one in 2010). There has been a pattern of major earthquakes occurring in sets of three, within a 50-year span, throughout the 17th, 18th, and 19th centuries. Historical writings show that these earthquakes have also produced considerable destruction, especially in Port-au-Prince and Cap Haitian. In fact, the Septentrional Fault was responsible for an earthquake in 1842 that destroyed the historical Sans-Souci Palace in northern Haiti (Scherer 1912). There was also widespread destruction in 1770 to Port-au-Prince due to a rupture of the Enriquillo Fault (Eberhard, et al. 2010). Given this historical data, and the fact that neither fault had significantly ruptured for over 100 years before 2010, it has been stated that the threat for another major earthquake is extremely high. Studies in 2008 showed that the Enriquillo Fault was capable of a $M_w$ of 7.2 at any time, which proved fairly accurate in 2010 (Manaker, et al. 2008). The Septentrional Fault was found to be capable of producing a $M_w$ of 6.9 at any time (Manaker, et al. 2008).

Figure 3.6 shows the shear-wave velocity ($V_{s30}$) for the island of Hispaniola. The regions surrounding Léogâne, Port-au-Prince, and Cap Haitian show risk of site amplification affects. Not only are these the regions with the softest soils, they also encompass the two major faults that run through the island, and are also the largest
population centers in the country. Figure 3.7 shows the seismic hazard maps for the island. The top figure is the expected peak ground acceleration (with site amplification effects included from Figure 3.6) for an event with a 10% probability of exceedance in the next 50 years. The lower figure shows the same data, except for an event with a 2% probability of exceedance over the next 50 years.
Figure 3.5: Crustal faults and subduction zones for the island of Hispaniola. Source: (Frankel, et al. 2010)

Figure 3.6: Hispaniola shear-wave velocity, averaged over the top 30 m of ground ($V_{s30}$). Source: (Frankel, et al. 2010)
Figure 3.7: Seismic hazard maps for Hispaniola, including site amplification effects (Figure 3.6), without aftershock hazard. Source: (Frankel, et al. 2010)
The second geophysical threat to housing in Haiti is poor soil stability. Due to its mountainous topography and rainy climate, landslides are common. Without forests and vegetation, topsoil is loose and susceptible to run-off. This threat is especially problematic in urban areas. Cities such as Port-au-Prince have significant populations residing on steep slopes outside of the city center (Figure 3.8). Some are wealthy and have situated themselves there to be removed from the denser areas of the city, and others are poor and are often stuck with living in the least favorable locations of the city. The risk of landslides constantly threatens to wipe out entire neighborhoods in the Port-au-Prince metropolitan area.

Figure 3.8: Hillside in Port-au-Prince, Haiti (Photo Credit: http://pamspen.wordpress.com).
3.2.1.3 Man-Made Hazards

The only hazard that falls under the man-made classification is urban security. Due to the sheer density of the population in Haiti and the anonymity that accompanies living among a population of two million people, theft is problematic. Whether the actual crime rate is high or not, or whether this has any correlation to extreme poverty, is really not relevant when discussing the impact of urban security concerns on housing. A perceived threat is just as impactful on the design and construction of homes in Haiti as the actual threat. Most of the urban population in Haiti perceives a threat of urban crime, and therefore urban security is a serious consideration in the design of urban housing. Haitians want to ensure that their limited resources are put towards constructing a home that will ensure the safety and security of its occupants. This topic has been especially at the forefront of Haitians’ minds since the earthquake because of the high crime incidence in the tent camps. The close living quarters and tarp structures left many vulnerable to crime. This has left many with a conscious concern about urban security when contemplating future housing options.

3.2.2 Economic Constraints

Haiti’s economy has been struggling since it went through a significant contraction during the 1980s, under the rule of dictator Jean-Claude Duvalier. Historically, Haiti’s economy has been driven by agricultural exports, dating back to its days as a French colony. Since the 1950s, overuse and abuse of the country’s agricultural lands have resulted in deforestation, soil erosion, and decreased agricultural
productivity (Haggerty 1989). This decrease led to a spike in manufacturing exports during the 1970s because of cheap, low-skill labor. However, the corruption and exploitation of the general population during the Duvalier years stripped the economy of large amounts of capital. Wealth became highly concentrated in a small segment of the population, driving the rest of the population into extreme poverty. Since Duvalier’s reign ended in 1986, the economy has continued to falter, due to a combination of civil and political unrest and an increased dependence on foreign aid.

Damaged agricultural land and the recent flood of international food aid and imports have further deflated the agricultural industry. This has driven much of the population of Haiti from the rural countryside to the urban city centers in search of employment. In the eyes of Haitians, the city centers are the only areas that hold hope for economic security. However, the economic situation in urban areas is not much better. There is an overabundance of cheap labor, little-to-no manufacturing activity, and a service sector thirsting for tourism. Over the last 20 years, city centers have transformed from lands of promise to slums trapped in the cycle of poverty. Per capita income remains the lowest in the Western Hemisphere and the forecast for widespread economic advancement is dim. With incomes and economic growth low, the purchase of housing remains an obstacle for the majority of urban Haitians.

3.2.2.1 Haitian Household Finances

In an effort to understand the financial reality of the average Haitian household, questions were included in the survey mentioned in Chapter 1. A few patterns emerged
from the data that have given insight into the finances available to a Haitian family for housing purposes, as well as the factors that can greatly influence cost. Firstly, in terms of demographics, households are typically four to five people, almost exclusively comprised of immediate family members. Additionally, the individuals surveyed exist as part of the informal economy, meaning that regular and predictable income is not common. Self-reported savings specifically dedicated to housing averaged 21 USD per month (Figure 3.9).

One notable characteristic of the housing savings data is that it is not uniformly distributed, but rather is characterized by a series of spikes. The survey asked the respondents to report in terms of a local currency called Haitian dollars², and when data is viewed on this scale, there are three spikes at the values 50, 100, and 200 Haitian dollars. While the mean of Haitian savings was 176 Haitian dollars, no person responded within 25 Haitian dollars of that amount. In fact, most Haitians responded at some increment of 50 or 100 Haitian dollars, suggesting the possibility that these incremental values have some greater social meaning, possibly a cultural pattern of talking about increments of money when the values are not specific. This may also be due to the values by which rent is usually discussed. Anecdotally, it has been observed that when a household builds a home, costs are rarely tracked during the process. This is in part due to the incremental construction model that most operate under and the long time spans

² The exchange rate is approximately 8.5:1, Haitian dollars to USD.
that elapse in the construction of a home. Practically, this means that most do not know what a typical home actually costs to build. On the other hand, many are familiar with rental rates in the Léogâne area, and therefore, savings may have been reported in terms of the denominations usually used when discussing rent pricing.

![Figure 3.9: Self-Reported Monthly Savings for Housing from IDPs in Léogâne, Haiti.](image)

This population also exists in an environment where access to capital and financing is severely limited. Lending is exclusively between family and friends, with no other formal channels for accessing cash. Mortgages are not an option, and even the microfinance sector, which is specifically built around serving individuals in this type of
environment, commonly does not provide financial services for the purchase of a home. However, of the 1390 individuals surveyed, over 98% reported some sort of home savings. Most do not hold these savings with a formal banking institution, but rather keep their savings as cash in their current living quarters. With such limited financial resources, these individuals are extremely frustrated by the lack of permanent housing options. The only permanent option that is both safe and available on the market is reinforced CMU construction. This option is priced far above the financial capacity of most of the population. For the typical home requested by those surveyed (four rooms, 600 square feet), the cost of a reinforced CMU or confined masonry home is approximately $12,000. At the average savings rate of $21 per month, and assuming no other forms of financing, one can see that the time frame to completion is unrealistic. This leaves the majority with only one option: the unreinforced CMU system that failed during the earthquake and left most of them homeless. A home of the same size, constructed from unreinforced CMU, costs approximately $4,500 (Gilholly 2012; Mix 2013). While still drastically expensive for this population, it is the most feasible option. There is a high demand among these individuals for an alternative option that optimizes safety and cost, while also catering to the functional needs of the household.

3.2.3 Capacity Constraints

3.2.3.1 Building Materials

Even if the purchasing power was present among Haitian households to purchase quality materials, their availability is extremely limited. There are essentially
three classes of building materials for any location: native, locally manufactured, and imported. Native materials are naturally occurring local materials that can be easily and cheaply transformed into useful building elements (e.g., wood). Locally manufactured materials are made from locally available raw materials, through a manufacturing process (e.g., concrete). Imported goods are simply native and/or manufactured materials from foreign countries. Unfortunately, Haiti has limited access to all three classes. In terms of native materials, the poor agricultural practices mentioned above, coupled with the use of wood charcoal as a primary fuel source, has led to over 90% deforestation in Haiti. Construction grade wood is difficult to find in meaningful quantities, and what is available is priced accordingly. Other potential native materials such as bamboo have yet to be available in scalable quantities. Turning to locally manufactured materials, the traditional examples are steel and concrete. There are no significant deposits of iron ore in Haiti, so local steel production is not viable. The ingredients for concrete (stone, sand, water, and cement) are widely available, although the cement is imported from the Dominican Republic. Other manufactured materials, such as plastics or synthetics, are currently unavailable, mostly due to a lack of collection and manufacturing capacity. It is possible that future investigations into recycling the massive amounts of plastic that litter the streets in Haiti might prove to be viable, but this has yet to be done. Finally, there are imported materials. This approach has historically been extremely cost prohibitive in Haiti due to high raw material import taxes and, perhaps even more expensive, the multiple layers of payoffs that must be made to get materials out of port. Corruption is a reality in Haiti and ports are an easy
opportunity for officials to demand extra money. The only materials that are imported relatively easily and at reasonable prices, presumably due to the long reliance upon them in the construction industry, are cement and steel rebar from the Dominican Republic. However, these are far and away the most expensive elements of any reinforced concrete system.

From the above inventory, it is clear that the dominant materials used are concrete and CMUs. Although simple, these materials are especially advantageous financially for Haitians because they offer the opportunity for progressive purchasing. As previously discussed, progressive construction is the core building model in Haiti and materials that facilitate that process are embraced and do well in the market. In particular, CMUs are able to skirt the problems of limited access to credit. They are relatively cheap due to their small nature, cheap ingredients (sand, water, and cement), and simple manufacturing process (can be hand pressed in the street). Customers can purchase them in quantities as low as a single block and assemble them one at a time, independent of the next block. Consequently, nearly all residential permanent housing in Haiti is reliant on a combination of reinforced concrete and CMUs.

3.2.3.2 Knowledge Base

In addition to the issues presented by limited financing options, there is also no system established to ensure basic safety and quality standards. As shown in Figure 3.10, in the developed world there is a dependence on a system of governmental public policy (i.e., building codes), education pipelines, engineering expertise, and economic
means to ensure that all vulnerabilities observed post-disaster are addressed and remedied. In Haiti, and many other developing countries, these basic underpinnings do not exist due to weak governments, education systems, engineering knowledge bases, and economies. Without these support structures, there is no way to diffuse good practices throughout the country in order to reduce vulnerability to hazards. Bad practices perpetuate, even after large-scale disasters, because no mechanism exists to formally identify and address vulnerabilities, and then diffuse, implement, and ensure best practices.

Figure 3.10: Developed world pipeline for construction standards development.

There is currently a knowledge gap in Haiti in many subjects pertinent to the housing sector. The subjects range from best practices in construction at the builder level, all the way to zoning and building standards at the national government level. The pipeline that exists in the developed world that transforms observed damages and failures into better design and construction practices are absent in the developing world. The post-secondary education system is not robust enough to identify and/or
investigate system wide deficiencies, as is done by universities in the developed world. Building codes are also rare, so no common standards of design or construction are applied throughout the housing sector. The engineering capacity of the country is also limited, mainly due to the restricted capability of the education system. Finally, the economy exercises influence over all of these systems, making investment in improving the pipeline extremely difficult.

3.2.3.3 Skill Sets

Acknowledging the larger system-wide flaws, this thesis will focus on capacity at a more local level. At this level, construction crew organization plays an important role in the housing sector. There are varying levels of skills and knowledge available, depending on the type of crew that is hired. There are three types of construction crews in Haiti. The first is the skilled foreman model, where one person has formal training from a university and instructs the rest of the team. This model works well, however because of the value placed on such expertise, these construction teams are often too costly for most residential construction and are mostly employed in commercial projects. The second model is an apprentice model, where the trade is passed down through generations of a family. This model is more evident in high-income residential construction. It is customary for crews in this category to be employed for short-term trial periods, and receive longer employment contracts when they have proven their skills. Finally, the last model employs a master builder, one person in a community that helps everyone to build their homes. This was the most prominent for low-income
housing. The master builder is rarely formally trained, but has some idea of construction techniques either through personal experience or experience passed down by family and friends. In residential construction, the latter two models are the most prevalent. While all three have accumulated considerable formal and informal expertise in concrete and masonry construction, they still must operate within the constraints created by limited financial resources. Even when adequate skills and knowledge are present, they are often underutilized because the financial resources are not available to use them to their full potential.

3.2.4 Cultural Constraints

Haiti’s history has long been defined by struggle and hardship. First settled by the Spaniards and later controlled by the French, Haiti’s past is wrought with political turmoil and revolt. Under French rule in the 1700s, the western part of the island of Hispaniola, called Saint Domingue at the time, became the most prosperous colony in the Western Hemisphere; exporting huge quantities of sugar products and coffee (Hallward 2007; CIA 2010). However, the success of the colony’s economy laid squarely on the shoulders of countless African slaves brought to the island during the height of the transatlantic slave trade. Dire living and working conditions plagued the slaves of the island; meanwhile plantation owners and French delegates grew increasingly wealthy from their labor. In the year 1791, the slaves reached a breaking point and started an organized rebellion under the leadership of Toussaint L’Ouverture that lasted for the next decade. By 1804, the now former slaves had successfully defeated the
French, plantation owners, and all other opposition and declared their independence as the nation of Haiti, becoming the first independent black republic (Hallward 2007; CIA 2010). As can be imagined, the success of the revolution was feared by Europe and the United States, as both economies and livelihoods still heavily relied on slavery at the time. This led to an economic isolation of Haiti from the rest of the world, accompanied by a F150 million debt imposed by France as compensation for the loss of men and the colony during the revolution. Arguably, this can explain how the island has struggled economically and politically since. Now infamously known as the poorest country in the Western Hemisphere, government of the country has been unstable. From occupation by the United States in the early 20th century, through the Duvalier reign, and especially during the last 20 years, during which two successful coup d’états against democratically elected president Jean-Bertrand Aristide occurred, governance of the island has a disrupted history. This instability, exacerbated by the presence and actions of countless international state and non-state actors, has created a nation with little institutional capacity for providing a basic, quality standard of living for its citizens.

3.2.4.1 Current State

Before the earthquake in January 2010, 80% of the population of Haiti lived in poverty and 54% were living in abject poverty (CIA 2010). The average annual income of a Haitian was around $600 before the earthquake, and is estimated to be $300-400 now. Unemployment rates are at a minimum of 40%, and the bulk of the labor force works in the informal sector. The provision of safe drinking water, sanitary sewage
systems, and electricity is rare at best. Through unsustainable farming practices throughout the country’s history and heavy use of timber for charcoal fuel, the island has also slowly become 95% deforested, decreasing the availability of timber for building materials and formwork and causing severe soil erosion problems throughout the country’s landscape. Health systems also struggle with providing affordable, accessible, and quality care to the Haitian people. Cost and location prohibit many people from seeking out treatment for even the simplest of health conditions. Education faces many of the same access and cost barriers, subsequently leading to a literacy rate of only 53% (CIA 2010). The January 12, 2010 earthquake only worsened the array of issues that already plagued Haiti, while simultaneously creating additional post-disaster complications.

3.2.4.2 Cultural Housing Preferences

Through the survey described previously in reference to the economic reality of Haitian households, other questions were posed to gain an understanding of the expectations for a home in Haiti. This survey (Appendix B), along with hours of tape-recorded anecdotal conversations, led to a series of important observations concerning what is expected of housing.

3.2.4.2.1 Single Family and Single Story Dwellings

It was clear that there is a desire for single-family homes in Léogâne. Over 53% of those surveyed responded with a preference toward single-family housing.
Historically, families live under the same roof, but for concern of security, it has not been the norm for multiple families to live in the same structure. Further, not only was a partiality shown towards single-family homes, but a desire for single story structures was also expressed. Over 58% of respondents preferred a single-story residence. This attitude was born almost exclusively out of the collapses of multi-story concrete buildings witnessed by the community during the 2010 earthquake. Multi-story structures in Haiti had considerable inherent vulnerability to earthquakes due to local construction practices and poor materials, and therefore many did not survive the earthquake. Exposure to this type of destruction has given the population a presupposition that multi-story buildings are fundamentally more dangerous than single story structures. Contrary to the fact that it is possible to design and construct multi-story buildings in a safe and hazard-resistant manner, the images and experiences of death and destruction from the earthquake have garnered an aversion to these types of structures, especially in the context of the residential sector.

3.2.4.2.2 Security and Privacy

As mentioned previously, security of homes is a prominent issue concerning Haitians. The fear and threat of trespassing and theft is constant in Haiti, especially in urban areas like Léogâne. As such, there is a disposition towards materials and structures that exude impressions of strength and security, which is one of the many reasons concrete and block homes are so prevalent in urban areas. It is not uncommon to see block walls built along the perimeter of land plots, usually topped with barbed
wire, sharp rocks, or glass, acting as a deterrent to potential intruders. Culturally, acceptance of building systems that utilize lightweight partitioning is difficult because there is an assumption that they are inherently more susceptible to intrusion. However, as shown by the January earthquake, there are also serious disadvantages to using heavy materials for partitioning, namely an increased vulnerability to earthquake forces and subsequent damage.

Along the same lines as security, Haitian families also place high priority on privacy within a home. Overwhelmingly, the sentiment continually expressed was a desire for homes with a layout that give subgroups within the family unit their own privacy. This was especially the case when discussing sleeping quarters. Most Haitian families asked that there be, at the minimum, separate sleeping quarters for children and parents, which translated into an average of over three rooms per home. This raises an issue, however, as it leads to highly partitioned homes. The desire for highly partitioned homes itself is not problematic, but when heavy materials are used to create the partitions, resiliency to earthquake damage can be greatly reduced. Once again, evidence of this can be seen in the destruction from the January 12th earthquake. Heavy materials were used for partitioning and steps were not taken to reinforce them properly to offset the vulnerabilities to earthquakes that their mass inherently creates.

In order to accommodate the desire for privacy within a household, alternative lighter partitioning systems must be explored, as there are rarely adequate funds available to a Haitian family to properly reinforce the heavy block system that is the current norm in Haiti.
3.2.4.2.3 Zoning and Ownership

During discussions with community members, it became apparent that people were not only concerned about the physical structure of a home, but also about how that home fit within the context of the rest of the community. Currently, urban sprawl is a growing problem in Léogâne. The city of Léogâne lies within a larger commune (similar to a state or province), also called Léogâne, which depends heavily on the agricultural sector for its livelihood. The production of sugarcane for raw sugar and rum is a large portion of the local economy. However, due to lack of government oversight and zoning laws, houses are haphazardly built wherever there is open land. This practice has led to over half of the land historically used for sugarcane production to be lost (Volcin 2011).

Without zoning laws and a system for determining ownership, this practice will only continue to, and could eventually dismantle the economic driver of the area. Zoning and ownership issues are also taking their toll within the city, as unstructured settlements are extremely common. Houses are built on areas of land that are not clearly claimed, with no regard for ownership or safety.

There was a large and clear call after the earthquake for the coupling of a housing paradigm with provisions for zoning laws and land ownership documentation. These issues are local governance issues and must be addressed through that system, however, any new housing paradigm must be cognizant of the issues that arise from public safety, urban sprawl, and zoning so as to create a system that fits within the framework built to address these additional community needs. For example, it is obvious that the aforementioned preference for single family and single story housing
directly conflicts with addressing the issue of urban sprawl within an expanding city, which in turn has an effect on local agriculture and the local economy. If the local economy suffers from lack of production, people within the community suffer financially and the price point of a home may no longer be appropriate to accommodate low-income residents. The interconnectedness of the problem is undeniable, hence the careful attention that must be taken in discovering the issues influencing housing from all angles.

3.2.4.2.4 Materials, Quality Control, and Building Codes

The nation of Haiti is heavily constrained in terms of natural resources available for use as construction materials. Timber products are all but extinct because of the overuse of timber for fuel and unsustainable farming practices that have damaged much of the nation’s soil. The island is not a source of iron ore, making steel only available through importation. Consequently, block and concrete have become the only sensible option for building in urban areas. They have proven effective in mitigating the effects of hurricane winds and also provide the security so adamantly demanded in Haiti. Yet, the quality of these materials leaves a lot to be desired in most instances and, when coupled with poor construction and engineering, the resulting structures are death traps. Exploration of new materials is almost a must for housing in Haiti. New materials that can be locally produced will have an impact on the nation that extends far beyond adequate housing. The ability to seed new industry, think carefully about sustainability, and use materials that mitigate against natural hazards in the most efficient and
effective way possible is an opportunity that must be seized. However, as conventional wisdom dictates, change is not easy. Public perception of lightweight materials is negative due to security concerns. Classic wattle and daub homes (1.4.1 Wattle and Daub) are often scoffed at, perceived as reserved for the poorest that can afford nothing better. Concrete and block construction is an institution, a status symbol, and in many cases, the goal. Ironically, for as prevalent as the technology is, the quality is extremely low. Construction practices are shoddy even outside of the context of natural disasters, and material standards and building codes do not exist. Understanding that public perception has to change on the materials front is essential, along with the realization that any new construction paradigm must be accompanied by a set of standards, acceptable practices, and most importantly, a vehicle by which to enforce them.
CHAPTER 4:
DEFINING AND ASSESSING THE SOLUTION SPACE

4.1 An Empowerment Model for Reconstruction

The previous chapter explained in depth, the various dimensions of the housing sector in Haiti and how each has contributed to the current model and its vulnerabilities. This understanding was based upon a four-dimensional breakdown of the housing sector: hazards and vulnerabilities, economic constraints, capacity constraints, and cultural constraints. In order to produce a new housing model, each of these constraints must be addressed in a holistic manner. The ultimate goal of such an approach is a model that can be sustained without reliance on foreign actors – which the author and his co-advisors have termed the Empowerment Model. By addressing all four dimensions in an integrated way, a housing system is created that not only meets the structural demands, but is also affordable, practically implementable, and culturally accepted. Building upon the housing sector model described in Figure 3.3, the Empowerment Model has four tenets, each tied to a category of constraints (Figure 4.1):

- **Resiliency**: can insure life-safety and protection against natural hazards and other environmental factors; requires an understanding of hazards and vulnerabilities

- **Sustainability**: can reconcile a household’s financial capacity with the market reality of labor, resources, and engineering expertise to create a
local model that can be supported indefinitely; requires an understanding of market constraints

- **Feasibility**: can be practically implemented using locally available technologies, capabilities, and materials; requires an understanding of capacity constraints

- **Viability**: can earn the support of most local stakeholders as culturally appropriate, so that the model is not just accepted, but embraced and promoted; requires an understanding of cultural context

Figure 4.1: Empowerment Model for post-earthquake housing reconstruction.

Although it is clear that the four tenets of the Empowerment Model are important to identify, they are only useful if they act as a tool to evaluate proposed solutions. Viewing the current models and proposed solutions through the lens of the Empowerment Model, it is evident that the inability of many proposals to carefully consider and/or satisfy one or more of the tenets explains why they have failed to
provide lasting solutions to the Haitian housing dilemma. Consider, for example, the solutions presented at the Building Back Better Communities Expo, sponsored by the Clinton Foundation. Many proposals presented failed to meet basic engineering requirements, retailed for $20,000-$30,000, and only an estimated 10 percent relied exclusively on local materials (MacDonald 2011). Had basic expectations for designs been communicated between vendors and Haitians in advance, and then scored with an appropriate assessment tool, not only would the submitters have been motivated to incorporate features that target long-term empowerment of the Haitian people, but there would have also been an objective framework to score and rank proposed models. This realization motivated the use of the Empowerment Model, combined with appropriate mechanisms for community input, to develop a process to score and assess how different reconstruction models meet the multidimensional needs explored here thus far. The Empowerment Model then serves as a framework for approaching urban housing problems, and also as the basis of an assessment tool to identify the strengths and weaknesses of proposed housing solutions.

