STRUCTURAL PERFORMANCE OF THE AMBIENTE HOUSING SYSTEM FOR USE IN NORTHERN COMMUNITIES OF CANADA

BY

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A Thesis

Submitted to the Faculty of Graduate Studies

In Partial Fulfillment of the Requirements for the Degree of

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FACULTY OF GRADUATE STUDIES

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STRUCTURAL PERFORMANCE OF THE AMBIENTE HOUSING STSTEM FOR USE IN NORTHERN COMMUNITIES OF CANADA

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ABSTRACT

Extreme weather and social circumstances impose tough challenges on the condition of homes and the health of their inhabitants in many Northern communities of Canada. Major problems arise from overcrowding and structural deterioration which result in poor indoor air quality and poor energy efficiency. Among the many housing systems specifically designed for the Canadian north is a new housing technology which has shown promising features and incorporates only advanced composite materials, the Ambiente Housing® system. An extensive research and development program was initiated at the University of Manitoba in order to evaluate this housing system for use in Canada and especially in Northern communities where the weather conditions are quite extreme. The evaluation process included both structural testing and material performance testing. In addition, factors affecting thermal efficiency were also addressed. A series of structural tests, as well as a real-time thermal monitoring of a scaled house unit were carried out. The results to date show that this technology can provide durable, energy efficient, and healthier homes for Northern communities. Other benefits of the Ambiente Housing include the speed and ease of construction. Furthermore, 80% of the house components are fabricated using recycled glass. In this thesis the results from structural and

material tests conducted on various components of the Ambiente Housing system are presented. The results, whenever possible, are compared to current standards and specifications. Since one of the objectives of the evaluation program is to ensure compliance of the Ambiente Housing System with Part 4 of the National Building Code of Canada, the design of a full-scale house was carried and is presented in this thesis. This house was designed taking into consideration some of the worst weather conditions encountered in Manitoba.

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TABLE OF CONTENTS

LIST OF FIGURES

Page

LIST OF TABLES

1. INTRODUCTION

1.1. HOUSING CRISIS IN NORTHERN CANADA

One of the fundamental living requirements for all human beings is shelter. Healthy, affordable and adequate housing is a primary human right that should be available to every community including the northern communities of Canada. In reality, however, the north is facing a great housing problem. Extreme weather and social conditions impose many challenges on the conditions of homes and the health of their inhabitants in the north. Major problems arise from overcrowding and structural deterioration which result into poor indoor air quality, and poor thermal performance. In addition to these problems, rapid deterioration of these homes makes financing options impossible. The lack of financing, combined with the fact that the average income in these communities is less than half the Canadian average, has resulted in poor construction, minimal maintenance and, overall, a severe housing shortage (INAC, 2002). According to the Assembly of First nations (AFN), there is a broad consensus that the condition of much of the housing stock in First Nation communities threatens the health and safety of community members, directly contributing to the social justice issues including child poverty, suicide, educational attainment, alcoholism, family break down and others (AFN, 2003). The fact that the northern

communities of Canada are experiencing a housing crisis is widely acknowledged not only by various government and other agencies but, most importantly, from the residents in those areas that are facing tough challenges in their day-to-day survival. The demand for engineered solutions in bringing affordable and adequate homes to these communities is of high priority.

1.1.1. Overcrowding

Among all communities in Canada, First Nations communities are the fastest growing, with a birth rate double the national average. Therefore the demand for housing is rapidly increasing. However the number of new homes available every year cannot adequately meet this demand. This gives no other option for some families but to have to share homes that were designed to accommodate fewer people. According to Indian and Northern Affairs of Canada, about 11% of the homes in First nations Communities are overcrowded, compared to 1% elsewhere in Canada (INAC, 2002). According to the Canadian Housing and Mortgage Corporation (CHMC), 36% of on-reserve homes do not satisfy minimum requirements of the National Occupancy Standards, compared to a 2.3% for Canada as a whole (CMHC, 1987). Overcrowding creates many social problems and has severe effects on the indoor air quality but also on the structural integrity and lifespan of the house.

1.1.2. Indoor Air Quality

Overcrowding, along with the fact that people spend most of their time indoors due to the extreme weather conditions of the north, have resulted in a poor indoor air quality. In the absence of adequate ventilation, constantly high levels of indoor relative humidity are produced which facilitate the growth of mould. In fact, mould growth in northern reserve homes is of great concern within the government and other agencies, due to the hazards it poses to human health (INAC, 2002).

1.1.3. Construction and Maintenance

The lifespan of First Nations homes in the north is severely affected by overcrowding and high levels of humidity which deteriorate wooden walls, floors and foundations due to rot and mould. In the absence of adequate funding, combined with low family incomes, the quality of northern housing is quite poor. In many communities, prefabricated trailers are used as homes. These are cheap to ship and construct. These trailers, however, are designed primarily for southern Canadian climates, and are not adequately designed to meet the insulation requirements for northern housing (McLeod 2004). Furthermore, it is very common in the north to find homes with broken windows and malfunctioning doors due to differential settlements of the foundations. This settlement can occur when the foundations are poorly designed or constructed. Inadequate insulation between the house and the underlying soil allows for melting of the permafrost,

which in turn causes excessive settlements of the ground and consequently severe damage to the house. The life expectancy of homes in the north has been reported to be five to six years (Williamson, 1996), yet many families have no other option but to continue to occupy substandard and unhealthy homes.

1.2. AN ALTERNATIVE HOUSING SYSTEM

Among the many efforts to build and deliver housing specifically designed for the Canadian north, a new housing technology developed by Ambiente Homes® has shown promising features. Ambiente Homes is a company that was established in Puerto Rico and belongs to a group of companies under Ambersham®, based in the United Kingdom. Under this group extensive research has been done on housing and its various components, in order to develop products that can improve the thermal and structural efficiency of homes. Ambiente Homes was created in lieu of a new award winning technology that incorporates composite materials to make homes that use no steel or wood in their construction.

Originally these homes were developed for the Caribbean market, as a solution to housing shortage. Benefits of this system include its ease and speed of construction, low costs and its resistance to hurricanes. The features of this technology and the materials incorporated were sought at the University of Manitoba as a promising solution to enlighten the current housing crisis in the northern communities of Canada. This technology provides evidence of much more durable and healthier homes than current construction practices in the

north. Furthermore, tests have shown that this system performs well in terms of energy efficiency (McLeod, 2004) and is much faster to install. Other beneficial features include that these homes are modular and easy to ship, they do not require skilled workmanship for installation and they are environmentally friendly. A more detailed description of this technology is given in chapter two.

1.3. OBJECTIVES

An extensive research program was established in 2003 under the University of Manitoba, in order to explore the Ambiente house and examine the potential of using this technology for designing a housing system that can provide an adequate structural and mechanical performance in order to meet the requirements of the climatic conditions and lifestyles of the north.

The purpose of this paper is to present the research performed with respect to the structural aspects of the Ambiente system, and to develop a design that can satisfy requirements as set out in the National Building Code of Canada (NBCC).

Key points addressed in this project are outlined as follows:

a) To review in detail the Ambiente technology and the structural testing that has been done in behalf of Ambiente.

b) To test a composite structural floor joist developed under this research program, as part of an elevated floor design required for homes built on permafrost.

c) To design and perform additional tests in order to provide supplementary information about the Ambiente system.

d) To analyze experimental results in order to check the suitability of the materials and to establish relationships that can be used to support the design of a house.

e) To outline possible required modifications as suggested by test results and observations.

f) To identify structural loads and to design a four-bedroom house with a predefined floor plan that will be built in the Smart Park research facility at the University of Manitoba.

1.4. SCOPE

This report is organized in seven chapters. Chapter one provides an introduction to the housing conditions in the north that have defined the need to seek for alternative housing solutions, as well as the objectives of this report.

Chapter two provides an overview of the Ambiente housing system including construction details, as well as other features that provide advantages with respect to traditional construction methods and materials.

The third chapter describes in detail the experimental tests that were conducted under the University of Manitoba, in an effort to provide additional evidence required to support the structural design of a complete structure.

The results of the experiments are presented in chapter four, organised in graphs and tables that provide meaningful interpretations of results.

Chapter five gives a brief description of the Smart House, a full scale experimental unit that is planned to be constructed at the Smart Park Research facility within the University of Manitoba. This chapter also provides in detail the calculation of specified loads and limit state design checks against the structural requirements imposed by the NBCC, using the worst-case conditions throughout Manitoba.

Finally, Chapter six summarizes the project and also includes discussions on the experimental results, observations from the design procedure, as well as recommendations for construction and future research.

2. THE AMBIENTE HOUSING SYSTEM

2.1. STRUCTURAL COMPONENETS

The Ambiente housing system is an award-winning technology that incorporates advanced composite materials, to produce a modular system that is easy to pack, ship, and assemble. This system essentially consists of four elements; the panels, which make up the walls and the roof; the pultruded fibreglass sections, which serve as connection links; a network of post-tensioning cables that tie together the panels and anchor the roof to the foundation; and, finally, the floor, consisting of composite joists. Originally, the Ambiente housing system was designed for the Caribbean market and is typically constructed on a slab-on-grade. However, due to the effect of permafrost in the north, homes need to be elevated in order to prevent melting of the permafrost which triggers excessive ground settlement. For this reason, Ambiente funded the development of a new composite structural floor which was tested at the University of Manitoba. Each group of components listed above is described in detail in the following sections. An overview of the assembly of the Ambiente housing system is given in section 2.2.

2.1.1. Panels

The Ambiente panel consists of two components, the core and the skin, as shown in Figure 2.1. The core provides bearing resistance, insulation, and includes an area where cables and wires run through. The core is contained by a fiberglass skin on either side that protects the core from impact forces, fire and also serves as a pre-finished surface. Currently, these panels are manufactured at a thickness of 152mm(6inch) and are 1.219m(4ft) wide with a variable height. By default, the panels come with a white finished surface; however, the colour can be changed by request. The roof panels have the same finish as the walls in the interior, yet there are two types of exterior finishes available. For southern climates, Ambiente provides a Spanish tile look, yet a shingled roof finish has been developed for the north. A 25mm(1inch) diameter duct runs at three locations through the panels to allow the passage of the prestressing cables and any other electrical wires. These are located at three heights, two below the windows and one above. Figure 2.2 shows sections from a wall panel and a roof panel with the Spanish tile look.

The structural performance of the Ambiente wall panels has been the focus of experimental work both at the University of Manitoba and the University of Puerto Rico. The experimental work at the University of Manitoba involved the testing of the Ambiente panel under racking loading, tests to obtain the tensile capacity of the post tensioning cables, testing the shear capacity along the joint interface between panels, as well as testing panels under four-point bending loading. Also, freeze-thaw durability tests were conducted on specimens

extracted from Ambiente panels. Ultraviolet radiation and condensation cycling tests were conducted on the skin of the panels. The experimental work at the University of Puerto Rico involved the testing of both roof and wall panels under flexure, compression and lateral loading. (Lopez, 2000)

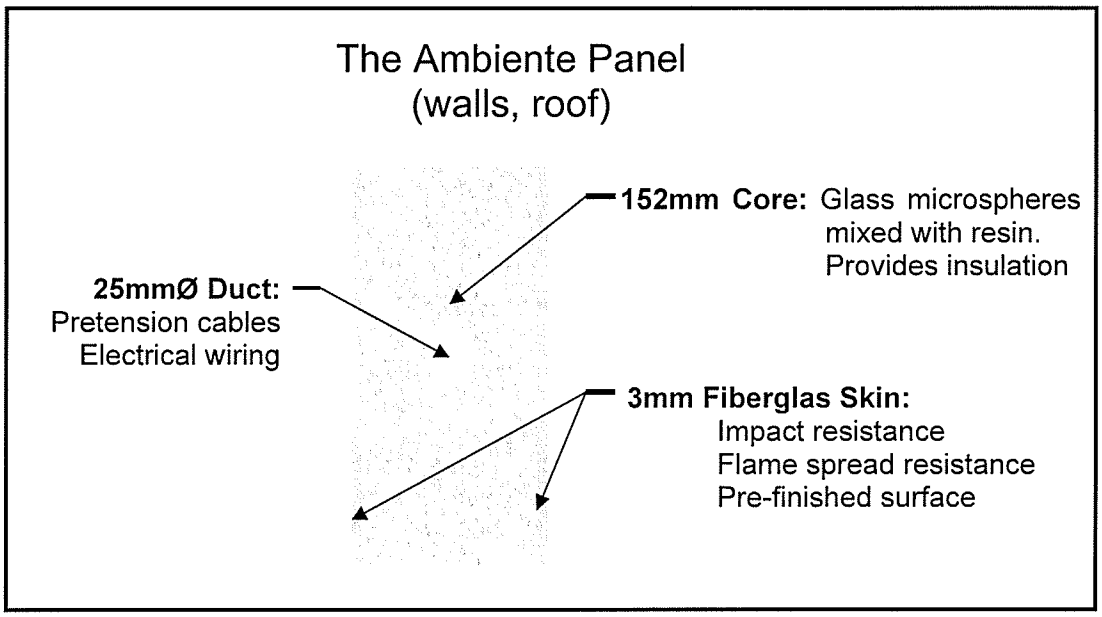


Figure 2.1: Schematic of the Ambiente Wall Panel

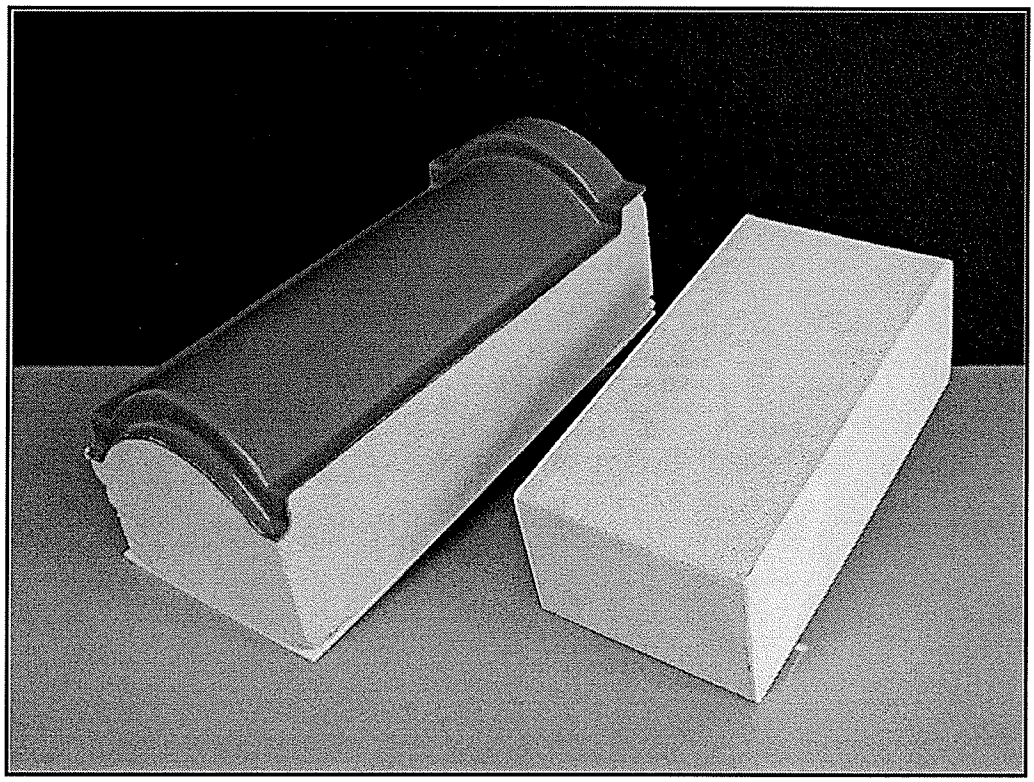


Figure 2.2: Sections of a roof and a wall panel.

