BUILDING PRESSURE TESTS OF FIBER-REINFORCED FOAM-BASED LIGHTWEIGTH CONCRETE PRECAST WALL PANELS

by

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ABSTRACT

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Instrumentation and procedures for building pressure tests were designed as a means of providing new wind pressure test methods for full-scale building models. Fiber-reinforced foam-based lightweight concrete precast wall panels were studied in this investigation. Three (3) full-scale prototype buildings were tested for this purpose.

To determine the material properties and design parameters for this targeted 90 pcf (1,446 kg/m^3) cellular lightweight concrete and to gain overall knowledge of its behavior, some tests such as compressive strength test, pull-out strength test, and flexural strength test were performed. The wind pressure tests performed provided

results for strength, maximum wind pressure the panels can sustain, and the deflection capacity of these lightweight concrete panels.

The results for each of the tests were the following: In Test #1 the maximum pressure sustained was 3.14 psi (0.22 kg/cm^2); a deflection of 0.15 inches (0.38 cm) and the failure was ductile. In Test #2 the panels sustained a maximum pressure of 4.5 psi (0.32 kg/cm^2); a deflection of 0.18 inches (0.46 cm) and failure was also ductile. The panels in Test #3 sustained a maximum pressure of 4.76 psi (0.33 kg/cm^2); a deflection of 0.57 inches (1.45 cm) and failure was ductile as well. Results from these tests provide information that will be later compared with information of Normal Weight Concrete.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

One of the basic and most important parameters to consider when designing any structure is the wind loads that act on it. Depending on the geographical location, wind loads can highly affect the design and behavior of a structure. The ASCE-7 Code provides the parameters and the procedures to calculate the wind loads acting on a structure. Knowing these parameters, the pressure acting on the structure can be calculated and a safe structure can be provided through a reliable design.

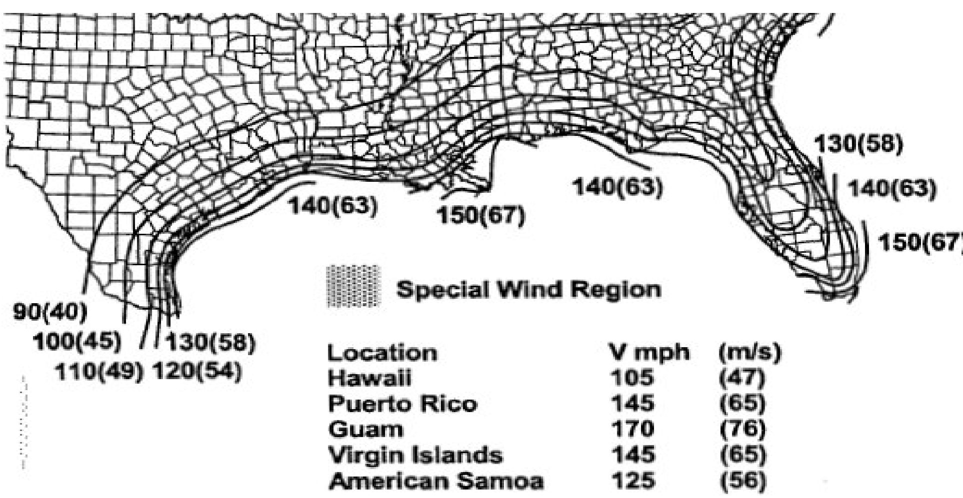

Figure 1.1 Wind Speed Parameters[1]

[1] Picture taken from the ASCE 7-02 Code.

Design of a structure usually depends on the materials available at or near the location where the structure will be erected. One of the most used construction materials is concrete.

Concrete is a composite material that consists of a binding medium within which particles or fragments of aggregates (usually a combination of fine and coarse aggregates) are embedded. The binding mixture is commonly composed of cement and water. Admixtures can also be added to achieve specific properties in the concrete. Concrete has proven to be a material with excellent compressive strength. However, it does not have good tensile strength and for this reason steel reinforcement is used in concrete structures to provide the necessary tensile strength. Even though concrete has this disadvantage, it is still a good choice as a building material since it is lighter and less expensive than steel.

Reinforced concrete structural elements made of normal weight concrete (having a density of 150 pcf [2400 kg/m^3]) have been tested and are designed everyday to resist wind loads and pressure in hurricane prone regions. Lightweight concrete composed of fine aggregate has also been tested and used in design for some hurricane prone regions such as Florida.

The Fiber-Reinforced Foam-Based Lightweight Concrete Precast Wall Panels produced for this research are lightweight structural elements used for structural applications. The average density of this lightweight cellular concrete is 90 pcf (1,446 kg/m^3). The purpose of producing a lighter concrete is to improve ductility and lower the dead loads, which at the same time lowers the need of steel reinforcement.

Precast concrete panels provide advantages such as a reduction in time of construction since panels can be put together in one day, optimizing the use of materials and reducing labor costs. This type of panel also provides temperature and sound insulation.

To determine the performance of this cellular concrete as a structural material, full-scale building prototype tests were performed subjecting these models to a wind speed higher than 150 mph, the critical design wind speed in hurricane prone regions in the United States. The mix design of the cellular concrete consists of water, cement, fine aggregate (sand), glass fibers and a protein-hydrolysis based foaming agent called Neopor.

Cellular concrete systems were introduced to the construction industry more than 50 years ago. The use of this concrete has been limited for non-structural void filling jobs, roof insulation and blocks. Due to insufficient physical properties, caused by instable foams and lack of scientifically undertaken research work, the use of such void-filling cellular concrete systems has not been allowed for structural applications.

Neopor was introduced more than 10 years ago. With the introduction of this system, it was possible for the first time to produce densities from 400-1,800 kg/m^3 (24-108 pcf) with an optimum density versus strength ratio. An important fact about Neopor is that it causes no chemical reaction in the concrete but that it only serves as wrapping material for the air entrapped. This foaming agent can be stored to at least 24 months, and its chloride content is less than 0.002%. Other advantages of this cellular concrete are that there is no need of vibration for compacting purposes and that traditional mixer and truck mixers can be used for the preparation of the mix.

This cellular concrete can be air-cured. Curing can also take place by accelerating it applying heat, steam, or chemicals. Building elements of cellular concrete may be stripped in approximately 6-10 hours after casting depending on the temperature of the casting location and the quality of the cement used.

Building prototypes were fabricated using this foamed lightweight reinforced concrete for the precast panels. These were subjected to internal pressure to determine its strength and stiffness when experiencing wind loads. These building prototypes were tested simulating possible wind loads in hurricane prone regions in the U.S. such as Florida, New Orleans and Texas.

1.2 Literature Review

Lightweight precast concrete wall panels have been used in building construction for many years. However, most of these have served as architectural building products instead of being used for structural capacity purposes. Lightweight precast wall panels can be used as a load-bearing structural member by increasing its bending moment and compressive strength capacity. Using these members as part of the structural body can help reduce the size, weight and cost of other members while providing a product that is easy to erect and install.

1.2.1 Historical Background

Since 1757, theory of buckling strength of compression members has been investigated. Euler derived the well known Euler load formula for a perfectly straight pinned-end column. This equation is applicable while material behaves linearly elastic. Other theories such as Engesser-Karman were derived for structural members with a nonlinear stress-strain relationship.

This theory presents the tangent modulus E_t for non-linear behavior. The reduced modulus theory was also proposed to solve column problems in the inelastic range. When an eccentric load is applied to a column, the secant formula, which considers the slenderness ratio, is used. All the results of these procedures are presented in the building code as requirements to design a safe structure.

The American Concrete Institute Standard Building Code Requirements (ACI-318) has been the authority in the United States for concrete practice since 1920. This code suggests the use of an approximate procedure based on the magnified moment concept in addition to the second-order method for slenderness effects on reinforced concrete compression members.

All the limits in this code are directed to normal weight concrete. Because the properties of lightweight concrete members are different from that of normal weight concrete and mostly unknown, there is a need to perform experimental investigations on full-scale lightweight precast concrete wall panels.

1.2.2 Wind Loads

Simiu et al. (2003) studied the precision of detail used in the calculation of gust wind speed used in the Peak-Gust Map of the ASCE Standard 7. The investigators studied the quality of adding different substations information into a station. It was found that for some locations nationwide this fact did not do much effect. On the other hand, in some cases, data varied greatly in the same state depending on the physical geography and its distance from the coast. It was concluded that the ASCE 7 peak-gust map division of the conterminous US into two main adjacent wind speed zones does not reflect correctly the differentiated extreme wind climate of the US. This map entails significant waste of material due to overestimated wind loads and losses due to underestimated wind loads. The investigators suggested the improvement of this map in future revisions.

Kallaby (2007) pointed out the need to have a viable concept and procedure that can be used by practicing engineers to address severe hurricanes such as hurricane Katrina in 2005. The author suggested that at or near coastlines, new structures be designed and existing ones be upgraded based on which hurricane category they can survive under impact of surge and wind. The survival of these structures is measured on the Reserve Capacity (RC). Since this would not be cost-effective, the best approach suggested by the author is to design using standard practice, or the Design Strength Level (DSL), for hurricanes two intensity levels less than the most intense hurricane expected. After this design, the structure should be investigated for RC and critical members should be strengthened as needed. After the above mentioned procedure is taken into consideration, inundation potential and foundation erosion should be investigated. The uses of these practices serve as good design criteria while providing more economically viable structures.

The University of Florida performed tests on Cast-in-Place (CIP) Concrete Walls to determine their performance in high wind loads of hurricanes and tornadoes of about 155 miles per hour. These walls were 6" thick, reinforced with #4 bars. It was found that concrete walls were more resistant to debris compared to steel frames and wood frames. A test was conducted simulating a 125 miles per hour storm to determine some properties of the concrete walls. The researchers found that the walls for an f'_c of 3,000 psi had an axial compression of 7,680 lbs/ft, a bending moment of 2,158 ft-lb/ft and a shear of 3,960 lb/ft. It was concluded that CIP Concrete Walls are effective for construction in hurricane prone areas.

The University of Florida also performed other tests for high wind loads performance, testing Insulated Concrete Form (ICF) Walls. ICFs are cast-in-place concrete walls that keep the forms permanently. The three types of ICFs that exist are: flat walls (create uniformly thick walls), waffle/grid walls (creates concrete of varying thicknesses), and post-and-beam walls (create discrete horizontal and vertical concrete columns). The walls that were tested were 6" thick, reinforced with #4 bars. As in the previous study, it was found that concrete walls were more resistant to debris compared to steel frames and wood frames. The researchers found that the walls for an f'_c of 3,000 psi had an axial compression of 6,000 lb/ft, a bending moment of 1,000 ft-lb/ft and a shear of 4,100 lb/ft. It was concluded that ICF walls are effective for construction in hurricane prone areas.

Ginger (2000) performed full-scale tests on low-rise buildings to study the effect of internal and external pressure on these. Ginger determined that internal pressure is dependent on the external pressure field, the position and size of all openings connecting the interior of the building to the exterior, the volume of the building, internal layout, and the flexibility of the envelope. When a building is flexible, the internal volume will expand and contract with the changes in internal pressure. Ginger studied two types of buildings: nominally sealed, and buildings with large openings. Ginger concluded that the mean and fluctuating internal pressure coefficients in a nominally sealed building are smaller in magnitude than the pressure coefficients on the external surfaces and increase with an increasing windward/leeward open area ratio.

Meanwhile, a dominant windward wall opening resulted in reduced net positive loads on the windward wall and increased net negative loads on the roof, sidewalls, and leeward wall compared to that of the nominally sealed building. The large positive internal pressure combined with the large external suction pressure at the roof windward edge generated a large net peak suction pressure.