4.2 Proposed Housing Solution Assessment Tool

Systems that have evolved and taken root organically in resource-constrained settings inherently satisfy the tenets of an Empowerment Model because they are born from the reality experienced by the community. Unfortunately, these are not informed by engineering knowledge and can prove vulnerable to infrequent extreme events, as was the case in the 2010 Haiti Earthquake. In order to objectively evaluate existing
housing systems and potential alternate systems, an assessment rubric was developed that measures compliance with the tenets of the Empowerment Model, by scoring specific attributes of homes.

This tool was created in a larger context of reconstruction in Léogâne, specifically during a community planning workshop held in March 2011, for which this research group was asked to represent the urban housing sector. The workshop was structured around nine different working groups, representing the following sectors:

- Roads, Water, Storm Water & Sewer
- Electricity & Communications
- Education
- Housing
- Planning & Zoning, Downtown Development, City Wide Development
- Economic Development
- Public Services (Trash, Police, Fire)
- Health and Medical Care
- Human and Cultural Capital

The objective of the workshop was for each working group to take an inventory of what already exists in Léogâne, what is needed, and extract a vision for the solutions to those needs from the community itself. Specifically, from the housing standpoint, the goal was to bring all interested parties together, including Léogâne residents from various demographics, housing non-governmental organizations (NGOs), international
organizations, and government representatives to discuss each party’s current activities, future plans, and to start cohesively confronting the urban housing issue.

While the rubric discussed here was developed during the community planning workshop specifically to assess urban permanent housing options in Haiti, it can be readily adapted to other applications in the developing world. The attributes selected for the rubric were informed by the reconnaissance fieldwork in Léogâne covered in Chapter 2, the IDP surveys, as well as feedback received from Léogâne citizens participating in the workshop. The rubric is shown in Table 4.1, and as can be seen, each of the tenets of the Empowerment Model materializes as two or more attributes. Most attributes can be rated with one of four choices scored on a scale from 0 to 100, where a score of 100 is the most favorable. In two instances, an attribute has only binary scoring options that are punitive in nature: a score of up to -100 points can be awarded for an undesirable behavior. Each attribute is then assigned a weighting as a simple way of reflecting items of high priority for the community being served – in this case the weights were derived from a brainstorming exercise with Haitians engaged in the community planning workshop. Participants indicated that security, seismic performance, and the use of local construction materials and processes were among the most important considerations. The weighted scores for each attribute are then summed to yield the composite score for a particular housing option.

Obviously the scores assigned to each choice and the weights assigned to each attribute are subjective and effective scoring does require some expertise. However, what is important is the development of a rational and consistent framework for
evaluating reconstruction options across a spectrum of attributes, specific to Haiti. As
such, it encompasses attributes such as cost and quality control, issues that are very
important to the Haitians currently in IDP camps, but also addresses issues like cultural
acceptability and construction time, attributes that may be important to more affluent
Haitians. Many solutions may be offered for permanent housing in Haiti, and based on
how they score in the above rubric, may be appropriate for certain demographics and
not others. Therefore, variations of this rubric could be developed with the scoring and
weighting adjusted to explicitly filter non-compliant designs outright. For example, for
the bottom of the pyramid demographic, material cost might be more heavily weighted,
with scores assigned to these choices that levy heavy penalties for exceeding a certain
price point. In fact, the current lack of solutions that satisfy all four tenets for this
particular demographic ultimately motivates the paradigm shift that will be discussed in
the next section of this thesis. Such use of the rubric to target the solutions best serving
a given demographic is encouraged, as it returns to the holistic approach of the
Empowerment Model. In this way, it becomes not only an assessment tool but also a
mechanism to support planning processes and design competitions to incentivize
certain desirable attributes and disincentivize certain undesirable practices.
### Table 4.1:

RUBRIC USED TO ASSESS HOUSING SYSTEM COMPLIANCE WITH EMPOWERMENT MODEL

<table>
<thead>
<tr>
<th>Tenant</th>
<th>Attribute</th>
<th>Choices and Point Values</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hurricane Resilience</td>
<td>Collapse Failure: Major Element 10, Failure: Minor Element 40, No Damage 70</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Seismic Resilience</td>
<td>Collapse Failure: Major Element 10, Failure: Minor Element 40, No Damage 70</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Durability</td>
<td>1 year 10, 1-5 yrs 40, 5-25 yrs 70, 25+ yrs 100</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Required Construction Skills</td>
<td>Individual 100, Master Builder 80, Trained Crew 60, Trained Crew &amp; Equipment 25</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Construction Process</td>
<td>Modular by Hand 100, Modular with Fasteners 70, Staged Construction 40, All at once 10</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Potential for Codification</td>
<td>Unlikely -50, Likely 0</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Need for Quality Control</td>
<td>Not Required 0, Necessary -100</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Use of Local Materials</td>
<td>Majority Imported 25, Potential Local Production 50, Likely Local Production 75, Majority Domestic 100</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Material Cost</td>
<td>Raw Hand Processed 100, Shop Processed 66, Mill Processed 33, 0</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Construction Time</td>
<td>1 Week 100, 1 Month 70, 3 Months 40, 6 Months 10</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Viability</td>
<td>Cultural Acceptance Exists Currently 100, Existed Historically 70, Possible with Programming 40, Unlikely 10</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Security</td>
<td>Manual Intrusion 10, Intrusion with Hand Tools 40, Intrusion with Power Tools 70, Intrusion Unlikely 100</td>
<td>1.75</td>
</tr>
</tbody>
</table>
4.3 Application of Assessment Tool to Current Haitian Housing Models

After the rubric was developed, the housing working group identified the typologies best suited to Haitian reconstruction and the spectrum of materials that could be used for these primary load-resisting systems, as well as their respective cladding and partitioning. As covered in Chapter 1, the primary load resisting systems include load bearing walls, confined masonry, moment-resisting frames, and hybrid systems. It should be noted that this exercise did not seek to evaluate the feasibility of any one system or material but rather to propose an exhaustive suite of options. While roof and flooring systems were also evaluated as part of this exercise [see complete report (Rebuild Léogâne 2011)], this discussion will focus on the primary load resisting systems only, whose results are presented in Table 4.1. Materials historically available in Haiti are indicated in bold, including CMU – the nation’s most popular and versatile modern building element. In fact, according to a government survey prior to the earthquake, these elements were used in walls of 76% and 97% of single-story and multi-story construction projects, respectively (IHSI 2003; IHSI 2010).
### TABLE 4.2:

SPECTRUM OF MATERIALS FOR RESIDENTIAL SYSTEMS IN HAITI

<table>
<thead>
<tr>
<th>Primary Load Resisting System</th>
<th>Primary Material Choices</th>
<th>Partitioning Choices</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Bearing Walls</td>
<td>• CMU</td>
<td>Same as primary material</td>
</tr>
<tr>
<td></td>
<td>• Earth</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Stone</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Wood</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Brick</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Manufactured Structural Panels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Recyclables</td>
<td></td>
</tr>
<tr>
<td>Confined Masonry</td>
<td>• CMU</td>
<td>Same as primary material</td>
</tr>
<tr>
<td></td>
<td>• Brick</td>
<td></td>
</tr>
<tr>
<td>Moment Resisting Frame</td>
<td>• Wood</td>
<td>• Woven Thatch</td>
</tr>
<tr>
<td></td>
<td>• Concrete</td>
<td>• Lathe</td>
</tr>
<tr>
<td></td>
<td>• Light Gauge Steel</td>
<td>• CMU</td>
</tr>
<tr>
<td></td>
<td>• Bamboo</td>
<td>• Brick</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Manufactured Non-Structural Panels</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Recyclables</td>
</tr>
</tbody>
</table>

Note: **Bold** indicates material either presently or historically was available in Léogâne and surrounding areas prior to the earthquake.

The housing working group at the community planning workshop applied the rubric to evaluate more than a dozen primary load resisting systems (Rebuild Léogâne 2011), a sampling of which are presented in Figure 4.2, with the exhaustive results provided in Appendix C. The rubric then provides a means to quantitatively assess and compare potential solutions and identify weaknesses and strengths of proposed systems and the tenets they best satisfy. This process was conducted for each system evaluated, and recommendations were made to help further enhance its compliance with the Empowerment Model (see Appendix C). Given that CMU is the only widely
available partitioning and cladding material in Léogâne, the scored attributes of “local availability” and “construction process” will drive the Empowerment Model toward such “lowest hanging fruits”; in this case, reinforced load bearing CMU walls and confined masonry systems. Unfortunately, the rubric cannot ultimately predict if the solution is truly resilient, sustainable, feasible, and viable for the community being served, particularly if the total price of the system dramatically exceeds the income available to households in that demographic. Rather, it facilitates objective comparison between solutions. In order to achieve this function, scores and weights need to be adjusted with a particular demographic in mind, as mentioned in the previous section.

![Figure 4.2: Composite scores from the assessment of several potential residential housing systems.](image)

It is not unreasonable that the rubric tends to bias toward existing technologies and systems, as it is difficult to score a proposed system that has not yet been vetted on the open market. Thus, despite the poor performance of CMU walls in the earthquake, CMU remains viable due to the lack of other local options that satisfy the cultural and environmental needs of the community. Load bearing CMU wall systems primarily
served low-income populations in Léogâne and did sustain damage in the earthquake.

However, due to their small size, single story modality, and typically light metal roof, they generally did not experience total collapse, but rather the loss of one or more walls in brittle shear failure. The reimplementation of this system should include reinforcement, better quality masonry units, and a ring beam to tie the system together. However, it must be stressed that lack of quality materials and certified construction practices, or the finances to procure them, will lead to houses that are just as vulnerable as the homes that were destroyed during the earthquake.

Unfortunately, this same concern surfaces as one evaluates options available to middle and upper income residents in Léogâne. As Figure 4.2 indicates, the formal introduction of confined masonry construction to Haiti scores well across all four tenets, as it is a direct extension of current practices employing materials already available locally. When executed properly, the system has the flexibility to support multi-story construction, which allows the potential for future expansion in dense urban zones – an attribute many Haitians desire. For organizations like Build Change, “minor, low-, or no-cost improvements to existing ways of building” often proves easier “than to introduce a completely new technology or reintroduce a traditional building method” (Hausler 2010). The rubric similarly supports this hypothesis. Unfortunately, engineering adequate seismic resilience through higher quality CMU and larger quantities of steel can cost up to $20,000 – well beyond the reach of the majority of displaced Haitians who have irregular sources of income (Phillips 2011). As a result, the current reconstruction in Léogâne shows limited examples of the successful implementation of
confined masonry. CMU construction has escalated in Léogâne and, while there is
evidence of increased quantities of transverse reinforcement in columns and vertical
reinforcement through CMU walls, other necessary features such as ring beams,
horizontal wall reinforcement, and effective keying of higher quality blocks are not yet a
consistent practice. In fact, the most consistent mitigation measure that has been taken
is to limit homes to a single story and employ lightweight metal roofs framed by timber.
Although Haitians still prefer the option to add floors to their homes in the future, the
aversion to concrete slabs after the earthquake has increased the use of metal roofs in
post-quake reconstruction.

4.4 Creating a New Structural System, Business, and Quality Control Mechanism

While the previous section assessed immediately implementable options, there
should be care in advocating solely these construction modalities for all Haitian urban
housing, as many do not have the financial resources necessary to make masonry
systems resilient against earthquakes. These are the preferred systems identified by the
Empowerment Model largely because there are no other legitimate market
competitors. For many of the other possible main force resisting systems discussed in
the workshop report (see Appendix C) and in Figure 4.2, the lack of native resources and
industries to manufacture engineered materials in country led to punitive scoring in the
evaluation process. However, many of these systems, including the one presented
subsequently, could be dramatically more sustainable, feasible, and resilient if the
materials and technologies could be locally produced and retailed on the Haitian free
market. If certain inputs were provided, alternative systems could fare well in the rubric, and could become viable options for the low-income demographic in Léogâne.

The surveys conducted throughout Léogâne, as well as the community planning workshop, provided valuable input, data, and anecdotes relating to the cultural expectations of a home in Haiti. In pursuing a new system, the cultural preferences underlie every aspect of the home and provide a foundation for buy-in. However, as the results of the assessment of current modalities reflect, there is also room for innovation in the other three tenants of the Empowerment Model that would ultimately increase access to safe, quality housing for the low-income demographic. With insights from the reconnaissance work, the IDP surveys, and the community planning workshop, the research group focused on innovating one primary aspect of housing in each of the other three areas. The general innovations are covered in the following section, and the next chapter will elaborate in much more detail on the formal system created from them.

4.4.1 Resiliency Innovation: Alternate Partitioning Systems

The dichotomy created by the wide availability of CMU and its role in creating considerable vulnerabilities in earthquakes makes partitioning one domain ripe for innovation. There is an opportunity for lightweight, partitioning elements that reduce vulnerability, can be produced locally with a high level of quality, and still satisfy cultural expectations for privacy and security. Interestingly, historical Haitian construction of “gingerbread houses”, many of which survived the earthquake despite their age and
condition (Figure 1.5), employed timber framing with plank walls. These lightweight structures were well adapted to the climate of Haiti and were sustainable prior to deforestation. Therefore, it is of interest to explore technologies sustainable in modern day Haiti that would permit load-bearing walls to be replaced with frames clad in non-structural partitioning systems. In fact, by recognizing that most households have limited resources and cannot afford the degree of reinforcement required to aseismically design every wall in a highly partitioned confined masonry home, the adoption of a frame system is preferable because it concentrates the structural resistance and thereby limited financial resources in a few select elements of the system.

While frames could be feasibly made of reinforced concrete using local materials and skill sets – this structural system is already employed in commercial structures in Haiti – an equally feasible panel technology is not currently available. As a result, the research team has proposed production of thin panels from a variety of local, raw material sources, including pre-cast concrete panels using a lightweight mix design reinforced with wire mesh (Kijewski-Correa, et al. 2012). These lightweight panels are isolated from the primary structural system and help to reduce the seismic demand on the home, while still maintaining adequate strength to bear the pressure of hurricane force winds and provide basic security from intruders. The panels can be attached to frames using a simply bolt assembly, the joints can then be sealed, and finally the walls continuously finished with stucco and paint. More importantly, since the panels are non-structural in nature, strict quality control on material properties is not required,
allowing them to be produced without the need for heavy machinery or technical training.

4.4.2 Viability Innovation: Creating a Business Model

For any alternative to succeed, it ultimately must offer households a value proposition that meets their needs and desires, within the confines of their financial capacity. While aid models are essential in the immediate aftermath of a disaster, businesses that offer both social goods and the benefits of economic development are much more likely to prove sustainable. Therefore, for any alternative to succeed in the long-term, it must be implementable through a business that can compete with the existing options on the open market. The economic reality for the bottom of the pyramid demographic makes this challenging; however, if achieved, it is one of the most powerful forces for widespread development.

Operating under a business model also achieves other goals. Firstly, it grants the ability to fold quality control into the production and construction process. Business protocols and oversight can be used to ensure quality, which will ultimately manifests itself as a competitive advantage in the market, incentivizing their adoption. Additionally, worries about raising enough donor funds year over year are no longer an issue, because operations are sustained through the sale of homes. Obviously, the creation of the business model that allows such success is crucial, but if approached in the same manner that has been laid out in this thesis for the design of the structure, it is entirely possible.
The elements that are essential to the success of a housing business in Haiti are (1) progressive construction, (2) alternative financing mechanisms, and (3) safe, functional, and aesthetic design. Catering to the target market with a construction process that allows for the progressive purchase of the home is essential. Additionally, leveraging the financial capabilities that are available, such as savings, mobile technology, and cooperative savings groups, can decrease the time it takes to move a household into their home. Finally, the structure itself must meet the needs of the market. Folding in opinions and desires expressed during reconnaissance, surveys, and workshops can ensure that the final design meets the safety, functional, and aesthetic expectations.

4.4.3 Feasibility Innovation: Engineering Quality Control

Although the elimination of CMU walls will significantly reduce the seismic demands on the structure, the resistance of the system is no longer distributed and is instead concentrated in the discrete elements of the frame. Therefore, a high degree of quality control is necessary to ensure the frame is properly designed and constructed, with columns and beams sized and reinforced appropriately. Deficiencies commonly encountered in reinforced concrete construction in Haiti, as discussed previously, include inadequately sized members, insufficient reinforcement (particularly transverse), and poorly mixed concrete. Achieving quality control in each of these facets is difficult in settings where building codes, certification, and inspection is non-existent. In fact, interviews with Haitians throughout the course of reconnaissance and the
Community planning workshop demonstrated an overwhelming desire for regulations on any type of construction, implying a clear recognition of the need for quality control and oversight, but also the admission that the government is currently ill equipped to provide it. As a result, several process innovations will be necessary under the business structure to support production of the alternative partitioning systems and engineer the quality of the frame.

Without a strong governmental infrastructure in place to ensure a high level of quality, capacity needs to be created organically through the business to fill this gap. Quality control must be brought directly into the production and construction process itself, as opposed to relying on traditional inspection processes common in the developed world. Therefore, the feasibility innovation is based on engineering a system to provide quality control. This system has three basic elements: standardization, prefabrication, and paraskiling.

4.4.3.1 Standardization

The first strategy to mitigate vulnerabilities introduced by poor construction practices is standardization of the housing model. Vulnerabilities are often created when custom designs require skill sets and expertise that exceed the knowledge of the designer and/or construction crew, resulting in a design that is not sufficient to mitigate the threat of damage or collapse. Alternatively, standardization generates a limited number of designs that meet all relevant requirements, whose behaviors are well understood, and are implementable using the skill sets of the local construction crews.
Creating a few designs that, when done correctly, offer a high level of quality and safety is a possible route to decreasing the vulnerabilities of the entire housing stock. Although this eliminates the ability to customize a home for each household, by engaging the local population (through surveys and workshops), it is possible to incorporate a majority of the needs and desires of the community. Standardization of design, fabrication, and construction processes delivers a high level of quality control in an environment where oversight and enforcement is lacking.

4.4.3.2 Prefabrication

In order to successfully implement standardization to achieve an adequate level of quality, as much production as possible needs to occur prior to being on site. Once on site, quality is much harder to control. Equipment is not always available, field “tweaks” are inevitable, and corners can be cut since no oversight is present. Alternatively, in the controlled environment of a production facility, investment can be put into equipment, far tighter tolerances can be achieved, and oversight is much more plausible to ensure. Prefabricating elements, such as rebar cages, can have a dramatic effect on safety, as many of the vulnerabilities exposed were due to steel reinforcement detailing. Providing templates, protocols, and having the ability to oversee the process within a business environment, could drastically increase resilience. Additionally, the demographic being targeted is almost always excluded from formal finance mechanisms, meaning that progressive construction is the norm. By prefabricating standardized components, the
structure can be reduced to a series of smaller elements, that when purchased over time, create a livable home.

4.4.3.3 Parasking

Finally, both standardization and prefabrication require human capacity. In an environment where formal education pipelines are lacking, and those that do have formal training are usually employed in the commercial sector, training a workforce to support residential construction is a challenge. However, building upon the elements of standardization and prefabrication opens up the ability to implement parasking techniques for training a workforce. Parasking is the practice of dividing large tasks into small, highly repeatable subtasks. These subtasks are then taught to a workforce, each of whom is solely responsible for learning one particular subtask. As individuals repeat their respective task over and over, they become highly efficient at it and can ensure a high level of quality. The larger task (or product) is then simply the sum of these subtasks. Since these subtasks already have a high level of quality, the larger task is also of higher quality. In this way, a workforce with minimal formal training can construct complicated engineered structures. When this parasking is placed within the context of prefabrication, quality is only elevated, and within the larger context of standardization, it is also widely replicable.
4.5 Introduction to the Frame and Panel System

Instead of simply documenting failures, vulnerabilities, and constraints, this thesis offers a new urban housing paradigm to fill the current void in the Haitian housing sector. Following the Empowerment Model, as well as the innovations covered in the preceding sections, this research offers a housing model that fundamentally shifts the structural system used for low-income residential construction in Haiti, while leveraging the existing materials and capabilities available locally. The system is standardized and relies heavily on prefabrication and paraskilling to deliver a level of quality not currently available. It has been designed over the course of three years, during which extensive time and effort was spent in Haiti to understand the hazards, vulnerabilities, capabilities, and cultural context of the urban housing problem, as covered thus far in this thesis. Built upon this research, the proposed model has been created to take advantage of the capacity that does exist, in order to deliver a safe, affordable, and appropriate urban housing solution.

Figure 4.3 shows the standard layout of the proposed home. The basic model is a one-story structure with four rooms. Each is 13’ x 13’ (3.96 m x 3.96 m), for a total of 689 ft² (64 m²) of living space. It has been designed to accommodate a family of four to five people. The home will include an open-air porch, as well as the option to add an indoor bathroom facility to the back of the home (note: at this time that the most common sanitation system for residences is separate outdoor structures). As most cooking is also executed outdoors, the open-air porch can be adapted to serve as an outdoor kitchen. Multiple door and window configurations will be available to the
customer based on personal preference. The home will also be available in two, two-room modules that can be constructed next to each other over time and connected through a shared hallway to create the four room basic unit (Figure 4.4). This option is especially attractive for families that lack the upfront capital to build the four-room basic model.

Figure 4.3: Proposed housing system: plan view (left), frame and panel system (right).
4.5.1 Structural System

The core structural system of the proposed model is a reinforced concrete moment resisting frame (RCMRF). The rationale for choosing this system was threefold. Firstly, from an engineering perspective, the moment resisting frame system provides the lateral resistance needed to handle the demands from seismic and to a lesser extent hurricane hazards. Unlike the current system, a RCMRF utilizes beams to tie columns together and engage all the structural elements in resisting gravity and lateral loads. Secondly, reinforced concrete is a material that can be procured and produced locally. Supply chains already exist, so economies of scale are not an issue. Finally, Haitian construction crews already have some exposure to the material and building techniques, making technology adoption less of a challenge.
In contrast to the current system, the moment resisting frame allows for walls to be non-structural elements. The RCMRF is capable of resisting all loads, meaning that the walls simply provide cladding and partitioning. The result is that the walls can be much lighter, reducing the overall mass of the structure. For example, the panels that will be further explored in Chapter 5 are made from lightweight reinforced concrete. At a weight approximately 17 pounds per square foot of wall, a wall system from the panels is nearly four times lighter than the CMU option, which registers approximately 63 psf (Brandow, et al. 2009). They can be isolated from the structural system, guaranteeing that failure of the walls will not affect the rest of the structure. The opportunity is available, therefore, to replace the CMUs that are responsible for many of the current vulnerabilities. This is a challenge however, because the materials usually used for lightweight, non-structural partitioning, such as wood, drywall or even sandwich composites, are not available locally. An alternative element needed to be created from locally available materials that possessed the characteristics of being lightweight and non-structural, but still addressed the security concerns of the Haitian people. The solution created is a lightweight concrete panel, reinforced with steel mesh. In a sense, it is simply a reconfiguration of the CMU. The material is essentially the same, with the exception of the steel mesh. However, because the walls do not need to support any loads, the panels can be much thinner and lighter elements. As was the case with the RCMRF, the materials are locally available and the manufacturing process is not all that different than block. The result is a wall system that is lighter and non-structural, but still falls within local cultural preferences and production capacity.
4.5.2 Addressing Constraints

For the proposed housing paradigm to be viable, it must exist within the current housing sector constraints. The major ones discussed here were economic conditions, building material availability, and knowledge base. The proposed model addresses each of these through material choice and construction practices. Realistically, economic conditions in Haiti are not going to improve in the near future. Therefore, any new housing paradigm must be price competitive with the current model in order to spur adoption. By utilizing locally available materials and labor, a home can be delivered at a competitive price point to what is currently available. Additionally, the proposed model allows for progressive construction in order to accommodate irregular incomes. The home is built in four stages: (1) the foundation, (2) the frame, (3) roof, and (4) walls and finishes, parts of which are shown in Figure 4.5.

![Figure 4.5: Construction stages of proposed home.](image)

The staged process is congruent with the ability of an average Haitian customer to pay. Related to the fact that the walls are non-structural elements, the home can also serve as temporary shelter after the second stage by cladding the frame in tarpaulin. In the current system, the CMU walls must be completely finished before the home becomes inhabitable. Again, in settings where access to up-front capital is non-existent,
this can take 5 to 10 years. By making the skeleton of the building a frame that is erected before the walls, the tangential issue of finding temporary living arrangements is solved, as the family saves to continue the construction process by adding panels over time.