2.1.1.1. Core

The two materials that are used to make the core of the wall panel are glass and resin. The glass is formed through a pressurized heating process from recycled stock into micro-spheres which contain entrapped air, most commonly referred to as cenospheres. Cenospheres have been known for their ability to provide diaphragms of superior insulation, due to the entrapped air that they contain. They have been used in applications where extreme insulation is required, such as equipment for space operations, for insulating extremely deep foundations of oil rigs, or to insulate the sensor on the warhead of heat-guided missiles (Lucida, 2004).

When forming the core, cenospheres are bound together with resin. The type of resin used can vary depending on requirements for density, viscosity and chemical properties. Ambiente currently uses the unsaturated polyester resin AROPOL® manufactured by Ashland Chemicals. In order to prevent off-gassing of the final product, the manufactured panels are heated in an oven to ensure full curing of the resin and then they go through the process of ionization in order to remove any entrapped gasses that may produce odour. Figure 2.3 shows a microscope picture of the core structure from an Ambiente panel slice. In order to improve fire rating, Alumina Trihydrate (ATH) is added to the resin as a fire retardant.

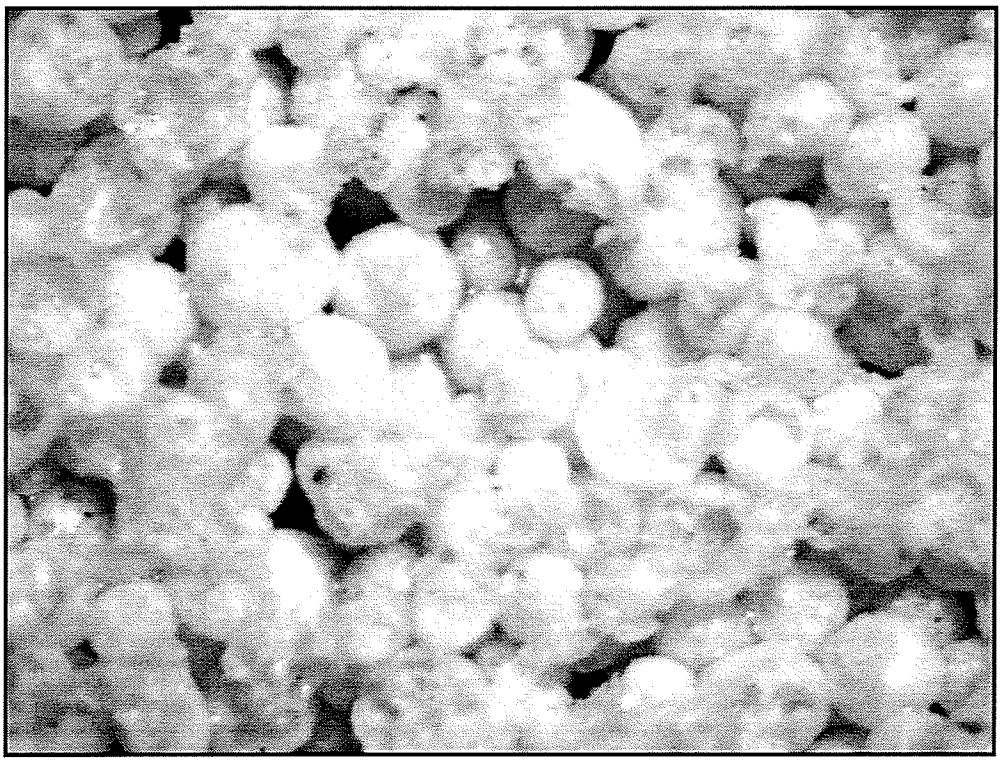


Figure 2.3: Microstructure of the core material (Magnification: x 30)

2.1.1.2. Skin

The core of the panel is protected by a 3mm (1/8 inch) fiberglass skin that provides protection from impact and fire. The skin is bonded to the core with an additional layer of ATH in between in order to create an additional fire barrier. Although the skin provides a finished surface, it can be painted or covered with wallpaper according to the desires of the owner. The exterior can be painted to give a stucco effect.

2.1.2. Pultruded Sections

The designation pultruded refers to a production technique widely used in the industry of composite materials. Many companies produce pultruded sections for applications in structural engineering, such as lattice towers, floors, structural beams, or even bridges. This production technique involves the continuous drawing of fibers through a heated dye where the fibers, pre-impregnated in resin, are instantaneously cured. Pultruded fiberglass sections are known for their low resin content and their good structural performance. Their purpose in the Ambiente system is to serve as connection links between panels and to distribute the loading over a large area, thus contributing to the overall performance of the house against external loads, since they are much stronger, yet more flexible than conventional materials. Various shapes of sections are used for the Ambiente house, as shown in Figure 2.4. These are produced by Bedford Reinforced Plastics, Inc., of Bedford, Pennsylvania. They also contain fire retardants such as ATH, as well as Decabrome, in order to bring flammability

to a non-burning rating, as per ASTM D635 (Bedford, 2003). Coupon properties of the sections used in the Ambiente House are listed in Appendix B.

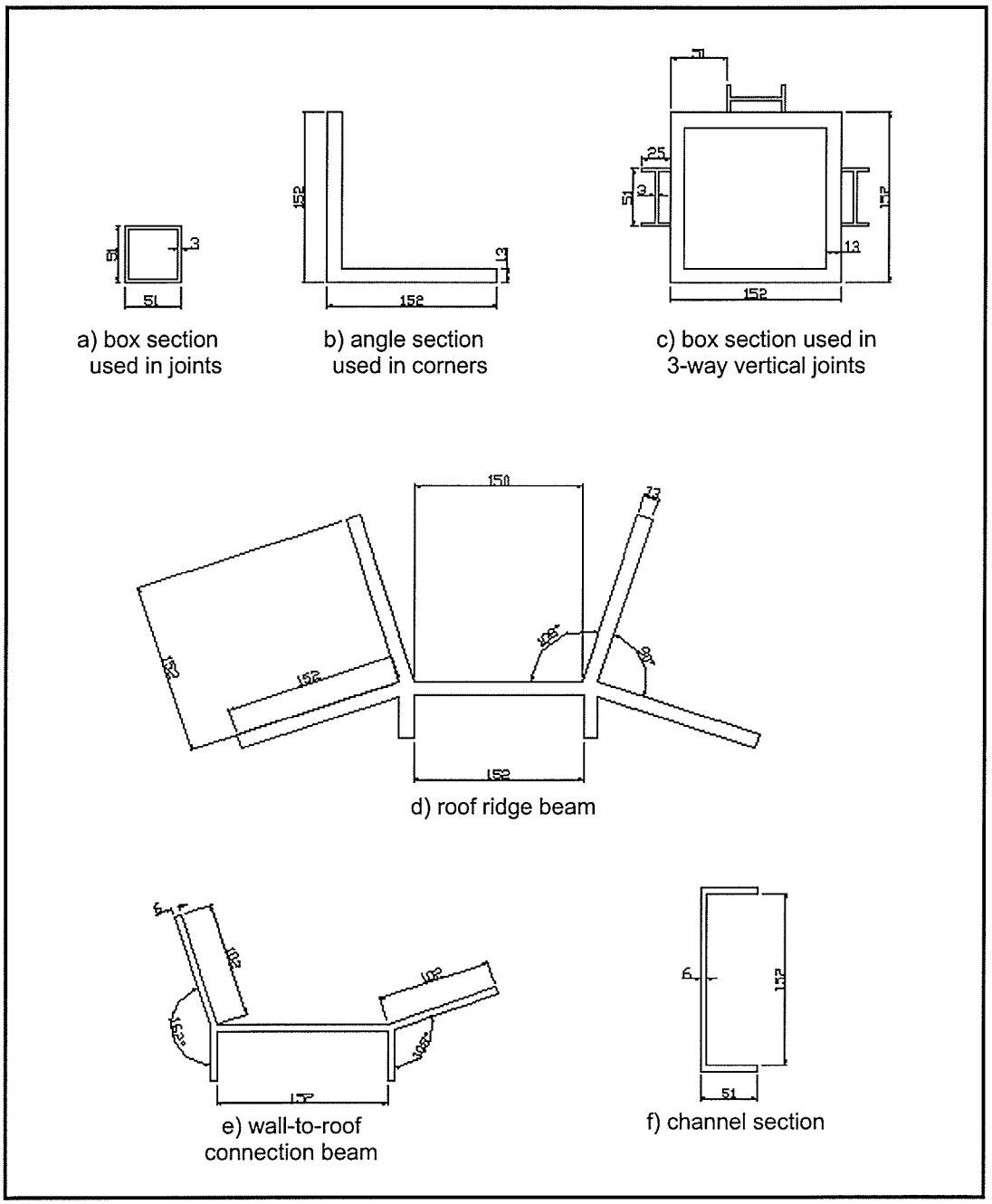


Figure 2.4: Pultruded Fiberglass Sections used in the Ambiente House

2.1.3. Post- tensioning Cables

The Ambiente technology incorporates a network of post-tensioned cables that run both horizontally and vertically through the panels of the wall and the roof, in order to create a rigid structure. The roof is also tied down to the foundation creating exceptional wind resistance for the Ambiente houses which were originally designed to resist hurricanes. The cables used in this system are manufactured by Marlow Ropes®, UK, are 12.7mm in diameter and consist of 3 helical strands of polyester, coated with a cover from a number of yarns which come together to form a plait. The cover protects the polyester strands and gives a round shape to the rope, so it can be used in pulleys and clutches, but also makes it stiffer than regular rope, so it can be easily fed through a hole by simply pushing from one side. Upon tensioning, the cables are fastened using wedge anchors, similar to the ones used for prestressing steel in concrete beams, as shown in Figure 2.5.

At the earlier stages of this project two alternatives for post tensioning were considered. One was the originally used cables and the other one was threaded fibreglass rods with thermoplastic nuts provided by Strongwell® as part of their Fiberbolt® series. However, early tests revealed that the threaded rods were not suitable due to the low shear capacity of their threads.

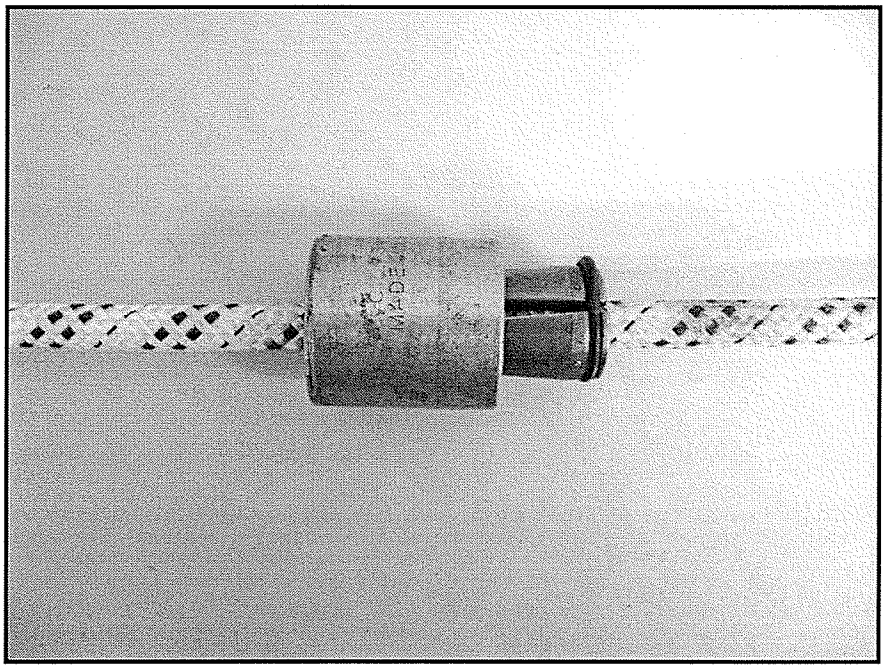


Figure 2.5: Post-tensioning Cable and Anchor

2.1.4. Floor

The first homes built for Puerto Rico were designed to be constructed with a concrete slab-on-grade floor. However, permafrost conditions in the north call for a well-insulated and elevated floor in order to prevent excessive foundation settlements. Ambiente, in partnership with the University of Manitoba, initiated a research program to develop an elevated structural floor. In current elevated wood floor structures in the north, rapid deterioration due to mildew, rot and insects is a common phenomenon. For this reason, it was decided to create a floor using only fiberglass polymers. Another advantage of using this composite material is that it is lighter than wood and steel and therefore easier to ship and construct.

The idea of an open-web joist was found to be the most suitable solution, since open space within the floor is required for the mechanical and plumbing system to pass through. Furthermore, it was decided to use the process of filament winding, in order to increase production speed and to control fiber content. The joists developed under this project were manufactured by Rhino Composites of Gimli, Manitoba, using a procedure that is new and unique to the composite industry. A single joist consists of rectangular filament-wound cells that are combined together in a longitudinal array, before curing is complete. A set of pultruded diagonal members are inserted during manufacturing, as shown in Figure 2.6. Early tests revealed that these diagonal members were necessary in order to prevent premature shear failure, of the joists. The joists were designed to be 356mm(14") deep. A conceptual prototype is shown in Figure 2.7. More details on the assembly are given in section 2.2.