Surry et al. (2005) designed full-scale mechanisms at model scale and found that these mechanisms provide a useful approach to study and capture wind pressure patterns in ultimate behavior. The method proposed by the investigators allowed for a wide range of configurations, based on results from real full-scale experiments performed after significant wind events. These investigators discovered that influence coefficients together with a novel loading system, influence surfaces and load paths can be determined for full-scale structures from low levels of loading to failure.

The importance of this investigation is that this information can be used to calibrate finite element models of the structural behavior, and together with wind tunnel data can be used to study other configurations other than those that can be tested at full-scale. This has the potential for significant forefronts in understanding how complex structural systems work, and how to improve these.

1.2.3 Fiber Reinforced Concrete

Boshoff and Van Zijl (2007) performed tests to study the response of fiber-reinforced concrete to creep and shrinkage. Specimens were tested at an age of 14 days. Tensile rate tests were performed as well as rate tests of beams in Three Point Flexure. Tensile creep tests were performed to characterize and quantify the tensile creep behavior. Both shrinkage and creep tests were performed for a duration of eight (8) months. Results showed that the failure mechanism at the ultimate load was not the matrix failure as in the case of the first cracking strength, but the fibers bridging a crack either pulling out or rupturing. Ductility and the E-modulus were found to be insensitive to the increase of the loading rate on the fiber-reinforced specimen. Flexural ductility did show a decrease with an increase of deflection rate. In the tensile creep tests, the cracked specimen showed a higher rate of creep compared to the uncracked specimen. The creep of the fibers bridging the crack, and the time-dependant fiber pull-out and/or the further time-dependant initiation of cracks are possible explanations of this increased creep. This study shows that the ultimate strength is less dependent on the loading rate than in ordinary concrete.

Balaguru and Foden (1996) investigated the behavior of fiber reinforced lightweight concrete with different fiber content. This concrete was tested for compressive strength, modulus of elasticity, splitting tensile strength, modulus of rupture (flexural strength), and toughness. Silica fume was added to some of the mixes tested. The investigators concluded the following: the addition of silica fume increases compressive strength substantially. Addition of fiber further improves strength. The increase in strength provided by fibers was much higher for concrete without silica fume. Also, increase in fiber content results in an increase in compressive strength. The conclusion for the splitting tensile strength tests was that the addition of fibers results in a substantial increase (more than 100%) in this strength and modulus of rupture. Fibers 50 mm (2.0 in) long that had a longer aspect ratio, provided better strength increases than 60 mm (2.4 in) long fibers. Findings for toughness were that fibers are very effective in improving toughness, indicating strain-hardening behavior, and sustaining loads equal to cracking load even under large deformations.

Yost and Gross (2002) evaluated safety in Fiber-Reinforced-Polymers (FRP) reinforced flexural members with respect to energy consumption and reserve. The investigators stated that where FRP is used as flexural reinforcement, it is not enough to establish design acceptance on strength alone because the structure will lack section and member ductility. After various experiments, it was concluded that a high level of reserve strength is necessary to ensure an appropriate level of energy dissipation through elastic straining in the FRP reinforcement. Also, it was found that increasing concrete strength will result in more efficient use of FRP.

Tureyen and Frosch (2002) investigated shear strength and behavior of concrete beams reinforced with glass fiber-reinforced polymer (GFRP) bars without transverse reinforcement. Their objective was to investigate the effect of differences in the modulus of elasticity of GFRP and steel reinforcing bars on the concrete contribution to shear strength of slender reinforced concrete flexural members without transverse reinforcement. The experiments showed that flexural concrete members reinforced with FRP bars in the longitudinal direction can fail in shear at loads lower than those reinforced with an equivalent area of steel bars. It was also concluded that the number of flexural cracks and their spacing were similar for all beams. However, the depth of flexural cracks, the size and the horizontal projection of inclined cracks prior to failure were different for GFRP and steel. The investigators recommended that due to these differences, stirrup effectiveness for concrete members reinforced longitudinally with FRP bars be studied in future investigations.

Sheikh et al. (2002) performed tests on the damage sustained by foundation walls and beams in a building repairing these with FRP, glass and carbon fiber, sheets and wraps and testing these to failure. Results showed that FRP were effective in strengthening for flexure and shear. However, over-reinforcing in flexure resulted in shifting the failure to shear which is not desirable. On the other hand, the strengthening of a member in shear resulted in an increase in the ultimate displacement and toughness factor. The investigators suggested that in order to avoid shear failure and to allow a more efficient use of FRP and a more ductile behavior, flexural strength enhancement should be limited.

Toutanji and Saafi (2000) made analytical experiments to propose new design equations to predict cracking behavior and deflection of GFRP reinforced concrete beams. As a result of the experiments performed it was observed that cracks were initiated when the applied moment reached the cracking moment. It was concluded that crack width decreases as the reinforcement ratio increases. Also, it was found that due to the low modulus of elasticity of GFRP, concrete members reinforced with GFRP exhibit large deflections and crack widths compared with concrete members reinforced with steel. Toutanji and Saafi proposed equations to predict this cracking behavior taking into consideration the lower modulus of elasticity of GFRP.

Nkurunziza et al. (2005) investigated the creep behavior of GFRP bars in different environments under sustained loads. The two environments studied were alkaline solution and de-ionized water, both at different stress levels. The investigators concluded that alkaline solutions tend to have a more harmful effect on the bars than the de-ionized water at higher stress levels. At lower levels, only natural pH water can migrate through microcracks and have a harmful effect. The modulus of elasticity of the bars was very stable and almost unaffected by the conditions and stress levels induced. This finding is very important because the modulus of elasticity is directly related to the crack width, deflection, and other serviceability concerns.

1.2.4 Lightweight Concrete

John August (1997) performed tests on 3” wide x 6” deep x 12” long lightweight concrete blocks. Some eight to sixteen common nails were hammered into the concrete blocks without predrilling. August mentioned: “In its lightest densities, concrete absorbs shock”. He also found that decreasing the weight and density produces significant changes that improve many properties of concrete, both in placement and application. In conclusion, lightweight composite concrete (LWC) construction can be a partial solution for several environmental problems since it can reduce timber construction, hence, reduce deforestation and the use of poison used for lumber treatment. August mentioned that considering all of LWC’s positive characteristics little attention has been given to its possibilities even though it is used in many third world countries.

The ESCI organization presented an article on the advantages of structural lightweight concrete. This article discussed that structural lightweight concrete (SLC) has strengths that are comparable to normal weight concrete, yet it is typically 25% to 35% lighter. SLC also offers design flexibility and substantial cost savings by providing less dead load, improved seismic structural response, longer spans, better fire ratings, thinner sections, decreased story height, smaller size structural members, less reinforcing steel, and lower costs in foundations and, reduced trucking and placement costs of SLC precast panels.

Shi et al (2006) tested ICF panels using self-consolidating concrete (SCC) to prevent honeycombing, which affects the structural performance of the concrete. Using

SCC provides advantages such as a decrease in construction time and labor costs, reduces voids in highly congested structural members, decreases permeability while improving durability of concrete, and facilitates constructability. Since high casting rates are usually used for SCC, formwork is usually designed for hydrostatic pressure. This fact made the researchers decide that using lightweight concrete had significant benefits. After tests were performed for material properties, it was concluded that SCC was a good choice for these panels since there were no signs of honeycombing or large voids, and no vibration was needed for consolidation purposes.

1.2.5 Bond Strength

Harajli and Mabsout (2002) evaluated the bond strength of reinforcing bars embedded in plain and fiber-reinforced concrete (FRC). Two types of failures were studied; pull-out and splitting of the concrete. Several parameters were investigated such as concrete compressive strength on the development and bond properties of reinforcing bars embedded in conventional concrete, and the effect of fiber reinforcement on the bond performance in comparison with plain unconfined concrete and concrete confined by transverse reinforcement. It was concluded that since fiber reinforcement does not affect the response for a pull-out type of bond failure, the analytical predictions of pullout development and anchorage strength characteristics are expected to be identical to those for plain concrete. Results using an analytical evaluation showed that presence of fiber reinforcement significantly increases bond strength of reinforcing bars and improves the ductility of bond failure.

1.3 Goals and Objectives

The main objective of this research is to design the test setup and instrumentation needed to conduct pressure tests to simulate the effect of wind pressure on precast lightweight concrete panels. The design of these tests and instrumentation were conducted as a means of detecting the structural capacity and behavior of this new material. To achieve this objective, the following are the forefronts:

1) To design pressure tests to simulate the effect of wind loads on these precast panels.
2) To identify a pressure bag suitable to resist high pressure in order to perform these tests.
3) To identify ideal equipment to measure pressure and deflection during the tests.
4) To evaluate the pressure resistance characteristics of precast lightweight wall panels. This includes the comparison of this lightweight concrete wall panels with regular wall panels when subjected to the same uniform pressure.

CHAPTER 2

EXPERIMENTAL PROGRAM I:
MATERIAL TESTING

2.1 Introduction

This chapter presents a description of the materials and the mix design of the cellular lightweight concrete prepared to perform the tests. A description of the reinforcement used and the diagrams of these panels are also presented in this chapter. A series of experimental tests developed to determine material properties of this cellular lightweight concrete are also presented.

Tests following ASTM Standards were conducted using small batch laboratory settings and large batch field mixing. The laboratory drum mixer (Figure 2.2) used had a capacity of 27 ft^3 (0.77 m^3), while the field mixing truck (Figure 2.3) had a capacity of 177.6 ft^3 (5 m^3). Equipment was used to control the foam mix while the concrete mix was being prepared. Plots of certain results obtained from these tests are presented throughout the chapter. A comprehensive description about this material information is found in Ake Pyamaikongdech (2007) investigation.

After the concrete mix was ready, it was poured over the forms as shown in Figures 2.5 through Figure 2.7.

Figure 2.1 Foam used as Lightweight Aggregate

Figure 2.2 Drum Mixer

Figure 2.3 Truck Mixing

Figure 2.4 Equipment used to Control Foam Mix

Figure 2.5 Formwork with Reinforcement

Figure 2.6 Pouring of Concrete Mix

Figure 2.7 Casting of Panels

2.2 Materials and Material Properties

2.2.1 Concrete (Mix Design Process and Preparation)

The cellular lightweight concrete mix used for the tests been performed contains sand as fine aggregate, water, cement, and Neopor (foam that offers no chemical reaction but serves as wrapping material for the air). Quantities of the foaming agent were optimized to achieve lower densities ranging between 87 to 105 pcf. A constant water cement ratio of 0.44 was used to maintain uniformity in results. Glass fibers were added to the concrete mix to improve ductility and to control creep and shrinkage.

For testing purposes, mixing took place in both laboratory and a mix truck to simulate actual construction conditions. The mix proportion used for one cubic yard of concrete is presented in Table 2.1.

Table 2.1 Concrete Mix Proportion for 1 yd^3

Material	**Proportion**
Cement	675 lbs (307 kg)
Water	300 lbs (137 kg)
Sand	1440 lbs (655 kg)
Foaming Agent (Neopor)	40 lbs (19 kg)
Glass Fibers	1 lb (0.45 kg)

2.2.2 Steel Reinforcement

Steel was used as the main reinforcing material for both strength and serviceability. Also, steel embeds were used for wall-to-foundation and wall-to-wall connection. The material properties of the reinforcing steel used in this study is presented in Table 2.2.