The proposed model is also compatible with the limited access to building materials. The entire home relies on materials that are already available and used in Haiti. By simply rethinking the use and configurations of existing building materials, the proposed model is capable of delivering the safety factor that the current model lacks. Logistically, using existing materials is also advantageous because import costs are eliminated and established supply chains can be leveraged. Scalability is not an issue because there is no lag time in procuring materials (as might be the case if a material such as bamboo was used). Again, the proposed model is purely transforming the shape and configuration of the already established system to deliver a product that better meets the demands of the surrounding environment. This is a powerful approach because many of the issues with introducing a new technology are avoided, such as manufacturing capabilities, scalability, and material procurement.

Finally, the proposed model specifically and purposely leverages the current expertise in the construction industry to confront the knowledge gap. Instead of attempting to drastically shift skills and expertise needs, incremental adjustments are made. This approach accelerates the training process by putting new practices in the context of familiar ones. The new paradigm is much less intimidating, not to mention much less expensive to implement. Training costs are minimal compared to a brand new
technology, and it is much easier to “train the trainers,” allowing for a network effect in knowledge uptake. It also allows the proposed model to utilize current labor supply and networks, again aiding in achieving scalability. Moreover, the standardized design and emphasis on prefabrication are vital components in improving quality of construction. Since building codes and enforcement are almost non-existent, and will remain that way for the foreseeable future, innovative quality control mechanisms must be used to ensure quality and safety. Prefabricating standardized components and creating a robust, repeatable construction process are the keys to increasing quality in an environment where classic quality controls are absent.

4.5.3 Addressing Vulnerabilities

In the same way that the proposed model must operate within the constraints of the Haitian environment, it must also improve upon the vulnerabilities of the current model. To be a product with a real competitive advantage, it must address the current deficiencies while maintaining the existing positive characteristics. The strengths of the system in place are its ability to mitigate hurricanes and environmental factors such as heat and moisture. Its deficiency is its inability to meet the demands imposed by moderate to severe seismic events. The ability to only resist one of the two major hazards is a common characteristic of structural systems. The mitigation techniques commonly used for each hazard directly compete with those of the other, making it difficult to efficiently handle both. However, Haiti is in the rare predicament of having to deal with the threat of two major natural hazards. Finding a system that performs
adequately in hurricanes and earthquakes is a challenge even in developed countries where resources are abundant, much less in the context of a developing country. The proposed model has been able to strike a balance between the competing interests of hurricane and earthquake mitigation, while making no sacrifices on environmental factors or urban security.

4.5.3.1 Mitigation of Vulnerabilities to Climatological Hazards

The positive attributes of the current Haitian housing model are its ability to withstand high winds and its resistance to heat, moisture, and sunlight. These benefits are derived from material choice. Heavy CMU and concrete construction mitigates the effects of high winds and is relatively unaffected by standing water, moisture, and sunlight. These materials also have good insulation properties, keeping homes cool in extreme temperatures. In order to maintain these advantages, the proposed model utilizes nearly identical materials. The concrete panels can withstand the wind pressures generated by annual tropical storms and hurricanes and preserve the water resistance of the home, as well as prevent issues with mold and mildew. Degradation of the walls and frame by the sun is not a concern because concrete is relatively unaffected by direct sunlight. Since the materials are identical to the CMU system, the insulation properties are also similar, keeping the interior of the home comfortable and livable. Thus, the proposed system is able to conserve every advantage of the CMU model.
4.5.3.2 Mitigation of Vulnerabilities to Geophysical Hazards

The most glaring deficiency of the Haitian CMU model is its failure to adequately withstand seismic events. The proposed model focuses the little resources available to the average Haitian and utilizes them in a way that significantly improves seismic performance. The overarching theory behind the system is to concentrate the resources in the frame to make it seismically resistant, and then make the wall elements non-structural. The combination of a frame and panel system addresses the four deficiencies of the current system.

Panels vs. CMUs: The underlying problem with CMU walls is that they are the core structural element in the home but incapable of handling the demands that accompany that role. Transforming the walls into non-structural elements reduces demands on them providing greater flexibility in the materials can be used and how they are configured. More importantly, isolating the walls from the rest of the structural system implies that if the walls do suffer damage or fail, there is no mechanism for force transfer to the primary structural system. In the CMU system, the failure of the block, while dangerous and causing injury, was not the critical liability; the transfer of forces to the columns upon failure was. The panels may crack or fail, but the damage will be sequestered from the primary structure. The wall elements as a whole also become nearly four times lighter with the use of the panels, reducing the threat of harm to inhabitants if failure does occur.

Moment Resisting Frame vs. Load Bearing Walls: Although the CMU system has columns, the walls carry the majority of gravity and lateral loads, essentially constituting
a load bearing wall system. The introduction of the RCMRF changes the paradigm, concentrating the structural resistance in a system of beams and columns rather than in the walls. Non-structural panels allow for the limited resources once dedicated to wall construction to transfer into the frame. Columns are over twice as large, in terms of cross-sectional area, as those in the CMU system and contain twice the amount of steel reinforcement. The addition of beams creates a mechanism to engage all the columns together in resisting lateral forces (recall that most unreinforced masonry homes in Haiti did not employ beams at all). This robust lateral system can survive regardless of wall failures and has a level of redundancy that can even withstand isolated damage to the RCMRF. The redundancy ensures that there are alternate load paths should individual elements fail. Although the system is by no means immune to damage or collapse, it is designed in such a way that multiple levels of damage must occur for the entire system to fail, which provides valuable time for the safe evacuation of inhabitants.

*Panels & Roof System vs. Excessive Mass:* As explained previously, using panels instead of CMUs drastically reduces the overall mass of the structure. Strictly specifying a sheet metal roof as opposed to a concrete slab roof also limits mass. With the specifications given, a home constructed from a frame, a panel wall system, and a roof with steel HSS framing and metal sheeting approximately 60% the total weight as the same structure made from CMUs and a slab roof. Table 4.3 details the assumptions used to arrive at this conclusion, based on the dimensions given previously in this chapter. Reducing the weight of the wall system and the roof significantly reduce the forces the structure will be subjected to during an earthquake. Therefore, not only does the
proposed home have a more robust structural system, but it also experiences smaller forces. This is the best-case scenario when attempting to improve the performance of a structure: increased capacity coupled with decreased demand.

**4.5.3.3 Mitigation of Vulnerabilities to Man-Made Hazards**

Urban security threats are mostly predicated on the appearance of the home. The projection of security is just as important as any actual security measure. The choice to primarily use concrete was vital to ensuring that the home portrayed a sense of security and discouraged intruders. From the exterior, the finish of the panels will be indistinguishable from CMU, which most Haitians strongly associate with safety, as well as the status symbol of a “proper home.” The physical composition of the panels also makes it extremely difficult to break in without considerable time and tools. Both of these characteristics make the home well equipped to exist in the urban environment of Haiti.

**TABLE 4.3: ASSUMPTIONS FOR WEIGHT COMPARISON BETWEEN STRUCTURAL SYSTEMS**

<table>
<thead>
<tr>
<th>Lateral System: CMU</th>
<th>Lateral System: Reinforced Concrete Frame + Panel</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Roof System:</strong> Reinforced Concrete Slab</td>
<td><strong>Roof System:</strong> Steel Frame + Metal Sheeting</td>
</tr>
<tr>
<td><strong>Element</strong></td>
<td><strong>Element</strong></td>
</tr>
<tr>
<td>CMU</td>
<td>Frame</td>
</tr>
<tr>
<td>16”x8”x6”, solid mortared cores, 63 psf</td>
<td>150 pcf, sized as shown in Ch. 5</td>
</tr>
<tr>
<td>Slab</td>
<td>Panel</td>
</tr>
<tr>
<td>4” thickness, 150 pcf</td>
<td>134 psf, sized as shown in Ch. 5</td>
</tr>
<tr>
<td></td>
<td>Roof</td>
</tr>
<tr>
<td></td>
<td>10 psf, 684 sq. ft.</td>
</tr>
<tr>
<td><strong>Total Weight</strong></td>
<td><strong>Total Weight</strong></td>
</tr>
<tr>
<td>101.3 kips</td>
<td>58.4 kips</td>
</tr>
</tbody>
</table>
The next two chapters will detail the structural design and construction of the frame system and the panels, respectively. This will include a summary of the loading conditions considered, the reinforced concrete frame design, the panel design, and the construction process. Additionally, an alternative frame joint detail will be explored utilizing headed steel rebar.
CHAPTER 5:
STRUCTURAL DESIGN AND CONSTRUCTION OF PROPOSED FRAME

5.1 Proposed Frame

The layout for the proposed frame, based on information gathered from the Léogâne community, called for a one-story, four-room structure with approximately 600 square feet of living space. Therefore the proposed structure, rendered in Figure 5.1 with plans in Figure 5.3, is a one-story structure with four equally sized rooms and is approximately 676 square feet in plan. The materials include reinforced concrete for the frame, lightweight reinforced concrete for the panels, and a corrugated steel roof (Figure 5.1). To minimize the up-front cost of the home, the foundation, much like the superstructure, will be a staged process. In Stage 1, as the frame is cast, so will a cellular foundation. This cellular foundation, shown in Figure 5.2, will be a stone and mortar base, topped by grade-level reinforced concrete beams that serve as lateral ties between columns, as well as a lower connection mechanism for the panels. This system of grade-level beams ensures that the structural system is fully engaged laterally, especially under earthquake loading, preventing the columns from displacing differentially and causing excessive damage to the panels. As opposed to a system consisting of a monolithic slab tied to footings, which can also achieve the same
mechanism, this system removes the cost of concrete within the cells, saving considerable money (Figure 5.2). As more resources become available after the rest of the structure has been constructed, the soil infill that created a temporary floor system can be excavated and concrete can be cast into each of the four cells to provide a finished floor for purely functional purposes.

Figure 5.1: Proposed frame and panel structural system. Labels A and B refer to the different wall types of the home, with respect to the gable roof. Note the foundation beams, serving to tie the columns together laterally, as well as a connection mechanism for the panels.
Figure 5.2: Example of shallow foundation used in residential construction. (Top) Photo courtesy of the organization Dwell Earth, showing their foundation consisting of a rock/mortar bed, topped by a concrete ring beam. (Bottom) Schematic showing approximate dimensions of a shallow foundation adapted for the proposed housing model.
5.2 Finite Element Model

The proposed frame system was modeled in the finite element software package, SAP2000. This finite element model (FEM) was used to determine the lateral stiffness and modal properties, and to compute member forces when subjected to the loadings prescribed in the following sections. The dimensional layout of the proposed four-room, single-story frame is shown in Figure 5.3. The finite element model, as shown in Figure 5.4, was assumed to be rigidly fixed to the foundation and thus modeled as fully restrained joints at the base. Full rigidity was also assumed for all beam-column joints. Based on an approximate analysis of the frame under the loads introduced in the following sections, initial members were selected, with dimensions and properties as shown in Table 5.1, deriving material strengths from the test data from samples collected in Haiti and reported in Chapter 2 (the model was later updated after the first iteration of design). As the table shows, the panels were not originally included in the finite element model, though their influence on the overall stiffness of the frame will be explored later in this chapter.

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimensions</th>
<th>Material</th>
<th>$f_c$</th>
<th>$f_y$</th>
<th>Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>12” x 12” x 156”</td>
<td>Reinforced Concrete</td>
<td>4 ksi</td>
<td>60 ksi</td>
<td>Longitudinal: 8 #3 Bars Transverse: #3 bars</td>
</tr>
<tr>
<td>Columns</td>
<td>12” x 12” x 120”</td>
<td>Reinforced Concrete</td>
<td>4 ksi</td>
<td>60 ksi</td>
<td>Longitudinal: 8 #3 Bars Transverse: #3 bars</td>
</tr>
</tbody>
</table>

$^1$ Concrete compressive strength
$^2$ Reinforcing steel yield strength
Figure 5.3: Proposed frame layout: plan view (left) and elevation view (right).

Figure 5.4: Finite element model of proposed frame in SAP2000: plan view (left) and 3D view (right). Naming conventions for beam (e.g., B1) and columns (e.g., C1), as well as X-Y axes definitions are also shown.
5.3 Loads

At the time of the 2010 Haiti Earthquake, there was no nationally adopted building code governing the design and construction of structures in Haiti. Since the earthquake, multiple international organizations have been working on creating a building code; however, those efforts have not materialized yet, nor has a plan been implemented that would enforce such a code. As such, the loads and load combinations used to design the proposed structure were taken from the ASCE 7-10 standard currently used in the United States. The following sections will give a summary of the loads considered and their application to the proposed structure.

5.3.1 Gravity Loads

Gravity loads were taken as a combination of dead and live loads as prescribed by ASCE 7-10, Chapters 3 and 4. Figure 5.5 shows the tributary areas used to distribute the dead and live loads to the beam elements in the finite element model. The following two sections detail these loading cases and report the resulting line loads used on the beams in the FEM model.
5.3.1.1 Dead Loads

The proposed structure is one story and residential; therefore the only contributing dead loads \( (D) \) originate from the self-weight of the structure, i.e., there is no fixed service equipment. As per ASCE 7-10, Section 3.1.1, the self-weight of the structure was assumed to have the following contributors:

- Beam cross-sectional dimensions of 12" x 12", with 150 pcf unit weight
- Column cross-sectional dimensions of 12” x 12”, with 150 pcf unit weight
- Partitioning unit weight of 135 pcf
- Roof unit weight of 10 psf

5.3.1.2 Live Loads

Live loads were calculated in accordance with ASCE 7-10, Chapter 4, Table 4-1.

Since the structure is one story, floor loads were not considered in the design of column and beam members; however, for foundation design purposes, a floor live load \( (L) \) of 40 psf was assumed. A roof live load \( (L_r) \) of 20 psf was assumed. Table 5.2 shows the resulting beam line loads, for both dead and live load cases.

**TABLE 5.2:**

<table>
<thead>
<tr>
<th></th>
<th>Dead</th>
<th>Live</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exterior Beams</td>
<td>Exterior Beams</td>
</tr>
<tr>
<td>( D_{roof} )</td>
<td>2.74 lbs/in</td>
<td>( L_{roof} )</td>
</tr>
<tr>
<td>( D_{beam, sw} )</td>
<td>12.50 lbs/in</td>
<td></td>
</tr>
<tr>
<td>( D_{panel, sw} )</td>
<td>6.92 lbs/in</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>22.16 lbs/in</td>
<td><strong>Total</strong></td>
</tr>
<tr>
<td></td>
<td>Interior Beams</td>
<td>Interior Beams</td>
</tr>
<tr>
<td>( D_{roof} )</td>
<td>5.48 lbs/in</td>
<td>( L_{roof} )</td>
</tr>
<tr>
<td>( D_{beam, sw} )</td>
<td>12.50 lbs/in</td>
<td></td>
</tr>
<tr>
<td>( D_{panel, sw} )</td>
<td>6.92 lbs/in</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>24.90 lbs/in</td>
<td><strong>Total</strong></td>
</tr>
</tbody>
</table>

5.3.2 Wind Loads

Wind loads \( (W) \) were calculated in accordance with ASCE 7-10, Chapters 26 and 28. Supplemental wind maps for the Caribbean, specifically the island of Hispaniola,
were taken from (Gibbs 2008). These maps were created to align with the new standards ASCE 7-10, namely with wind speeds taken under the assumptions of a 3-second gust, recorded at a height of 33 feet, in Exposure C, with a 7% probability of exceedance in 50 years (equivalent to a mean recurrence interval of 700 years). The MWFRS Envelope Procedure, detailed in Chapter 28, Part 1, was used to compute the wind loads on the structure. To utilize this procedure, it is assumed that the roof does not provide a rigid diaphragm. The proposed structure meets all other requirements outlined in ASCE 7-10 Section 28.1 as necessary for this method to be valid.

Wind pressures were calculated in accordance with the procedure outlined in Table 28.2-1 in ASCE 7-10, and applied to the structure according to Figure 28.4-1, which is also shown in Figure 5.6 here. Table 5.3 below shows the various factors used in the analysis for this particular structure and location (definitions consistent with those use in ASCE 7-10). The occupancy category is II, as defined in Chapter 1 of ASCE 7-10. The category permits the use of a basic wind speed for wind load calculations with a mean recurrence interval of 700 years, which is 150 mph for the city of Léogâne (see Figure 3.4). Exposure Category B was used, under the assumption that the structure will be located in an inland, suburban area.

Based on these parameters, the velocity pressure at the roof height of the structure ($q_h$) was determined. This velocity pressure is then combined with a series of internal ($GC_{pi}$) and external ($GC_{pf}$) pressure coefficients. The internal pressure coefficients are shown in Table 5.3 and assume an enclosed structured, as the vented roof design does not meet the minimum “open area” requirements to qualify as a partially enclosed structure. The
external pressure coefficients, under the Envelope Procedure, were determined from ACSE 7-10 Table 28.4-1, which also defines the loading cases for low-rise structures.

ASCE 7-10 defines multiple zones of the home based on the assumed direction of the wind in relation to the structure. These zones include both typical loading zones, as well as “End Zones”, which areas of increased pressures due to increased velocities at bluff corners and edges of the structure (Figure 5.6). The external pressure coefficients, and their resulting wind pressures are outlined in Table 5.4 and Table 5.5 for each zone and the two load cases defined in ASCE 7-10 Figure 28.4-1. These pressures are assumed to act uniformly over the zone to which they are applied.
Figure 5.6: Basic load cases for the envelope procedure used in Chapter 28 of ASCE 7-10. Figure shows the various definitions for the “End Zones” based on wind loading direction, used to determine the external pressure coefficients.
TABLE 5.3:
ASCE 7-10 MWFRS ENVELOPE PROCEDURE WIND LOAD PARAMETERS FOR PROPOSED STRUCTURE

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>26.00 ft</td>
</tr>
<tr>
<td>H</td>
<td>8.42 ft</td>
</tr>
<tr>
<td>a</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>End Zone Length</td>
<td>6.00 ft</td>
</tr>
<tr>
<td>Eave Height</td>
<td>2.00 ft</td>
</tr>
<tr>
<td>$\theta$</td>
<td>8.85 $^\circ$</td>
</tr>
<tr>
<td>H_roof</td>
<td>13.15 ft</td>
</tr>
<tr>
<td>L_roof</td>
<td>26.00 ft</td>
</tr>
<tr>
<td>$V$</td>
<td>150.00 mph</td>
</tr>
<tr>
<td>$K_d$</td>
<td>0.85</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>B</td>
</tr>
<tr>
<td>I</td>
<td>1.00</td>
</tr>
<tr>
<td>$K_2$</td>
<td>0.70</td>
</tr>
<tr>
<td>$K_3$</td>
<td>1.00</td>
</tr>
<tr>
<td>$K_4$</td>
<td>1.00</td>
</tr>
<tr>
<td>$K_{zt}$</td>
<td>1.00</td>
</tr>
<tr>
<td>$q_h$</td>
<td>34.27 psf</td>
</tr>
<tr>
<td>$GC_{pi1}$</td>
<td>0.18</td>
</tr>
<tr>
<td>$GC_{pi2}$</td>
<td>-0.18</td>
</tr>
<tr>
<td>$2.5 * eave height$</td>
<td>5.00</td>
</tr>
</tbody>
</table>
### TABLE 5.4:

WIND PRESSURES FOR PROPOSED STRUCTURE: LOAD CASE A

<table>
<thead>
<tr>
<th>ZONE</th>
<th>GCₚᶠ</th>
<th>p₁ (GCₚᵢ₁)</th>
<th>p₂ (GCₚᵢ₂)</th>
<th>Governing p (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Windward Wall)</td>
<td>0.53</td>
<td>12.00</td>
<td>24.33</td>
<td>24.33</td>
</tr>
<tr>
<td>2 (Windward Roof)</td>
<td>-0.69</td>
<td>-29.82</td>
<td>-17.48</td>
<td>-29.82</td>
</tr>
<tr>
<td>3 (Leeward Roof)</td>
<td>-0.48</td>
<td>-22.62</td>
<td>-10.28</td>
<td>-22.62</td>
</tr>
<tr>
<td>4 (Leeward Wall)</td>
<td>-0.43</td>
<td>-20.91</td>
<td>-8.57</td>
<td>-20.91</td>
</tr>
<tr>
<td>1E (Windward Wall - End Zone)</td>
<td>0.80</td>
<td>21.25</td>
<td>33.59</td>
<td>33.59</td>
</tr>
<tr>
<td>2E (Windward Roof - End Zone)</td>
<td>-1.07</td>
<td>-42.84</td>
<td>-30.50</td>
<td>-42.84</td>
</tr>
<tr>
<td>3E (Leeward Roof - End Zone)</td>
<td>-0.69</td>
<td>-29.82</td>
<td>-17.48</td>
<td>-29.82</td>
</tr>
<tr>
<td>4E (Leeward Wall - End Zone)</td>
<td>-0.64</td>
<td>-28.10</td>
<td>-15.77</td>
<td>-28.10</td>
</tr>
</tbody>
</table>

3 The pressures labeled p₁ and p₂ were computed using GCₚᵢ₁ and GCₚᵢ₂, respectively.

### TABLE 5.5:

WIND PRESSURES FOR PROPOSED STRUCTURE: LOAD CASE B

<table>
<thead>
<tr>
<th>ZONE</th>
<th>GCₚᶠ</th>
<th>p₁ (GCₚᵢ₁)</th>
<th>p₂ (GCₚᵢ₂)</th>
<th>Governing p (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Sidewall)</td>
<td>-0.45</td>
<td>-21.59</td>
<td>-9.25</td>
<td>-21.59</td>
</tr>
<tr>
<td>2 (Windward Roof)</td>
<td>-0.69</td>
<td>-29.82</td>
<td>-17.48</td>
<td>-29.82</td>
</tr>
<tr>
<td>3 (Leeward Roof)</td>
<td>-0.37</td>
<td>-18.85</td>
<td>-6.51</td>
<td>-18.85</td>
</tr>
<tr>
<td>4 (Sidewall)</td>
<td>-0.45</td>
<td>-21.59</td>
<td>-9.25</td>
<td>-21.59</td>
</tr>
<tr>
<td>5 (Windward Wall)</td>
<td>0.40</td>
<td>7.54</td>
<td>19.88</td>
<td>19.88</td>
</tr>
<tr>
<td>6 (Leeward Wall)</td>
<td>-0.29</td>
<td>-16.11</td>
<td>-3.77</td>
<td>-16.11</td>
</tr>
<tr>
<td>1E (Windward Sidewall - End Zone)</td>
<td>-0.48</td>
<td>-22.62</td>
<td>-10.28</td>
<td>-22.62</td>
</tr>
<tr>
<td>2E (Windward Roof - End Zone)</td>
<td>-1.07</td>
<td>-42.84</td>
<td>-30.50</td>
<td>-42.84</td>
</tr>
<tr>
<td>3E (Leeward Roof - End Zone)</td>
<td>-0.53</td>
<td>-24.33</td>
<td>-12.00</td>
<td>-24.33</td>
</tr>
<tr>
<td>4E (Leeward Sidewall - End Zone)</td>
<td>-0.48</td>
<td>-22.62</td>
<td>-10.28</td>
<td>-22.62</td>
</tr>
<tr>
<td>5E (Windward Wall - End Zone)</td>
<td>0.61</td>
<td>14.74</td>
<td>27.07</td>
<td>27.07</td>
</tr>
<tr>
<td>6E (Leeward Wall - End Zone)</td>
<td>-0.43</td>
<td>-20.91</td>
<td>-8.57</td>
<td>-20.91</td>
</tr>
</tbody>
</table>
5.3.2.1 Wind Load Application to Structural Model

The loads above were applied to the FEM using the tributary areas shown in Figure 5.7 and Figure 5.8. Figure 5.7 represents the scenario for wall A (see convention in Figure 5.1), and Figure 5.8 represents the scenario for wall B (see convention in Figure 5.1). The red shaded areas are distributed as lateral line loads to the columns, the blue shaded areas are distributed as lateral line loads, and the green shaded areas are assumed to be transferred through the panels to the grade-level foundation beams. All line loads are uniformly distributed loads, in accordance with the pressures applied as shown in Figure 5.6.