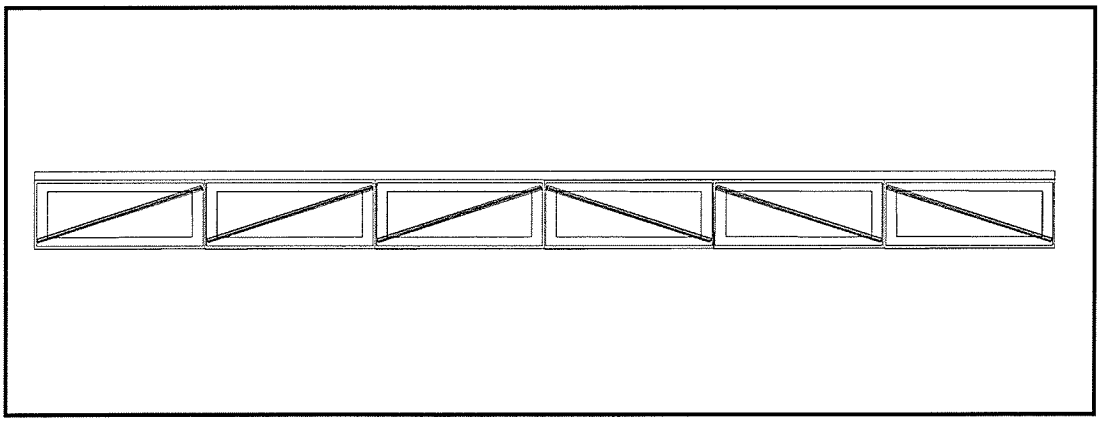


Figure 2.6: Schematic of a Single Joist

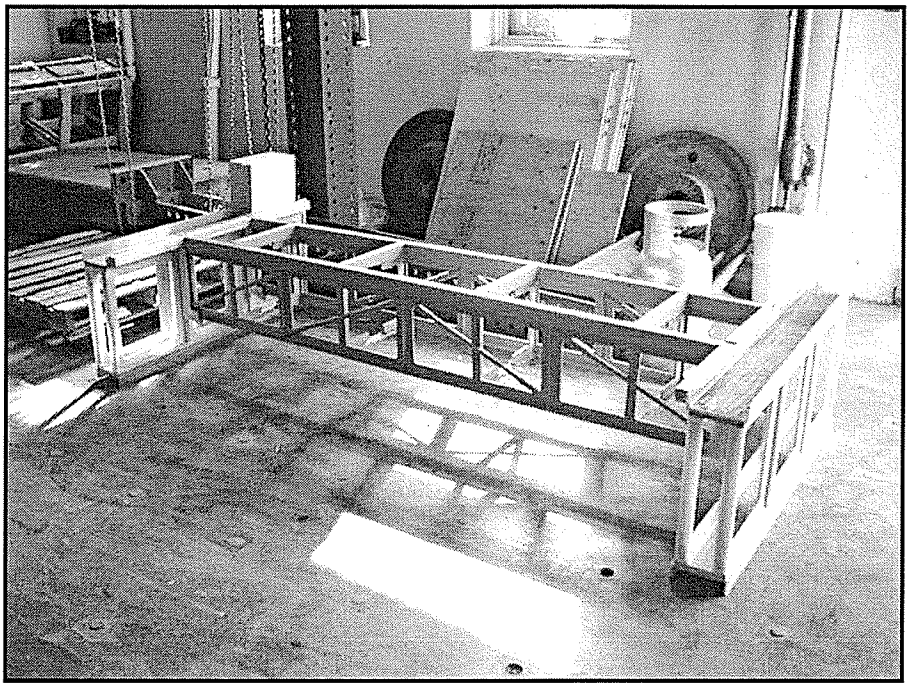


Figure 2.7: Floor Joist Prototype

2.2. STRUCTURAL ASSEMBLY

Perhaps the most unique feature of the Ambiente house is its assembly. Its quick installation is attributed to the simple connections between panels, which do not require skilled workmanship. Once the floor is constructed, a track of pultruded channels is laid on the floor. Then the panels are inserted in these tracks, which provide enough stability for the panels to stand vertically during construction. At specific locations such as corners and joints where two or three panels meet orthogonally, special pultruded sections are inserted vertically. Once all vertical panels are placed, a network of horizontal cables runs through the panels to secure them from tilting. Then another set of pultruded sections is placed horizontally along the top of the wall panels and the interior main wall,

where the roof panels will be placed. These pultruded sections are also fastened through vertical cables that run in between wall joints down to the floor. The roof panels are then placed on either side of the main interior wall, bearing against the horizontal pultruded sections. Horizontal cables fasten the roof panels at orthogonal directions as well. Caulking is provided along all contacts between panels and pultruded sections to cover the seams and to prevent leaks. The following sections explain in detail various assembly elements of the Ambiente house.

2.2.1. Foundation and Floor

Figure 2.8 shows a schematic of the foundation of the Ambiente house developed for the North, which was designed by Wardrop Engineering Inc., of Winnipeg, Manitoba. According to this design, all panels of the exterior walls are extended to bear directly upon a concrete strip footing. The composite floor joists are supported by a second line of Ambiente pony walls, allowing for a 610mm(2ft) crawl space beneath the joists, in order to have access to the mechanical system and the plumbing of the house. Sufficient insulation is necessary in order to prevent permafrost melting below the house.

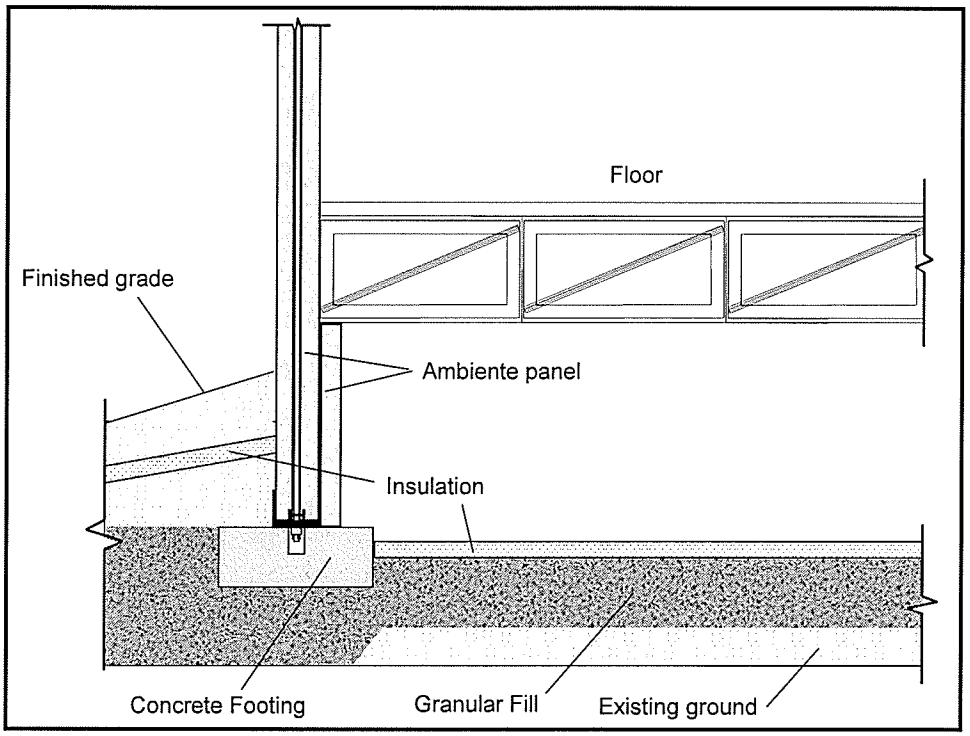


Figure 2.8: Schematic of Foundation and Sub-floor Components

2.2.2. Panel Joints

The direct jointing of panels is the same for the walls and the roof. As shown in Figure 2.9, the panels have a 25mm(1□) deep vertical groove along the edge where a 51mm by 51mm (2□by 2□) pultruded hollow square section is inserted between the two panels as a shear key. The vertical cables that tie down the roof run through the void space of the pultruded square sections. The edge of the panels is also covered with a rubber gasket in order to prevent air and water leaks. Once the tension is applied through the horizontal cables, the compressed gaskets provide a tight joint.

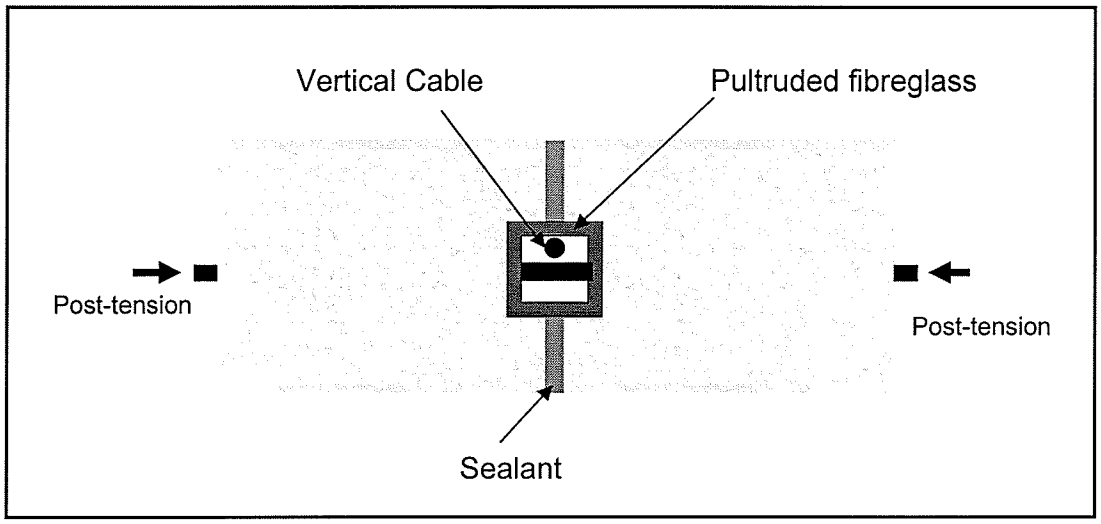


Figure 2.9: Schematic of Panel-to-Panel Joint

2.2.3. Connections

Figure 2.10 shows a plan view of the corner connection where two wall panels meet orthogonally. A pultruded L-shaped section is used in order to be able to tighten the cables from both sides, but also to provide a strong bearing surface. Upon construction, the void space is filled with insulation and finally is covered with a finished fiberglass covering. In the case where three panels meet, a three-way connection exists, which is very similar to the corner joint, as shown in Figure 2.11.

An elevation view of the connection between the roof and the interior and exterior walls are shown in Figure 2.12 and Figure 2.13. The pultruded sections used in these joints have a more complex shape, yet the essential elements are similar to the other types of joints.