Table 2.2 Steel Properties for #4 Reinforcing Bars

Properties	**Value**
Modulus of Elasticity	29,000 ksi (200,000 MPa)
Yielding Strength	60 ksi (413.7 MPa)

2.2.2.1 Roof Panel

The roof panels of the prototype building had dimensions of 12 ft x 12 ft x 8 in (3.66 m x 3.66 m x 20.32 cm). Weld plates of dimensions 6 in x 4 in x 3/8 in (15.24 cm x 10.16 cm x 0.95 cm) with two (2) No. 4 bars, 16 in (40.64 cm) long were placed at every 3 ft-6 in (1.0 m) on the roof panels. Utility anchor lifters were placed at 3 ft (0.91

m) from the edge of the panel for lifting and assembly purposes. Figure 2.8 presents the typical roof panel of the building prototype.

2.2.2.2 Floor Panel

The floor panels of the prototype building had dimensions of 12 ft x 12 ft x 10 in (3.66 m x 3.66 m x 25.4 cm). Weld plates of dimensions 6 in x 6 in x 3/8 in (15.24 cm x 15.24 cm x 0.95 cm) with four (4) ½ in (1.27 cm) thick and 6 in (15.24 cm) long studs were placed at every 3 ft-6 in (1.0 m) on the floor panels. Utility anchor lifters were placed at 3 ft (0.91 m) from the edge of the panel for lifting and assembly purposes. The floor panel and the wall panels were connected by angles with dimensions 1 ½ in x 1 ½ in x 3/8 in (3.81 cm x 3.81 cm x 0.95 cm) and 6 in (15.25 cm) long. A typical drawing of the wall panels is shown in Figure 2.8.

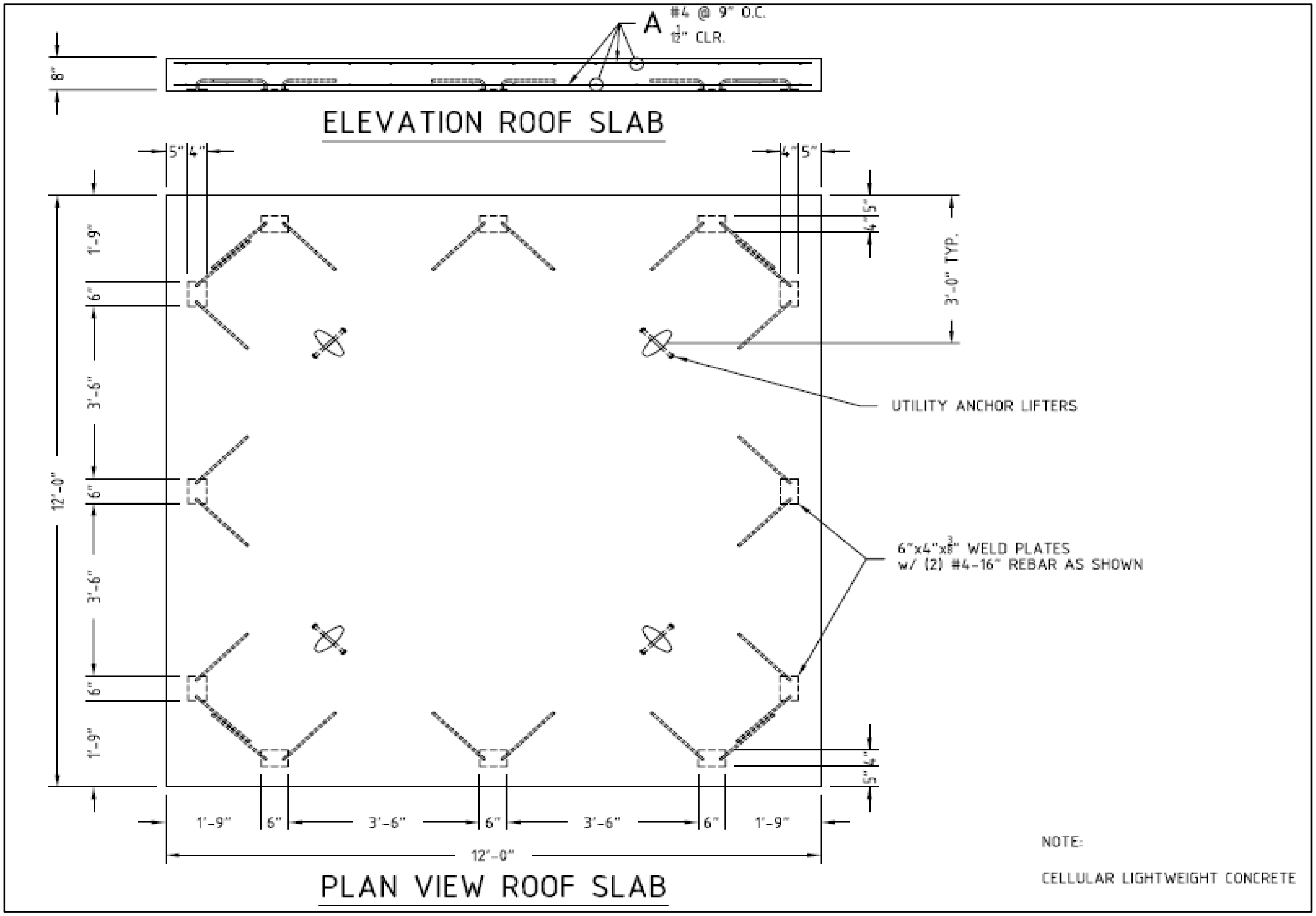

Figure 2.8 Typical Roof Panel

2.2.2.3 Side Wall Panels

The side wall panels of the prototype building had dimensions of 8 ft-1 in high x 10 ft-7 in wide x 8 in thick (2.44 m x 3.06 m x 20.32 cm). Figure 2.9 shows the typical wall panel used for these tests. Weld plates of dimensions 6 in x 4 in x 3/8 in (15.24 cm x 10.16 cm x 0.95 cm) with two (2) No. 4 bars, 16 in (40.64 cm) long were placed every 3 ft-6 in (1.0 m) on the side wall panels. There were two (2) Meadowburke RL-6, four (4) ton lifters embedded on the panels for assembly purposes. The floor panel and the wall panels were connected by angles of dimensions 1 ½ in x 1 ½ in x 3/8 in (3.81 cm x 3.81 cm x 0.95 cm) and 6 in (15.25 cm) long.

Steel welded wire meshes of size W3.0/W2.0 – 2/6 were used throughout the entire panels for the model. Number 4 bars (area of 0.20 in^2) were used as reinforcement between roof-wall connection, wall-floor slab connection, as well as steel angles of 2 in x 2 in x 3/8 in (5.08 cm x 5.08 cm x 0.95 cm) dimensions, welded between the panels.