Figure 5.7: Tributary areas for lateral wind loads on wall A. Red shaded areas are attributed as lateral line loads to columns, blue shaded areas are attributed as lateral line loads to beams, and green shaded areas are assumed to be transferred to the grade-level foundation beams via the panels.
5.3.3 Seismic Loads

Seismic loads were computed in accordance with ASCE 7-10, Chapters 11 and 12. The Equivalent Lateral Force procedure (ASCE 7-10 Chapter 12, Section 12.8), as allowed by ASCE 7-10 Table 12.6-1, was used to compute the seismic base shear on the proposed structure. Spectral acceleration values for Léogâne were taken from the United States Geological Survey, through their Worldwide Seismic “DesignMaps” Web Application (Figure 5.9) (United States Geological Survey 2012). Based on maps compiled after the earthquake by HaitiData.org, in conjunction with data from the World Bank, it was determined that Léogâne, Haiti, has a shear wave velocity (V_{s30}) of approximately 352 m/s, placing it in Site Class D according to Chapter 20 of ASCE 7-10 (NATHAT 2011).
Figure 5.9: USGS Web Application used for spectral accelerations for Léogâne, Haiti.

The initial fundamental period of the structure was computed from a modal analysis done in SAP2000, reported in Table 5.6 with values of additional parameters used to compute the base shear.
### TABLE 5.6:

ASCE 7-10 SEISMIC EQUIVALENT LATERAL LOAD PROCEDURE PARAMETERS FOR INITIAL PROPOSED STRUCTURE

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structure Mass</strong></td>
<td>54.6 kips</td>
</tr>
<tr>
<td><strong>Lateral Stiffness (K)</strong></td>
<td>135.1 kips/in</td>
</tr>
<tr>
<td><strong>T (fundamental period)</strong></td>
<td>0.11 s</td>
</tr>
<tr>
<td><strong>S_s</strong></td>
<td>1.43 g</td>
</tr>
<tr>
<td><strong>S_1</strong></td>
<td>0.49 g</td>
</tr>
<tr>
<td><strong>Site Class</strong></td>
<td>D</td>
</tr>
<tr>
<td><strong>F_o</strong></td>
<td>1.00</td>
</tr>
<tr>
<td><strong>F_v</strong></td>
<td>1.51</td>
</tr>
<tr>
<td><strong>S_MS</strong></td>
<td>1.43</td>
</tr>
<tr>
<td><strong>S_M1</strong></td>
<td>0.74</td>
</tr>
<tr>
<td><strong>S_DS</strong></td>
<td>0.95</td>
</tr>
<tr>
<td><strong>S_D1</strong></td>
<td>0.49</td>
</tr>
<tr>
<td><strong>T_o</strong></td>
<td>0.10 s</td>
</tr>
<tr>
<td><strong>T_s</strong></td>
<td>0.52 s</td>
</tr>
<tr>
<td><strong>T_L</strong></td>
<td>12.00 s</td>
</tr>
<tr>
<td><strong>S_a</strong></td>
<td>0.95</td>
</tr>
<tr>
<td><strong>Occupancy Category</strong></td>
<td>II</td>
</tr>
<tr>
<td><strong>Importance Factor</strong></td>
<td>1.00</td>
</tr>
<tr>
<td><strong>Seismic Design Category</strong></td>
<td>D</td>
</tr>
<tr>
<td><strong>Structural System Selection</strong></td>
<td>Ordinary Reinforced Concrete MRF</td>
</tr>
<tr>
<td><strong>R</strong></td>
<td>3.00</td>
</tr>
<tr>
<td><strong>Ω_o</strong></td>
<td>3.00</td>
</tr>
<tr>
<td><strong>C_d</strong></td>
<td>2.50</td>
</tr>
<tr>
<td><strong>C_u</strong></td>
<td>1.40</td>
</tr>
<tr>
<td><strong>C_t</strong></td>
<td>0.02</td>
</tr>
<tr>
<td><strong>h_n</strong></td>
<td>10.00</td>
</tr>
<tr>
<td><strong>x</strong></td>
<td>0.90</td>
</tr>
<tr>
<td><strong>T_o</strong></td>
<td>0.13 s</td>
</tr>
<tr>
<td><strong>Period Upper Limit</strong></td>
<td>0.18 s</td>
</tr>
<tr>
<td><strong>C_s</strong></td>
<td>0.32</td>
</tr>
<tr>
<td><strong>V (Base Shear)</strong></td>
<td>17.34 kips</td>
</tr>
</tbody>
</table>
There are a few parameters that deserve some attention and explanation. Firstly, the long-period transition period, $T_L$, was assumed to be the same as shown in ASCE 7-10 for Puerto Rico. This parameter, however, is not vital to the base shear calculation as the proposed structure has a very short period, and its loading is not affected by this parameter. Secondly, as listed in Table 5.6, an ordinary reinforced moment resisting concrete frame was chosen for the structural system classification. Technically, this type of system is not permitted for structures located in an area with a Site Class D classification. However, due to the unusual nature of this reinforced concrete structure (i.e., one story, residential structure with a lightweight roof), it was decided that this classification was the most accurate representation of the system. Furthermore, this classification leads to a more conservative loading scenario as the reduction factor is 3, instead of 8, as permitted by the use of a special reinforced concrete moment resisting frame classification. To ensure that this assumption is safe, later sections in this chapter will compare the final design, consistent with the ordinary MRF classification, with the requirements of the special MRF classification to determine how closely they match. From that comparison, further comments will be made on the assumption made here.

The base shear calculated was then applied to the structural FEM model. It was assumed that the dynamic response of the structure is dominated by the fundamental mode. Since the design of short period structures is dominated by strength requirements (i.e., accelerations), which remain relatively constant at short periods, while the participation factor decreases in higher modes, these modes are assumed not
to participate significantly in the response. This fundamental mode is assumed to have a linear mode shape, consistent with assumption of a shear-dominated building and the behavior from the FEM model. This is then used to translate the base shear to a lateral loading on the structure. The mass has been assumed to be concentrated at the first floor level. Therefore, lateral concentrated loads have been assumed on the joints to represent the equivalent static seismic loading scenario. Due to the symmetry of the stiffness of the structure, the base shear was assumed to be distributed to each lateral frame equally. Figure 5.10 shows this representation.

Figure 5.10: Assumed distribution of seismic base shear from equivalent static lateral load procedure.

5.4 Frame Analysis

The finite element model was then analyzed in SAP2000 under all above loadings applied to the structure following the strength design load combinations and load factors listed in Chapter 2 of ASCE 7-10. Table 5.7 shows the governing forces for each
member. Table 5.8 lists the displacements in the six degrees of freedom for all joints at the roof level. The labels used in these tables are consistent with the labels shown in Figure 5.4. The deflections in particular confirm that the rigid diaphragm assumption was indeed not valid for this structure.

**TABLE 5.7:**

MAXIMUM ELEMENT FORCES OF PROPOSED INITIAL STRUCTURE

<table>
<thead>
<tr>
<th>Element</th>
<th>P</th>
<th>V2</th>
<th>V3</th>
<th>T</th>
<th>M2</th>
<th>M3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>kips</td>
<td>kips</td>
<td>kips</td>
<td>kip-in</td>
<td>kip-in</td>
<td>kip-in</td>
</tr>
<tr>
<td>Beams</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>-6.90</td>
<td>3.52</td>
<td>-0.57</td>
<td>-1.72</td>
<td>-13.35</td>
<td>-149.79</td>
</tr>
<tr>
<td>B2</td>
<td>-3.21</td>
<td>-3.48</td>
<td>0.49</td>
<td>1.85</td>
<td>-11.77</td>
<td>-147.93</td>
</tr>
<tr>
<td>B3</td>
<td>-0.72</td>
<td>3.13</td>
<td>-0.62</td>
<td>1.63</td>
<td>14.87</td>
<td>-92.08</td>
</tr>
<tr>
<td>B4</td>
<td>-1.32</td>
<td>4.20</td>
<td>0.07</td>
<td>0.31</td>
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</tr>
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<td>1.88</td>
<td>-3.06</td>
<td>74.97</td>
<td>-203.36</td>
</tr>
</tbody>
</table>
TABLE 5.8:

MAXIMUM JOINT DISPLACEMENTS FOR PROPOSED INITIAL STRUCTURE

<table>
<thead>
<tr>
<th>Joint Name</th>
<th>U1 (in)</th>
<th>U2 (in)</th>
<th>U3 (in)</th>
<th>R1 (rad.)</th>
<th>R2 (rad.)</th>
<th>R3 (rad.)</th>
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<tr>
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<td>0.066</td>
<td>0.011</td>
<td>0.002</td>
<td>0.000</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td>C7</td>
<td>0.069</td>
<td>0.011</td>
<td>0.001</td>
<td>0.000</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td>C8</td>
<td>0.067</td>
<td>0.017</td>
<td>0.002</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C9</td>
<td>0.066</td>
<td>0.011</td>
<td>0.001</td>
<td>0.000</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td><strong>Maximum</strong></td>
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<td><strong>0.017</strong></td>
<td><strong>0.003</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.001</strong></td>
<td><strong>0.000</strong></td>
</tr>
</tbody>
</table>

5.5 Frame Design

The beams and columns of the frame were designed using a capacity-based approach. Sizing and reinforcement detailing are in accordance with ACI 318-11, including seismic provisions. Material properties assumed are shown Table 5.9, again in accordance with the material data collected during reconnaissance and presented in Chapter 2. It should be noted that in all reinforcement detailing, the ACI 318-11 minimums governed. This was due to the relatively low forces on the structure (due to its small size and mass). Beams were originally sized at 8” wide and 14” deep, and columns at 8” square. Figure 5.11 shows the cross-sectional detailing of both the beams and columns after the initial design iteration. Figure 5.12 shows the transverse reinforcement spacing for both the beams and columns. The critical end sections are
shown. Spacings shown near the section cut in this image are representative of those found in all non-critical sections of the elements.

**TABLE 5.9:**

**MATERIAL PROPERTIES USED IN FRAME DESIGN**

<table>
<thead>
<tr>
<th>Material</th>
<th>Strength Properties</th>
<th>Modulus of Elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$f_c = 4$ ksi</td>
<td>3,605 ksi</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>$f_y = 60$ ksi</td>
<td>29,000 ksi</td>
</tr>
</tbody>
</table>

![Figure 5.11: Initial column (left) and beam (right) cross sectional sizing and reinforcement detailing.](image)

**COLUMN**

**BEAM**
Figure 5.12: Initial column (top) and beam (bottom) transverse reinforcement spacing. Only element ends (i.e. critical sections) are shown; spacings shown next to section cut are representative of the entire non-critical section.

5.5.1 Joint Design

Due to the small size of this reinforced concrete frame, spacing between reinforcement is a concern, especially in the beam-column joints. Due to the high level of quality that must be met to make this proposed system viable, the joint detailing
needs to be as simple as possible to construct. Overly crowded rebar cages in this area will counteract this goal. This constraint led to the consideration of using headed bars in the beam-column joints, as a substitute for traditional joint reinforcement detailing. Additionally, due to the ease of fabrication and installation of headed bars, as opposed to the labor-intensive practice of bending joint reinforcement in the field, the use of headed bars is attractive to support the prefabrication of elements. This section will outline the traditional joint reinforcement design, and then detail the headed bar joint reinforcement design.

5.5.1.1 Traditional Joint Reinforcement Detailing

Traditional joint detailing in monolithic reinforced concrete structures has revolved around two main requirements: (1) adequate confinement and (2) proper anchorage and development. The first is essential for maximizing the contribution of the concrete to the joint’s performance, especially critical in seismic events, and preventing the vertical reinforcement in the column from buckling (Nilson, et al. 2010). The latter is important for taking full advantage of the presence of the steel in the members and the joint. Without proper anchorage mechanisms or development length, the reinforced concrete behaves much more like plain, unreinforced concrete.

5.5.1.1.1 Joint Classification

Before actually detailing the confinement and anchorage-development steel, the performance level of the joint must first be specified. ACI 352R-02 does this with two
broad categories, Type 1 and Type 2 connections (Joint ACI-ASCE Committee 352 2012).

They are defined as follows:

**ACI 352R-02, Section 2.1:**

*Type 1 — A Type 1 connection is composed of members designed to satisfy ACI 318-02 strength requirements, excluding Chapter 21, for members without significant inelastic deformation.*

*Type 2 — In a Type 2 connection, frame members are designed to have sustained strength under deformation reversals into the inelastic range.*

Seismic demands are of concern for this structure and this would normally call for a Type 2 classification in order to achieve adequate inelastic deformations and energy dissipation capacity. However, due to the unique nature of the structure being designed, namely the fact that it is a one-story reinforced concrete frame with minimal gravity loads, inelastic deformations were not considered to be as important. It should be noted that in the design of the beams and columns to the ACI 318-11 code, code minimums for steel reinforcement governed the design due to the fact that the loads on the structure are relatively minor (ACI Committee 318 2011). Taking all of this into account, it was decided that the joints in the proposed structure would be designed as Type 1 connections, looking to only satisfy strength requirements.

After the required joint performance level was determined, the next step is to determine joint configurations. There are six basic joint configurations, the three most critical to this design are shown in Figure 5.13: roof interior, roof exterior, and roof corner. This is due to the fact that the structure is only one story (note, if future iterations of the design would include a second story, all six joint types would be
applicable). These classifications will come into play when calculating joint shear capacity later in this process.

Figure 5.13: ACI 352R-02 typical beam-to-column connections relevant to the proposed design.

5.5.1.1.2 Determination of Member Flexural Strengths

ACI dictates that joints be designed for the forces equal to the nominal strengths of all connected members, as opposed to the forces determined by frame analysis of the loading scenarios. Therefore, nominal flexural strengths of the beams must be determined in order to calculate shear demand. Refer to Appendix D for the hand calculations of member strengths and shear demand.

5.5.1.1.3 Joint Transverse Reinforcement

To satisfy the confinement requirements of the joint, transverse reinforcement must be provided in both the vertical and horizontal directions, through the joint. For Type 1 connections, at least two layers of reinforcement must be provided horizontally between the top and bottom longitudinal reinforcement of the deepest beam, with spacing not exceeding 6 in. Additionally, for joints with free horizontal column faces
(a.k.a. roof columns), vertical transverse reinforcement must also be provided. The guidelines for number of layers and spacing are the same as the horizontal transverse reinforcement. In order to facilitate placement of this reinforcement, inverted U-shaped stirrups may be used. Transverse steel area requirements are outlined in ACI 352R-02, Section 4.2. The calculations of amounts and placement of this transverse reinforcement are shown in Appendix D. It should be noted that all stirrups must be closed with 135-degree hooks to satisfy seismic requirements. There is a minimum length for the legs of these hooks, also reported in Appendix G, pages 1-2.

5.5.1.1.4 Joint Shear

The next step in joint detailing is verifying joint shear capacity exceeds the design shear force. As noted previously, this design shear force is calculated based on nominal flexural strengths of the beams framing into the column. Nominal joint shear strength is determined by Equation 4.7 in ACI 352R-02:

\[ V_n = \gamma b_j h_c f'_c \]  
Equation 5-1

where \( \gamma \) is a constant governed by joint configuration (Figure 4.4 in ACI 352R-02), \( b_j \) is the effective joint width (governed by Equation 4.8 of ACI 352R-02), \( h_c \) is depth of the column in the direction that shear is being considered, and \( f'_c \) 28-day compressive strength of the concrete. The joint must satisfy Equation 4.6 in ACI 352R-02:
\[ \Phi V_n \geq V_u \]  \hspace{1cm} \text{Equation 5-2}

where \( V_u \) is the design shear calculated from nominal strengths of beams that frame into the joint. Free body diagrams and calculations for these values are shown in Appendix D. It should be noted, that due to the symmetry of the columns and beams in the proposed structure, a corner joint and exterior joint were the only joints analyzed. Since shear has to be considered in each direction, all other joint configurations are a superposition of these two joint types.

5.5.1.1.5 Development of Reinforcement

In order to achieve composite behavior between the concrete and steel, proper anchorage and bond forces must be achieved. In traditional joint detailing, this is achieved either through sufficient development length of a straight bar or through the use of terminating steel hooks in the joint. In most joints there is not enough room for proper development length of a straight bar, and this is all but impossible in roof, corner, and exterior joints. Therefore, in the case of the proposed structure, a combination of proper development length and proper anchorage hook length will be used. For Type 1 joints the former is governed by ACI 318-11 Section 12.5.2, (assuming normal weight concrete and uncoated reinforcement):

\[ l_{dh} = \frac{0.02\psi_e f_y d_b}{\sqrt{f'_c}} \] \hspace{1cm} \text{Equation 5-3}

where \( \psi_e \) is a constant, taken as 1.2 for epoxy-coated reinforcement and 1.0 for all other cases. The term \( f_y \) is yield stress of steel reinforcement and \( d_b \) is the bar diameter.
This length is as measured from the face of the column. For the initial proposed structure the $l_{dh}$ is required to be greater than 7.5” for both the columns and the beams.

Anchorage length is governed by ACI 318-11 (Figure 5.14), as measured from the free end to the start of the 90-degree bend:

$$ l \geq 12d_b \quad \text{Equation 5-4} $$

![Diagram of development length and anchorage length](image)

Figure 5.14: Definition of development length ($l_{dh}$) and anchorage length ($l$) for traditional joint reinforcement.

As referenced throughout this section, Appendix D contains the supporting calculations for traditional joint detailing described herein. The traditional joint detailing for the initial design of the proposed frame is illustrated in Figure 5.15.
5.5.1.1.6 Issues in Traditional Detailing: Initial Proposed Structure

As already stated, the original sizing of the columns and beams in the structure were 8 in. x 8 in. and 8 in. x 12 in., respectively. When the above procedure was completed for elements with those dimensions, issues arose with spacing in the joint. Both the development length and hook length required by code for the columns and beams are too large to fit the dimensions of the joint itself (see Figure 5.15). Cover and spacing requirements from other sections of ACI 318-11 were not met with the initial configuration. Consequently, column and beam sizes had to be increased if a traditional joint detail was to be used. Beam column dimensions were increased to 10” x 15”, and column dimensions were increased to 10” x 10”. The schematic for the resulting detail, using the same traditional joint procedure outlined above, is shown in Figure 5.16, which meets all ACI code requirements. Additionally, the constructability of the reinforcement scheme shown in Figure 5.16 is much more feasible than the initial proposed design. The calculations for this joint are located in Appendix E.
Figure 5.15: Traditional joint detail for initial proposed structure. Note that the development length ($l_{dh}$) of the reinforcement from the beam is less than that required by code (7.5").
Figure 5.16: Traditional joint detail for revised proposed structure, (Prototype Structure) which meets all ACI code requirements.

5.5.1.2 Headed Bar Literature Review

Research into the use of headed bars in practical applications dates back to the late 1980s/early 1990s. Multiple studies have demonstrated their effectiveness in developing anchorage forces, as well as a particular interest in their performance under seismic and cyclical loading. The motivation for the study of headed bar use in joints is less about enhanced performance and more about increased constructability (Wallace, et al. 1998). The speed and ease of installing rebar cages without hooks is highly attractive to the construction industry, as well as joint detailers. Beam-column joints become much less congested, increasing speed of assembly, while easing concrete
placement. The aforementioned study by Wallace partnered with two headed bar manufacturers and was focused on the seismic performance of joints that utilized headed bars. Moreover, there is general consensus among researchers (Chutarat and Aboutaha 2003; Lee and Yu 2009; Kang, et al. 2009; Kang and Mitra 2012) that seismic performance of headed bars meets, and in some cases, exceeds that of traditional hooks. No data was found to the contrary. Due to the clear advantages of using headed bars, all literature recommended their use in real applications. There were a few issues that were brought to light, that must be addressed in actual design:

R1. For corner and roof type joints, restraining the rebar and heads is crucial. Therefore, the transverse reinforcement that is required in traditional joint detailing is still recommended.

R2. Pushout failure of the heads when used in an exterior joint may be a problem if shear forces are high and loading is cyclic. Arresting this failure mode is possible with the use of double anchorage systems.

R3. Clear spacing between adjacent bars with headed reinforcement is not as important as once thought, provided that the joint has sufficient transverse reinforcement to ensure adequate confinement.

The research performed throughout the 1990s and 2000s led to the inclusion of headed bar detailing in the design recommendation document ACI 352R-02. Further validation of performance and the design recommendations led to the eventual inclusion of a section in ACI 318-08, called “Development of headed and mechanically anchored deformed bars in tension”. This section outlines requirements for development length, bar size, head bearing area, clear cover, and spacing.
5.5.1.3 Design Guidelines for Headed Bar Detailing

For the most part, the switch from traditional joint reinforcement detailing to headed bars does not vary the process of design much. The main differences are in the calculation of development length, and instead of hook length, head size must be calculated. Referring back to the previous section on traditional joint reinforcement detailing, all steps are identical except for the calculations of development lengths.

5.5.1.3.1 Development of Reinforcement – Headed Bars

First, development will be calculated, according to ACI 318-11 Section 12.6.2:

\[ l_{dt} = \frac{0.016 \psi f_y d_b}{\sqrt{f'_c}} \]  

Equation 5-5

For this case, \( l_{dt} \) must be greater than the larger of \( 8d_b \) and 6 in. Once this calculated, the bearing area of the head must also be checked against ACI 318-11, 12.6.1(f):

\[ A_{brg} \geq 4A_b \]  

Equation 5-6

where \( A_{brg} \) is the net bearing area of the head (in\(^2\)) and \( A_b \) is the cross sectional area of the steel reinforcement. There are additional conditions that must be checked in terms of placement of the bars, clear spacing, and cover that can be found in Section 12.6 of ACI-318-12.

5.5.1.4 Issues in Headed Bar Detailing: Initial Proposed Structure

As was the case in the traditional detailing, the original sizing of the columns and beams proved inadequate to meet the requirements set forth by the code for headed
bar detailing. Although the development and anchorage lengths are met, spacing between heads and stirrups is an issue. There is not enough physical space to fit the heads along with the transverse stirrups needed inside the joint for the beam element. The detail for this initial structure is shown in Figure 5.17. Just as in the traditional detailing procedure, beam and column sizes were increased to the sizes stated previously. When this configuration is detailed following the preceding procedure, all requirements are again met. This detail is shown in Figure 5.18. Therefore, the proposed structure has been revised, regardless of the use of traditional or headed bar joint detailing, to beams sized at 10” x 15” and columns at 10” x 10”. The detailing of these elements is shown in Figure 5.19. Spacing of the transverse reinforcement within the members themselves does not change due to this increase in the cross sectional dimensions (the only changes are within the joints) from those prescribed for the initial proposed structure (refer back to Figure 5.12 for these details). This structure will be referred throughout the rest of this thesis as the “Prototype Structure”.

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Figure 5.17: Headed bar joint detail for initial proposed structure. Note that the heads interfere with the stirrups confining the beam within the joint.
Figure 5.18: Headed bar joint detail for revised proposed structure (Prototype Structure), which meets all ACI code and constructability requirements.
5.6 Influence of Panels on Frame Behavior

The modeling of the proposed frame, as described in the previous sections, assumes there is no contribution to the behavior of the structure from the lightweight concrete panels. While these panel elements are intended, and as will be shown in the next chapter, designed to act as non-structural elements, because of the novelty of this system and its constituent materials, it was decided that a more thorough investigation was warranted. Additionally, quantifying the influence of the panels to the overall system behavior provides a point of comparison with the traditional unreinforced masonry system, allowing the author to make subsequent claims about the difference in structural performance between the two systems. This section will propose a few
models to include the influence of the panels on the overall behavior of the frame, and then draw some conclusions about the actual system based on these models.

5.6.1 Bare Frame

In order to understand the influence of the panels, the bare frame first needs to be modeled and analyzed in terms of system behavior, to set a baseline. A linear modal analysis was conducted for the bare frame model, with properties equal to those of the final design iteration. Table 5.12 lists the periods for the first three modes. The table also lists a lateral stiffness, computed by imparting a unit load at each joint of the roof level in the global X-direction. This produced a deflection, from which the stiffness was calculated.