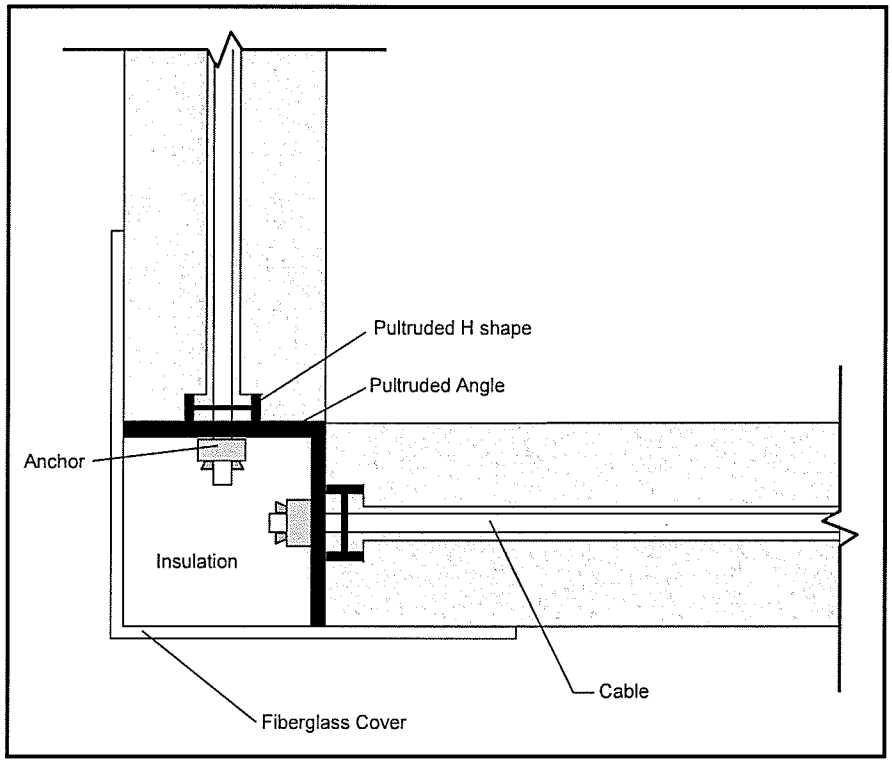


Figure 2.10: Plan View of Corner Connection

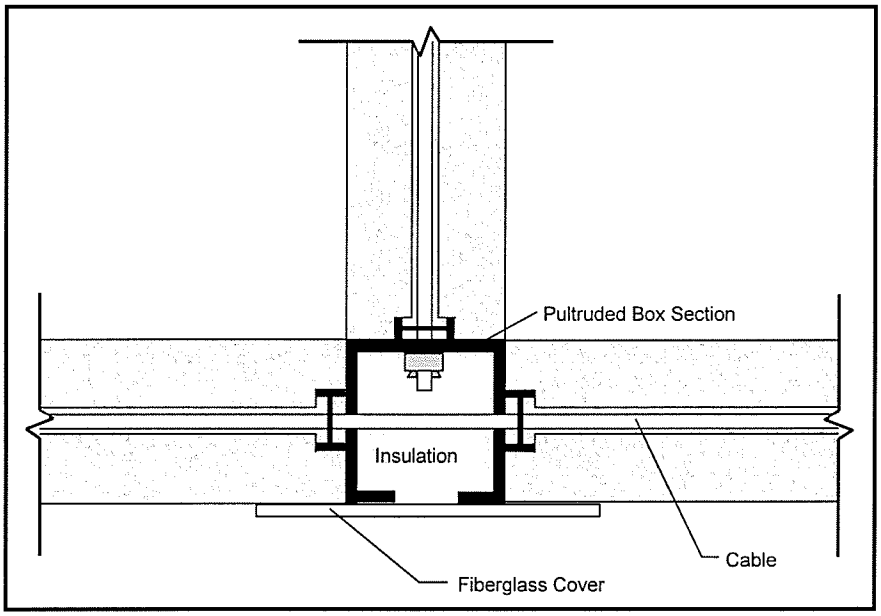


Figure 2.11: Plan View of Three-way Connection

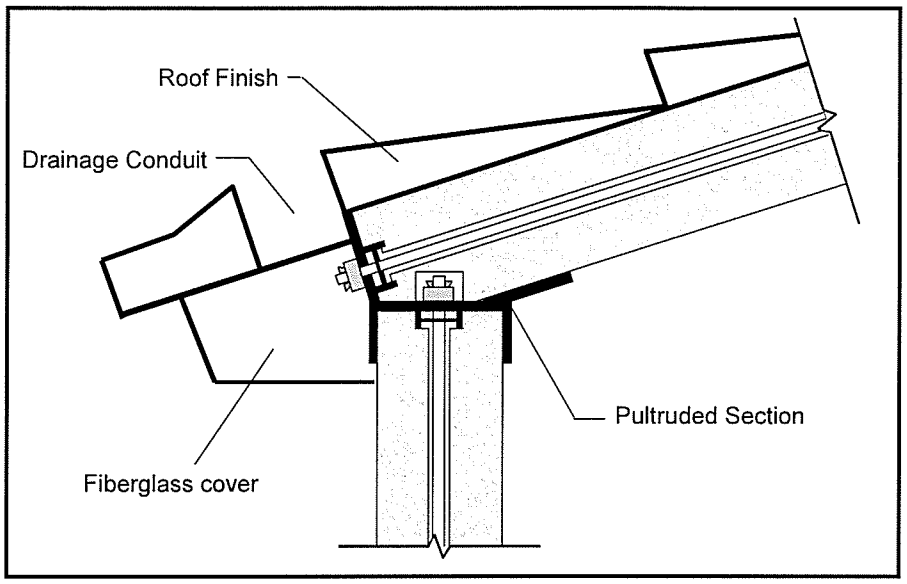


Figure 2.12: Connection between Exterior Wall and Roof

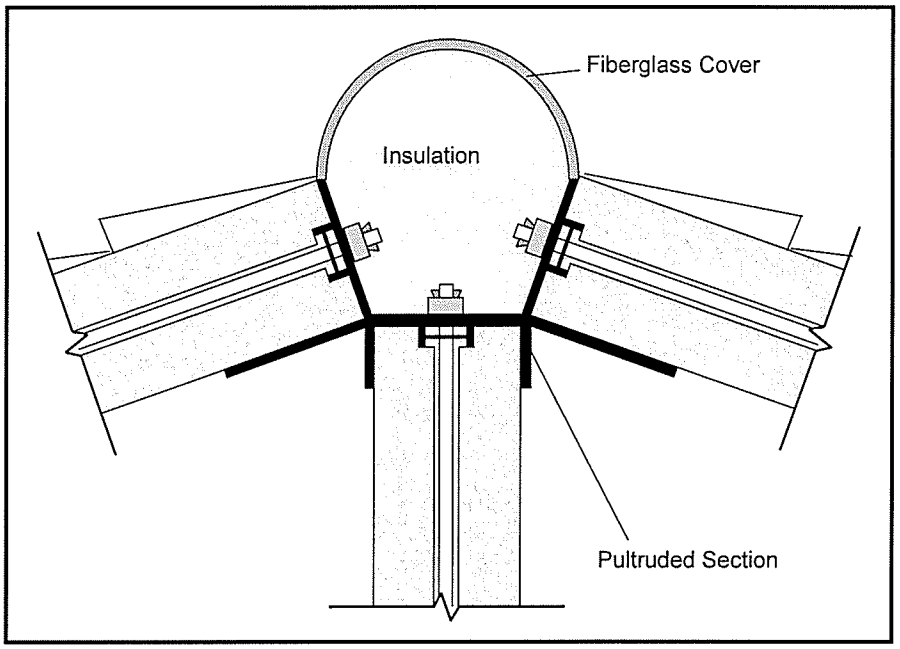


Figure 2.13: Connection between Roof and Main Interior Wall

2.2.4. Post-Tensioning Cables

The feature that adds superior wind resistance and overall rigidity to the Ambiente house is the network of post-tensioning cables, which run in two directions across every panel. In the horizontal direction, they run at three levels equally spaced for the roof panels, but at uneven intervals for the wall panels in order to allow for window openings. In the vertical direction, cables run at every panel joint through the void space of the 2□x2□pultruded section. Figure 2.14 shows the location of cables on a typical Ambiente house.

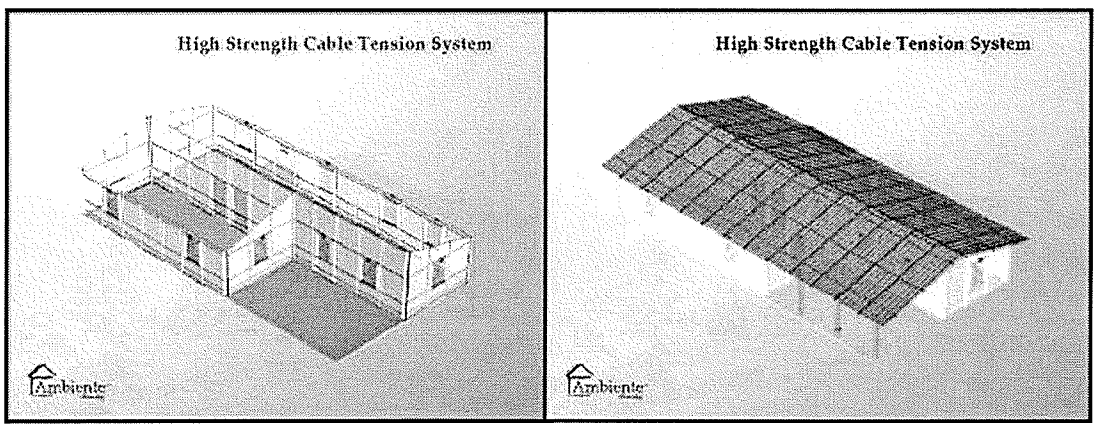


Figure 2.14: Post-tensioning Cable Orientation in Walls and Roof (Ambiente 2002)

2.3. PERFORMANCE

The promising features of the Ambiente system have been recognized by the American Composites Manufacturers Association, by awarding this technology with the ACE award for most innovative design and best use of composite materials in the year of 2000. Along with the many features presented by Ambiente, an extensive experimental and theoretical study on the structural

and thermal performance of this system was conducted under the University of Manitoba as part of this research project.

2.3.1. General

Ambiente has conducted various tests on components to evaluate structural performance, thermal insulation and fire resistance. Testing at Omega Point laboratories at Elmendorf, Texas, has shown that the panel skin has a class C flame spread and smoke rating, has a zero flame spread, it will not sustain combustion and has a low smoke emission when burned (Ambiente Housing, 2003).

An attractive feature of this system is the ease and speed of assembly. Ambiente claims that a typical twelve hundred square foot house can be built within a week, without a need for skilled labour or specialty equipment. Shipment is also easy since the components of a whole house can be packed and shipped into one container. Every single component can be predefined and cut to the exact lengths; therefore the amount of materials used can be precisely estimated.

According to Ambiente, 80% of the raw materials used for their house come from recycled waste glass. This can potentially enlighten the problem that many municipalities are facing today, having to dispose an excess amount of waste glass, which is virtually non-degradable. This fact combined with the fact that the Ambiente house eliminates the use of wood and the need to cut down trees also shows an environmental contribution.

The durability of the panels is of particular interest for housing in northern Canada. Since the materials used are non-organic they will not sustain mould, which is a great problem in the north, as already discussed. Furthermore the high levels of humidity that exist in northern homes will not degrade the materials used in the Ambiente system, since they will not corrode or rot. The rigidity of the house attributed to the post-tensioning cables is also a factor expected to reduce damages against any possible differential settlement of the foundation.

2.3.2. Thermal Performance Tests

In terms of thermal performance, Ambiente assures that the use of 6-inch panels will exceed an equivalent R-30 rating (Ambiente, 2002) for the house. Using a material like this combined with high performance windows and doors can provide a system of superior thermal performance that can save a very significant amount of energy for heating.

In order to monitor the thermal performance of the Ambiente technology, a small test unit (9ft x 9ft) was built at the University of Manitoba Campus (Figure 2.15). This test unit was equipped with several instruments that monitored real-time values for temperature, pressure and relative humidity at various locations inside and outside the unit, as well as inside a panel and a joint (Figure 2.17). Values were obtained over a three-week period, during the cold winter period between February and March of 2003. The amount of energy used to heat the unit as well as the instrument readouts were digitally recorded and saved for further analysis and observations. Other tests on this unit included infrared

scanning (Figure 2.16) as well as blower door testing for air tightness. Test results led to the conclusion that the thermal resistance of the wall and roof sections satisfied perspective requirements of the R-2000 specification for thermal efficiency. The air tightness also showed compliance with this standard. Although humidity levels were kept very high, no mould growth was observed on any Ambiente component, besides the wooden windowsill below the window. The potential for condensation, which encourages mould growth, did not appear within the walls and the joints, with the exemption of the roof joints. A more detailed analysis of the results is given by McLeod (2004).

Overall, evidence show that the Ambiente system can perform better, in terms of energy conservation, than housing systems currently used in the north.

2.3.3. Canadian Approval

In order to bring the Ambiente housing system into the Canadian market, Ambiente Canada® was established. One of the first priorities of this company was to obtain approval from various agencies. Wardrop Engineering, Inc., from Winnipeg, has been involved in the design of the Ambiente housing system to ensure compliance with the National Building Code of Canada (NBCC), as well as requirements that reflect local needs. Testing done under this research project, as described in Chapter three, has provided supplementary information for this process.

2.3.4. Test House Unit Construction

The test unit (Figure 2.15) was constructed at the University of Manitoba (in September 2002). No specialty tools, screws nails or other fasteners were required for this construction. The dimensions of this unit are 2743mm(9ft) by 2896mm(9.5ft) with a wall height of 2438mm(8ft) at the ends and 2743(9ft) at the gable. The unit includes one window and one door. An internal partition with a door wall was located along the gable. A floor plan of the test unit is shown in Figure 2.19. All joints were sealed using typical caulking for exterior use. The floor for this unit was constructed from a double-walled wooden slab with rigid insulation and vapour barriers encased.

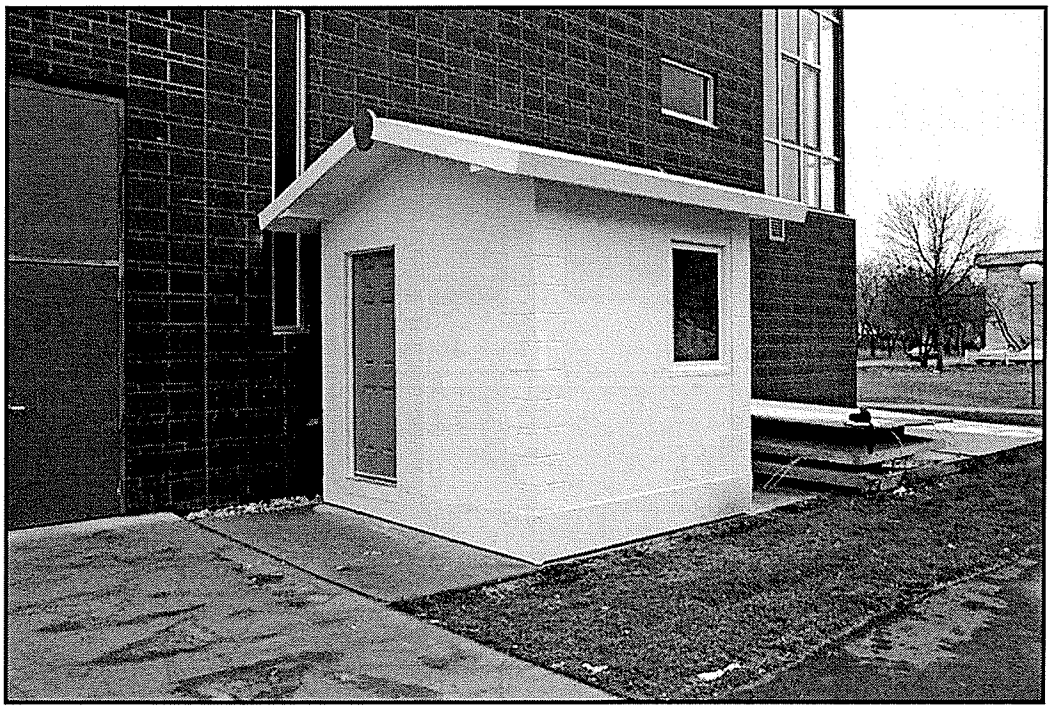


Figure 2.15: Experimental Unit

Figure 2.16: Infrared Picture of Test Unit

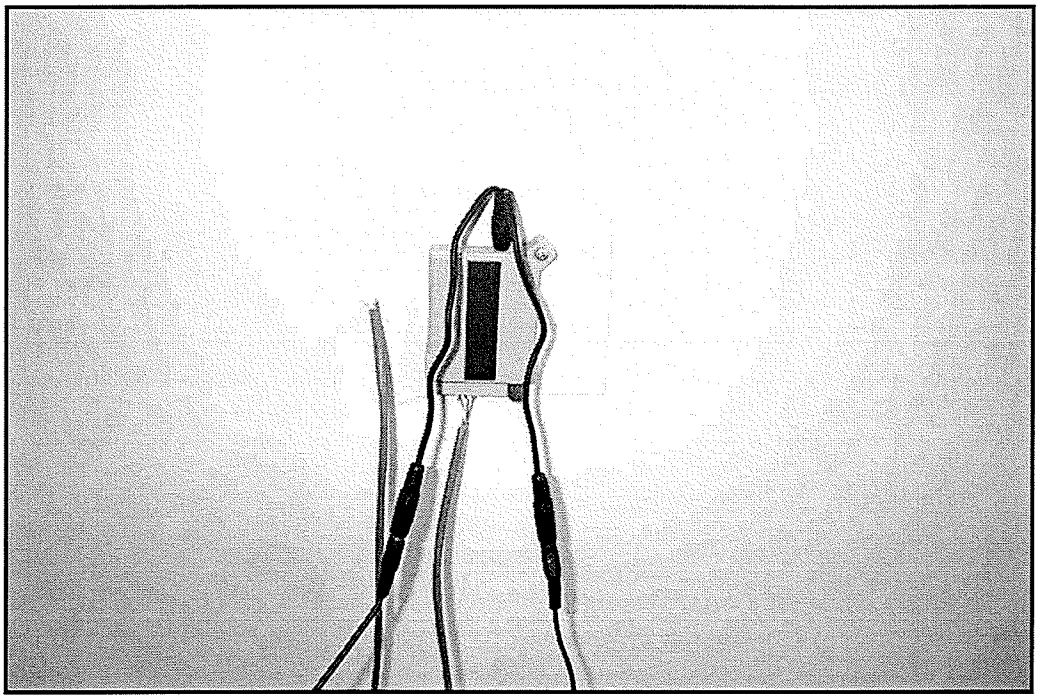


Figure 2.17: Monitoring the Temperature and RH Profile of a Wall

Figure 2.18: Mould Growth on Wooden Windowsill

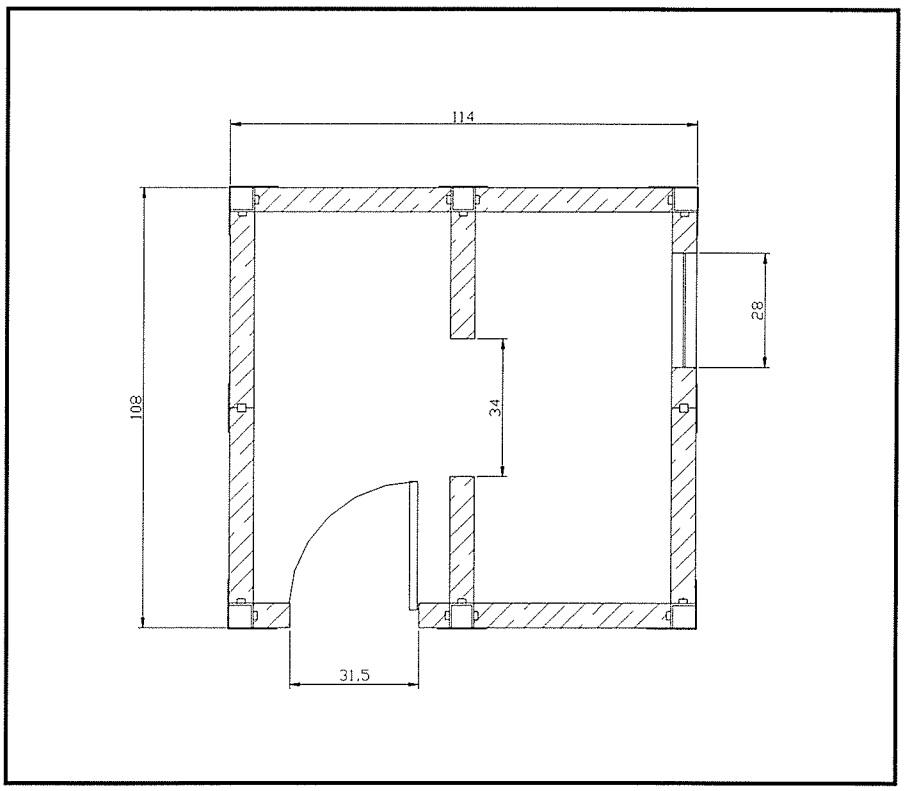


Figure 2.19: Plan View of Test Unit (Dimensions are in Inches)