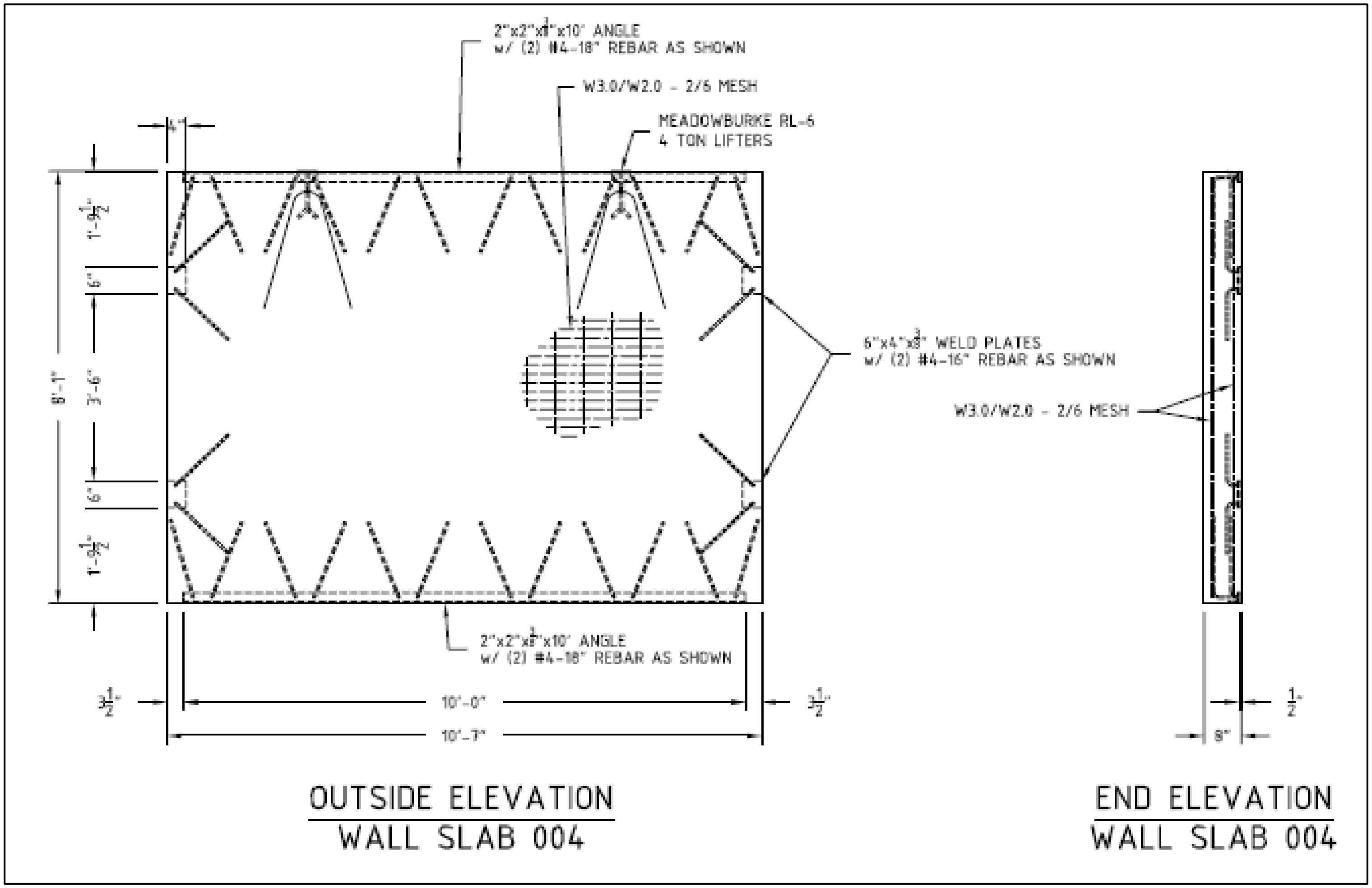

Figure 2.9 Typical Wall Panel

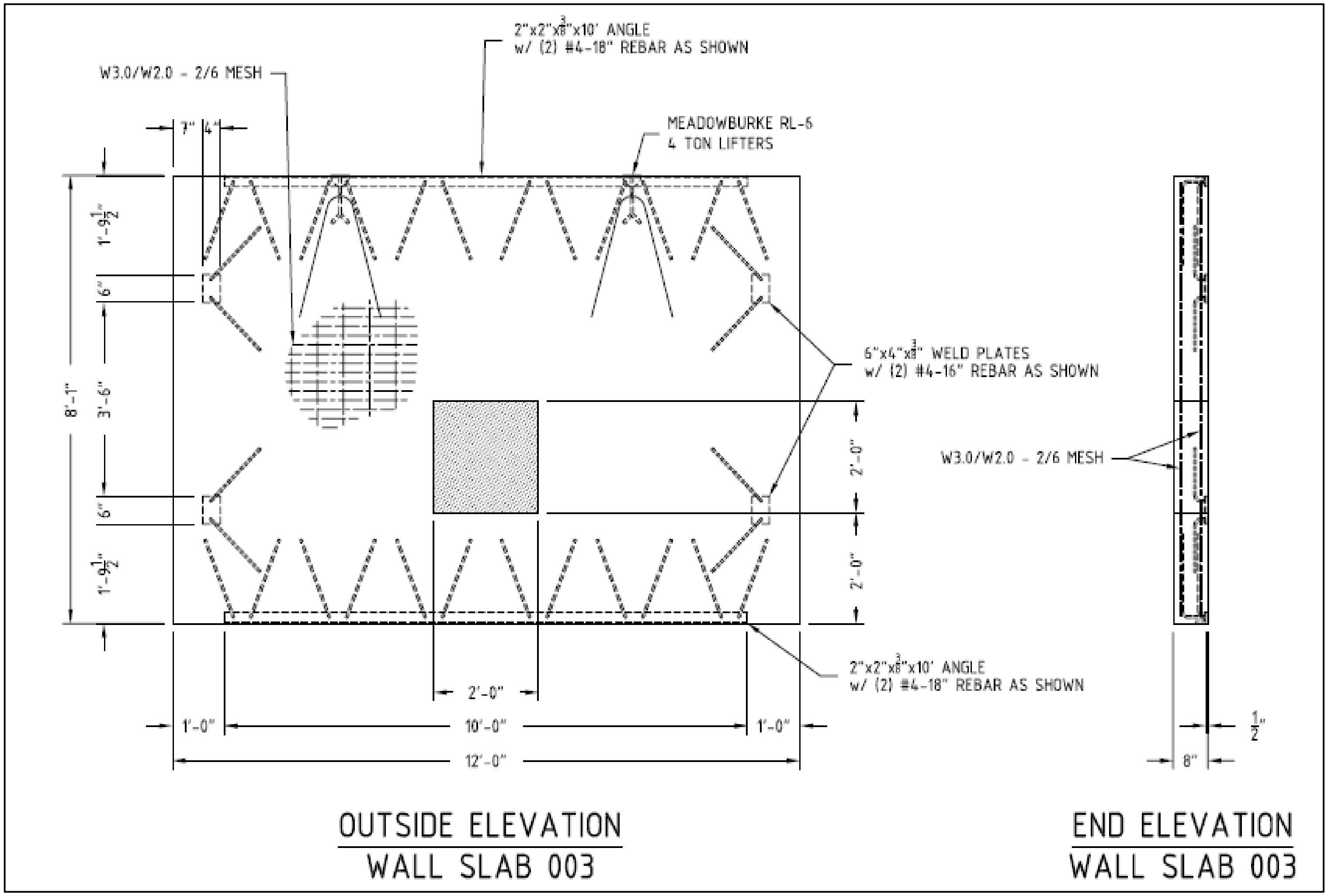

Figure 2.10 Typical Door Panel

2.2.2.4 Door Panel

The door panel of the prototype building (as shown in Figure 2.10) had dimensions of 8 ft-1 in high x 12 ft wide x 8 in thick (2.44 m x 3.66 m x 20.32 cm). Weld plates of dimensions 6 in x 4 in x 3/8 in (15.24 cm x 10.16 cm x 0.95 cm) with two (2) No. 4 bars, 16 in (40.64 cm) long were placed every 3 ft-6 in (1.0 m) on the door panel. There were two (2) Meadowburke RL-6, four (4) ton lifters embedded on the panels for assembly purposes. The floor panel and the door panel were joined together by angles of dimensions 2 in x 2 in x 3/8 in (5.08 cm x 5.08 cm x 0.95 cm).

Steel welded wire meshes of size W3.0/W2.0 – 2/6 were used throughout the entire panel for the model. Steel angles of 2 in x 2 in x 3/8 in (5.08 cm x 5.08 cm x 0.95 cm) dimensions, welded between the panels were used as reinforcement between roof-wall connections, wall-floor slab connections. The door opening of dimensions 2 ft x 2 ft (0.61 m x 0.61 m) was placed 5 ft (1.52 m) from the edge and 2 ft (0.61 m) from the bottom of the panel.

2.2.2.5 Back Wall Panel

The back wall panel of the prototype building had dimensions of 8 ft -1 in high x 12 ft wide x 8 in thick (2.44 m x 3.66 m x 20.32 cm), the same as shown in Figure 2.10, without the opening. Weld plates of dimensions 6 in x 4 in x 3/8 in (15.24 cm x 10.16 cm x 0.95 cm) with two (2) No. 4 bars, 16 in (40.64 cm) long were placed every 3 ft-6 in (1.0 m) on the back wall panel. There were two (2) Meadowburke RL-6, four (4) ton lifters embedded on the panels for assembly purposes. The floor panel and the back wall panel were connected by angles of dimensions 2 in x 2 in x 3/8 in (5.08 cm x 5.08 cm x 0.95 cm).

Steel welded wire meshes of size W3.0/W2.0 – 2/6 were used throughout the entire panel for the model. Steel angles of 2 in x 2 in x 3/8 in (5.08 cm x 5.08 cm x 0.95 cm) dimensions, welded between the panels were used as reinforcement between roof-wall connection, and wall-floor slab connection.

2.2.3 Information on Neopor

Basic characteristics based on previous testing have given the following information about Neopor:

1. Increasing the quantities of sand and cement and decreasing the amount of foam yield higher densities and consequently higher strength.
2. For structural elements, a density of 1,000-1,600 kg/m^3 (62-100 pcf) is recommended.
3. Tests performed on slabs of densities between 1,000-1,400 kg/m^3 have been performed to monitor corrosion in reinforcement. After four months duration tests there was no corrosion on a 1,200 kg/m^3 specimen.
4. A wall panel with a density of 1,200 kg/m^3 (75 pcf), for example, cured in open air, after 28 days showed a shrinkage of 0.215 mm/m. Between the 28th and 90th day the shrinkage was even less than with traditional concrete.

Physical properties that can be achieved using cellular concrete made with Neopor:

1. Density 400-1,800 kg/m^3 (24-108 pcf)
2. Achievable strength 1-25 N/mm^2

3. Shrinkage behavior 1,200 kg/m^3- 0.215 mm/m
4. Thermal conductivity 0.082-0.555 (W/m K)
5. Fire rating non combustible DIN 4164
6. Water absorption approximately 5% @ density of 1,200 Kg/m^3; no condensation closed cellular concrete
7. If one kilogram of Neopor is diluted in 40 liters of water; it yields approximately 540 liters of foam.

2.3 Material Tests

2.3.1 Compressive Strength

Different types of tests have been conducted with the purpose of obtaining material and structural properties of this lightweight concrete. All the tests performed followed the specifications of American Standards for Testing Material (ASTM) Specifications. To perform compressive strength tests, specimens of dimensions of 6 in (152.4 mm) x 12 in (304.8 mm) cylinders were used.

Figure 2.11 Compressive Strength Test Setup

Results from the compressive strength tests for the 28th day at a unit weight of approximately 91 pcf (1,463 kg/m^3) showed that the strength for this cellular concrete is approximately 1,117 psi (77.93 kg/cm^2). The following relationship between compressive strength and unit weight for this lightweight concrete at an age of 28 days was found.

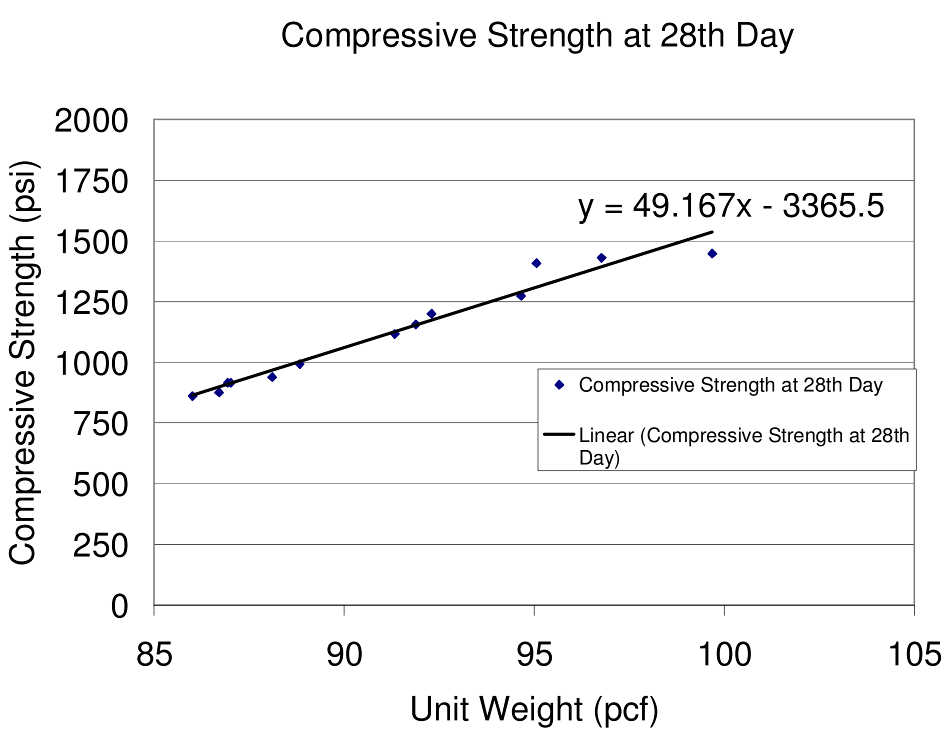

Figure 2.12 Relationship between f'_c and Unit Weight of Cellular Concrete at 28th Day

2.3.2 Tensile Strength

Following the compressive strength tests and using the same mix design batches, flexural strength of beams was tested to measure the tensile strength of the concrete. This test was performed following the specifications of ASTM C78 (Third Point

Loading Test). The test was performed for both beams with and without steel reinforcement. Both types of beams contained glass fibers to improve ductility, and control creep and shrinkage. Beam specimens of dimensions 6 in (152.4 mm) width x 6 in (152.4 mm) depth x 24 in (609.6 mm) were tested.

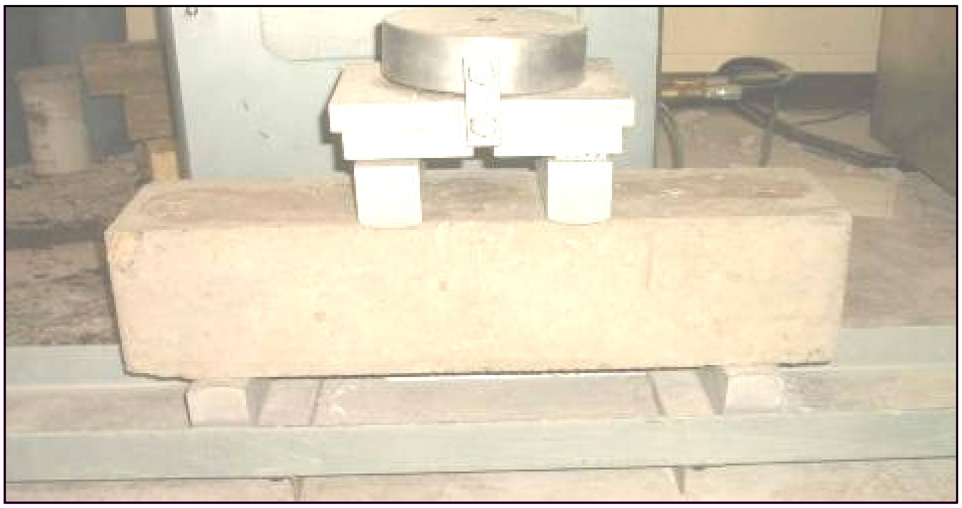

Figure 2.13 Tensile Strength Test Setup

Results from the tensile strength tests at the 28th day for a unit weight of approximately 94 pcf (1510 kg/m^3) showed that the strength for this cellular concrete is approximately 259 psi (18.06 kg/cm^2). This strength represents approximately a 23% of the strength the concrete provides in compression. The following relationship between compressive strength and unit weight for this lightweight concrete at an age of 28 days was found.