5.6.2 Equivalent Lateral Bracing

Infill concrete and masonry panels are commonly modeled as diagonal compression struts for seismic analysis of structures (Mainstone 1971). Other models have also been used, based on finite element representations of the infill (Maneetes and Memari 2009), however, the diagonal strut model has proven effective in matching system wide properties. The premise of the diagonal strut model is that when loaded laterally, the surrounding frame engages the infill panel for discrete lengths at two corners of the infill undergoing compressive forces, creating an effective strut, as shown in Figure 5.20. These areas of contact then create a strut diagonally across the panel that is responsible for the contribution to lateral stiffness of the system. If the modulus of elasticity and the thickness of the panel are assumed to remain constant, an
equivalent strut can be calculated simply by determining a width for the strut. Early models simply assigned a constant factor, such as one-third of the diagonal length of the panel. With time, more sophisticated models were explored, and it is now widely accepted that the width of the strut is related to the relative stiffness of the frame (Teeuwen 2009).

This basic model has been adopted by the Federal Emergency Management Agency in *FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 356 2000). This model will be adopted for modeling of the panels as structural elements. Although the panels are not infill, this model will show the maximum influence they could have on the frame, and an appropriate reduction factor can be assumed to take into account that the panels are attached outside of the plane of the frame.
Figure 5.20: The equivalent strut concept. $W$ represents the width of the panel engaged in the lateral system.

The cross sectional area of the strut is calculated from the following equation:

\[
\alpha = 0.175 (\lambda_1 h_{col})^{-0.4} r_{inf}
\]

Equation 5-7

where $\lambda_1$ is a parameter calculated based on the relative stiffness of the infill to the columns, $h_{col}$ is the height of the columns, and $r_{inf}$ is the diagonal length of the infill.

The $\lambda_1$ factor is calculated as follows:

\[
\lambda_1 = \left[ \frac{E_{me\inf} \sin 2\theta}{4E_{fe\col} h_{inf}} \right]^{\frac{1}{4}}
\]

Equation 5-8

where the parameters are defined in Table 5.10.
TABLE 5.10:
PARAMETERS AND DEFINITIONS FOR EQUIVALENT STRUT CALCULATION

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Assumed Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{inf}$</td>
<td>Height of infill panel, in.</td>
<td>96 in.</td>
</tr>
<tr>
<td>$E_{fe}$</td>
<td>Modulus of elasticity of frame material, ksi</td>
<td>3605 ksi</td>
</tr>
<tr>
<td>$E_{me}$</td>
<td>Modulus of elasticity of infill material, ksi</td>
<td>4009.5 ksi</td>
</tr>
<tr>
<td>$I_{col}$</td>
<td>Moment of inertia of column, in$^4$</td>
<td>100 in$^4$</td>
</tr>
<tr>
<td>$t_{inf}$</td>
<td>Thickness of infill panel and equivalent strut, in.</td>
<td>1.5 in. (See Ch. 6)</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Angle whose tangent is the infill height-to-length aspect ratio, rad.</td>
<td>Varies with model (see next section)</td>
</tr>
</tbody>
</table>

Given that the panels are not exactly infill, two variations on the model will be used. The first (Model 1) will assume that the four panels per wall will act as one element across the frame. One diagonal strut will be calculated from this assumption and applied to the frame (Figure 5.21). The second (Model 2) will assume that each of the four panels acts as its own strut. The strut properties will be calculated (results in Table 5.11) based on this assumption and four diagonal braces will be used per frame, at the locations of the four panels (Figure 5.21). The only difference in calculating these two models is the width of the infill used.
Figure 5.21: Equivalent strut models used to model panel behavior. (Left) Model 1: assumes the four panels work as one element. (Right) Model 2: that assumes each panel works as a separate strut.

### TABLE 5.11:
EQUIVALENT STRUT PROPERTIES FOR MODELS 1 & 2

<table>
<thead>
<tr>
<th>Equivalent Strut Model</th>
<th>Cross-Sectional Area</th>
<th>Length</th>
<th>Axial Stiffness</th>
<th>Lateral Stiffness Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>26.39 in²</td>
<td>185.8 in.</td>
<td>563.1 kip/in.</td>
<td>396.8 kip/in.</td>
</tr>
<tr>
<td>Model 2</td>
<td>15.85 in²</td>
<td>108.3 in.</td>
<td>580.6 kip/in</td>
<td>75.3 kip/in.</td>
</tr>
</tbody>
</table>

5.6.3 Implementing Equivalent Lateral Bracing in SAP2000

Once the equivalent strut properties were determined, they had to be translated into elements in SAP2000 in order to run analyses. Two different types of elements were used in order to run two different analyses. The first element, a general bracing member, was used to run a linear modal analysis. This element was defined with the properties of the panel, i.e. modulus of elasticity, and with the cross sectional
dimensions of the equivalent diagonal strut. These elements were only defined in one direction, assuming to work in both compression and tension. The second type of element used was a nonlinear gap spring element. This element was used in nonlinear time history analyses on the structure. The element was simply defined by the axial stiffness listed in Table 5.11. The gap distance was taken as zero. These parameters ensured that the gap element worked only in compression (under any compression force), therefore allowing the use of springs in both directions. The matrix schematic in Figure 5.22 shows the four different configurations used for both diagonal strut models mentioned and for both analysis cases.

![Figure 5.22: Analysis matrix of SAP2000 models.](image-url)
5.7 Analysis Cases

5.7.1 Linear Modal Analysis

Table 5.12 shows the results of the linear modal analysis completed for the systems shown in the first row in Figure 5.22. The results are as expected, with the bare frame having the longest fundamental periods and the lowest lateral stiffness. Model 1 has the highest fundamental period and the highest lateral stiffness. Referring back to Table 5.11, it’s clear that this model has the largest lateral stiffness contribution. Physically this also makes sense, as a singular infill would have a much higher stiffness contribution than four individual elements. The actual contribution of the panels is most likely to fall somewhere between Model 2 and that of the bare frame. The panels are expected to act more like individual elements than one composite panel, but the fact that the panels are not actually infill elements means that their stiffness contribution is less than that of Model 2. In terms of the effect of this increased stiffness on the loading, per ASCE equivalent static load procedure, this increase actually decreases the design spectral response acceleration, meaning no increase in loading. Additionally, because all code minimums were enforced for the design requirements, sizing of the members would not change.
TABLE 5.12:

STRUCTURAL SYSTEM PROPERTIES OF MODELS WITH AND WITHOUT INFLUENCE OF PANELS

<table>
<thead>
<tr>
<th>Model</th>
<th>Mode 1 Period (s)</th>
<th>Mode 2 Period (s)</th>
<th>Mode 3 Period (s)</th>
<th>Lateral Stiffness (kip/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare Frame</td>
<td>0.140</td>
<td>0.140</td>
<td>0.120</td>
<td>84</td>
</tr>
<tr>
<td>Model 1</td>
<td>0.055</td>
<td>0.053</td>
<td>0.052</td>
<td>769</td>
</tr>
<tr>
<td>Model 2</td>
<td>0.085</td>
<td>0.076</td>
<td>0.067</td>
<td>313</td>
</tr>
</tbody>
</table>

5.7.2 Nonlinear Time History Analysis

Both models with nonlinear gap spring elements were subjected to a nonlinear time history analysis. Due to the fact that there was no seismic instrumentation in Haiti at the time of the 2010 earthquake, simulated ground motions were used (Mavroeidis and Scotti 2013). Ground motions were taken from those simulated for the city of Léogâne, under the variable-size subevent simulation parameters. Three motions were selected out of a class of 30. Figure 5.23 and Figure 5.24 show the ground acceleration response spectra for the three ground motions in the two horizontal directions. The figures also show the average of the three motions, and the design spectral acceleration determined by ASCE 7-10. In the period range defined by ASCE 7-10 16.1.3.1, the average of the three ground motions exceeds the design response spectrum in both horizontal directions, making the suite of three ground motions acceptable for use in analysis.
Figure 5.23: Acceleration response spectra for three simulated ground motion time histories for horizontal component 1 (top). Additionally, a close up of the comparison between the design response spectrum from ASCE 7-10 and the mean of the three ground motions is shown (bottom).

Figure 5.24: Acceleration response spectra for three simulated ground motion time histories for horizontal component 2 (top). Additionally, a close up of the comparison between the design response spectrum from ASCE 7-10 and the mean of the three ground motions is shown (bottom).
The maximum displacements and rotations for each model under the suite of ground motions for the roof joints are listed in Table 5.13, Table 5.14, and Table 5.15.

**TABLE 5.13:**

MAXIMUM JOINT DISPLACEMENTS AND ROTATIONS FOR BARE FRAME UNDER THREE GROUND MOTIONS

<table>
<thead>
<tr>
<th>Joint</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>in</td>
<td>in</td>
<td>in</td>
<td>rad.</td>
<td>rad.</td>
<td>rad.</td>
</tr>
<tr>
<td>C1</td>
<td>0.303</td>
<td>0.250</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>C2</td>
<td>0.303</td>
<td>0.362</td>
<td>0.001</td>
<td>0.002</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C3</td>
<td>0.303</td>
<td>0.250</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>C4</td>
<td>0.441</td>
<td>0.250</td>
<td>0.001</td>
<td>0.000</td>
<td>0.002</td>
<td>0.000</td>
</tr>
<tr>
<td>C5</td>
<td>0.441</td>
<td>0.362</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C6</td>
<td>0.441</td>
<td>0.250</td>
<td>0.001</td>
<td>0.000</td>
<td>0.002</td>
<td>0.000</td>
</tr>
<tr>
<td>C7</td>
<td>0.303</td>
<td>0.250</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>C8</td>
<td>0.303</td>
<td>0.362</td>
<td>0.001</td>
<td>0.002</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C9</td>
<td>0.303</td>
<td>0.250</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.441</td>
<td>0.362</td>
<td>0.001</td>
<td>0.002</td>
<td>0.002</td>
<td>0.001</td>
</tr>
</tbody>
</table>
### Table 5.14:

**Maximum Joint Displacements and Rotations for Model 1 Under Three Ground Motions**

<table>
<thead>
<tr>
<th>Joint</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>in</td>
<td>in</td>
<td>in</td>
<td>rad.</td>
<td>rad.</td>
<td>rad.</td>
</tr>
<tr>
<td>C1</td>
<td>0.012</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C2</td>
<td>0.012</td>
<td>0.021</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C3</td>
<td>0.012</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C4</td>
<td>0.020</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C5</td>
<td>0.020</td>
<td>0.021</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C6</td>
<td>0.020</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C7</td>
<td>0.012</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C8</td>
<td>0.012</td>
<td>0.021</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C9</td>
<td>0.012</td>
<td>0.011</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Maximum</td>
<td><strong>0.020</strong></td>
<td><strong>0.021</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
</tr>
</tbody>
</table>

### Table 5.15:

**Maximum Joint Displacements and Rotations for Model 2 Under Three Ground Motions**

<table>
<thead>
<tr>
<th>Joint</th>
<th>U1</th>
<th>U2</th>
<th>U3</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>in</td>
<td>in</td>
<td>in</td>
<td>rad.</td>
<td>rad.</td>
<td>rad.</td>
</tr>
<tr>
<td>C1</td>
<td>0.017</td>
<td>0.016</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C2</td>
<td>0.017</td>
<td>0.030</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C3</td>
<td>0.017</td>
<td>0.016</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C4</td>
<td>0.030</td>
<td>0.015</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C5</td>
<td>0.030</td>
<td>0.030</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C6</td>
<td>0.030</td>
<td>0.016</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C7</td>
<td>0.018</td>
<td>0.015</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C8</td>
<td>0.018</td>
<td>0.030</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C9</td>
<td>0.018</td>
<td>0.016</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>Maximum</td>
<td><strong>0.030</strong></td>
<td><strong>0.030</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
<td><strong>0.000</strong></td>
</tr>
</tbody>
</table>
Again, as with the modal analysis, the response is the highest for the bare frame model, and the lowest under Model 1. Since it is assumed that the actual system lies between the bare frame model and Model 2, it is safe to say that the bare frame model provides the worst-case scenario in terms of joint displacements. The maximum displacement of any joint under any simulated ground motion was 0.44”, and thus tolerances of the panels. This drift value will be used later in the design of the panels themselves to ensure that drift levels do not cause a fracture of the panels or crushing due to contact between adjacent panels.

5.8 Constructability

In addition to the challenges that arise from lack of materials, engineering design expertise, and building codes and enforcement, the actual construction of a home can also be problematic. Lack of formwork, on-site quality control measures, and tools and equipment limit the types of implementable systems, as well as the quality of those systems. The proposed frame and panel system is reliant on a high level of quality, especially in the frame designed in the previous sections, to ensure the life safety it is intended to deliver. Therefore, a three-fold strategy has been created to guarantee that the high level of quality is present. This chapter will explore these three elements, namely reusable formwork, the use of prefabrication and paraskilling, and tools and equipment tailored to the developing world, and explain how their utilization will allow for the implementation of the frame and panel housing system.
5.8.1 Reusable Formwork

One of the most problematic constructability issues facing Haiti is the lack of available formwork for concrete work. The most obvious reason for this shortage is the deforestation that has plagued the country over the past decades. Poor agricultural practices, combined with the heavy use of timber as charcoal for cooking purposes, has left Haiti over 90% deforested, leaving little construction grade wood. The manifestation of this deficiency is the construction practice discussed in detail in Chapter 2, where CMUs had to serve as partial formwork for columns, due in part to a lack of wood. As mentioned in that chapter, this led to undersized columns, which were at the same time, highly integrated into the lateral system. As walls failed during the earthquake, loads were transferred to these undersized columns, which were ultimately ill equipped to handle the loads. One mitigation strategy to ensure proper column sizing is to solve the formwork deficit.

If an alternative to wood is to be offered for formwork, much like the housing system itself, it must be economically viable and available. The most obvious way to think about making formwork economical is to make it reusable. In fact, wood is already used in this way in Haiti, however the number of reuses is limited. Materials such as aluminum, steel, and plastics are more durable, not to mention the capacity to be built into a customized formwork set. Therefore, the proposed model will incorporate such a customized system. By utilizing reusable formwork (in the form of aluminum, steel, or plastic), two problems are addressed: (1) proper column sizing can be achieved and (2) a high level of quality can be ensured from job to job. The repetition of using the same
formwork from home to home increases the quality of the product because the crew becomes accustomed to a singular system, and additionally, the tolerances on a manufactured formwork set can be set extremely low. The formwork set can also be designed so that it only becomes usable if erected in the intended configuration, such “foolproof” mechanisms again filling the quality control void.

5.8.2 Formwork Design

In light of the use of standardization and a reusable formwork system, the design of the formwork itself becomes very important. The system must accommodate the construction constraints present locally, but also be conducive to repetition and paraskilling. There are a few constraints in Haiti that impact the design of the formwork. Firstly, due to a lack of heavy equipment, the concrete frame must be cast monolithically. Precast concrete elements are not viable because of the inability to lift beams into place, as well as limited connection mechanisms. Therefore, the formwork must be light enough to assemble as a complete frame, without the use of heavy equipment. Secondly, the fact that the frame must be monolithically cast also raises concerns over consolidation of the concrete in the bottom of the columns. Since concrete will be poured from the top of the column formwork, and without the vibrating equipment typically used in the developed world, consolidation at the bottom of the column could become a problem. Consequently, the formwork design must make accommodations for ensuring proper consolidation at the bottom of the columns.
The above constraints led to the determination that the formwork needs to be manufactured from the lightest available material, even if that means importation. Because of the importance of the formwork, and the fact that it will be a reusable tool, importing it is not as big of a deterrent as it is with other elements of the housing model. This naturally leads to plastic, aluminum, or fiberglass. The final choice will depend on the pricing of such materials when the time to fabricate the formwork comes. Secondly, to address the issues concerning consolidation, the formwork design has incorporated a tiered system for the columns. As shown in the rendering in Figure 5.25, one side of the column is split into three separate panels. This allows the column to be cast in three lifts, making consolidation easier to ensure at the bottom of the column. Formwork systems similar to this have been used in Malaysia, such as the one pictured in Figure 5.26, by the company TAC Contracts Sdn Bhd.
Figure 5.25: Column casting sequence, utilizing reusable formwork.
5.8.3 Prefabrication and Paraskilling

Chapter 4 touched on the use of prefabrication and paraskilling in order to elevate the level of quality currently in the Haitian housing sector. The practical implementation of these two theories will take on many forms throughout the construction process of the proposed housing model. The practices are best used in tandem, with the paraskilling of tasks directly related to the prefabrication of certain elements of the home. There is one element of the proposed model that will adopt both of these practices – the steel reinforcement cages. These cages will be bent, tied, and outfitted with threaded heads, at an off-site facility by a crew specifically trained in
these tasks. The panel production, which will be done in a tilt-up method, will adopt paraskilling, with one crew responsible for casting and attaching the panels on-site. The erection of the formwork and subsequent casting of the beams and columns will be done by another crew, specifically trained in this task. A general construction crew will carry out foundation and roof construction, and finishes, as these are tasks that are already familiar to most crews. The following subsections will outline how prefabrication and paraskilling will specifically be applied to the tasks mentioned.

5.8.4 Steel Reinforcement Cages

There are two dominant deficiencies in the columns of the housing system employed currently in Haiti: (1) they are undersized and (2) they are underreinforced. Properly sized and detailed steel reinforcement cages are essential to solving both problems. In light of this, it was determined that it is best that the cages are prefabricated. The prefabrication process allows the cage assembly to occur in a highly controlled environment, with access to tools to ensure proper bends and spacing, as well as maintaining the quality and integrity of the steel. The paraskilling process will be broken into three distinct steps. One crew will be solely responsible for the task of creating and assembling the cages. The first stage will be bending the steel confinement hoops. These hoops, in large part, determine the cross sectional dimensions of the beams and columns. Templates will be provided for these hoops, one for columns and

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4 Note: This is not referring to “underreinforced” in the reinforced concrete behavior sense, but rather that there was simply an inadequate amount of reinforcement.
one for beams (Figure 5.27). These templates ensure that the task is repeatable and provide the high quality level needed. Once these hoops are bent, cage assembly is the next task. This involves fishing the longitudinal steel through the hoops, all at the proper spacing, and tying them together. The spacing of the hoops is critical, especially near the ends of the columns and beams, as this confinement steel provides ductility in these high moment regions. In order to ensure the proper spacing, another type of template will be used. This template will consist of slots, into which the hoops are positioned, with the slots preconfigured to correspond to the proper spacing introduced earlier in this chapter (Figure 5.28). Once these hoops are in position, the longitudinal steel can be fished through, and the subsequent ties made.

Finally, the steel heads are attached. The longitudinal steel will need to be firstly threaded, then the heads attached. The threading of the bars requires a special type of machinery, most likely not currently available in Haiti. This machine will require training for the local work force to use. Once the longitudinal bars are threaded, heads are attached (see Figure 5.29 for an example) and the cages are complete and ready to be cast into the foundation.
Figure 5.27: Reinforcing ties template. The holes are drilled to ensure that the size of the tie is correct. The bolts are used in an interchangeable manner as leverage points to bend the reinforcing steel, and then subsequently hold in place.
Figure 5.28: Steel reinforcement cage template and assembly sequence.
5.8.5 Tools and Equipment

In addition to training, processes, and procedures, achieving a high level of quality in construction is also predicated on having the appropriate tools and equipment. The Haitian construction sector, especially the residential subgroup, lacks many of the most important tools and equipment. Therefore the proposed model must be accompanied by investment in tools and equipment that meet the unique constraints of Haiti, but also allow the construction crews to carry out their job effectively. Some of the tools needed have already been covered previously, as they relate to the many elements in the proposed model. However they will be listed here for thoroughness, with an important additional piece of equipment:

R1. Reusable concrete frame formwork
R2. Reusable steel reinforcement hoop bending template
R3. Reusable steel reinforcement cage tying template
R4. Reusable concrete panel formwork
R5. Threading machine
R6. Portable mechanical concrete mixer
The first five have already been discussed, so the rest of this section will be dedicated to the last item listed. From research completed by (DesRoches, et al. 2011), it was shown that typical Haitian concrete only achieves a compressive strength of 1,300 psi. This is in large part due to poor ingredients, lack of mechanized mixing equipment, and a general lack of knowledge pertaining to the intricacies of concrete production. As shown in Figure 5.30, most residential concrete production is done on-site, mixed manually with shovels. Piles of aggregate, cement, and buckets of water are stockpiled, and proportions are based solely on instinct, with no formal measurement system. Concrete production quality is therefore based entirely on experience and workability, as opposed to a minimum strength standard. Investing in a mechanized system, that also has built-in paraskilling aids, could not only improve quality, but also increase efficiency and decrease cost.
Figure 5.30: Typical on-site, manual concrete production in Haiti.

Traditional concrete mixing systems, however, are not directly transferable to Haiti. Large-scale operations would be extremely costly to implement, not to mention the human capital that would have to be invested. Small batch mixers, used for D.I.Y.-type projects in the developed world are much more applicable, however they are not optimized for the constraints for the developing world. A few concrete equipment manufacturers are creating small batch mixers more tailored to the developing world, one of which will be used in the proposed housing model. The “Concrete MD”, created by Cart-Away Concrete Systems, Inc. is a 0.65 yd$^3$ small batch mixer, specifically designed to operate within Haiti’s construction sector. The mixer is shown in Figure 5.31. There are five prominent features that make this mixer appropriate for the residential construction sector in Haiti.
R1. Batching Buckets: The mixer is equipped with batching buckets, used to guarantee that proper aggregate and cement proportions are used. The mixer comes with a standard concrete mix design, reflected in the size of the batching buckets. Customizations can be made by simply prescribing amounts of aggregates and cement in denominations of batching buckets. This gives the construction crew a tool to measure their inputs and correlate those to concrete quality.

R2. Screen: In addition to the batch buckets, the mixer is also equipped with a screen, complete with a cement bag breaker. The screen ensures that the aggregate used in the mix is not too large, while both of these features also help to limit the number of cement bags that get thrown into a concrete batch; a common practice in Haiti.

R3. Mechanized Auger: Arguably the most important feature, the mixer is powered by either a gasoline or diesel engine. This power source operates an auger in the bottom of the mixer, which actually does the mixing. Compared to the typical shovel-based approach, this mechanized system creates much better consistency across batches, as well as a higher quality mix.

R4. Discharge Valve: The auger in the bottom of the mixer feeds into a discharge valve off of the side of the mixer. This discharge valve, combined with the two-way auger, allows buckets and wheelbarrows to be filled and transported to the appropriate location on the job site. Unlike other small batch mixers, this valve allows laborers to take the concrete in the batch size convenient for that particular job site.

R5. Wheels and Hitch: Finally, the mixer is on wheels, complete with a truck hitch. This makes the mixer portable between job sites, as well as within a job site itself. The unit is light enough to be picked up and wheeled across a job site by hand, decreasing further reliance on heavy equipment.
Figure 5.31: Concrete MD mixer from Cart-Away Concrete Systems, Inc. The mixer includes batching buckets used to ensure proper proportions (top left and right), a screen used to ensure proper aggregate sizing (bottom left), and a discharge valve designed for buckets (bottom right). (Photo Credit: Cart-Away Concrete, Inc.)
CHAPTER 6:
DESIGN AND CONSTRUCTION OF PROPOSED LIGHTWEIGHT REINFORCED CONCRETE PANELS

6.1 Introduction

Within the frame and panel system, there are a number of options for the material used to fabricate the panels, as discussed in Chapter 4. Although the solution space is varied, few options meet the constraints in Haiti, and specifically in Léogâne. While materials such as agricultural waste and recycled plastics have potential to be used in the future, the infrastructure to produce them is currently not in place. Therefore, the most viable option is concrete. In essence, concrete panels are simply a reconfiguration of the CMUs already so popular in Haitian construction. However, by making the panels non-structural elements, two advantages over CMUs are gained. Firstly, the panels can be much smaller in dimension and therefore lighter, and secondly, they do not have to meet the same strength requirements as load-bearing CMUs. This allows for the use of thin, lightweight concrete panels, reinforced with a simple steel mesh.

Due to the unique nature of these elements, how they behave is not well understood. The first part of this chapter will be dedicated to exploring the behavior of these panels under different particular loading cases, as well as detailing possible failure
modes. The insights gathered from these two analyses will then be used to identify and explore the variations of the parameters that can be controlled in finalizing the panel design.