3. EXPERIMENTAL PROGRAM

3.1. GENERAL

This chapter describes all the tests performed under this research program, in order to collect supplementary information on the structural behaviour of the Ambiente system. Static load tests were carried out in order to evaluate the structural performance of the floor joists under bending; the shear capacity of wall panels under in-plane loading; the tensile capacity of the cables; the interface shear capacity between panels; and, the bending capacity of the wall panels. In order to evaluate weathering effects due to sunlight and rain on the panel surface, as well as extreme temperature fluctuations, two types of durability tests were performed. All tests were conducted at the McQuade research and development structural testing laboratory at the University of Manitoba. More detailed discussion of the tests and the results is given in the following sections.

3.2. FLOOR JOIST BENDING

The purpose of this test was to evaluate the performance of a prototype fiberglass floor joist .The tested floor joist assembly was manufactured by Rhino Composites, Gimli, MB, as part of the latest product developed through a series of trial specimens in an effort to balance performance, cost and production time.

Figure 3.1 shows the setup used in order to test the assembly under a 4-point bending load. The specimen was 3658mm(12ft) long and consisted of two composite joists spaced at 610mm(2ft) apart. A 38mm(1.5□) wood strip was laminated on top of the joists in order to provide a surface to attach a 19mm(3/4□) plywood sheet across the two joists. A similar sheet was also attached at both ends of the joists to provide lateral stability. As Figure 3.2 shows, the load was applied through a steel beam excreting two point loads at 1219mm(4ft) away from each simple support. The load was applied through an MTS hydraulic actuator at a load rate of 2mm/min. Deflection was monitored at midspan using two Linear Voltage Displacement Transducers (LVDT□s). The results from this test are given in Section 4.1.

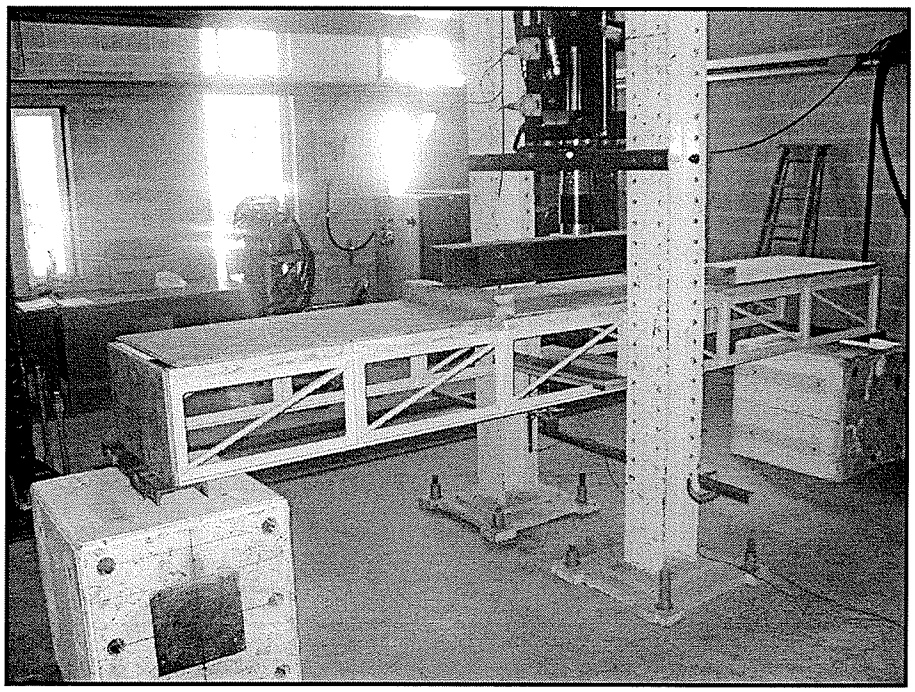


Figure 3.1: Double Joist Test Setup

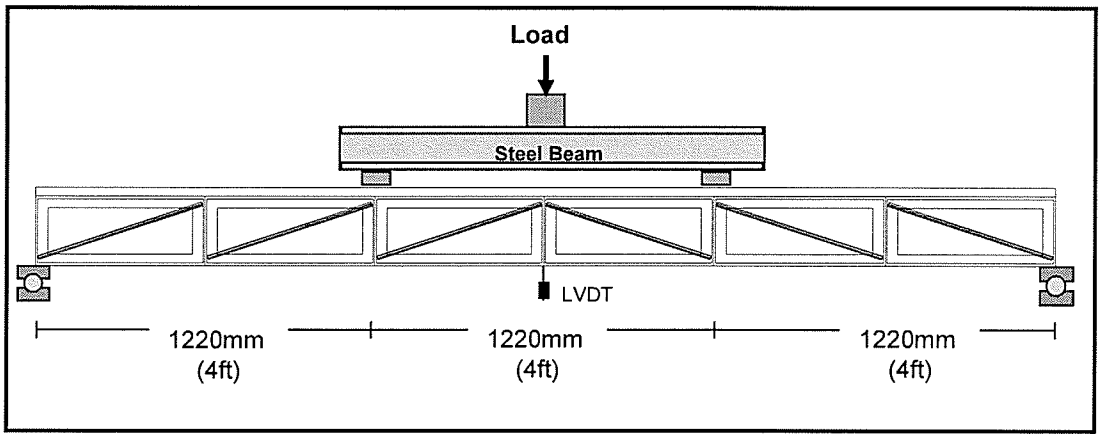


Figure 3.2: Schematic of 4-point Bending Test

3.3. RACKING TEST

In order to evaluate the structural performance of the wall panels under in-plane lateral loads, typically induced by wind loads, a racking test was performed according to ASTM E72-80 (Figure 3.3). The assembly consisted of three panels, one 1219mm by 2438mm (4□by 8□) and two 610mm by 2438mm (2□by 8□) panels on either side. This configuration allowed the inclusion of two vertical joints in the assembly. In the horizontal direction, three rows of cables were used at elevations of 305mm(1ft), 915mm (3ft) and 2134mm (7ft), which is the same spacing used in actual construction. The vertical cables were anchored to a pultruded channel cap at the top and a steel channel section at the bottom. All cables were fastened using a hydraulic fixture, typically used in prestressing steel for concrete structures. The hydraulic fixture was calibrated in order to establish a relationship between the pump pressure and the actual tension in the cable. The post-tension force was maintained at 13.3kN (3000lbs). A steel channel was used at the base in order to allow the assembly to be tied down to the structural

floor. Natural rubber was used as gaskets between vertical panel joints. Pultruded sections were provided at either end of the wall assembly to provide bearing surface for the horizontal cables (Figure 3.4) and to simulate actual construction practice. Horizontal slip at the base and overturning were restrained, as required by the standard. The load was applied at one corner through a manually operated hydraulic jack (Figure 3.5) at an average rate of 2mm/min. A steel frame, consisting of two vertical steel columns and two bracing angles, was used to support the hydraulic jack. Lateral deflection of the assembly was monitored at the top end opposite corner of the load application.

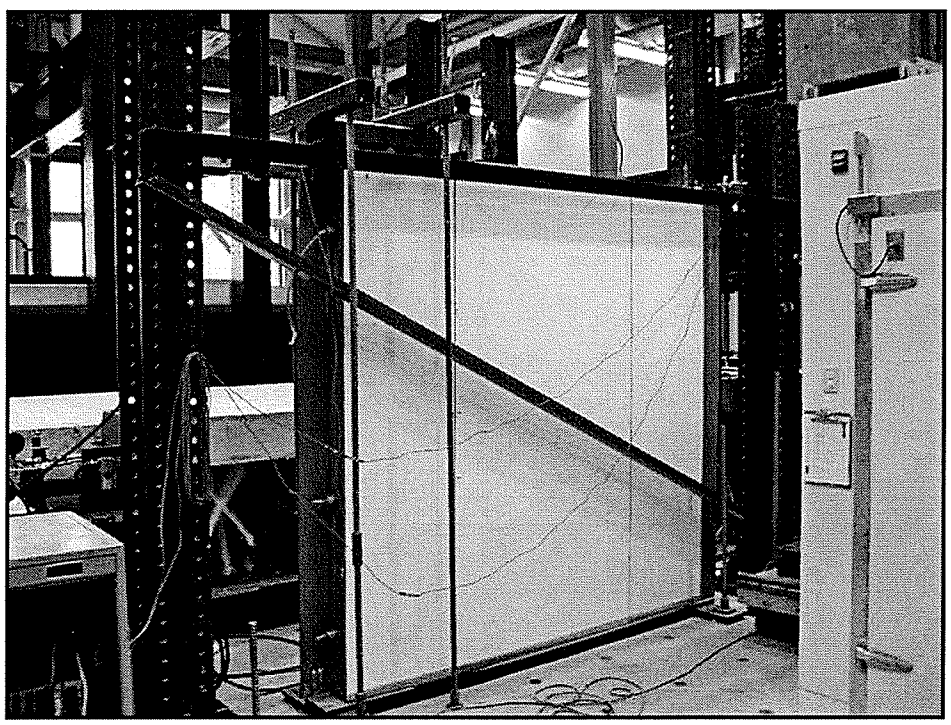


Figure 3.3: Racking Test Setup

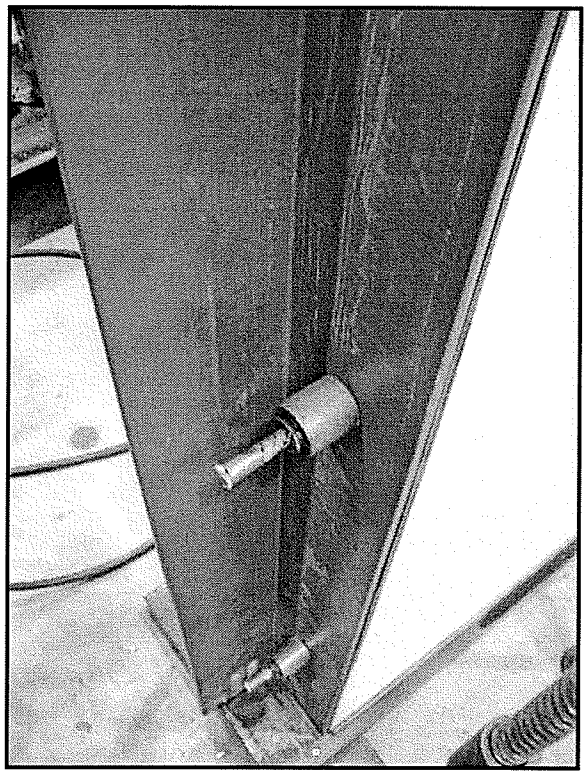


Figure 3.4: Anchored Post-Tension Cables

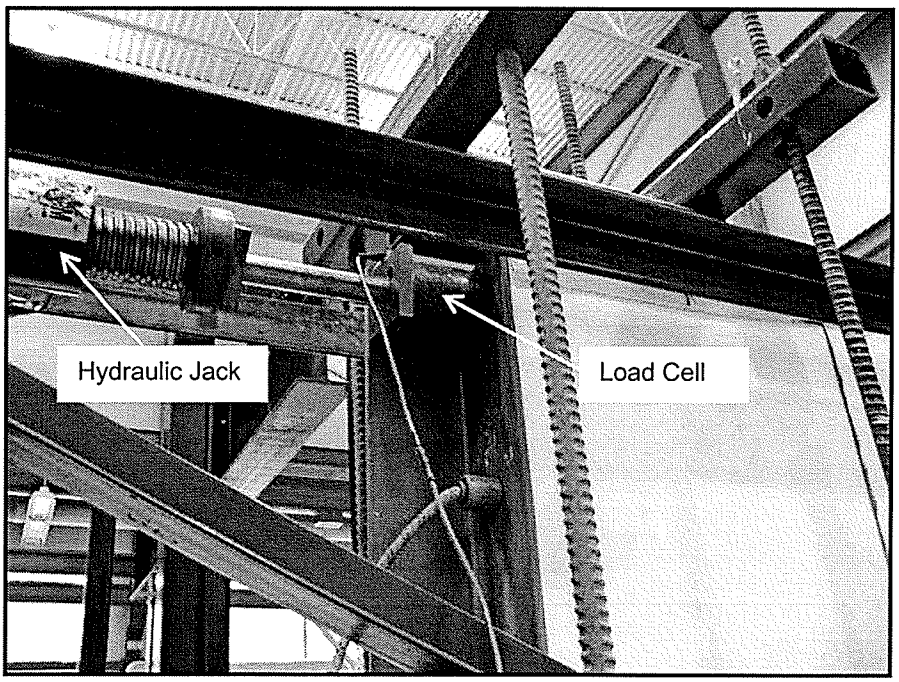


Figure 3.5: Loading Mechanism and Load Cell

3.4. TENSILE CAPACITY OF CABLE

The cable used in the post-tensioning of the panels was tested under two types of tension. One was testing the cable in pure tension and the other was fastening the cables with the steel anchors and applying tension. In order to achieve pure tension, the cable was tested while wrapped around a circular pipe, as shown in Figure 3.6. This allowed for minimal concentration of stresses at the ends. Load was applied at 4mm/min, using a 10,000lbs ATS machine. The second configuration using the steel anchors is shown in Figure 3.7. The same loading machine as in the previous configuration was used with a load rate of 2mm/min. In both tests deflection was monitored based on load versus crosshead movement to the nearest 0.01mm. Furthermore, for the tests using anchors, two types of specimens were used. In the first type, the cable was tested as is, while in the second type the cable ends were coated with epoxy in the region of the anchor to reduce the slip between the gripping anchors and the core of the cable. Three specimens were tested for each type for a total of nine specimens.