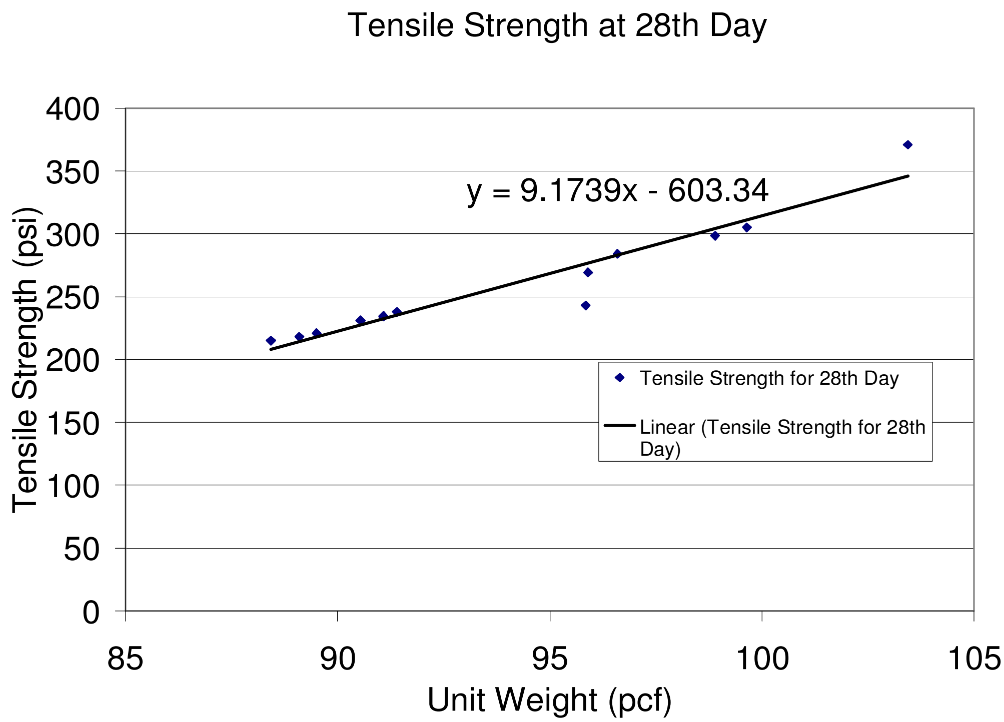

Figure 2.14 Relationship between f_t and Unit Weight of Cellular Concrete at 28^{th} Day

2.3.3 Pull-Out Strength

Pull-out tests were performed by Ake Pyamaikongdech using the same mix design batches to determine pull-out strength of the cellular concrete. This test is very important because the chemical properties of the concrete mix can affect the bond between the rebar and the concrete. Pull-out tests were performed following the specifications of ASTM C234 standard. The specimens tested were concrete cylinders with dimensions of 6 in (152.4 mm) diameter x 12 in (304.8 mm) length. The steel rebar used were No.4 bars placed at an embedded length of 4 in (10.16 cm) and 12 in (30.48 cm) during the casting process.

Figure 2.15 Specimen used for Pull-Out Strength Test

Results from the pull-out strength tests for the 28^{th} day for a unit weight of approximately 92 pcf (1,479 kg/m^3) showed that the bond strength for this cellular concrete is approximately 246 psi (17.16 kg/cm^2) as shown in Figure 2.16. Bond stiffness was also examined in these tests. Results from the tests for the 28^{th} day for a unit weight of approximately 93 pcf (1,494 kg/m^3) showed a bond stiffness of approximately 13,132 psi (916 kg/cm^2) as shown in Figure 2.17.

Pyamaikongdech (2007) concluded that bond strength and stiffness parameters increase with the increase of the concrete unit weight in a linear fashion. The following relationship between bond strength and unit weight for this lightweight concrete at an age of 28 days was found.

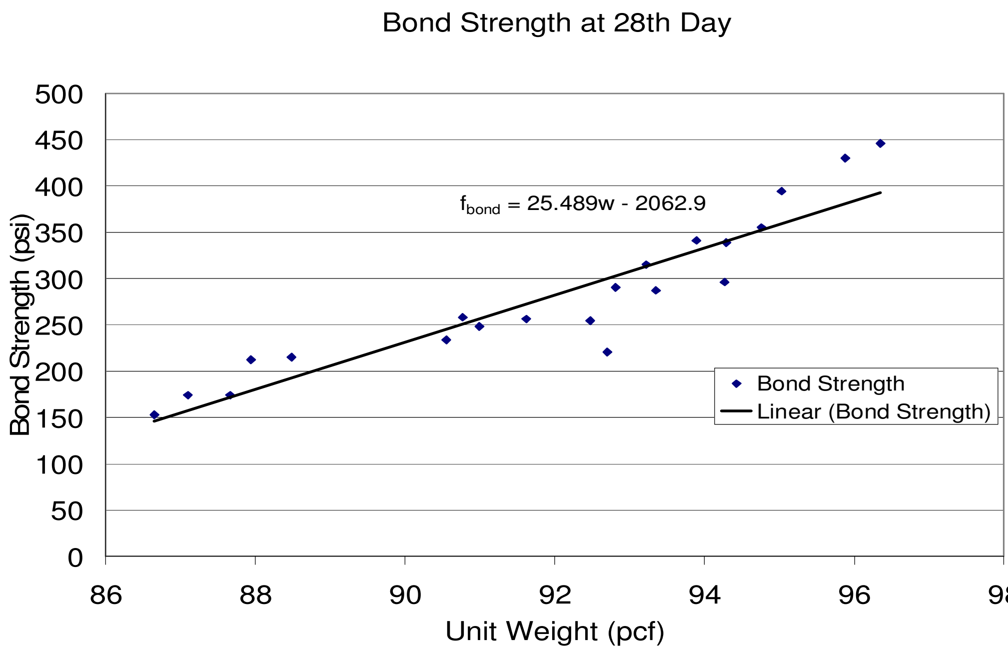

Figure 2.16 Relationship between Bond Strength and Unit Weight of Cellular Concrete at 28th Day

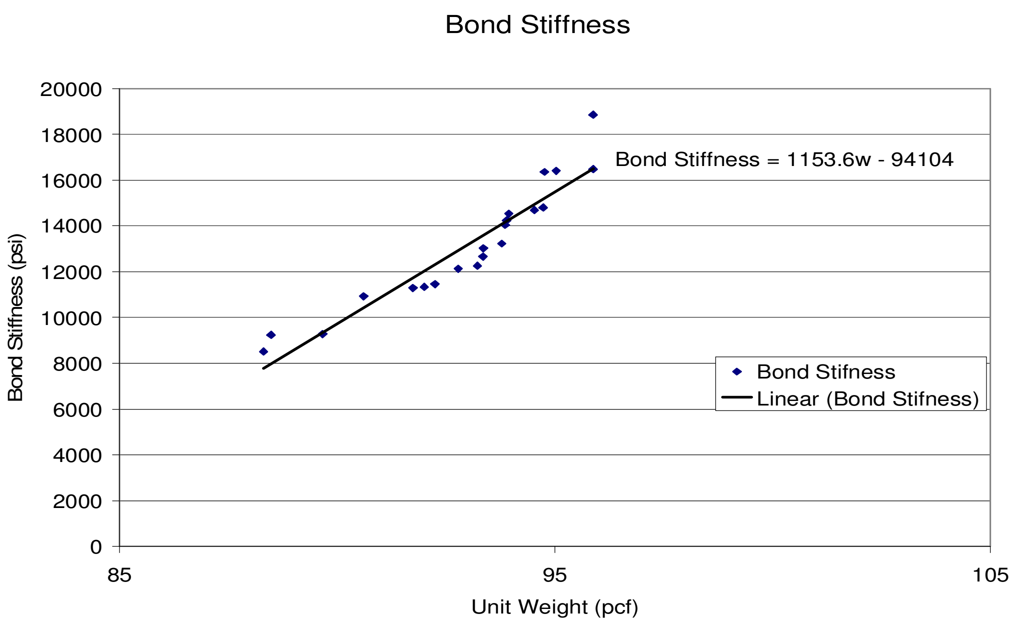

Figure 2.17 Relationship between Bond Stiffness and Unit Weight of Cellular Concrete at 28th Day

2.3.4 "α": Relationship between Compressive Strength and Tensile Strength

There is no research data about the relationship between compressive and flexural strength for this foamed-based lightweight concrete. For this reason, during the experiments, data was collected to determine this relationship. Values were obtained using this data and the equation, as reported by Pyamaikongdech (2007), presented below.

$$f_t = \alpha\sqrt{f'_c}$$

Where,

α = coefficient defining the relationship between tensile and compressive strength of concrete.

The results obtained from Pyamaikongdech (2007) research tests indicated an average value of α of 7 for this cellular concrete. These values ranged from 5 to 8 for an average unit weight of 93 pcf (1,495 kg/m^3) as shown in Figure 2.18. The values obtained from the tests indicate that the range is similar to those reported by the American Concrete Institute (ACI) for normal weight concrete in which the values of α fluctuates approximately from 5.13 to 8.22.

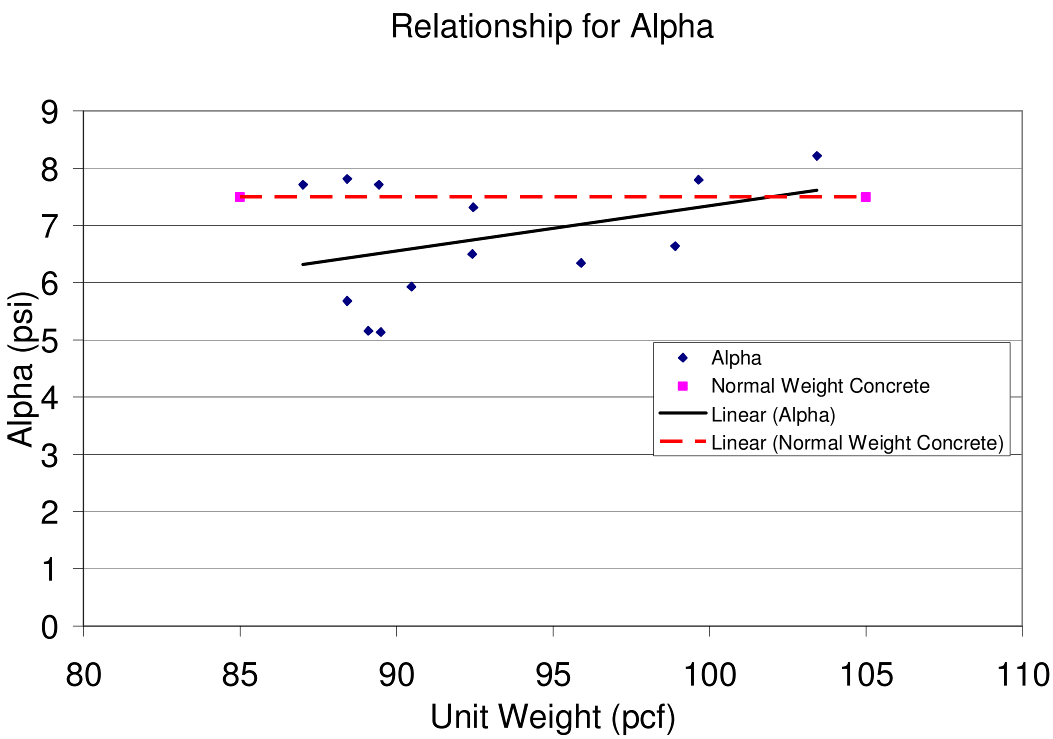

Figure 2.18 Relationship between Alpha and Unit Weight of Cellular Concrete at 28th Day

CHAPTER 3

EXPERIMENTAL PROGRAM II:
PRESSURE BUILDINGS

3.1 Introduction

Internal pressure tests were performed to simulate the effect of strong wind force and wind pressure on buildings. The purpose of performing these tests was to study the wind load capacity and structural behavior of the cellular lightweight concrete precast panels using steel as the main reinforcement. Pressure tests were performed following a procedure designed by the investigators because this type of full-scale test has not been studied before and therefore, does not appear on the ASTM standards.