6.2 Panel Layout

The panels, as initially proposed, are 96” (H) x 36” (W) x 1” (D). Both the width and depth can be varied as needed for weight and stiffness demands. The panels, when installed, bear on the foundation and connect at the top and bottom of the frame. There are three connections to each beam, consisting of a threaded bolt cast into the beam, which pass through a hole in the panels, and are constrained by two washers and a nut (Figure 6.1). The holes are oversized, for both constructability and to accommodate differential displacements as the frame deforms. The inside of the holes will be lined with a stainless steel sleeve, that will help to distribute any point stresses due to a bearing of the panel on the bolts.
6.3 Loads

The panels, although non-structural elements, undergo three types of loading: (1) gravity due to self-weight, (2) wind, and (3) seismic. This section will detail each loading type, to be used in the subsequent sections for exploring the various failure modes.
6.3.1 Gravity Loads

The only gravity loads acting on the structure are due to the self-weight of the panel. This load obviously depends both on the dimensions of the panel as well as the density of the concrete. For initial purposes, the dimensions given at the beginning of this section will be used and a concrete density of 135 lbs/ft³ will be used. The self-weight gravity load is 270 lbs under these assumptions.

6.3.2 Wind Loads

The non-structural panels fall into the *Components and Cladding* designation in ASCE 7-10. Therefore, the procedure outlined in Chapter 30, Section 30.4 was used to compute wind loads. The overall procedure is identical to that used for the frame analysis, except different external pressure coefficients are used, as well as different definitions of the windward, leeward, and end zones. All other parameters we identical to those used in the frame analysis. Table 6.1 shows the different zones, along with their external pressure coefficients (as defined by ASCE 7-10 Figure 30.4-1) and governing wind pressures.

<table>
<thead>
<tr>
<th>Zone</th>
<th>GCpf</th>
<th>Governing p (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 (Windward)</td>
<td>0.8</td>
<td>33.59</td>
</tr>
<tr>
<td>5 (Windward – End Zone)</td>
<td>0.8</td>
<td>33.59</td>
</tr>
<tr>
<td>4 (Leeward)</td>
<td>-0.9</td>
<td>-37.01</td>
</tr>
<tr>
<td>5 (Leeward – End Zone)</td>
<td>-0.95</td>
<td>-38.73</td>
</tr>
</tbody>
</table>
6.3.3 Seismic Loads

As explained in the previous chapter, the influence of the panels on the lateral stiffness of the structure is not fully known. Therefore, it is difficult to estimate the forces transferred to the panels from the frame. Using the connections detailed at the beginning of this chapter, including the oversized holes, it will be assumed that the effect of seismic loading relevant to the panels in the form of lateral deflections (interstory drift of the frame). Therefore, deflections of the roof level found from the time history analysis in the previous chapter will be used as the limit state under seismic loading for the panels. These drift demands are shown in Table 6.2.

<table>
<thead>
<tr>
<th>Frame Analysis Model</th>
<th>Maximum Interstory Drift (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare Frame</td>
<td>0.441</td>
</tr>
<tr>
<td>Model 1</td>
<td>0.041</td>
</tr>
<tr>
<td>Model 2</td>
<td>0.051</td>
</tr>
</tbody>
</table>

6.4 Panel Failure Modes

There are three classes of failures that the panels can undergo based on the proposed configuration. The first is out-of-plane flexure due to wind pressure, the second is edge failure due to the self-weight of the panel bearing on the foundation, and the third is a multitude of connection failures due to all three load types. This section will detail all failure modes and their governing parameters and equations.
6.4.1 Out-of-Plane Flexure

The flexural failure mode can occur under both positive and negative wind pressure acting on the panels. In order to understand the behavior of the panels under uniform pressure, the panel was modeled in SAP2000. Two models were created, one for the case that the panel lies in the middle of the wall and is only supported by the upper and foundation beams and another for the end case where the panel is supported by both beams as well as a column. The panel was modeled as an area element, specifically a thin plate shell with a 20x20 mesh. It was assumed, based on the connection detail illustrated previously in this chapter, that all connections and edge constraints be modeled as simple supports.

Figure 6.2 shows the first model, as well as the deformed shape under a uniform area load equal to the absolute maximum wind pressure listed in Table 6.1. As shown, the panel acts much like a simply supported beam. The maximum deflection reported by SAP2000 under these load conditions is 1.17”. If modeled as a simply supported beam, with thickness equal to 1” and width equal to 12”, the maximum deflection from the same loading is equal to 1.15”, a difference of 1.7%. Therefore, it can be reasonably stated that the analysis and design of the panel can be treated in the same fashion as a one-way slab, where a representative width can be chosen and then analysis and design completed in accordance with procedures used for beam flexural design.

Figure 6.3 shows the second model, as well as the deformed shape under a uniform area load equal to the absolute maximum wind pressure listed in Table 6.1. It also behaves as a one way slab would behave at with a constraint added for an edge
boundary. The maximum deflection under this case was 0.41”, which is less than the maximum deflection of the first model, thereby making the first model the governing case with respect to the design of a standardized panel.

![Figure 6.2: SAP2000 modeling of panel, case one. (Left) Undeformed shape of panel modeled as a thin plate shell element. (Right) Deformed shape of thin plate shell under uniform pressure. Notice the similarity to flexural behavior of a one-way slab.](image1)

There are two loading scenarios that must be considered in this failure mode: (1) positive wind pressure and (2) negative wind pressure. For this analysis, it is assumed
that the panel behaves identically in both cases, thus only the absolute maximum wind pressure will be used for this failure mode analysis. Utilizing an effective width of 12”, the maximum moment on the panel for the governing case is 3718 kip-in.

6.4.2 Bearing – Panel Edge

A bearing failure due to concrete crushing of the panel could occur at the interface of the panel and the foundation, under the self-weight of the panel.

6.4.3 Connections

There are a series of failure modes that could occur due to the connection detailing. Seven such failure modes are analyzed in this section.

6.4.3.1 Bearing – Panel Crushing

In addition to a failure induced by the self-weight of the panel on its bottom edge, should the panel not be in contact with the foundation, the self-weight would be supported by the bolt connections. If it assumed that either the upper or lower bolts would carry this load, the self-weight would be distributed over three bolts. The connection detail described earlier relied on the use of a stainless steel sleeve that will serve to distribute the point stresses of this type of bearing. For this analysis it will be assumed that an area equal to the panel thickness multiplied by one-eighth of the circumference of the panel hole (d = 1”) will bear these stresses.
6.4.3.2 In-Plane – Bolt Shear

The above self-weight could also cause a shear failure of the bolts. The panel self-weight for this failure mode would be distributed as a shear force over the cross-sectional area of three bolts.

6.4.3.3 In-Plane – Panel Shear Breakout

The self-weight of the panel could also cause shear concrete breakout failure in the panel, consisting of the bolt breaking out of the top edge of the panel.

6.4.3.4 Out-of-Plane – Panel Crushing

In the out-of-place direction, a negative pressure (i.e., suction) could cause a concrete crushing failure around the connections. In this case the load imposed by the wind pressure would bear on the washers on the exterior face of the panel. The maximum wind pressure, 38.73 psf, would cause a total force of 930 lbs on a panel in the end zone. Distributed over six connections, the load bearing on each washer of the connection is 155 lbs.

6.4.3.5 Out-of-Plane – Bolt Tensile

The same load as described in the out-of-plane, panel crushing failure mode could also cause tensile failure in the bolts. This would impart a tensile load of 155 lbs on each bolted connection.
6.4.3.6 Out-of-Plane – Bolt Pullout Beam

A third type of failure could be caused by out-of-plane loading described in the previous two sections, namely a pullout failure of the bolt from the beam. This would impart a load of 155 lbs on each bolted connection.

6.4.3.7 In-Plane Drift

The final failure mode relates to seismic loading. As stated in the seismic loading section of this chapter, the loading caused by a seismic event on the panels is difficult to theoretically quantify. However, given the oversized nature of the holes, as well as the stiffness contribution analysis done in Chapter 5, it is expected that the limit state for seismic loading will be deflection controlled. Therefore, the panels must be able to undergo the maximum expected interstory drift without failing. This maximum interstory drifts for the various frame and panel models was listed previously in Table 6.2.

6.5 Panel Design

The failure modes listed in the previous sections are a portion of the constraints on the panel design. The other portion is a product of the constructability of the panels in Haiti. Limited materials for the panels, connections, and formwork also influence the panel design. In particular, the limited availability of concrete ingredients heavily restricts the panel design. Dimensions can be manipulated rather easily to meet the limit states, however, concrete mix design is much less malleable. Since weight plays
such a large role in the panel design, strategies to lower the weight are important. However, typical strategies for lightweight concrete design, e.g. admixtures, pumice aggregates, etc., are not widely available in Haiti, particularly in Léogâne. When they are available, cost becomes a limiting factor to their implementation. Therefore, for the purposes of this research application, achieving a lightweight mix is very important. Due to material availability, this goal had to be met purely through a mix design using standard concrete ingredients. The design process for the panels was approached by first exploring the concrete strengths and densities that are possible, followed by dimensioning the panels to meet the limit states of the previous sections. The following sections will outline both parts of this design process.

6.5.1 Panel Concrete Mix Design

There are four parameters that can be manipulated to create a lightweight concrete mix in Haiti. These four parameters are proportion of water, proportion of cement, proportion of fine aggregate, and proportion of coarse aggregate. In addition to the aim of creating a lightweight mix, keeping the panel thickness at minimum is also a factor in reducing weight. This limits the dimension of coarse aggregate that may be used, as there will have to be adequate room for cover, as well as consolidation around the steel reinforcing mesh. Materials typical of those available in Haiti were used in a materials laboratory at the University of Notre Dame to trial mix designs and arrive at a final design. The following table (Table 6.3) details the properties of the material ingredients used in this mix design process.
Mix designs were created and tested in the materials laboratory at the University of Notre Dame. Batch volumes of 0.12 ft³ were used. Trial cylinders were made and tested using the guidelines in ASTM C192/C192M – 13: “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory” and ASTM C39/C39M - 12a: “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens”, respectively. Three-inch (diameter) by six-inch (height) cylinders were used for compressive strength tests. Testing was done on Instron SATEC Systems equipment in the University of Notre Dame Materials Laboratory, following all relevant guidelines and procedures outlined in ASTM C39/C39M.

As shown in Table 6.3, the two ingredients with the highest densities are cement and coarse aggregate. Therefore, it logically follows that reducing the proportions of these two ingredients, while increasing the proportions of fine aggregate and water, will produce a lighter concrete. However, increasing water content and decreasing cement content increases the water-to-cement ratio, usually correlating to strength reduction. Therefore a balance had to be struck between reducing the densest ingredients, while still maintaining strength adequate for the panel elements. Based on this general
strategy, three trial mixes were created as a starting point for the panel concrete design.

These three mixes are outlined in Table 6.4 below, with weight and volumes representative of a 0.12 ft³ batch. As the table shows, Mix 1 and Mix 3 completely eliminated the coarse aggregate in an attempt to decrease weight. Mix 2 and Mix 3 greatly increased water content and decreased cement content with the same goal in mind. There is a range in densities between the three mixes, with Mix 3 being the lightest and Mix 2 being the heaviest. Although Mix 2 is in the weight range of typical reinforced concrete, it provides a benchmark for the group in terms of strength versus weight comparisons.

**TABLE 6.4:**

**BATCH WEIGHTS FOR INITIAL PANEL CONCRETE MIX DESIGNS**

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (lbs)</td>
<td>16.55</td>
<td>6.07</td>
<td>7.45</td>
</tr>
<tr>
<td>Fine Aggregate (lbs)</td>
<td>1.70</td>
<td>10.10</td>
<td>6.63</td>
</tr>
<tr>
<td>Coarse Aggregate (lbs)</td>
<td>0.00</td>
<td>21.14</td>
<td>0.00</td>
</tr>
<tr>
<td>Water (lbs)</td>
<td>3.22</td>
<td>1.95</td>
<td>2.56</td>
</tr>
<tr>
<td>Total Weight (lbs)</td>
<td>21.47</td>
<td>39.26</td>
<td>16.64</td>
</tr>
<tr>
<td>Mix Density (lbs/ft³)</td>
<td>146.75</td>
<td>157.26</td>
<td>138.71</td>
</tr>
<tr>
<td>W/C Ratio</td>
<td>0.19</td>
<td>0.32</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 6.5 shows the results of the cylindrical compressive testing completed on the above three mix designs. The average maximum stress, standard deviation, and average 28-day strength were all calculated using the three cylinders tested for each mix design. As the results show, Mix 3 was by far the strongest of the mixes, but also the
lightest. This made it a natural candidate for iterations to further balance weight versus strength.

**TABLE 6.5:**

INITIAL PANEL MIX DESIGN COMPRESSIVE STRENGTH TEST RESULTS

<table>
<thead>
<tr>
<th>Test Results</th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Maximum Stress (psi)</td>
<td>1,920.10</td>
<td>1,943.20</td>
<td>7,936.70</td>
</tr>
<tr>
<td>Standard Deviation (psi)</td>
<td>398.10</td>
<td>174.30</td>
<td>690.99</td>
</tr>
<tr>
<td>Average 28-Day Strength (psi)</td>
<td>2,743</td>
<td>2,776</td>
<td>11,338.14</td>
</tr>
</tbody>
</table>

Two alternative mix designs were created with Mix 3 as the baseline. These two mix designs are shown in Table 6.6. In both designs, the cement content and fine aggregate content were lowered, while the water content was increased. This brought the water-to-cement ratios for both mix designs rather high for typical concrete mixes. Additionally, as in the original Mix 3, no coarse aggregate was used. The densities in both designs were lower than the original Mix 3, with Mix 3 V3 having the lowest density at approximately 131 pcf. Table 6.7 details the results of the compressive tests for the iterated designs. The following section will use the three iterations of Mix 3, combined with the failure mode limit states, to create an array of panel dimensions that meet the criteria. This array will then be analyzed for trade-offs between strength and weight.
TABLE 6.6:

BATCH WEIGHTS FOR PANEL MIX DESIGNS – MIX 3 ITERATIONS

<table>
<thead>
<tr>
<th>Batch Weight (lbs)</th>
<th>Mix 3 V2</th>
<th>Mix 3 V3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>6.98</td>
<td>6.78</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>6.20</td>
<td>6.02</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Water</td>
<td>2.89</td>
<td>3.01</td>
</tr>
<tr>
<td>Total Weight</td>
<td>16.07</td>
<td>15.81</td>
</tr>
<tr>
<td>Mix Density (lbs/ft³)</td>
<td>133.72</td>
<td>131.75</td>
</tr>
<tr>
<td>W/C Ratio</td>
<td>0.41</td>
<td>0.44</td>
</tr>
</tbody>
</table>

TABLE 6.7:

MIX 3 ITERATION PANEL MIX DESIGN COMPRESSIVE STRENGTH TEST RESULTS

<table>
<thead>
<tr>
<th>Test Results</th>
<th>Mix 3 V2</th>
<th>Mix 3 V3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Maximum Stress (psi)</td>
<td>5873.00</td>
<td>4818.80</td>
</tr>
<tr>
<td>Standard Deviation (psi)</td>
<td>1538.90</td>
<td>1085.60</td>
</tr>
<tr>
<td>Average 28-Day Strength (psi)</td>
<td>8390.00</td>
<td>6884.00</td>
</tr>
</tbody>
</table>

6.5.2 Panel Dimensioning

The panel thickness is a parameter that can be easily manipulated to meet the limit states. Therefore, this is a natural starting point in panel design. The thickness is directly related to the moment of inertia, and therefore the flexural failure mode was explored first. Note that even though the panels will be reinforced with a steel mesh, because of their thin nature, the mesh will have to be placed at the center of the panel. In terms of flexural resistance, the mesh will contribute negligibly. As such, for the panel flexural design, it is assumed that the concrete will be the only material resisting tension.
forces (i.e., no reliance on the steel mesh). It was assumed that the tension strength of concrete is 10\% of compressive strength.

Using the three mix designs in the previous section, it is possible to find the needed moment of inertia to resist the maximum flexural moment on the panel, discuss in the previous section By approaching the design of the panel in flexure like the design of a one-way slab, the width of the effective beam is fixed at 12”\,. Therefore the only variable to solve for is the thickness of the panel. Table 6.8 lists the resulting thicknesses, moments of inertias, and panel weights based on the maximum moment and concrete strengths (from the last section).

**TABLE 6.8:**

<table>
<thead>
<tr>
<th>Mix Design</th>
<th>28-Day Compressive Strength (psi)</th>
<th>Tension Strength (psi)</th>
<th>Maximum Moment (lb-in)</th>
<th>Panel Thickness (in)</th>
<th>Moment of Inertia (in^4)</th>
<th>Panel Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix 3</td>
<td>11,338</td>
<td>1,134</td>
<td>3721</td>
<td>1.29</td>
<td>2.15</td>
<td>358</td>
</tr>
<tr>
<td>Mix 3 V2</td>
<td>8,390</td>
<td>839</td>
<td>3721</td>
<td>1.50</td>
<td>3.38</td>
<td>401</td>
</tr>
<tr>
<td>Mix 3 V3</td>
<td>6,884</td>
<td>688</td>
<td>3721</td>
<td>1.65</td>
<td>4.49</td>
<td>435</td>
</tr>
</tbody>
</table>

Due to the practical restrictions on production capabilities in Haiti, it is not reasonable to assume that panels could be made to a precision of a tenth of an inch. It is assumed that a precision of half inch is possible. Therefore, Mix 3 and Mix 3 V2 would be sized at 1.5” to make them viable, and Mix 3 V3 would be sized at 2”\,. This changes
the weights of the panels, and actually makes Mix 3 V2 the most efficient design with a weight of 401 lbs. Mix 3 sized at 1.5” would weigh 416 lbs, and Mix 3 V3 sized at 2” would weigh 527 lbs. The panels will be constructed in a tilt-up fashion, meaning that the 401 lb panel may be manageable by four men. Further iteration, and possible use of admixtures, will be discussed in Chapter 7. For the purposes of this design, this will be the assumed panel concrete mix and thickness.

6.5.3 Limit State Checks

The following sections will check the above panel design with the governing failure modes. The material properties and dimensions listed in Table 6.9 will be used for these checks.

<table>
<thead>
<tr>
<th>Material</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength (f′c)</td>
<td>8.4 ksi</td>
</tr>
<tr>
<td>Length (lp)</td>
<td>96”</td>
</tr>
<tr>
<td>Width (wp)</td>
<td>36”</td>
</tr>
<tr>
<td>Thickness (tp)</td>
<td>1.5”</td>
</tr>
<tr>
<td>Modulus of Elasticity (Ep)</td>
<td>4,636 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Stainless Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Tension Strength (fu)</td>
<td>90 ksi</td>
</tr>
<tr>
<td>Yield Tension Strength (fy)</td>
<td>42 ksi</td>
</tr>
<tr>
<td>Ultimate Shear Strength (fs)</td>
<td>48 ksi</td>
</tr>
<tr>
<td>Bolt Diameter (db)</td>
<td>0.50”</td>
</tr>
<tr>
<td>Washer Inside &amp; Outside Diameter (dw1, dw2)</td>
<td>0.50”, 2.00”</td>
</tr>
<tr>
<td>Modulus of Elasticity (Eb)</td>
<td>28,000 ksi</td>
</tr>
</tbody>
</table>
6.5.3.1 Bearing – Panel Edge

The bearing capacity of the panel is equal to its compressive strength, \( f_c' \). The bearing stress caused by the panel is its self-weight divided by its cross-sectional area, or 7.4 psi, which is less than the compressive strength of 8.4 ksi. Therefore, this failure mode is mitigated.

6.5.3.2 Connection: Bearing – Panel Crushing

The maximum bearing stress on the concrete surrounding the connections, assuming that all the panel self-weight is bearing, is 340 psi. The compressive strength of the concrete is 8.4 ksi, therefore mitigating this failure mode.

6.5.3.3 In-Plane – Panel Shear Breakout

The maximum shear force caused by the panel self-weight on the three upper connections is 133 lbs. The nominal concrete breakout strength in shear, as defined in ACI 318-11, is governed by the following equation:

\[
V_{cb} = \frac{A_{vc}}{A_{vco}} \psi_{ed,v} \psi_{c,v} \psi_{h,v} V_b \quad \text{Equation 6-1}
\]

where \( \psi_{ed,v} \) is the modification factor for edge effects, \( \psi_{c,v} \) is the modification factor for the presence of cracking, and \( \psi_{h,v} \) is the modification factor for member thickness. \( A_{vc} \) is the projected cross-sectional area of failure, \( A_{vco} \) is the same projected area for a deep member, and \( V_b \) is the basic concrete breakout strength. The following equations are those used to calculate these parameters.
where \( c_{a1} \) is the distance from the connection to the edge of the member, in the same direction of the shear force and \( c_{a2} \) is the distance to the edge perpendicular to \( c_{a1} \).

\[
\psi_{c,v} = 1.4 \quad \text{Equation 6-3}
\]

for members that are uncracked under service loads, which applies to the panels.

\[
\psi_{h,v} = \sqrt{\frac{1.5c_{a1}}{h_a}} \quad \text{Equation 6-4}
\]

for a member where \( h_a < 1.5c_{a1} \), where \( h_a \) is the thickness of the member.

\[
A_{vc} = 2(1.5c_{a1})h_a \quad \text{Equation 6-5}
\]

\[
A_{vco} = 4.5(c_{a1})^2 \quad \text{Equation 6-6}
\]

The basic concrete breakout strength, \( V_b \), is the smaller of Equations 6-7 and 6-8:

\[
V_b = \left( 7 \left( \frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \right) \lambda_a \sqrt{f_c'(c_{a1})} \quad \text{Equation 6-7}
\]

\[
V_b = 9\lambda_a \sqrt{f_c'(c_{a1})}^{1.5} \quad \text{Equation 6-8}
\]

where \( l_e \) is the embedment length, \( d_a \) is the diameter of the bolt, and \( \lambda_a \) is the modification factor for lightweight concrete.

Based on the above equations the breakout strength in shear of the panel, \( V_{cb} \), is 4,410 lbs, which is greater than the maximum shear load of 133 lbs, and therefore, this failure mode is mitigated.
6.5.3.4 Connection: In-Plane – Bolt Shear

The maximum shear stress on the bolts from panel self-weight is 680 psi. The shear strength of the stainless steel bolts is 48 ksi, which is greater than this maximum shear stress, thereby mitigating this failure mode.

6.5.3.5 Connection: Out-of-Plane – Panel Crushing

The maximum bearing stress on the concrete panels (transferred by the washers of the connections) at the connections due to a negative wind pressure is 353 psi. The maximum compressive strength of the panels is 8.4 ksi, which is greater than the maximum stress, thereby mitigating this failure mode.

6.5.3.6 Connection: Out-of-Plane – Bolt Tensile

The maximum tensile load transferred to the six bolts due to a negative wind pressure causing the panel to bear on the connections out-of-plane is 5.3 ksi. The ultimate strength of the bolts is 90 ksi and the yield strength is 42 ksi, which are both greater than the maximum tensile stress, thereby mitigating this failure mode.

6.5.3.7 Connection: Out-of-Plane – Bolt Pullout Beam

The maximum pullout force caused by a negative wind pressure is equal to 1,038 lbs. The concrete breakout strength of a anchor in tension in the beam is equal to:

\[ N_{cb} = \frac{A_{nc}}{A_{nco}} \psi_{ed,n} \psi_{c,n} \psi_{cp,n} N_b \]  
Equation 6-9
where $\psi_{ed,n}$ is the modification factor for edge effects, $\psi_{c,n}$ is the modification factor for the presence of cracking, and $\psi_{cp,n}$ is the modification factor for post-installed anchors (not applicable here). $A_{nc}$ is the projected cross-sectional area of failure, $A_{nc0}$ is the same projected area for a deep member, and $N_b$ is the basic concrete breakout strength. The following equations are those used to calculate these parameters.

$$\psi_{ed,n} = 1.0 \quad \text{Equation 6-10}$$

where $c_{a,min}$, the minimum distance from the connection to the edge of the member, is greater than $1.5h_{ef}$, where $h_{ef}$ is the embedment length.

$$\psi_{c,v} = 1.25 \quad \text{Equation 6-11}$$

for cast-in anchors.

$$\psi_{cp,v} = 1.0 \quad \text{Equation 6-12}$$

for cast-in anchors.

$$A_{nc} = (c_{a1} + 1.5h_{ef})(2 * 1.5h_{ef}) \quad \text{Equation 6-13}$$

$$A_{vco} = 9(h_{ef})^2 \quad \text{Equation 6-14}$$

The basic concrete breakout strength, $N_b$, is:

$$N_b = k_c \lambda_a \sqrt{f'_c(h_{ef})^{1.5}} \quad \text{Equation 6-15}$$

where $k_c$ is 24 for cast-in anchors.
Based on these parameters, the breakout strength in tension for the anchor bolts is 6,500 lbs, which is greater than the maximum tension force of 1,038 lbs, and therefore this failure mode is mitigated.