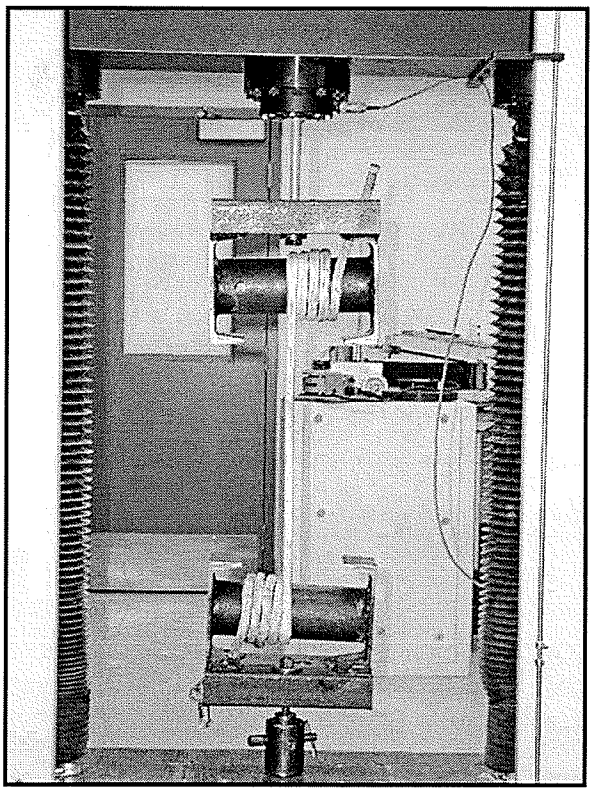


Figure 3.6: Pure Tension Test

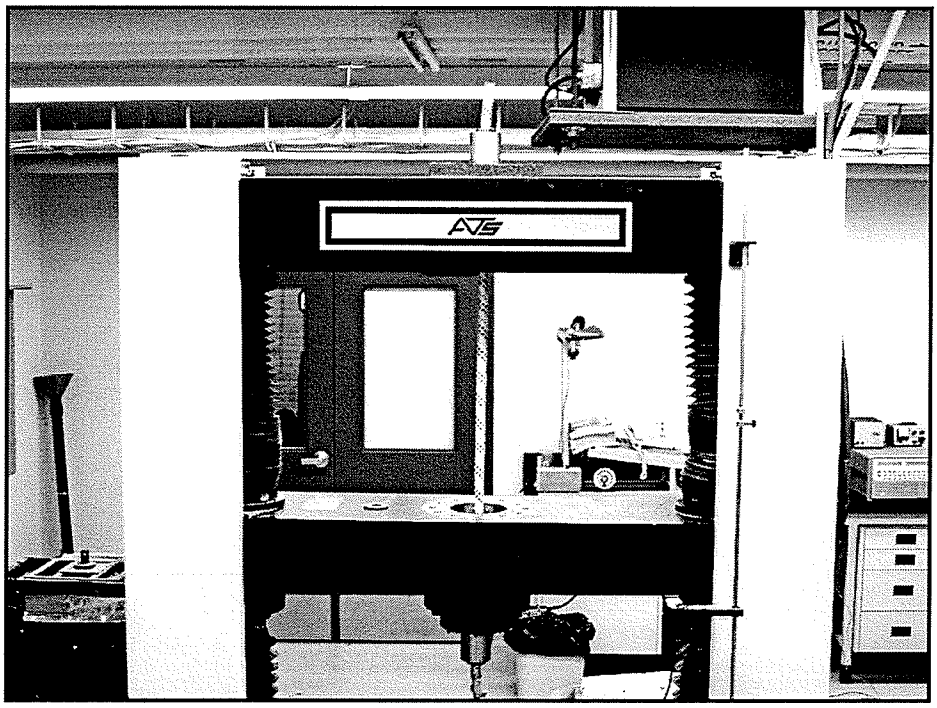


Figure 3.7: Tension Test using Steel Anchors

3.5. INTERFACE SHEAR BETWEEN PANELS

The relationship between interface shear and post-tensioning force between panels, was investigated by conducting two types of tests, one simulating the panel-to-channel interface and another simulating the panel-to-panel interface. The tested panels were 305mm by 305 mm by 152mm thick (1ft x 1ft x 6□). For practical reasons, the panels were subjected in double shear, as shown in Figure 3.8. A 3mm(1/8□) natural rubber gasket was used at the panel-to-panel interface, as it is required in the actual construction, to allow for building tolerances but also thermal efficiency requirements. One FRP rod was used at mid-height of the panel. The rods were 13mm(½□) in diameter and continuously threaded, provided by Strongwell®. The rods were fastened by means of a 22mm(7/8□) thermoplastic nut, also provided by Strongwell. These tests were not required to be repeated using cables since the only parameter of interest was the relationship between the applied tension force and deflection. Both types of specimens are shown in Figures 3.9 and 3.10.

Each type of test was performed for different magnitudes of tension in the rod, by adjusting the torque on the nut. Prior to the shear test, a separate test was performed in order to establish a relationship between the applied torque on the nut and the resulting tension in the rod. This was done by attaching a hollow load cell between the nut and the bearing surface of the specimen in order to measure the load output for given amounts of torque. The torque on the nut was controlled by a torque wrench.

Load was applied at 1mm/min using a 10,000lbs ATS machine, with deflection monitored through the crosshead movement of the machine, to the nearest 0.01mm.

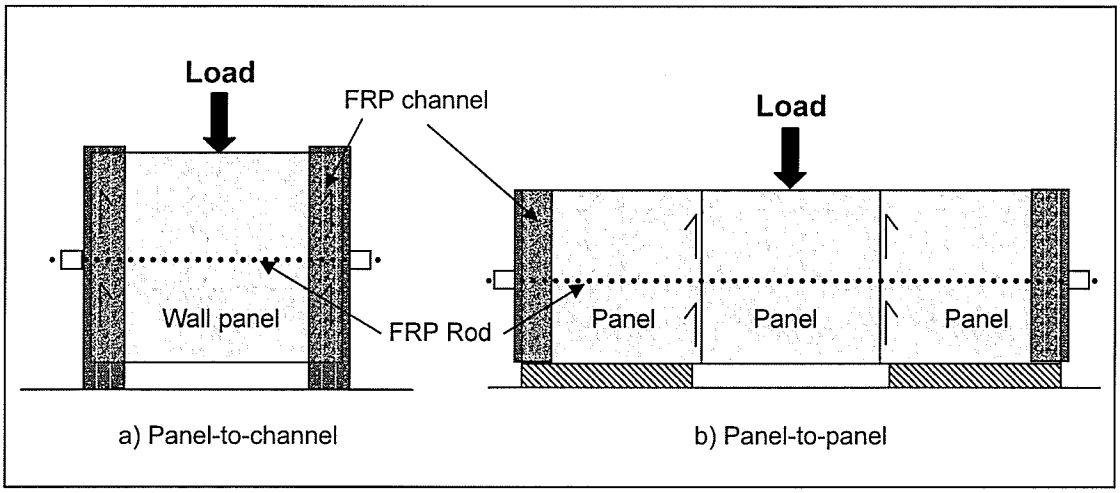


Figure 3.8: Schematic of the Interface Shear Tests

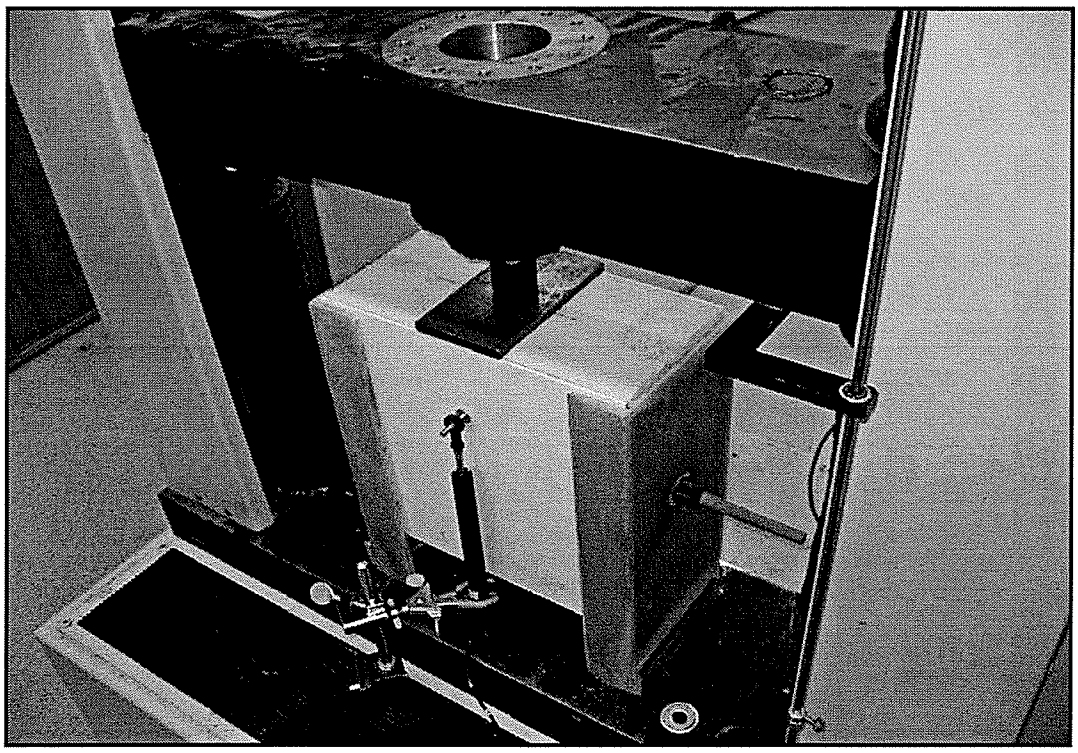


Figure 3.9: Panel-to-channel shear test

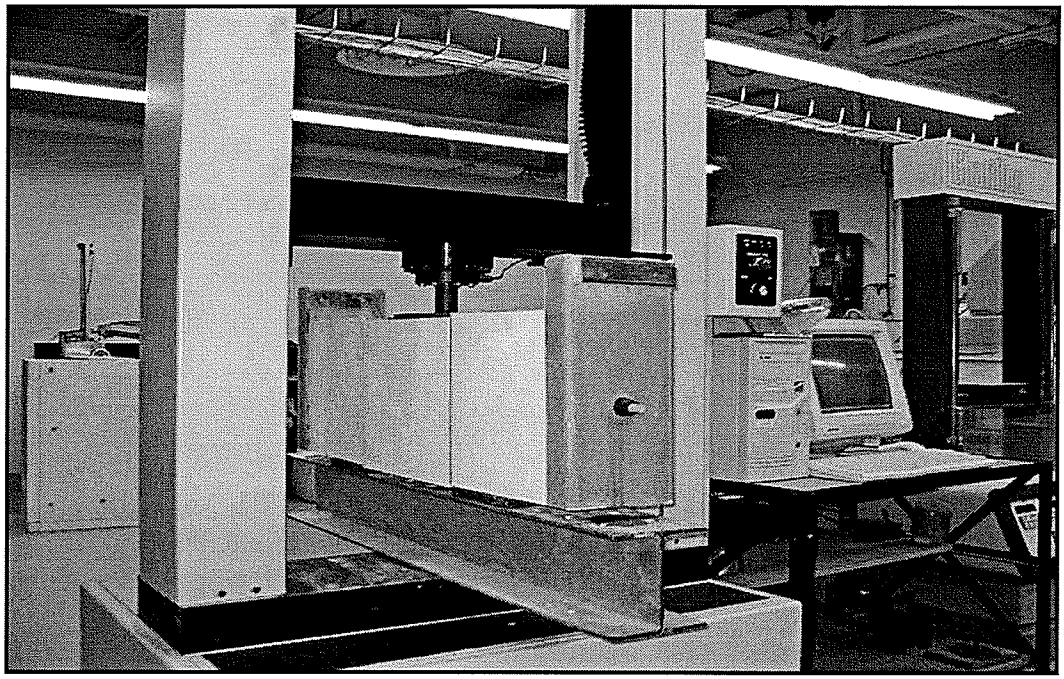


Figure 3.10: Panel-to-panel shear test

3.6. IN-PLANE PANEL BENDING

The purpose of these tests was to investigate the performance of the Ambiente panel as a supporting pony wall for the floor joists. The pony wall was considered to be either directly supported on a strip footing or supported on pads at regular intervals. For the second case, the pony wall would be subjected to a bending moment applied by floor joists resting on top at presumed 610mm (2ft) intervals.

Two pony walls were used in this test; both were 2438mm (8ft) long. The first was 559mm (22in) deep while the second was 610mm (24in) deep. These were cut from a 4x8 panel, which by default, has three ducts for the prestressing cables, running perpendicular to the long dimension.

The first pony wall, which was 559mm (22in) deep, was tested with no FRP channel sections attached to the edges. Bearing support was provided at the loading points and the supports using sections of a 5mm thick pultruded channel, as shown in Figure 3.12. This specimen also included a 25 by 51 mm (1"x2") edge groove at the compression flange, as this is also a feature of the Ambiente panel.

The second pony wall (Figure 3.13) was 610mm (2ft) deep, and had no edge groove. Furthermore, a 1.6mm-thick (1/16in) channel was bonded along the compression and tension flange, using West System® epoxy. No additional bearing support was provided for this specimen.

Both specimens were tested in four-point bending with an effective span of 2337mm(7'-8"). The load was applied at a rate of 4mm/min, at two points, each at 305mm(1ft) away from the centre of the beam, as shown in Figure 3.11. Two LVDT's placed at midspan of the bottom edge were used to measure deflection.