The building prototype tested consisted of precast panels joined together using embedded steel angles. The steel reinforcement details were discussed earlier in Section 2.2.2 of Chapter 2. The detailed procedure on how to perform these tests is described in the following chapters.

This chapter presents a description of the test setup and instrumentation used to perform the pressure tests on the prototype building constructed with lightweight panels. It also presents the process of designing testing procedure and achieving the ideal instrumentation and pressure building setup. The process of fabrication of the panels is summarized together with the test setup.

3.2 Instrumentation

Following is a list of the equipment and instrumentation used in this research:

3.2.1 Pressure Transducers

Pressure transducers were used to measure the pressure inside the bag placed in the building prototype during the test (see Figure 3.1). Two pressure transducers were placed inside the airbag. For the first test, one of the pressure transducers malfunctioned. For the other tests, two transducers were used to compare the results. These transducers measured the increasing pressure while the airbag was inflated and indicated the loss of pressure when failure occurred. Since these transducers are highly sensitive to excessive movement, they were placed inside a foam cube (see Figure 3.2) to provide cushion while the transducers moved inside the airbag as a consequence of the inflating process.

Figure 3.1 Pressure Transducers[2]

[2] Picture was obtained from the Omega Company website.

Figure 3.2 Foam Cover for Pressure Transducer

3.2.2 Wire Potentiometers

Wire potentiometers (see Figure 3.3) were used to measure the deflection of the wall while pressure was applied inside the test building. Two wire potentiometers were used for each test, which were placed on different panels of the building prototype on the side walls and the roof. Measurements of deflection were recorded using the data acquisition system.

Figure 3.3 Wire Potentiometer[3]

[3] Picture was taken from the Celesco Company website.

The potentiometers were secured to a wood stand (see Figure 3.4) at the mid-height of the wall. The wire cable was attached to the wall using a screw. This way, the wire potentiometer read the length of the cable at the beginning, and every second during the test which indicated the deflection of the wall.

Figure 3.4 Wire Potentiometer Setting

3.2.3 Pressure/Vacuum Pump

The pressure/vacuum pump was used to fill the airbag placed inside the building prototype. A hose was connected from the air pump into the airbag. A gage valve was connected to this air pump to control the rate at which air was being pumped (see Figure 3.6). Controlling this rate of air helped the investigators notice cracks.

Figure 3.5 Pressure/Vacuum Pump

Figure 3.6 Gage Valve

3.2.4 Data Acquisition System

The data acquisition system was used to transfer and convert the data readings from the pressure transducers and wire potentiometers to computer language and later into graphs.

3.2.5 Airbag

The airbag was used to apply pressure to the walls of the building prototype simulating wind pressure. In order to find the ideal airbag several trials were performed. The following subsections present a description of each of the bags.

3.2.5.1 Airbag #1

The first airbag was hand made by research students. It consisted of several 10 mil tarps put together with large amounts of duct tape. Even though this airbag was resistant, it did not work because the airbag dimensions were too big. During the tests trial, the airbag did not inflate completely and eventually air leakage occurred.

Even though this airbag did not work, it showed that there was one detail that needed to be taken into consideration when choosing an airbag: either the airbag material should be resistant enough to not be affected by the rough edges of the concrete and steel angles inside the building prototype, or measures should be taken to cover the rough edges inside the building prototype.

Figure 3.7 Airbag #1

3.2.5.2 Airbag #2

The second airbag was machine made by a manufacturer. It consisted of a cube-shaped airbag made of thick tarps sewed together. This airbag was thicker and more resistant than Airbag #1 but it did not work because air leaked through the seams so the airbag did not inflate completely during the tests trial.

Once again the investigators found more parameters to be taken into consideration when designing the airbag. For the next trial, dimensions and sealing method would be the priority.

3.2.5.3 Airbag #3

The third and final airbag was machine made by a manufacturer also. It consisted of thick tarps sealed together by heat treatment. This airbag was thicker and more resistant than Airbag #1 and #2. Airbag #3 showed resistance and worked until the tests were finished as shown in Figure 3.8.

Figure 3.8 Airbag #3

3.2.6 Building Prototype

Building Prototypes were used to represent a relatively large test building with common panel sizes with dimensions of 12 ft (3.66 m) wide x 12 ft (3.66 m) long x 8 ft (2.44 m) high. These building prototypes were made of precast lightweight concrete panels.

3.2.6.1 Sealants

During the first test trials different sealants were used to try to completely seal the spaces between the panels. To seal the inside of the building and cover any sharp edges that could damage the airbag, a bituminous material, Ram-Nek, was used. Ram-Nek is a pre-formed joint sealant that is commonly used to provide a watertight bond to fresh and cured concrete surfaces. Ram-Nek did not serve its purpose to seal between the roof-to-wall, and wall-to-wall panels because it did not adhere to the surface.

Possible causes for not adhering could have been that the panel surfaces were not cleaned and primed before placing the Ram-Nek.

Figure 3.9 Building Prototype

The second sealant used as an attempt to seal these spaces was a silicon-based sealant. This sealant was placed in the spaces between the panels, but on the outside of the building prototype (see Figure 3.9). This sealant did not work because when air pressure was applied to it, it formed bubbles that eventually popped and let the air out.

After trying these sealants, it was determined that sealing hermetically the building prototype was not indispensable. The ideal condition in order for the test to work properly was to find an airbag that covered entirely the inside surface of the building and these spaces between the panels.

3.2.6.2 Door Setup

A wood panel was used to close the door opening of the building prototype. A small hole was drilled through the panel to put the pressure transducers and air pump hose inside the airbag. As the airbag inflated, the wood panel was pressed against the wall maintaining it in place. Figure 3.10 presents this setup.

Figure 3.10 Door Setup

3.3 Description of Test Setup

3.3.1 Casting and Curing of Panels

The wall panels were cast in steel forms after arranging the steel reinforcement cage and bars. After pouring the concrete, the panels were cured at ambient temperature for 24 hours. After the panels were removed from the form, these were stored outside and shipped to the precast concrete plant were the tests took place 28 days after casting.

3.3.2 Setup of Building Prototype

The first step in the testing procedure was to place pressure transducers inside the airbag. These transducers are used to measure actual internal pressure during the test. The airbag used to apply the pressure uniformly to the walls and roof panels was placed inside the building prototype. The 2 ft (0.61 m) x 2 ft (0.61 m) wood door of the building prototype was sealed to keep uniform pressure inside.

Wire potentiometers were placed on the outside of the side walls, back wall, and roof of the house were used to measure deflection during the test. A data acquisition system was programmed and connected to the pressure transducers and wire potentiometers in order to create the pressure vs. deflection plot as the test progressed.

3.4 Testing Procedure

After all the equipment was set up (see Figure 3.11), the air pump was used to inflate the airbag inside the building prototype. When the airbag was inflated, the pressure was continued to be applied in smaller increments using a pressure gage. While the walls and roof were being pressurized, this data was being recorded for each second through the data acquisition system.

As internal pressure and panel deflection increased, small cracks started to form on the walls and/or roof. As cracks started to become larger and wider, the air was pumped more slowly until complete failure occurred.

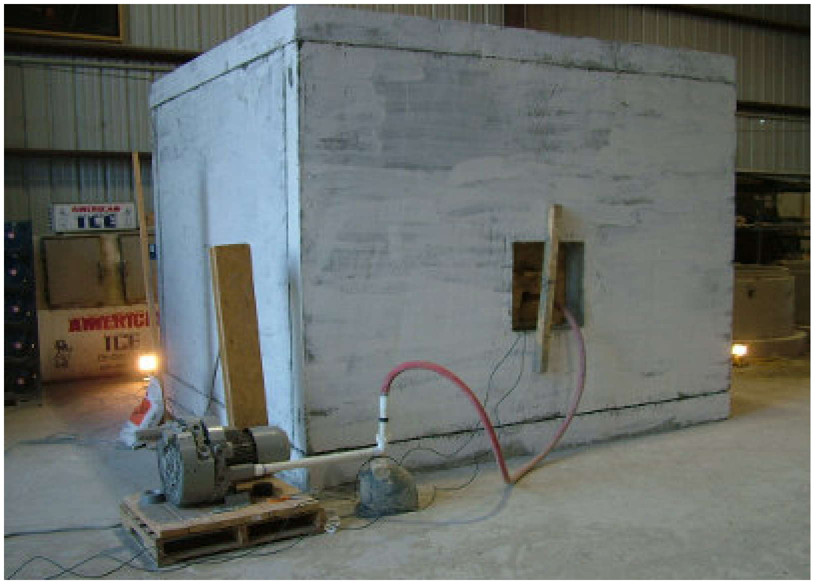

Figure 3.11 Setup of Building Prototype Test

CHAPTER 4

TESTS RESULTS

4.1 Introduction

This chapter presents the results of the building prototype tests performed at the Hanson Grand Prairie Precast Plant. The information obtained includes parameters such as ultimate pressure capacities, wall and roof deflection values, pressure versus deflection curves, failure locations, and failure modes. The relationship between the wind speed and maximum pressure experienced inside the building prototypes before failure is also presented in this chapter.

4.2 Wind Loads and Internal Pressure

4.2.1 Summary

The ASCE 7 Code and the IBC 2006 Code show that critical wind speed in the southern area of the United States is 150 mph (241.4 km/hr), as shown in Figure 4.1. After applying internal pressure to the building prototype, the first cracks started to appear at an approximate pressure of 2.0 psi (0.14 kg/cm^2). This pressure indicated a resistance to an approximate wind speed of 424 mph (682 km/hr). The calculations for the wind speed-pressure relation are shown in Section 4.2.2.

The results of these tests indicated that the test prototype resisted an average internal pressure of 4.13 psi (0.29 kg/cm^2) before failure while deflecting 0.30 inches

(0.76 cm) in the weakest panel. This pressure is equivalent to an approximate wind speed of 610 mph (982 km/hr).