6.5.3.8 In-Plane Drift

The interstory drift is 0.441”, under the bare frame model. The actual interstory drift will lie somewhere between this value and that of Model 2, which was 0.051”. The connection detailed previously has an oversized hole with a diameter of 1”, thereby allowing for a movement of 0.25” in either direction before the bolt engages the panel. Assuming that the frame will act as a bare frame until the panels are engaged (a deflection of 0.25”), and will act as Model 2 once engaged, the panels will have to undergo 0.051” of deflection once in contact with the connections. Interstory drift limit considerations are discussed in Appendix C of ASCE 7-10 for non-structural components. Recommendations are to limit drifts to 1/600 to 1/400 of story height. In this case, that means drift should be limited to 0.17” to 0.25”. This puts the connection at the very upper boundary of deflection limits for the frame. Damage to the panels could be reduced by slotting the oversized holes for the connections to make sure to mitigate this failure mode.

6.6 Panel Production

The panels can be constructed in much the same way as the currently popular CMUs – locally and by professionals specifically trained in the production process. There
are two main stages to their production, which will be performed on-site by a crew
solely responsible for these tasks. Due to the weight of the panels and the
unpredictability of road and travel conditions, it is recommended that the panels be cast
on-site, utilizing a process known as tilt-up construction. This process is used widely in
the commercial sector in the developed world, and is based on a two-stage process.
Firstly, the panels are cast with a form, flat on grade, next to the location where they
will eventually be attached to the frame. Once the panels have cured, they are “tilted”
into place and an attachment is made. Although this usually involves large-scale panels
and heavy machinery in the developed world, the same principle will apply to the
proposed housing model. Panels will be cast on-site, and then tilted into place, using
manual labor. As the panels are tilted into place, the slots in the top of the panel will
overlap the bolts, and a washer and nut will be used to secure it into place. The entire
process is depicted in Figure 6.4.
6.6.1 Panel Formwork

There are three major panel formwork design objectives. The first is ensuring that the mesh remains flat throughout the length of the panel, the second is creating the oversized, slotted holes for the connections, and the third is creating a system to remove the formwork once the panel is cast. This section will discuss potential formwork designs to meet these objectives.

The proposed model is a three-element formwork. The first element is a bottom that has attached to it studs that will create the oversized holes in the panel for the connections, as well as four pop-out panels that will be used to detach the formwork once the panel is cast (Figure 6.5). As the panel is tilted into place, these panels will be
removable from the back, allowing the laborers to punch out the panel from the formwork and onto the bolt connections on the frame.

The second element is similar to a picture frame. This layer serves as a form to cast the first half of the thickness of the panel. On this layer (Layer 1 in Figure 6.5) are studs that serve as anchors for the mesh. These studs allow the mesh to be attached at one end, then stretched over the length of the panel, ensuring that it remains in place. This layer is placed over the bottom (Figure 6.6), the first half of the thickness of the concrete is cast, and the mesh is then laid over the mesh anchors. Finally, a second border is placed over the first (Layer 2 in Figure 6.5), allowing for the remainder of the panel to be cast.

Figure 6.5: Panel formwork elements: plan view.
6.6.2 Panel Reconfiguration

Another feature that is important to making the proposed model as modular and progressive as possible is the ability to reconfigure the panels to create different spaces within the house. Due to the weight of the panels, a construction crew would be called upon to do such reconfiguration. Therefore, it would be beneficial to create a system to handle the panels as they are moved. A proposed tool is shown in Figure 6.7.
Figure 6.7: Panel reconfiguration tool. Consists of a long handle with three studs that fit into the panel connection holes. Stoppers are used to hold the handle on the panel.

The proposed tool is a long metal handle, with three studs. These studs are spaced to match the dimensions of the holes in the panels. There are threaded stoppers on the ends of the studs, which serve to keep the panel on the handle once it is lifted. These tools are attached to the panel as shown in Figure 6.8, and make the job of reconfiguring the panels much easier for the construction crew.

Figure 6.8: Panel reconfiguration tools as attached to panels: (left) front side of the panel, showing the handle and (right) back side of the panel, showing the threaded stoppers.
CHAPTER 7:
FUTURE WORK AND CONCLUSIONS

7.1 Introduction

This thesis outlined the framework for a new urban housing model in Haiti and resulted in the design of a novel reinforced concrete frame and panel system. Focusing on the engineering aspects, there are three major areas of research that could be the subject of future investigation: panels, foundations, and connections. Each of these areas currently has a preliminary design (all discussed in this thesis), however, improvements in both engineering and constructability warrant further research. For purposes of this particular research project, the approach to investigating these topics will be two-fold. The first step will be to prototype the current design, as presented in this thesis. The process of prototyping will shed light on areas on improvement in both engineering and constructability. Several of the iterations outlined in Chapters 5 and 6 will be used in the prototype, in order to understand which is most effective, and which can be the subject of further research. The construction process will also be reviewed, particularly in the context of what is possible in Haiti, as outlined in Chapters 3 and 4.

This chapter will explain the prototype concept, discuss the three areas of research mentioned above, and finally draw some conclusions to this thesis in its entirety. As the earlier part of the thesis has demonstrated, an understanding of market, capacity, and

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cultural constraints is crucial. As such, it is important to note that any future work should also revisit these issues in order to ensure the sustainability of any new proposed solution.

7.2 Campus Prototype

In order to gain a better understanding of the structural system and its practical implementation, it was decided that a partial prototype would be constructed on the campus of the University of Notre Dame. The process of constructing the prototype would shed light into areas available for improvement both in the design and the implementation of that design. Figure 7.1 illustrates the partial prototype concept. It was decided to construct a partial prototype in the interest of time and cost. The portion chosen for construction encompasses the most critical areas of the design, still granting the ability to analyze the advantages and disadvantages of the proposed system. The prototype is an L-shape, representative of one corner of the proposed model. Each side of the prototype will be outfitted with four full-size panels, as shown in Figure 7.1. The frame will include a freestanding foundation, consisting of footings at the base of the columns, and connected through a foundation ring beam. The following sections will discuss major elements of the prototype.
7.2.1 Foundation

The prototype foundation will be cast above ground, in order to fully understand the best detailing options. The detail will include footings at the base of each column. These footings, which typically would be below grade, will be shallow footings, due to the high water table that is present in Haiti. The footing itself will be 10” deep and 20” square in plan. These will be cast above grade for the purposes of the prototype, on top of an existing concrete slab. In addition to the footings, the foundation ring beams will also be cast. These beams serve two primary purposes. Firstly, they are the element that ties the foundation of the structure together. Due to the high water table, the foundations must be limited in depth (no more than three feet). As such, with the seismic risk present in Haiti, differential deformations and settlements are of concern.
The foundation ring beam ensures that the bases of the columns are tied together, much in the same way that the upper beams constrain the columns. This mitigates the risk of differential deformations, and ensures that the system-wide behavior discussed in Chapter 5 is maintained.

7.2.2 Formwork for Frame

Due to cost limitations and the customization needed for the partial prototype, the formwork has been constructed from wood. Three column forms and two beam forms were built in order to cast a monolithic frame. The columns will simply sit on the slab where the frame is cast, and the beams will likewise rest on top of the columns. The formwork will be outfitted with the bolt connections needed for the upper panel attachment. The formwork was built from a combination of ½” plywood and 2x4s. Each form had polyurethane applied and will also be coated in form release to ensure a high quality finish on the frame. Figure 7.2 shows an example of the wooden formwork for a beam.
Figure 7.2: Wooden formwork constructed for the beams of the campus partial prototype.

7.2.3 Panels

The prototype will allow for experimentation with many aspects of the panels and their interactions with the frame. Three aspects of the panels will be experimented with across the eight panels that will be needed for the prototype. Mix design, formwork design, and the process of panel reconfiguration will all be varied across the eight panels to find the best practices for each aspect. Although all three of these have already been designed, the prototype will allow those designs to be field tested and iterated, creating opportunity for improvements.

Two parameters will be explored in panel design, both in an attempt to optimize the strength-to-weight ratio. The two parameters are the use of lightweight filler
materials and chemical admixtures. The lightweight filler materials will be used to reduce the density of the concrete, and therefore the weight, and the chemical admixtures will be used to replace cement (the densest of the concrete ingredients) in order to maintain strength while reducing weight. These variations will allow one of two things to occur. Either the panel concrete strength remains constant, but the weight reduces, allowing us to have a panel of the same thickness but lighter, or the concrete strength increases, allowing for a thinner, and therefore lighter, panel. In both cases, the goal of reducing panel weight for constructability reasons is achieved.

Three issues will be explored for the panel formwork. Firstly, the design of the formwork to ensure that the mesh lies flat and in place will be iterated upon. Secondly, the mechanism used to tilt the panel up from the ground to the frame (whether this be in the formwork or not), will be investigated. Finally, the way in which the formwork is stripped from the panel, in order to be able to reuse the formwork will be researched.

Finally, panel reconfiguration will be further investigated, in conjunction with the two previous aspects of the panel casting and installation process. The way that the panels are reconfigured will depend both on the weight and size of the panel, as well as how the reconfiguration process must be included in the formwork design. Additionally, the prototype allows the research team to analyze how the systems (a.k.a. foundation, frame, panels) interact with each other, especially in terms of the construction process, standardization, prefabrication, and paraskilling.
7.3 Future Research

In addition to the investigations that will be done for the purposes of the prototype, future research could also be completed in the areas of shallow foundations and frame-panel interaction.

7.3.1 Shallow Foundations

Due to the extremely high water table in Haiti, particularly in the city of Léogâne, typical foundations for residential structures are not viable with the resources at hand. The current foundation type is a simple bed of rock and mortar, with a concrete slab topping. Due to the frame system in the proposed model, this type of foundation, although probably sufficient for gravity loads, does not address the lateral loading condition and in particular, the effects of seismic loading with regard to differential deformations. Further investigation into other small structure shallow foundations, followed by a proposed design for the proposed above-grade structure detailed here would further advance this research.

Not only is the investigation into the structural aspect of the foundation important, but cost to the homeowner also becomes very important. An on-grade slab is a considerable portion of the cost of the home, however, if the current foundation type is used, it can be necessary before further construction on the frame can begin. Creating a foundation system that can also be staged; similar to the rest of the house is ideal, as it allows the homeowner to build the frame as fast as possible. Since the frame can provide temporary shelter as the rest of the home is completed, this is a very important
milestone to reach as quickly as possible. A foundation design that can marry this
constraint with the engineering constraints could improve the effectiveness of this
housing model.

7.3.2 Frame-Panel Interaction

Chapter 5 began a discussion on the interaction of the frame and panel system,
especially as it affected the lateral stiffness and modal period of the structure. Two
models were offered to model the panels, although as noted, these models are based
on the modeling done in the past of infill masonry and concrete panels. Because of the
unique nature of these panels, namely that they are attached outside of the plane of the
frame, the infill models are not entirely accurate. The expected behavior of the frame-
panel composite system is assumed to lie somewhere between the bare frame and the
models explored in this thesis, however, it is unclear precisely where that behavior falls.
Further analytical and possibly experimental research into the interaction of these two
elements, especially as it relates to the connection type, would increase the
understanding of the system, and therefore possibly allow for a more efficient design,
ultimately reducing costs. This type of investigation could aid in making the housing
model more widely available, as well as produce new connection types that may
improve the constructability of the system.

7.4 Conclusions

The objective of this thesis was to propose a housing model that met not only
the engineering constraints of residential urban housing in Haiti, but also the economic,
capacity, and cultural constraints as well. The objectives laid out in Chapter 1 were organized into four core areas that reflect these constraint categories: resiliency, sustainability, feasibility, and viability. This section will restate those objectives and review those that were achieved in this thesis. The objective achieved will be reported throughout the narrative in parentheticals. Note that, as stated in Chapter 1, some of these objectives, in particular those dealing with sustainability, are the subject of another thesis by the author, reference to which can be found in the bibliography.

7.4.1 Resiliency Objectives

R1. Understanding of the current urban housing stock, their properties, and vulnerabilities to natural hazards.

R1. Spectrum of viable materials that could be used for different elements of the structure.

R2. Selection of possible structural systems, including materials, dimensions, and reinforcement details based upon modeling and capacity evaluation.

7.4.2 Sustainability Objectives

R3. Discuss a potential plan for creating and seeding new industries seeded by alternative building material production.

R4. Cost breakdown of proposed urban housing model and research proving its availability to the informal housing market demographic.

R5. List of possible funding mechanisms for the procurement of materials and labor required to build a house conforming to the proposed paradigm.

7.4.3 Feasibility Objectives

R1. Inventory of technical skill set capacity of Haiti, specifically of the city of Léogâne.

R2. Urban housing paradigm that utilizes and expands pre-existing skills and local capacity.
R3. Set the foundation for training and education program to disseminate knowledge that enhances technical skill sets, including quality control, inspection, and basic seismic and hurricane resistant design/construction practices.

7.4.4 Viability Objectives

S1. Structure for community engagement and feedback that values the opinions and desires of the Léogâne community, while also providing an avenue through which the proposed paradigm can be explained and understood by the Haitians from a technical perspective.

S2. An urban housing solution that fits into the Haitian history and culture and becomes utilized, embraced, and promoted in the future.

7.4.5 Thesis Work

Firstly, the 2010 Haiti Earthquake was studied in detail, complete with on-the-ground reconnaissance work. This reconnaissance work provided valuable information as to the vulnerabilities present in Haitian housing (R1), the construction practices and material constraints that create them (F1), and the economic and cultural influences that perpetuate them. Based on the information gathered from this reconnaissance, a problem statement was created, built upon the four types of constraints named previously. Further investigation into these constraints and how a solution could be crafted while meeting them called for the creation of an evaluation process and rubric (R2). This process consisted of community engagement, again in Haiti, and produced a list of attributes and constraints that any proposed housing model in the future could be judged against (V1). This rubric was then detailed with a scoring system that provided an objective way to compare housing models.

The outcomes of the reconnaissance work and evaluation rubric led to the proposal of a novel frame and panel residential urban housing model (R3). The
remainder of the thesis was devoted to moving the model from concept to design, and finally to a construction process. Chapter 5 of this thesis explained the loading scenarios present in Haiti and the design process for the reinforced concrete moment resisting frame. It explored two types of joint details, a traditional detailing, as well as a headed bar detailing. The headed bar detailing was explored for its practicality and ability to improve the standardization of the model (F2, F3). The chapter concluded with a discussion and investigation into the influence of the panels, considered non-structural elements, on the lateral stiffness of the frame. A series of simulated ground motions from the 2010 earthquake were used to evaluate different models for representing the influence of the panels. Both models were based on infill masonry and concrete models, leading to overly conservative results, but providing a benchmark and range within which the behavior of the proposed system could be placed (R3).

Chapter 6 explored the design of the panels themselves. The design approach was based on the expected loadings and failure modes. Several failure modes were considered, although the out-of-plane flexural failure governed. The panel design was an exercise in balancing strength versus weight, as the panels must remain light so they can be manually installed in Haiti (F2, F3). A panel design was completed, as well as a connection detailing, which fits all the engineering constraints, although it seems improvements could be made in order to further optimize the strength-to-weight ratio.

Chapter 7 concluded with future work and research that could be built upon this thesis work. The next step is the construction of a partial prototype, in which multiple issues will be explored. Foundation design, frame formwork, and panel design and
formwork will all be prototyped, some in multiple configurations to find what is most
effective in terms of constructability. Further research topics could also include shallow
foundation design, as well as a deeper look into the interaction between the frame and
the panels, and the effects that both have on the other.

In conclusion, a novel urban housing model for the developing world has been
presented, catering to the engineering, economic, capacity, and cultural constraints at
hand. The model is not only built on engineering and construction expertise, but also
upon knowledge of the societal factors that play a role in housing (V2). The model,
although the subject of much time and research at the University of Notre Dame, will
ultimately only survive if accepted, embraced, and implemented by the community in
Haiti. The purpose of including all of the additional constraints usually ignored by the
international community that addresses housing in the developing world is to ensure its
long-term sustainability. Empowerment is baked into the design process, producing an
end product that is not only resilient, but also sustainable, feasible, and viable in the
local community.
APPENDIX A:

RECONNAISSANCE TOOLS AND AIDES

A.1 Haiti Earthquake Damage Assessment Form

<table>
<thead>
<tr>
<th>Reference no.</th>
<th>Date/Time</th>
<th>Structure name</th>
<th>Location/Address</th>
<th>GPS position</th>
<th>Function</th>
<th>Primary Structural System</th>
<th>In Fill Wall Type</th>
<th>Roof Type</th>
<th>No of Stories Above Ground</th>
<th>Plan Dimensions [m] (BxD)</th>
<th>Column Dimensions [m] (BxDxH)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Residence, Hotel, Shop, School, Other</td>
<td>RC, Wood, Steel, Other</td>
<td>CMU, Wood, Other</td>
<td>Cast RC, RC/Masonry, Tile, Corrugated Metal, Wood</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column Reinforcement</td>
<td>Total No. of Columns</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------------------</td>
<td>----------------------</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Column Damage Level</strong></td>
<td>D0  D1  D2  D3  D4  D5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Beam Dimensions [m]</strong> (BxDxL)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Beam reinforcement</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Beam damage</strong></td>
<td>No damage  Cracking  Spalling  Collapse</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Wall Damage</strong></td>
<td>No Damage  In Plane Damage  Out of Plane Damage  Complete Collapse</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**UNOSAT Overall Damage Rating Scale**
- 1 = Destroyed
- 2 = Severe Damage
- 3 = Moderate Damage
- 4 = No Visible Damage

**EERI Overall Damage Rating Scale**
- 1 = No to Minor Damage
- 2 = Requires repair, but no primary structural elements damaged
- 3 = Loss of at least one column, wall or beam resulting in damage
A.2 Concrete Crack Scale and Classification

<table>
<thead>
<tr>
<th>TABLE A.2:</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE CRACK SCALE</td>
</tr>
</tbody>
</table>

| 1 mm |  |
| 2 mm |  |
| 3 mm |  |
| 4 mm |  |
| 5 mm |  |
| 6 mm |  |
| 7 mm |  |
| 8 mm |  |
| 9 mm |  |
| 10 mm |  |
TABLE A.3:

CONCRETE CRACK CLASSIFICATION

<table>
<thead>
<tr>
<th>AIJ (2002) Crack Classification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No cracks</td>
</tr>
<tr>
<td>1</td>
<td>Crack width &lt; 0.2 mm; cracks are visible only at close range</td>
</tr>
<tr>
<td>2</td>
<td>Crack 0.2 mm ≤ w &lt; 1.0 mm; cracks are obvious</td>
</tr>
<tr>
<td>3</td>
<td>Crack 1.0 mm ≤ w &lt; 2.0 mm; wide cracks with concrete spalling</td>
</tr>
<tr>
<td>4</td>
<td>Crack width ≥ 2.0 mm; concrete spalling and exposed reinforcement</td>
</tr>
<tr>
<td>5</td>
<td>Buckling of reinforcement, crushing of concrete core, or residual deformation is obvious</td>
</tr>
</tbody>
</table>

A.3 Steel Reinforcement Scale

TABLE A.4:

STEEL REINFORCEMENT SCALE

<table>
<thead>
<tr>
<th>BAR DESIGNATION</th>
<th>DIAMETER [in]</th>
<th>DIAMETER [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.375</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>0.500</td>
<td>13</td>
</tr>
<tr>
<td>5</td>
<td>0.625</td>
<td>16</td>
</tr>
<tr>
<td>6</td>
<td>0.750</td>
<td>19</td>
</tr>
</tbody>
</table>
# APPENDIX B:

## POST-EARTHQUAKE INTERNALLY DISPLACED PERSONS SURVEY

## TABLE B.1:

POST-EARTHQUAKE INTERNALLY DISPLACED PERSONS SURVEY

<table>
<thead>
<tr>
<th>Surveyor Name:</th>
<th>Zone:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date/Time:</td>
<td>Zone:</td>
</tr>
</tbody>
</table>

| # | Side: | A | B |

| 1) What is your name and address? | ☐ |
| 2) How many people live in your house? | ☐ |
| 3) Do you have a cell phone number you want to give us so we can call you again? | ☐ Telephone #: |
| 4) What is the name of this neighborhood? | ☐ |
| 5) Do you own the land where you currently live? If not, who is the owner? | ☐ Yes ☐ No |
| 6) When did you receive this shelter? | ☐ |
| 7) How long do you think you will live in this shelter? | ☐ |
| 8) What are the things you like about this shelter? | ☐ |
| 9) What are the things you do not like about this shelter? | ☐ |
| 10) Do you have plans for building a new home? If yes, what are they? | ☐ Yes ☐ No |
TABLE B.1 (CONTINUED)

<table>
<thead>
<tr>
<th>Question</th>
<th>Answer</th>
</tr>
</thead>
<tbody>
<tr>
<td>11) If you could save money each month for building a house, about how much could you save each month?</td>
<td></td>
</tr>
<tr>
<td>12) How many rooms would you like to have in your house?</td>
<td></td>
</tr>
<tr>
<td>13) Would you be willing to live in a multi-family home (a house with little houses inside)?</td>
<td></td>
</tr>
<tr>
<td>14) Do you receive money from a family member working in the US?</td>
<td></td>
</tr>
<tr>
<td>15) Do you think there will be another earthquake?</td>
<td></td>
</tr>
<tr>
<td>16) Do you feel concrete block houses are safe?</td>
<td></td>
</tr>
<tr>
<td>17) Why do you think houses fell down in the earthquake?</td>
<td></td>
</tr>
<tr>
<td>18) What features would be most important in a permanent home?</td>
<td></td>
</tr>
<tr>
<td>19) Is it important to you to be able to build a one-story house that you could one day expand to two stories (stages)?</td>
<td></td>
</tr>
<tr>
<td>20) Which door type do you prefer?</td>
<td>#1</td>
</tr>
<tr>
<td>21) Which window type do you prefer?</td>
<td>#1</td>
</tr>
</tbody>
</table>

B.1 Legend

- This represents an answer that is recorded on tape, to be transcribed later.
- This represents an answer that is written into the survey.