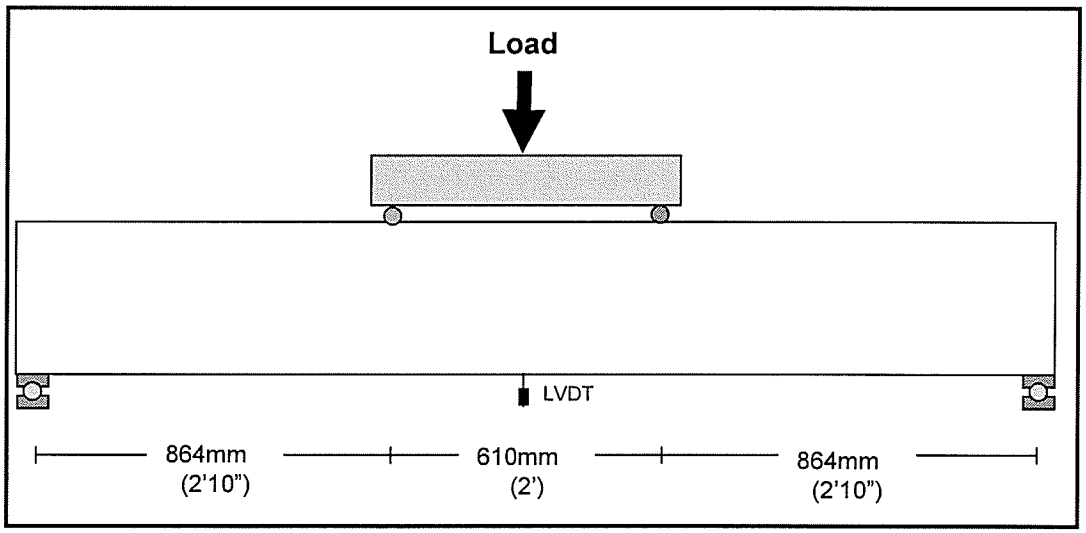


Figure 3.11: 4-point Bending of Ambiente Panel

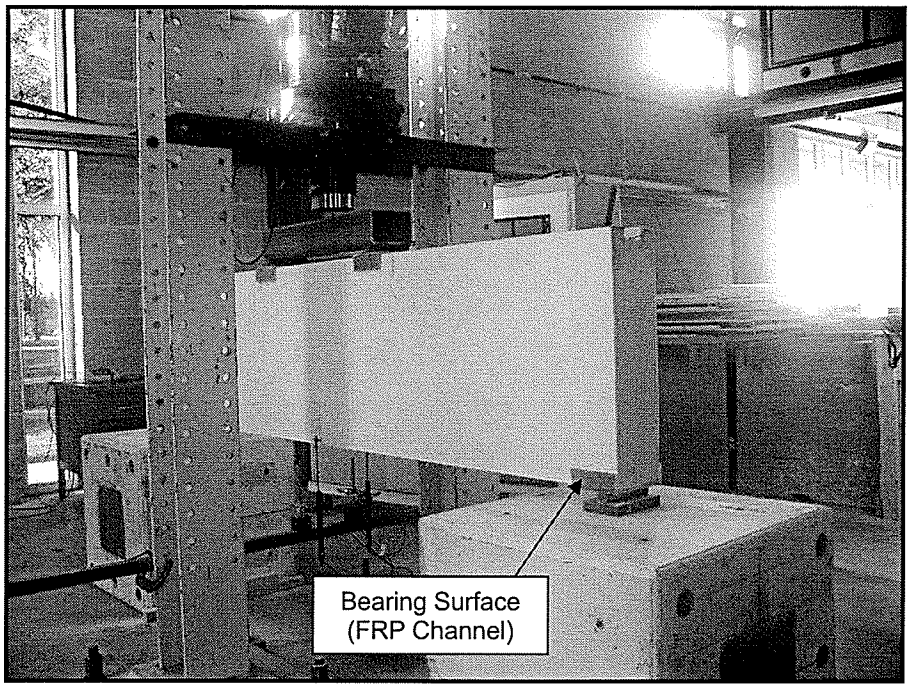


Figure 3.12: Bending Test of Beam without Channel Section along Flanges

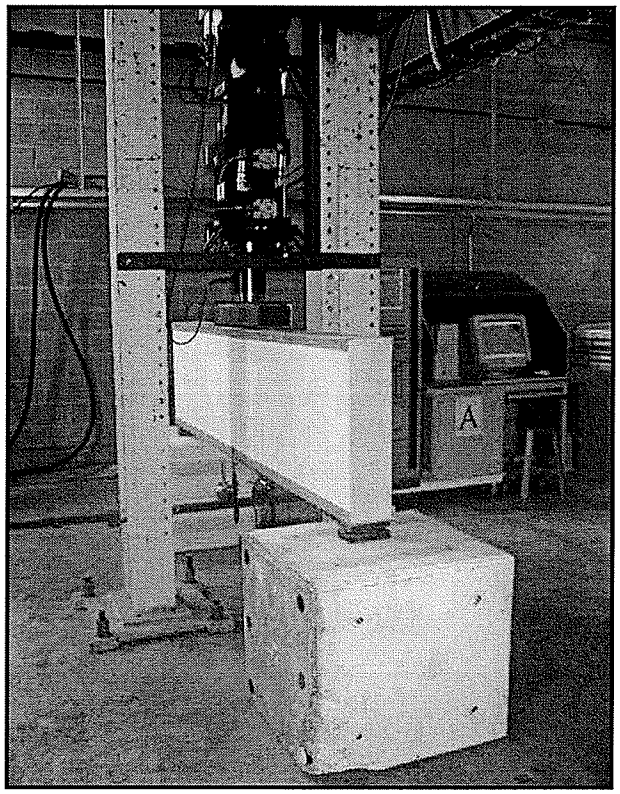


Figure 3.13: Bending Test of Beam with Channels Bonded along Flanges

3.7. UV RADIATION AND CONDENSATION DURABILITY

In order to asses whether there is any deterioration of the performance of the fiberglass skin of the wall panels due to sunlight and condensation, durability tests were performed on nine fiberglass skin specimens designated as E1 to E9. The specimens were subjected to accelerated effects of sunlight due to ultraviolet (UV) radiation, followed by a high humidity environment to simulate rain. The apparatus (Figure 3.14) used complies with the of ASTM G 53-91 standard. The specimens were subjected to 12 daily cycles consisting of 16 hours of UV exposure at a temperature of 65^0C and 8 hours of condensation at a temperature of 60^0C.

After 12 days of exposure, the specimens were tested according to ASTM D3039 (Figure 3.15) in order to obtain their tensile strength. Another nine control specimens (U1 to U9) were tested using this method in order to provide a basis of comparison in assessing whether accelerated UV and condensation exposure had an effect on the ultimate tensile strength of the material. Three tensile specimens of each group were also monitored for strain, using a 10mm strain gauge mounted in the middle of the specimen longitudinally. The results of these tests are discussed in section 4.7

Figure 3.14: UV and Condensation Apparatus (Model: QUV, Q-Panel Inc.)

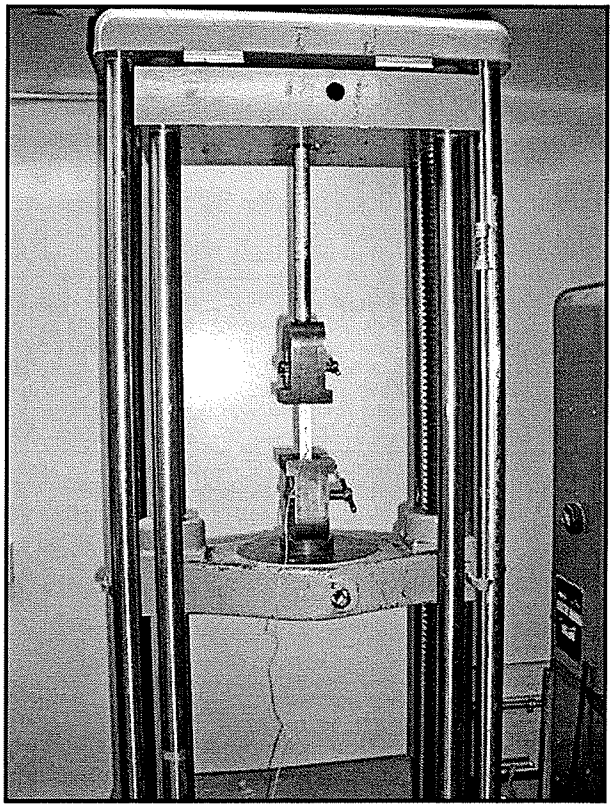


Figure 3.15: Tensile Strength Test on Fiberglass Skin Coupon

3.8. FREEZE-THAW DURABILITY

Freeze-thaw durability tests were performed according to ASTM C1262-98 in order to evaluate the performance of the panels under extreme temperature changes. Testing was conducted on two groups of samples extracted from an 8□x4□wall panel, saw-cut at dimensions of 110x110x32mm ± 1mm, with a sample weight of 182g ± 15g. Each sample included the fiberglass skin material on its top 110x110mm side. The first group of samples was tested with no modifications on the samples. The second group included samples that where coated with an epoxy resin on each edge, excluding the fiberglass skin side, with a coating thickness of approximately 1mm. The second group was incorporated in the test program in order to asses whether a reduction in the absorption of water would improve the overall durability performance of the wall panels.

Each group of samples consisted of four subgroups containing five samples each, for a total of twenty samples per group. Each subgroup was designated with the numbers 1 to 4. Each sample of a group was designated with the letters □A□to □E□for the first group and □A□to □E□for the second group. For example, specimen 3D□was sample D of subgroup three of the second group. All subgroups where subjected to 0(control), 20, 25 and 30 freeze-thaw cycles, respectively (Table 3.1). After proper conditioning and weighing, each specimen was immersed in water at a depth of 13 mm with the skin facing up. Each specimen was confined to individual plastic containers with air-tight covers. Each cycle was defined by subjecting a specimen from 23^{0}C temperature to -17^{0}C for a period of 4 hours and then back to 23^{0}C for a period of 3 hours. The freeze-

thaw apparatus used (Figure 3.16) was capable of controlling the temperature within 1^0C.

The objectives of this test method are to measure any weight loss and to asses any changes in the compressive strength due to freeze-thaw cycling. After subjecting each subgroup to the specified number of cycles, the residue of each specimen was collected by passing the water in each container through a pre-weighed paper filter. After oven-drying the specimens and the paper filter, the weights were recorded again in order to determine the individual weight loss for each sample. Finally, the samples were subjected to a compression test (Figure 3.17). Due to the fact that the tested specimens included the skin material, the conducted compressive tests were not in compliance with the ASTM C140 standard. These values, however, can be used to evaluate any changes in the compressive strength, since a control group was also tested.

Table 3.1: Freeze-Thaw Specimen Designation

Group 1 (Uncoated)	Group 2 (Epoxy Coated)	No. of Testing Cycles
1A	1A□	0 (Control)
1B	1B□	
1C	1C□	
1D	1D□	
1E	1E□	
2A	2A□	20
2B	2B□	
2C	2C□	
2D	2D□	
2E	2E□	
3A	3A□	25
3B	3B□	
3C	3C□	
3D	3D□	
3E	3E□	
4A	4A□	30
4B	4B□	
4C	4C□	
4D	4D□	
4E	4E□	

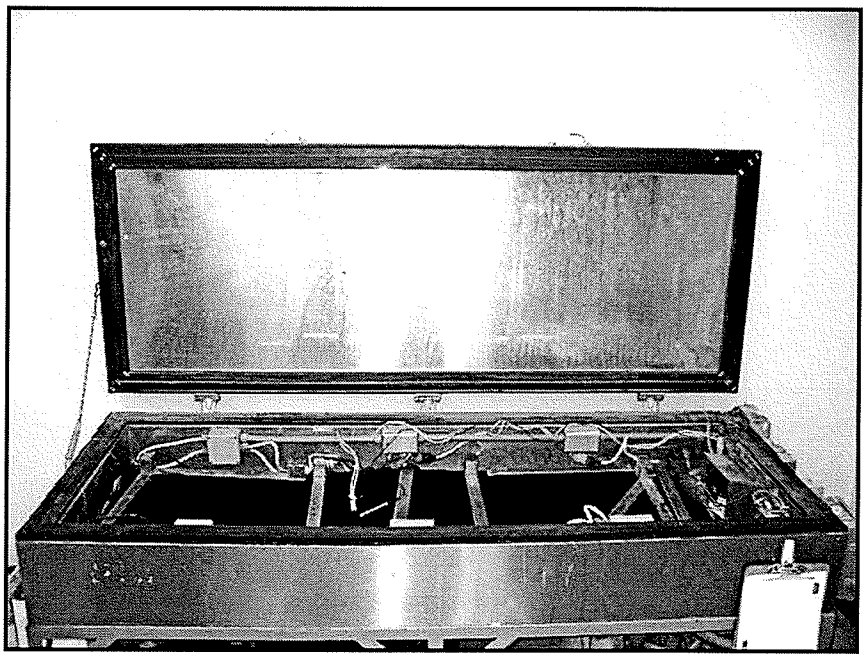


Figure 3.16: Free-Thaw Testing Chamber

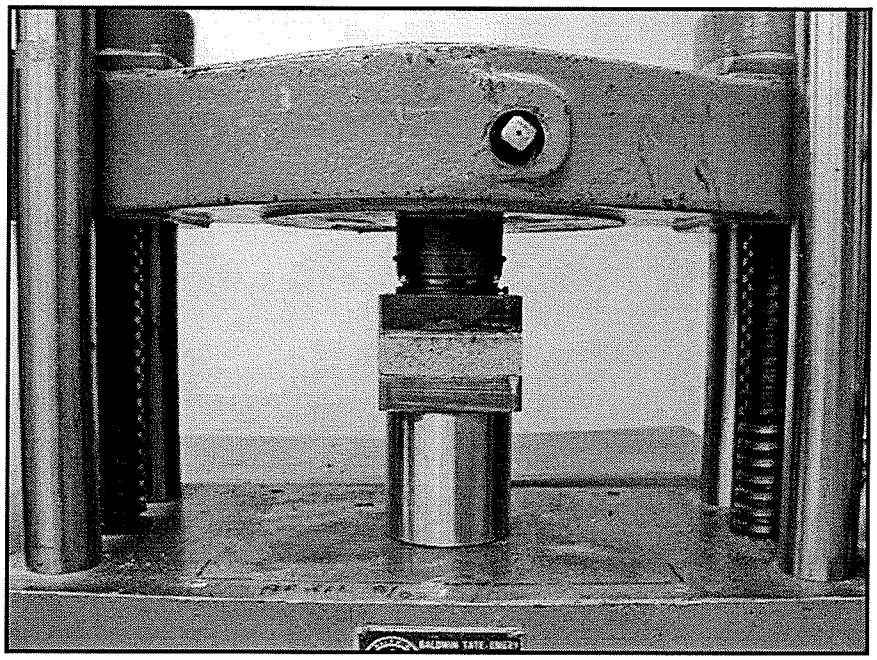


Figure 3.17: Compression Test of Freeze-Thaw Tested Sample

4. EXPERIMENTAL RESULTS AND DISCUSSION

4.1. SIGNIFICANCE OF TEST RESULTS

The purpose of the structural tests was to ensure that the specific components of the Ambiente system exhibit sufficient strength and stiffness to satisfy structural requirements, as specified by Part four of the National Building Code of Canada (NBCC), for the design of a full-scale test-house in Manitoba. A detailed description of the house and its design is given in Chapter five.