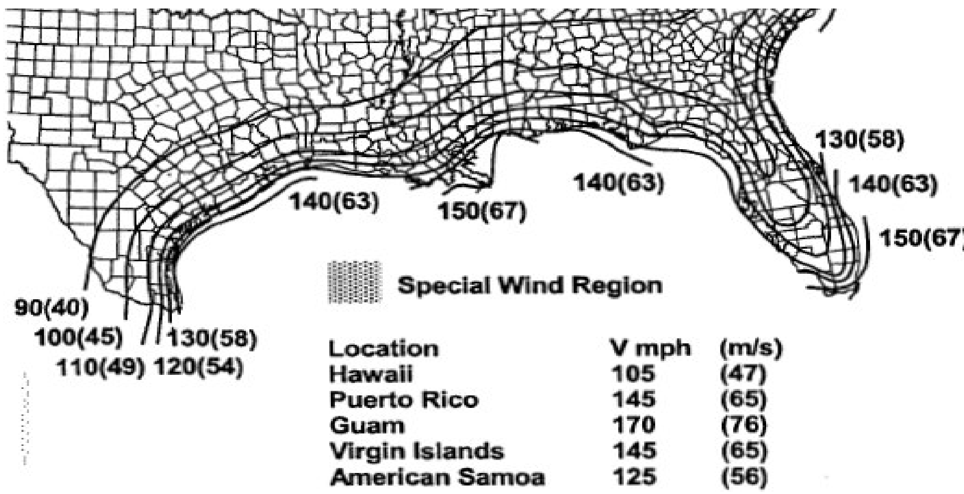

Figure 4.1 Wind Speed Parameters[4]

4.2.2 Wind Speed-Pressure Relation

A wind speed-pressure relation was determined to provide information on the resistance of these precast panels to hurricane winds. The ASCE 7 Code was used for this purpose. To determine the wind speed-pressure relation, a 3-second gust wind speed for Louisiana and other hurricane prone states was used. This wind speed was 150 mph (241 km/hr) as shown in Figure 4.1. Equation 6-15 of the ASCE 7-02 code was used to determine the velocity pressure in the building prototype.

$$q_{ext} = 0.00256 k_z k_{zt} k_d v^2 I \qquad \text{(ASCE 7-02 EQN 6-15)}$$

[4] Picture taken from the ASCE 7-02 Code.

Where,

$$k_z = 0.85$$
$$k_{zt} = 1.0$$
$$k_d = 0.85$$
$$v = 150 mph$$
$$I = 1.0$$

$$q_{ext} = 0.00256 \times 0.85 \times 1.0 \times 0.85 \times 150^2 \times 1.0 = 41.62 psf$$

Where,

k_z= velocity pressure exposure coefficient

k_{zt}= topographic factor

k_d = wind directionality factor

v= basic wind speed in mph

I=importance factor

The values for these parameters were determined from tables provided in the ASCE 7-02 Code. An Exposure C was assumed to include a variety of topographic conditions. The importance factor, I = 1.0 was determined from Table 6-1 of the ASCE 7-02 Code. This value was obtained by assuming a building and structure classification category II for hurricane prone regions with wind speed higher than 100 mph.

The value of 0.85 for the parameter k_z was determined from Table 6-3 of the ASCE 7-02 Code, for a height of 12 ft (3.66 m), Exposure C and components and cladding. The value of 0.85 for k_d was determined from Table 6-4 of the ASCE 7-02

Code for a structure type: Building- Components and Cladding. A value of 1.0 was assumed for the parameter k_{zt} from Figure 4-6 of the ASCE 7-02 code.

To determine the net pressure exerted on the building prototype, equation 6-17 from the ASCE 7-02 was used.

$$p_{net} = qGC_p - q_i(GC_{pi}) \qquad \text{(ASCE 7-02 EQN 6-17)}$$

Where,

$G = 0.85$ *Gust Factor*

$$q = q_i = 41.62\,psf$$
$$G = 0.85$$
$$C_p = 0.8$$
$$C_{pi} = (-0.18)$$

$$p_{net} = (41.62 \times 0.85 \times 0.80) - (41.62 \times -0.18) = 35.79\,psf$$
$$= 0.25\,psi$$

Where,

q = *pressure for windward walls evaluated at height z above the ground*

q_i = *pressure for windward walls, side walls, leeward walls and roofs of enclosed buildings*

G =*Gust factor*

C_p = *external pressure coefficient*

GC_{pi} =*internal pressure coefficient*

The value of 0.85 corresponded to the G parameter for Exposure C. The value of 0.8 for the parameter C_p was determined from Figure 6-6 of the ASCE 7-02 Code for the windward wall. The value of -0.18 for GC_{pi} was determined from Figure 6-5 (ASCE Code) for enclosed buildings. The values for q and q_i are the same for the case where height does not vary, and were determined above.

To convert the maximum pressures obtained from the tests to equivalent wind speeds, the following relation was derived:

$$\frac{v^2}{150mph} = \frac{pressure_{test}}{p_{net}} \tag{4.1}$$

$$v = \sqrt{\frac{pressure_{test} * 150^2}{0.25}} \tag{4.2}$$

The values obtained for each test are presented in the following section.

4.3 Tests Results

4.3.1 Test No.1

The failure in this test was sudden and unexpected. Failure occurred at a pressure of 3.14 psi (0.22 kg/cm^2) which is equivalent to a wind speed of 532 mph (856 km/hr). While small cracks started to appear in the north wall at a pressure of 2.5 psi (0.18 kg/cm^2), failure occurred by the explosion of the east wall breaking in half due to pull-out of the steel reinforcement at the joint connecting the bottom of the east wall to the roof slab and the floor slab.

Figure 4.2 Building Prototype Before Test No.1

Figure 4.3 Building Prototype after Test No.1

Figure 4.4 Explosion at East Wall

Figure 4.5 Failure of the Wall

Figure 4.6 Failure at Roof Slab

Figure 4.7 presents the plot of internal pressure versus the panel deflection. The plot shows that the east wall was much more flexible than the south wall which was casted using regular weight concrete.

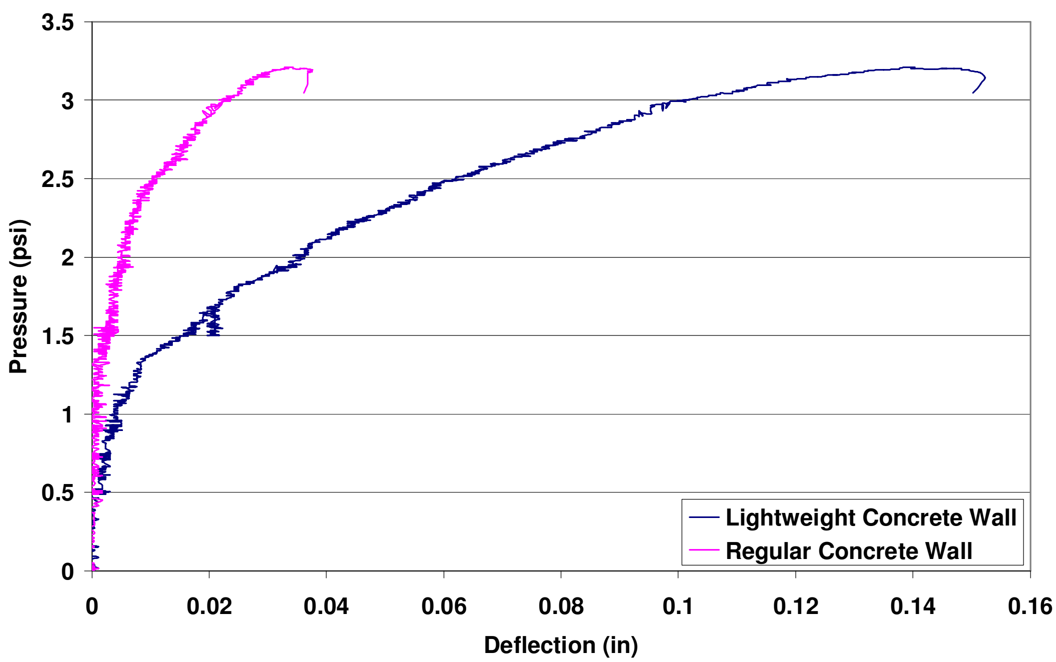

Figure 4.7 Pressure versus Deflection for Test No.1

4.3.2 Test No.2

The second test was performed using the same procedure as Test No. 1. However, failure occurred in a different setting. Cracks started appearing near the roof panel at a pressure of 2 psi (0.14 kg/cm^2), equivalent to a wind speed of 424 mph (682 km/hr).

Figure 4.8 Building Prototype Before Test No. 2

Failure occurred in this panel, the roof, as shown in Figure 4.9 at a pressure of 4.5 psi (0.32 kg/cm^2). This failure was not explosive though it occurred considerably fast. The ultimate pressure experienced was equivalent to an approximate wind speed of 636 mph (1,024 km/hr). Figure 4.9 presents the plot of internal pressure versus the panel deflection. Further modifications to the structural design of the roof panels were proposed after this test was performed. These modifications are out of the scope of this study. This basically confirms the variability in material, indicating more ductile roof panel when compared to wall panel.

Figure 4.9 Building Prototype After Test No. 2

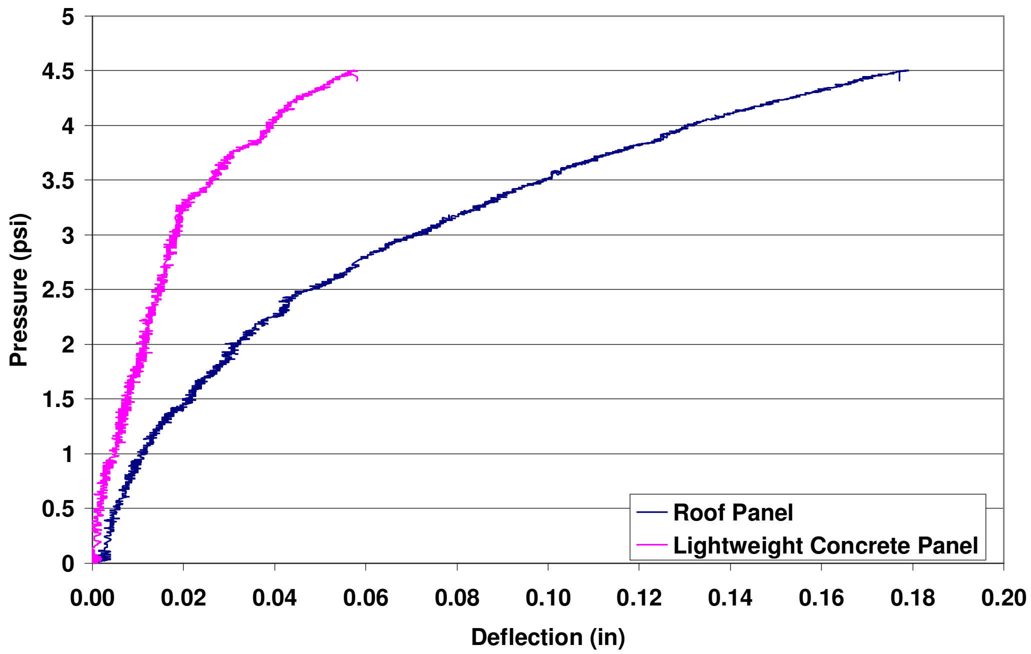

Figure 4.10 Pressure versus Deflection for Test No. 2

4.3.3 Test No. 3

The third building prototype test was performed using the same procedure as Test 1 and 2 with the exception that a valve was installed to the air pump to control the air entering the building prototype. Cracks started appearing near the roof panel at a pressure of 2 psi (0.14 kg/cm^2). The failure occurred in the joints between the bottom of the west panel and the floor slab as shown in Figures 4.13 through 4.17 at a pressure of 4.76 psi (0.33 kg/cm^2). This failure was as explosive as in Test No. 1. However, the wall did not pull-out completely as shown in the aforementioned figures. The pressure at which failure occurred is equivalent to an approximate wind speed of 655 mph (1,054 km/hr).