B.2 Notes

Questions 20 and 21 were asked while the participant was presented with the following pictures:
Figure B.1: Pictures shown to survey participants. (Left) Door options and (right) window options.
APPENDIX C:

DETAILED COMMENTARY ON POTENTIAL LATERAL SYSTEMS

This appendix presents each lateral system evaluated in the Léogâne master planning workshop, a brief commentary on each system, its composite score, noted weaknesses, possible improvements to enhance its score and its utility readiness. Target markets are identified as low income ($), middle income ($$) or high income ($$$) residents of Léogâne. The higher the score a system receives, the more viable it is as an option for residential reconstruction in Léogâne. The research team operated under the assumption that the system would be located in downtown Léogâne (urban zone) with a footprint of approximately 60-80 m². Since attitudes and feasibility of construction will vary within the two distinct contexts of rural and urban development, selected systems are evaluated in both zones. Note that in many cases, systems are not extendable to multi-story developments, which is consistent with the overall national trend in Haiti where government surveys prior to the earthquake documented that 73% of all buildings are single story. This is largely attributed to both technical capacity and limitations of the pre-existing systems as well as Haitian cultural preferences to be “close to the ground.”
C.1 Structural System: Load Bearing Walls

**TABLE C.1:**

LOAD BEARING WALLS USING CMU

<table>
<thead>
<tr>
<th><strong>System</strong></th>
<th>Load Bearing Walls using CMU</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>Although these systems are present in Léogâne and did sustain damage in the earthquake, due to their small size, single story modality and usually light metal roof, they generally did not experience total collapse</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>Urban = 79.9, Rural = 80.2</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single story and relatively small footprint (square footage)</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Addition of ring beams to tie walls together</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Feasible with proper planning</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$</td>
</tr>
</tbody>
</table>
### TABLE C.2:

**STONE AND MORTAR LOAD BEARING WALLS (RURAL)**

<table>
<thead>
<tr>
<th>System</th>
<th>Stone and Mortar Load Bearing Walls (Rural)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>This style of construction has been historically observed in the commune; it is completely organic and can be overseen by individuals with little to no training; however, it is difficult to find appropriate stones and fit them cleanly together; system is heavy and highly non-ductile under earthquakes</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>73.6</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, small floor plans</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Encasement in wire mesh as a jacketing to improve ductility</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Not feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$</td>
</tr>
</tbody>
</table>

### TABLE C.3:

**EARTHEN LOAD BEARING WALLS (RURAL)**

<table>
<thead>
<tr>
<th>System</th>
<th>Earthen Load Bearing Walls (Rural)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>33% of rural construction uses earthen systems currently; this mode of construction is completely organic and can be overseen by individuals with little to no training; system is not ductile under earthquakes; durability remains the primary issue</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>70.8</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, small floor plans</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Systems required reinforcement to enhance ductility, perhaps using bamboo or organic materials; erosion must be limited for life cycle</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Not feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$</td>
</tr>
</tbody>
</table>
### TABLE C.4:

**TRASH/RECYCLABLES AS LOAD BEARING WALLS**

<table>
<thead>
<tr>
<th><strong>System</strong></th>
<th>Trash/Recyclables as Load Bearing Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>This system is currently not being considered in Léogâne, though green industries have advocated for such practices that could press trash into membranes or recycle plastic bottles into block for construction—addressing a secondary issue of trash removal in Léogâne; however the connection between these elements for proper seismic performance and the even larger issue of how to facilitate local production of these “green” elements is yet to be addressed. This option may also meet considerable cultural acceptance and security issues.</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>57.5</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, small floor plans</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Systems required some reinforcement to enhance ductility, again local production program would need to be established; method of attachment/interconnection would need to be identified.</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Not feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$</td>
</tr>
</tbody>
</table>
## TABLE C.5:

WOOD SHEAR WALLS

<table>
<thead>
<tr>
<th>System</th>
<th>Wood Shear Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commentary</td>
<td>System would require both wood framing and plywood sheets to form rigid shear walls capable of providing lateral resistance in earthquakes; since this would be completely imported it is highly unsustainable; nailing schedule must also be closely regulated to insure proper system behavior; however system is very light and would perform well in earthquakes; would raise security and even cultural acceptance concerns in urban zones</td>
</tr>
<tr>
<td>Score</td>
<td>62.3</td>
</tr>
<tr>
<td>Limitations</td>
<td>System does not accommodate openings well (windows/doors)</td>
</tr>
<tr>
<td>Improvements</td>
<td>To be sustainable, would require successful reforestation program that still would not produce mature wood necessary for this system in the near term</td>
</tr>
<tr>
<td>Utility Rating</td>
<td>Feasible with proper planning</td>
</tr>
<tr>
<td>Target</td>
<td>$$</td>
</tr>
</tbody>
</table>
C.2 Structural System: Confined Masonry

**TABLE C.6:**

**CONFINED MASONRY USING CMU**

<table>
<thead>
<tr>
<th>System</th>
<th>Confined Masonry using CMU</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>Although the progressive construction practice in Léogâne currently mimics confined masonry, it performed very poorly in the earthquake and led to most of the fatalities. These practices must be dramatically improved (requiring training and codification) to achieve the requisite seismic resilience this system is capable of delivering, including reinforcement of walls, keying of blocks, and inclusion of beams to tie the system. This all assumes budget for sufficient reinforcing steel and quality CMU. The system has the flexibility to support multi-story construction and larger footprints.</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>78.8</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>May be insufficient for larger floor plates in multistory homes where system cannot adequately engage the perimeter elements or carry the larger seismic demands.</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>None necessary if properly implemented</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Feasible with proper planning</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$$</td>
</tr>
</tbody>
</table>
**TABLE C.7:**

**CONFINED MASONRY USING BRICK**

<table>
<thead>
<tr>
<th>System</th>
<th>Confined Masonry using Brick</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>Although the progressive construction practice in Léogâne currently mimics confined masonry, it performed very poorly in the earthquake and led to most of the fatalities. These practices must be dramatically improved (requiring training and codification) to achieve the requisite seismic resilience this system is capable of delivering, including reinforcement of walls, keying of blocks, and inclusion of beams to tie the system. This all assumes budget for sufficient reinforcing steel and quality bricks. Construction time and effort would lengthen in this option, due to the larger quantity of smaller bricks to be placed. The system has the flexibility to support multi-story construction and larger footprints.</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>63.5</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>May be insufficient for larger floor plates in multistory homes where system cannot adequately engage the perimeter elements or carry the larger seismic demands.</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>None necessary if properly implemented</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Feasible with proper planning</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$$</td>
</tr>
</tbody>
</table>

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C.3 Structural System: Frames With Non-Structural Partitioning

TABLE C.8:

REINFORCED CONCRETE FRAME WITH CMU INFILL

<table>
<thead>
<tr>
<th>System</th>
<th>Reinforced Concrete Frame with CMU Infill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commentary</td>
<td>Buildings in Léogâne that properly implemented this system (such as the Notre Dame Residence) performed extremely well in the earthquake. The major issue with this system is the need for formwork to erect free-standing frames. Again completely presumes budget for engineered design, skilled construction team, formwork and larger reinforcing steel. System has flexibility to accommodate wide ranging floor plans and numerous stories.</td>
</tr>
<tr>
<td>Score</td>
<td>69.9</td>
</tr>
<tr>
<td>Limitations</td>
<td>None for low-rise construction</td>
</tr>
<tr>
<td>Improvements</td>
<td>None if properly implemented</td>
</tr>
<tr>
<td>Utility Rating</td>
<td>Highly feasible</td>
</tr>
<tr>
<td>Target</td>
<td>$$$</td>
</tr>
</tbody>
</table>
TABLE C.9:

REINFORCED CONCRETE FRAME WITH MANUFACTURED PANELS

<table>
<thead>
<tr>
<th><strong>System</strong></th>
<th>Reinforced Concrete Frame with Manufactured Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>The partition system here has not been observed in Léogâne to date and would be imported, limiting its sustainability; the major issue with this system is the need for formwork to erect free-standing frames and the need for proper fastening techniques to attach the panels to the frame. Again completely presumes budget for engineered design, skilled construction team, formwork and larger reinforcing steel. System has flexibility to accommodate wide ranging floor plans and numerous stories. Potential security concerns and cultural acceptance issues may surface with panels when used as cladding</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>51.3</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>None for low-rise construction</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Seeding of new industry to press panels locally from agricultural waste (sugar cane, corn stalks) would enhance sustainability</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Highly feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$$$</td>
</tr>
</tbody>
</table>
## TABLE C.10:

**REINFORCED CONCRETE FRAME WITH DRYWALL PARTITIONS**

<table>
<thead>
<tr>
<th>System</th>
<th>Reinforced Concrete Frame with Drywall Partitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commentary</td>
<td>The partition system here has only been observed in Léogâne through importing and requires an underlying framing in wood or light gauge steel further limiting its sustainability; the major issue with this system is the need for formwork to erect free-standing frames and the need for proper fastening techniques to attach the panels to the frame. Again completely presumes budget for engineered design, skilled construction team, formwork and larger reinforcing steel. System has flexibility to accommodate wide ranging floor plans and numerous stories. Drywall could not be used for cladding, requiring CMU or manufactured panels.</td>
</tr>
<tr>
<td>Score</td>
<td>45.3</td>
</tr>
<tr>
<td>Limitations</td>
<td>None for low-rise construction</td>
</tr>
<tr>
<td>Improvements</td>
<td>Could enhance sustainability if drywall could be manufactured locally</td>
</tr>
<tr>
<td>Utility Rating</td>
<td>Highly feasible</td>
</tr>
<tr>
<td>Target</td>
<td>$$ $$</td>
</tr>
</tbody>
</table>
## TABLE C.11:

**LIGHT GAGE STEEL FRAME WITH MANUFACTURED PANELS**

<table>
<thead>
<tr>
<th>System</th>
<th>Light Gage Steel Frame with Manufactured Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>Prototypes of this system had been evaluated by Spanish Red Cross but were rejected due to lack of cultural acceptance of the panels. The system is completely reliant on imported materials, limiting its sustainability; however the system can be constructed very quickly</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>56.8</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, relatively modest floor plans; has security/acceptance issues</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Sustainability can be enhanced if local production of the steel framing or panels could be achieved</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Highly feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$$</td>
</tr>
<tr>
<td><strong>System</strong></td>
<td>Light Gage Steel Frame with Lathe Partitions (Rural)</td>
</tr>
<tr>
<td>---------------------</td>
<td>------------------------------------------------------</td>
</tr>
<tr>
<td><strong>Commentary</strong></td>
<td>This is the semi-permanent option being used by Spanish Red Cross, as lathe uses pre-existing local capacity. The system is reliant on imported materials for its framing, limiting its sustainability; however the system can be constructed very quickly</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>62.3</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, relatively modest floor plans</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Sustainability can be enhanced if local production of the steel framing could be achieved</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Highly feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$$</td>
</tr>
</tbody>
</table>
TABLE C.13:

WOOD FRAME WITH LATHE PARTITIONS

<table>
<thead>
<tr>
<th>System</th>
<th>Wood Frame with Lathe Partitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Commentary</td>
<td>Spanish Red Cross is using this partitioning approach in their semi-permanent houses using pre-existing local capacity; the framing is still reliant on imported materials and wood has some durability issues, including pests; unclear if system would raise security issues in urban deployments</td>
</tr>
<tr>
<td>Score</td>
<td>53.9</td>
</tr>
<tr>
<td>Limitations</td>
<td>Limited to single-story, relatively modest floor plans</td>
</tr>
<tr>
<td>Improvements</td>
<td>Sustainability for development decades from now can be enhanced if reforestation plans are successful; wood could also be a promising solution to rapidly re-generate wood without need for replanting</td>
</tr>
<tr>
<td>Utility Rating</td>
<td>Highly feasible</td>
</tr>
<tr>
<td>Target</td>
<td>$$</td>
</tr>
</tbody>
</table>
### TABLE C.14:

**BAMBOO FRAME WITH MANUFACTURED PANELS**

<table>
<thead>
<tr>
<th><strong>System</strong></th>
<th>Bamboo Frame with Manufactured Panels</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Commentary</strong></td>
<td>This would be a completely new system for Léogâne; at present both the framing and panels would need to be imported. System is lightweight, for good seismic performance, but connections in the framing and attachment of panels would have to be carefully thought through; system should be fairly quick to erect; durability issues would need to be explored (including pests); security and cultural acceptance can be issues in urban deployments</td>
</tr>
<tr>
<td><strong>Score</strong></td>
<td>Rural = 66.6, Urban = 60.7</td>
</tr>
<tr>
<td><strong>Limitations</strong></td>
<td>Limited to single-story, relatively modest floor plans</td>
</tr>
<tr>
<td><strong>Improvements</strong></td>
<td>Sustainability could be enhanced fairly quickly since bamboo is already being grown in the commune and has quick growth cycle (3-5 years); seeding of new industry to press panels locally from agricultural waste (sugar cane, elephant grass, corn stalks) would enhance sustainability</td>
</tr>
<tr>
<td><strong>Utility Rating</strong></td>
<td>Highly feasible</td>
</tr>
<tr>
<td><strong>Target</strong></td>
<td>$</td>
</tr>
</tbody>
</table>
C.4 Summary Of Structural System Assessments

Figure C.1: Composite scores of lateral systems evaluated by the rubric.

Based on the rubric and system combinations that we evaluated, there was a spectrum of scores that reflect the applicability of the systems to the current situation in Léogâne (Figure C1). It can be seen that the systems that scored the highest were load bearing walls made of CMU (both in rural and urban neighborhoods) and also confined masonry systems utilizing CMU. In the complete context of the situation in Léogâne, this
seems logical because it utilizes local materials, builds upon current practices and skill sets and meets the cultural and environmental needs of the community – which have centered on CMU almost exclusively in recent decades. As long as these types of construction are carried out properly and with higher quality materials, they are more than capable of being resistant to both hurricanes and earthquakes, though it remains questionable that privately financed home construction would have the financial resources to do so. However, it is important to note that if these systems are to be employed, there is a serious need for educational and vocational training and oversight to ensure that these systems are employed properly. It must be stressed that lack of quality of materials or the finances to procure them, sound construction practices, and supervision will lead to houses that are just as vulnerable as the homes that were destroyed during the earthquake. For many other systems, the lack of native resources and industries to manufacture engineered materials in country led to punitive scoring due to the heavy reliance on imported materials. The viability of many of these systems within this evaluation framework could be dramatically improved by identifying and seeding new industries to produce new construction materials locally, particularly if these can be green construction technologies that harness the agricultural strengths of the commune.
APPENDIX D:

INITIAL JOINT DESIGN
1. Joint Transverse Reinforcement

A. Horizontal (from column)

RET'EST: AT LEAST (2) LAYERS BETWEEN TOP & BOTTOM LAYERS OF LONGITUDINAL REINFORCEMENT. MAXIMUM SPACING IS 2".

BEAM

Column

CHECK STEEL AREA:

A_{th} = \frac{0.35 \cdot b_{w} \cdot f_{y}}{f_{th}} (A_{g} - 1)

A_{th} = \frac{0.075 \cdot b_{w} \cdot f_{y}}{f_{th}}

\frac{0.04(1.9)(65)(4.64)}{200} = 0.061

A_{th} = 0.3(1.9)(3600)(4.64) / (200 - 1)

A_{th} = 0.243 in\^2 > 0.061

A_{th} = 0.8(0.01in\^2) = 0.08 in\^2 \sqrt{\alpha}

\Rightarrow USE 4 6 STRIUPS ON COL.

Strap Hook Length:

1 \times (d_{a} = 3"

1 \times (0.375\) = 3"

2 \times 2\frac{1}{2} \times 3"
B. Vertical (From Beam)

**Foot**: At least (2) layers between top & bottom layers of longitudinal reinforcement. Max. spacings 6".

\[ x = 8" - 2d_b - 2d_b = 2c_v = B'' + \sqrt[4]{(0.399) - 2(0.5')} = 3.5" \]

- (2) stirrups: \[ \gamma = \frac{3.5}{3} = 1.17" \]
- \[ S_v = \min(4\text{ (clear}), (cd_v, u)) = 2" \]
- \[ S_v = 1.17 < 2" \]

**Check Steel Area**

\[ A_{st} = \frac{0.85b_s f_y}{f_y} \left( \frac{A_s}{A_{st}} - 1 \right) \]

\[ A_s = 0.09 \left( S_v b_s f_y \right) \]

\[ S_v = 1.17" \]
\[ b_s = 11" \]
\[ f_y = 3600 \text{ psi} \]
\[ f_{cu} = 60 \text{ ksi} \]
\[ A_{st} = 66''^2 \]
\[ A_s = 25''^2 \]

\[ A_{new} = 4 \left( 0.11''^2 \right) = 0.44''^2 \geq 0.36''^2 \]

**Use (2) (3) stirrups in beam**

**Stirrups Hook Length**

\[ l_b (4d) = 3" \]
\[ l_b (6d) = 3.75" \geq (2) \]
2. Development Lengths

→ Hooked Bars

A. Columns

1. $l_{h} < 8d_{n} > 6''$
   
   $l_{h} > 8(0.375) > 6''$
   
   $l_{h} > 3'' > 6''$
   
   $\Rightarrow l_{h} > 6''$

2. Type 1 Joint

   $l_{h} = \frac{60000(0.375)}{5000} = 7.5'' < General$

B. Beams

1. $l_{b} < 8d_{n} > 6''$
   
   $l_{b} > 8(0.375) > 6''$

   $l_{b} > 3'' > 6''$

   $\Rightarrow l_{b} > 6''$

2. Type 1 Joint

   $l_{b} = \frac{60000(0.375)}{5000} = 7.5'' < General$

→ Headed Bars

A. Columns

1. $l_{h} > 8d_{n} > 6''$
   
   $l_{h} > 8(0.375) > 6''$

   $l_{h} > 3'' > 6''$

   $\Rightarrow l_{h} = 6''$

2. $l_{h} = \frac{0.01\psi_{L}d_{n}L_{n}}{476}$

   $\psi: 1.0$

   $= \frac{0.01(0.01)(6000)}{476} 0.375$

   $l_{h} = 6'' < General$
3. Joint Shear

A. Corner Joint

\[(x = 12)\]

\[\phi V_u \geq V_u\]

\[\phi = 0.55\]

\[V_u = \frac{f_y \cdot t^2}{2} \cdot b_2 \cdot h_c\]

\[f_y = 60,000\text{ psi}\]

\[h_c = 8\text{ in}\]

\[b_2 = b_3 = \frac{b_1^2 \cdot b_2}{b_1^2 + b_2^2}\]

\[b_1 = 9\text{ in}\]

\[b_2 = 4\text{ in}\]

\[V_u = 12 \times \sqrt{60,000 \times 9^2} = 4,600\text{ kips}\]

\[\phi V_u = 0.55 \times 4,600 = 39,200\text{ kips}\]

**Shear Forces**

**V-dir:**

**Gravity Loads**

\[V_u = T_u = A_t f_y\]

\[V_u = 3(0.11 \times 27) = 19.8\text{ kips}\]

**External Loads**

\[V_u = C_e = A_t f_y\]

\[V_u = 3(0.11 \times 27) = 19.8\text{ kips}\]

\[\phi V_u = 39.2\text{ kips} > V_u = 19.8\text{ kips}\]

\[\frac{V_u}{V_u}\]

**K-dir:** same as V-dir.
**E. Ext. Joint (Y = 15)**

\[ V_u = \frac{b h c}{2} \]

\[ V_u = 75 \left( \frac{F_C}{b} \right) h c \]

\[ Y = 15 \]

\[ A = 3000 \]

\[ h_c = 8^\circ \]

\[ b_c = 2.5 \times 10^{-3} \]

\[ \delta = 0.05 \times 10 \]

\[ \phi V_u = 0.85 \times \frac{F_C}{b} = (8)(9) \]

\[ \phi V_u = 48.9 \text{ kips} \]

---

**Shear Forces**

**Y-Die:** (Same for (i) & (ii))

1. **Gravity Loads**
   \[ V_u = T_{bc} - T_{bc} = A_b f_y - A_b f_y \]
   \[ A_b = A_{bc} \]
   \[ V_u = 0 \]

2. **Lateral Loads**
   \[ V_u = T_{bc} = A_b f_y + A_b f_y \]
   \[ V_u = 2 A_b f_y = 2 (30.11) \text{ kips} \]
   \[ V_u = 39.6 \text{ kips} \]

\[ \phi V_u = 48.9 \text{ kips} > V_u = 39.6 \text{ kips} \]

**X-Die or (ii) is same as common joint**
APPENDIX E:

REVISED JOINT DESIGN
1. Joint Transverse Reinforcement

A. Horizontal (Column)

\[ \text{Seat: At least (2) LINES ET-W TOP Beam Reinforcement} \]

\[ \text{(ACI 318-02 9.3.1.5)} \]

\[ \text{MIN Stresses} \]

\[ x = 15 - 2d_{c_e} - 2d_{w} - 20' z + 4(0.325 \times 2(1.5)) = 10.5' \]

\[ S_{h} = \text{min} \left( \frac{f_{c}}{(f_{c} - 2.25)} \right) \]

\[ S_b = 2.25' \]

\[ S_{h} = \text{min} \left( \frac{f_{c}}{2.25} \right) \]

\[ S_{b} = 10.5' \]

\[ \geq 3 \text{ Bars (4) Stresses} \]

\[ S_{b} = 10.5' - 2.1' < 2.25' \]

\[ \sqrt{3} \]

Check \( A_{n} \)

\[ A_{n} = 0.09 \frac{S_{b} b_{w} z}{f_{y}} \left( A_{n} - 1 \right) \]

\[ A_{n} = 0.09 \frac{S_{b} b_{w} z}{f_{y}} \left( A_{n} - 1 \right) \]

\[ \sqrt{3} \]

\[ b_{w} = 10'' - 2c_{e} = 7'' \]

\[ f_{y} = 30000 \text{ psi} \]

\[ S_{h} = 2.25' \]

\[ S_{b} = 10'' \times 10'' = 100 \text{ in}^2 \]

\[ A_{n} = (10 - 2c_{e}) \times 49 \text{ in}^2 \]

\[ A_{n} = 0.09 \left( 2.1 \times 3 \right) \times 0.09 \times 2.25 \]

\[ = 0.235 \text{ in}^2 \]

\[ \geq 0.074 \text{ in}^2 \]

\[ \text{Note:} \]

\[ A_{n} = 0.08 \times 3'' = 0.27 \text{ in}^2 \]

\[ \sqrt{3} \]

1. USE (4) 3/8 STIRRUP'S ON COLUMN. 190 3/16"
Title: Vertical (Beam)

Conditions:
- Beam: 15" long, 1.5" wide
- Column: Unknown width
- Beam: 10" long, 4.5" wide
- Stress: 2.25 ksi

Calculations:
- \( S_v = \min \left( \frac{d}{2300}, \frac{d}{1000}, \frac{d}{8} \right) \)
- Beam: \( S_v = 2.25 \) ksi

Check for Stress:
- Beam: \( \frac{5.0}{3} = 1.67 \) ksi < 2.25 ksi

Conclusion:
- Use (2) stirrups in beam instead of (3).
2. DEVELOPMENT LENGTH

A. Hooked Bars

(i) Columns

\[ l > B_d > 0.1 \gamma_c \]

\[ 60000 \times 0.335 \times 7.5'' = 2745'' \]  

\[ \frac{60000 \times (0.335)}{50000} > 7.5'' \]

\[ l_h = 7.5'' \]

(ii) Beams

\[ l > 12d_e \]

\[ l = 12(0.335) \]

\[ l = 4.2'' \]

B. Heaved Bars

(i) Columns

\[ l_h = 0.014d_e \gamma_c \]

\[ l_h = 1.0 \]

\[ 0.014 \times 1.0 \times (0.335) \times (0.335) \]

\[ l_h = 0.17'' \]

\[ l_h = 0'' \]

(ii) Beams

\[ l_h = 0'' \]
3. **Joint Shear**

   \( A_s1 \ 350\text{ksi} \ \frac{l/5}{l/3} \)

   **A. Corner Joint**
   \( l = 12 \)
   \[
   V_a = \frac{1}{6} b f c \quad \phi = 0.65
   \]
   \[
   V_a = \frac{2}{3} f c \quad b f c
   \]
   \( f_c = 6000 \text{ psi} \)
   \( b f c = 10^{-3} \)

   \[
   \phi V_a = 0.1200 \text{ kips}
   \]

   \[
   \phi V_a = 0.12 \text{ kips}
   \]

   **Shear Forces**

   **Y-Dir:**

   **Gravity Loads**
   \[
   V_a = T_b = A_s f c \quad V_a = 4(0.12)(60 \text{ kips})
   \]
   \( V_a = 24.4 \text{ kips} \)

   **Lateral Loads**
   \[
   V_a = C_a \approx T_a = A_s f c \quad V_a = 24.4 \text{ kips}
   \]
   \[
   \phi V_a = 0.12 \cdot 24.4 = 29.3 \text{ kips}
   \]
   \[
   V_a^{\phi} = 29.3 \text{ kips}
   \]

   **X-Dir:**
   \[
   \text{Same as Y-Dir} \quad V_a^{\phi}
   \]
B. Extension Joint:

\[ \phi_{V_e} \leq \phi_{V_n} \]

\[ V_n = Y \cdot f_e \cdot b \cdot h_c \]

\[ f_e = 15 \]

\[ b = 10'' \]

\[ h_c = 10'' \]

\[ \phi_{V_n} = 0.85 \cdot (15) \cdot 10000 \cdot (10) (10) \]

\[ \phi_{V_n} = 76.5 \text{ kips} \]

\[ \phi_{V_n} = 76.5 \text{ kips} \]

\[ V_n = 52.8 \text{ kips} \]

\[ V_{ok} \]

Shear Forces:

Y-DIR:

\[ \begin{bmatrix} \text{Grav. Load} \end{bmatrix} \]

\[ T_x \rightarrow T_y \]

\[ V_x = T_{x,1} - T_{x,2} = A_{x,1} - A_{x,2} \]

\[ A_{x,1} = A_{x,2} \]

\[ V_x = 0 \]

LATERAL LOADS:

\[ C_{b,1} \rightarrow T_y \]

\[ V_y = T_{y,1} - C_{b,1} \cdot T_{y,2} \]

\[ V_y = 2 \cdot A_{y,1} = 2(4(0.11))(60) \]

\[ V_y = 52.8 \text{ kips} \]

X-DIR:

Same as corner joint V_{ok}
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