Figure 4.11 Building Prototype before Test No. 3

Figure 4.12 Building Prototype during Test No. 3 (Northwest Side)

Figure 4.13 Failure of Wall Sequence 1

Figure 4.14 Failure of Wall Sequence 2

Figure 4.15 Failure of Wall Sequence 3

Figure 4.16 Failure of Wall Sequence 4

Figure 4.17 Failure of Wall Sequence 5

The following figure presents the plot of the values of pressure and deflection measured during the test. It can be noticed that even though the first cracks appeared on the roof panel, the west wall deflected much more than the roof.

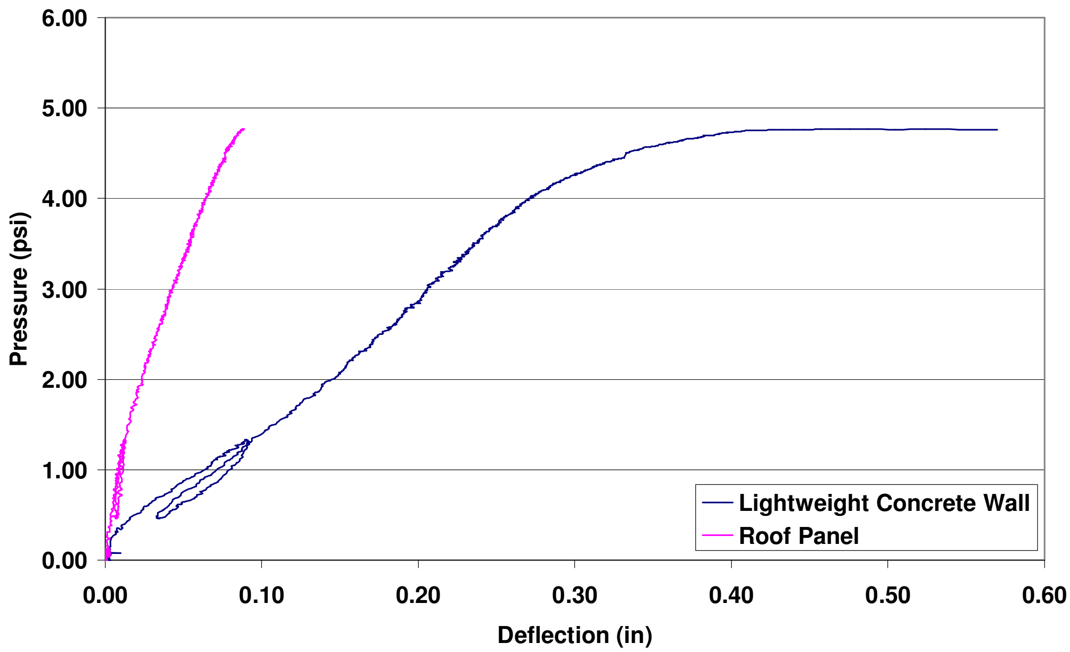

Figure 4.18 Pressure versus Deflection for Test #3

4.3.4 Cracking Patterns and Failure Loads

The following table presents a comparison between the cracking patterns and failure loads resulting from each test. It is shown that even though failure occurred at different pressures, cracks appeared in each of the tests at approximately the same pressure.

Table 4.1 Comparison of Tests Results

Test	Failure Pressure	Representative Wind Speed	Panel where Failure Occurred	Pressure at which Cracks Appeared
#1	3.14 psi ($0.22\ kg/cm^2$)	532 mph	East Wall Panel (Lightweight Concrete)	2.5 psi
#2	4.5 psi ($0.32\ kg/cm^2$)	636 mph	Roof Panel (Lightweight Concrete)	2.0 psi
#3	4.76 psi ($0.33\ kg/cm^2$)	655 mph	West Wall Panel (Lightweight Concrete)	2.0 psi
Average	4.13 psi ($0.29\ kg/cm^2$)	610 mph	Lightweight Concrete Panels	2.17 psi

Table 4.2 presents a summary of the results of all the tests performed. In this table the maximum pressure and deflection recorded before failure occurred are shown.

Table 4.2 Test Results for Pressure and Deflection before Failure

		Dmax (in)				
Test	Pmax (psi)	East Wall	South Wall	West Wall	Roof	Ultimate Deflection (in)
1	3.14	0.15	0.04	*	*	0.15
2	4.5	*	*	0.18	0.06	0.18
3	4.76	*	*	0.57	0.09	0.57

* No wire potentiometers were placed in these panels to measure deflection.

CHAPTER 5

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The conclusions of this study advances in the following forefronts:

- Design the instrumentation needed to conduct pressure tests to simulate the effect of high wind pressure on these fiber-reinforced foam-based lightweight concrete precast panels.
- This research was conducted as a means of creating a test procedure for wind loads for full scale building prototypes.
- This lightweight concrete presented material properties and characteristics similar to those of normal-weight concrete.
- In some cases, the time required to cure the panels is about 24 hours before removal from the form. This provides a material that can be used where fast construction is needed.
- The tests showed ductile behavior of the panels before failure.
- The low bond strength of the foam-based lightweight concrete caused the pull-out failure in most of the tests.
- Both, material and panels indicated that consistent mixes were not attainable.

- The results from Test No. 1 showed brittle failure even though the building prototype experienced wind speeds that greatly exceed those form hurricanes. This test showed that attention should be given to the bond between the steel reinforcement and the concrete. In this test, the building prototype experienced a maximum pressure of 3.14 psi (0.22 kg/cm^2) before failure. The results from this test could not be compared to normal weight concrete because this testing procedure was developed for this research. No other tests for wind loads have been performed in this manner before this research.
- The results from Test No. 2 showed that the weakest panel was the roof panel. This test showed a more moderate failure than the other tests. It experienced a maximum pressure of 4.5 psi (0.32 kg/cm^2).
- The results from Test No. 3 confirmed the weak bond between steel reinforcement and concrete. Even though the behavior was ductile, the sudden detachment of wall panels indicates a need to improve the reinforcement in the joint between the floor slab and wall panels. This building prototype experienced a maximum pressure of 4.76 psi (0.33 kg/cm^2) before failure. The results from this test were compared to normal weight concrete and it showed that the developed panels behaved more ductile than the normal weight concrete.

- The internal pressure tests showed a relationship between the pressure applied to the walls and the capacity of deflection of the wall before reaching failure. Information about the behavior of the wall panels produced of this type of lightweight concrete was not available prior to this research.
- Even though the structural design should be revised for modifications to the steel reinforcement, the lightweight concrete showed high resistance to high wind induced pressures.

5.2 Recommendations

This study recommends the following future research studies to complement the work presented here:

- Chemical properties of this concrete should be studied to see how it reacts with the steel reinforcement.
- Corrosion effects should be studied and taken into account to determine the quality of the proposed concrete with time.
- This lightweight concrete should be produced and tested using different foaming agents for performance-based comparison.
- The building pressure tests with different test specimen sizes are recommended.
- A detailed finite element model analysis of the test specimen is recommended.
- More material properties should be investigated such as: shrinkage, durability, creep, and temperature effects on strength, and freeze-thaw properties when using this foaming agent.
- Development of informed mix design procedures to produce this foam-based lightweight concrete with consistent material properties is highly recommended.

APPENDIX A

PICTURES OF MIXING AND CASTING

Figure A.1 Slump Test

Figure A.2 Small Batch Mixing

Figure A.3 Formwork for Panels

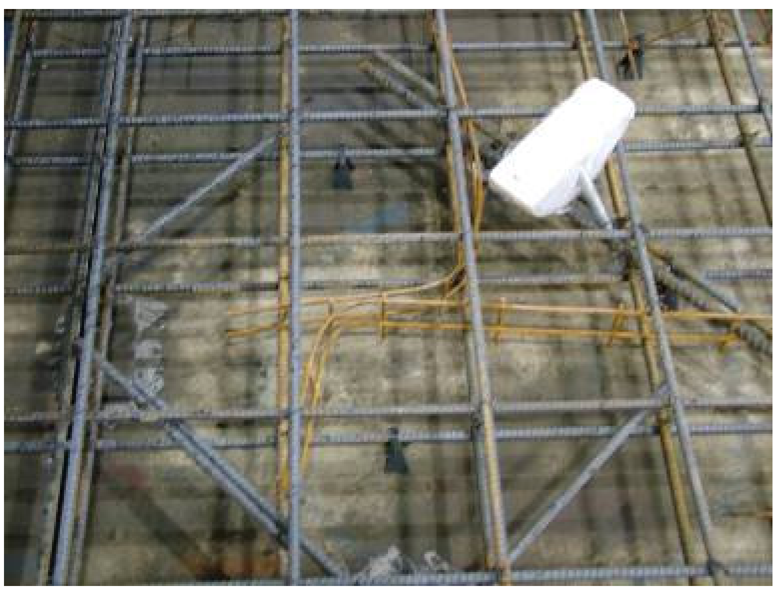

Figure A.4 Reinforcement

Figure A.5 Reinforcement at 45 degrees

Figure A.6 Reinforcement at 45 degrees

Figure A.7 Formwork for Door Panel

Figure A.8 Formwork with Wire Mesh

Figure A.9 Casting of Panels

Figure A.10 Pouring of Concrete

Figure A.11 Casting of Panels

Figure A.12 Casting of Panels

APPENDIX B

PICTURES OF TEST #1

Figure B.1 Setup for Test

Figure B.2 Failure of Wall

Figure B.3 Failure of Wall

Figure B.4 Left Half of Broken Wall

Figure B.5 Right Half of Broken Wall

APPENDIX C

PICTURES OF TEST #2

Figure C.1 Setup for Test

Figure C.2 Failure at Roof

APPENDIX D

PICTURES OF TEST #3

Figure D.1 Cracks at Door before Testing

Figure D.2 Cracks at Door before Testing

Figure D.3 Crack at Roof before Testing

Figure D.4 Setup for Test

Figure D.5 Failure at West Wall

Figure D.6 Reinforcement of Panel

Figure D.7 Left Upper Corner of West Wall

Figure D.8 Roof-West Wall View

Figure D.9 Lower Right Corner of West Wall View

Figure D.10 West Wall

Figure D.11 Upper Right Corner View of West Wall

Figure D.12 Failure of Wall Panel

Figure D.13 Upper Middle Section of West Wall

Figure D.14 Lower Left Corner of West Wall

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BIOGRAPHICAL INFORMATION

Francheska E. Seijo Montes was born February 5, 1984 in San Juan, the capital city of Puerto Rico. She finished her school degree in 2001 from Colegio Puertorriqueño de Niñas in Puerto Rico. During summer of 2005, Francheska attended a summer research program called Research Experience for Undergraduates Program (REU) at University of Texas at Arlington (UTA) where she and two more teammates wrote a technical paper titled “Experimental Investigation of Shear Capacity of Precast Reinforced Concrete Box Culverts”. This paper won this team the Superior Performance Award from this program. In July 2006, she graduated Magna Cum Laude from her Bachelor’s of Science in Civil Engineering degree from the Polytechnic University of Puerto Rico. After her graduation, on August 2006, Francheska entered the graduate program in Structural and Applied Mechanics at the University of Texas at Arlington (UTA) where she did research under Dr. Ali Abolmaali instruction to complete her Master of Science in Civil Engineering degree. In November 2006, she presented her research in the National Hispanic Engineers Conference and won the First Prize in the Technical Poster Competition under the graduate students’ category presenting her research “Experimental Analysis of Fiber-Reinforced Foam-Based Lightweight Concrete Precast Wall Panels”. In the present she works for the design company CMA Architects & Engineers LLP in Guaynabo, Puerto Rico.