4.2. FLOOR JOIST BENDING

Figure 4.1 shows the resulting load versus midspan deflection response of the joists. During testing it was noted that noise due to cracking and distortion was not noticeable until the joists had deflected over 13mm. Failure occurred on a diagonal member of the joist, directly above the end support of the assembly by splitting and punching through the flange, as shown in Figure 4.2, at a load corresponding to a midspan moment of 21.6kN.m or, 10.8kN.m per joist. As described in Section 5.4.4.4.1, this yields a factored resistance of 6.5kN.m. The factored moment was found to be 4.0kN.m, which shows that the floor joist is structurally adequate. In terms of serviceability requirements, Figure 4.1 suggests that under specified loads producing a moment of 5.4kN.m, the floor is expected

to deflect about 6mm. For a design span of 3810mm, the allowable deflection limit is 10.6mm which is greater than 6mm. Therefore, test results show that the floor joist is structurally adequate.

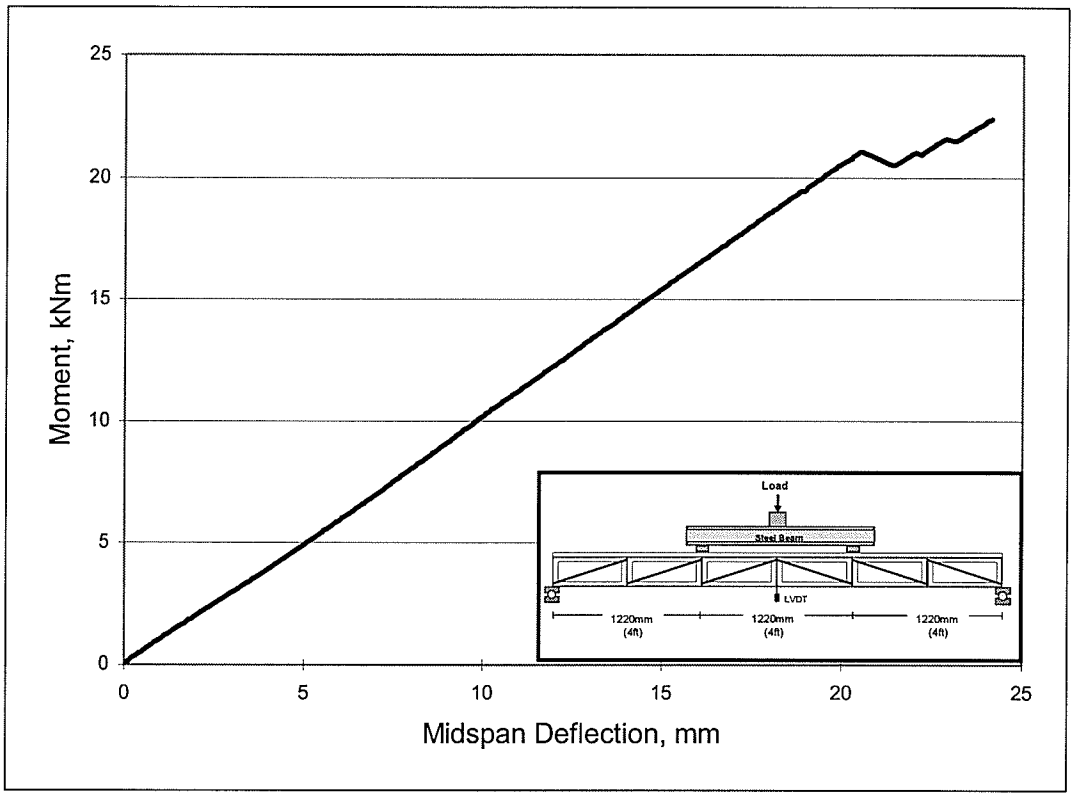


Figure 4.1: Moment due to Applied Load versus Midspan Deflection

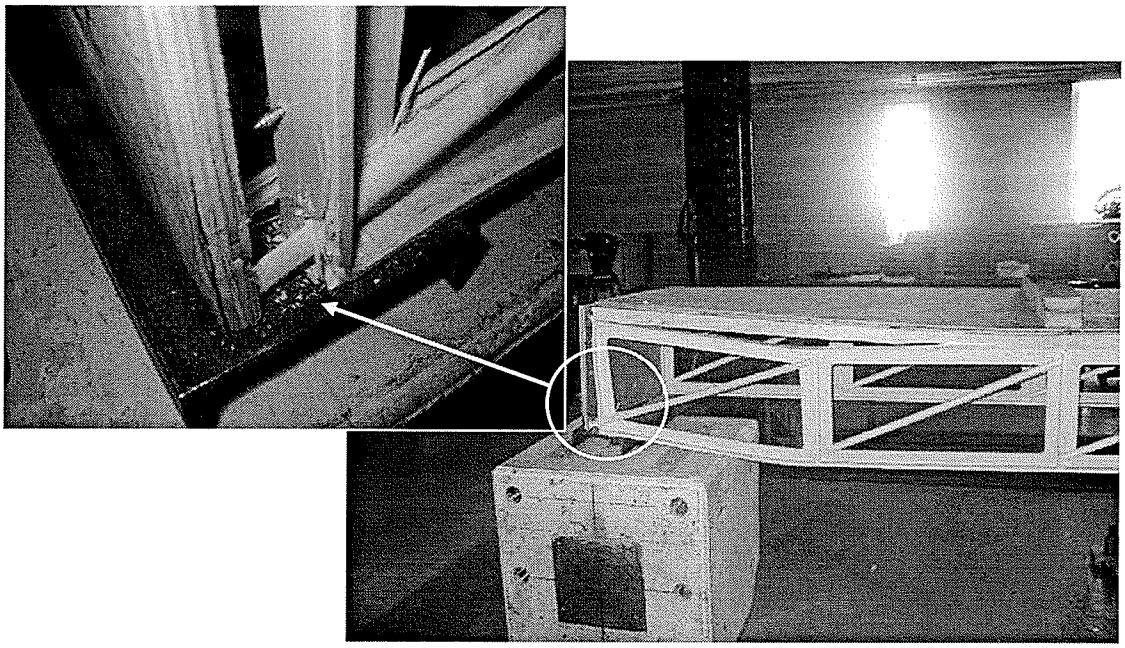


Figure 4.2: Punch-through Failure of Diagonal Member at 35.4kN

4.3. RACKING TEST

Figure 4.3 shows the load versus lateral deflection results of the racking test assembly. The ultimate load was recorded to be 39.5kN at a lateral deflection of 68.2mm. Failure was due to local buckling (Figure 4.4 and Figure 4.5) of the fiberglass skin at two locations. No failure was observed in the post-tensioning cables. The factored resistance was found in Section 5.4.4.6 to be 31.6kN. The factored lateral load due to wind was calculated in Equation 5.45 to be 16.3kN, which is less than the factored resistance. The specified load to be resisted from six wall panels was found to be 10.6kN. Figure 4.3 suggests that two panels under this specified load will produce a deflection of about 7mm, while the maximum allowable deflection was found to be 22mm. Therefore, test results indicate that the walls are structurally adequate under lateral wind loads.

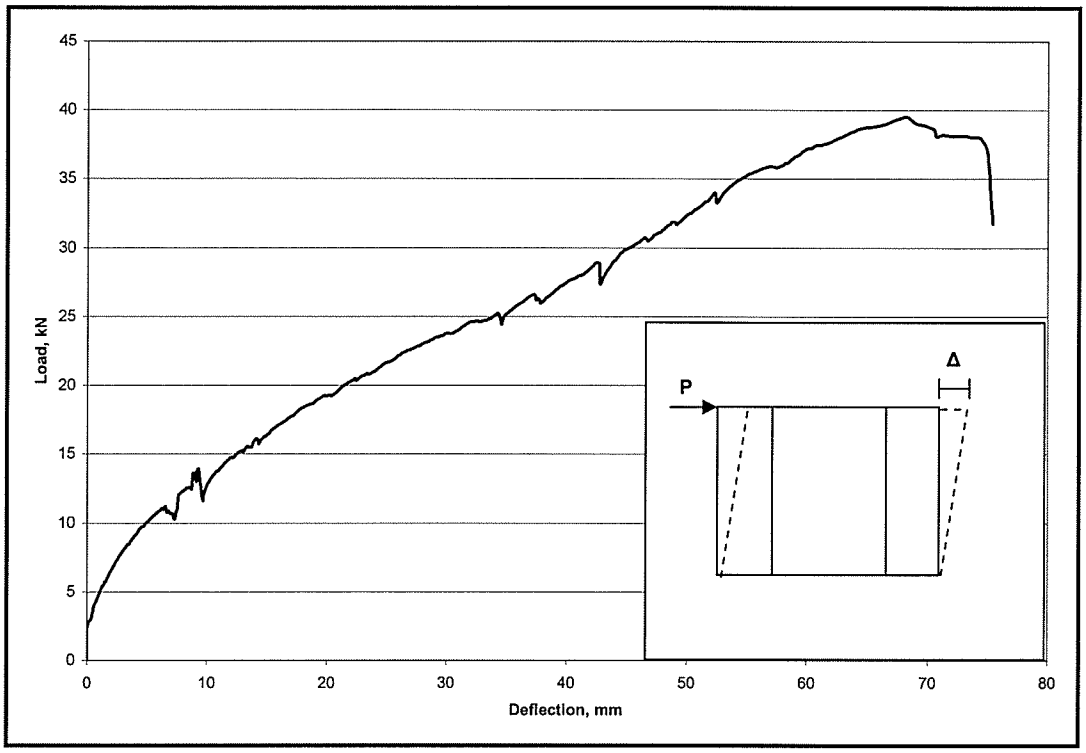


Figure 4.3: Load versus Lateral Deflection of Racking test Assembly

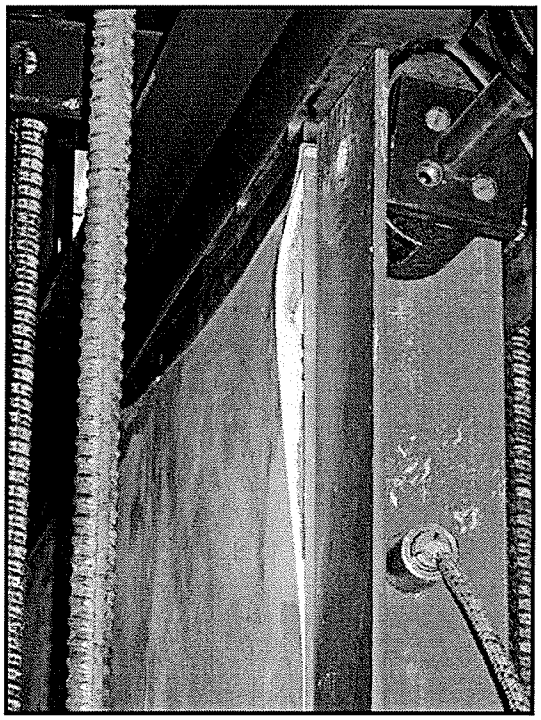


Figure 4.4: Local Buckling of Fiberglass Skin under Direct action of Load

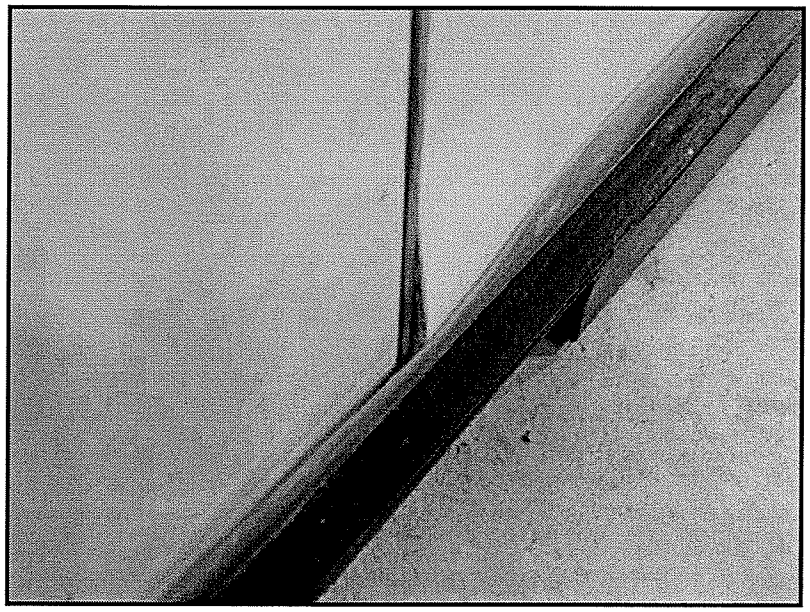


Figure 4.5: Loacal Buckling of Fiberglass Skin at Bottom of Vertical Joint

4.4. TENSILE CAPACITY OF CABLE

A summary of the results for the three specimens tested under pure tension is shown in Table 4.1. Although this test setup was designed for obtaining the pure tensile strength, the corresponding strain could not be obtained directly, since successive slip occurred as the cable was tightening around the circular steel end fixture (Figure 4.6). Tensile failure occurred at 20mm above the middle of the effective length (Figure 4.7). In order to obtain a measure of true ultimate deflection the test data file was imported into a spreadsheet and processed in order to remove the slip effect. The resulting average effective modulus of elasticity was found to be 2,125 MPa. The average ultimate stress was 231MPa.

The specimens that were tested with anchors and no resin coating failed prematurely, as shown by the load-deflection plot (Figure 4.8). As Table 4.2

shows, the average ultimate strength was only 49 MPa, which is about 20% of the pure tensile strength of the cable. Failure occurred directly below the anchor with the exterior cover of the cable failing and allowing the inner fibers to slip through the anchor (Figure 4.9).

The specimens coated with epoxy at the ends performed much better, since the epoxy at the ends provided a better transfer of stresses from the exterior coating to the inner fibers of the cable. As Table 4.3 shows, the average ultimate strength was 216 MPa, corresponding to a force of 29.3kN, which is very close to the pure tensile strength of the cable. Failure of these specimens was also slip at the inner fibers through the anchor.

It can be concluded that coating the cable ends with resin along the length of the anchorage can significantly enhance the tensile capacity of the cables. The resin coating allows for a better transfer of stresses from the inner fibers to the outer ones, bringing the cable capacity to a value near its pure tensile strength. With respect to design, the worst case loading, combined with a specified post-tensioning force of 13.3kN, will produce a total tensile force of 20.2kN, which occurs within the roof panels under diaphragm action. The factored resistance was found to be 20.5kN. Therefore, results show that the cables exhibit sufficient strength, provided that an epoxy coating is applied along the ends.

Table 4.1: Test Result Summary for Pure Tension Test

SPECIMEN	ULT. STRESS, MPA	EFFECTIVE MOE, MPA
PT1	223	2,120
PT2	239	2,114
PT3	231	2,140
AVERAGE	231	2,125

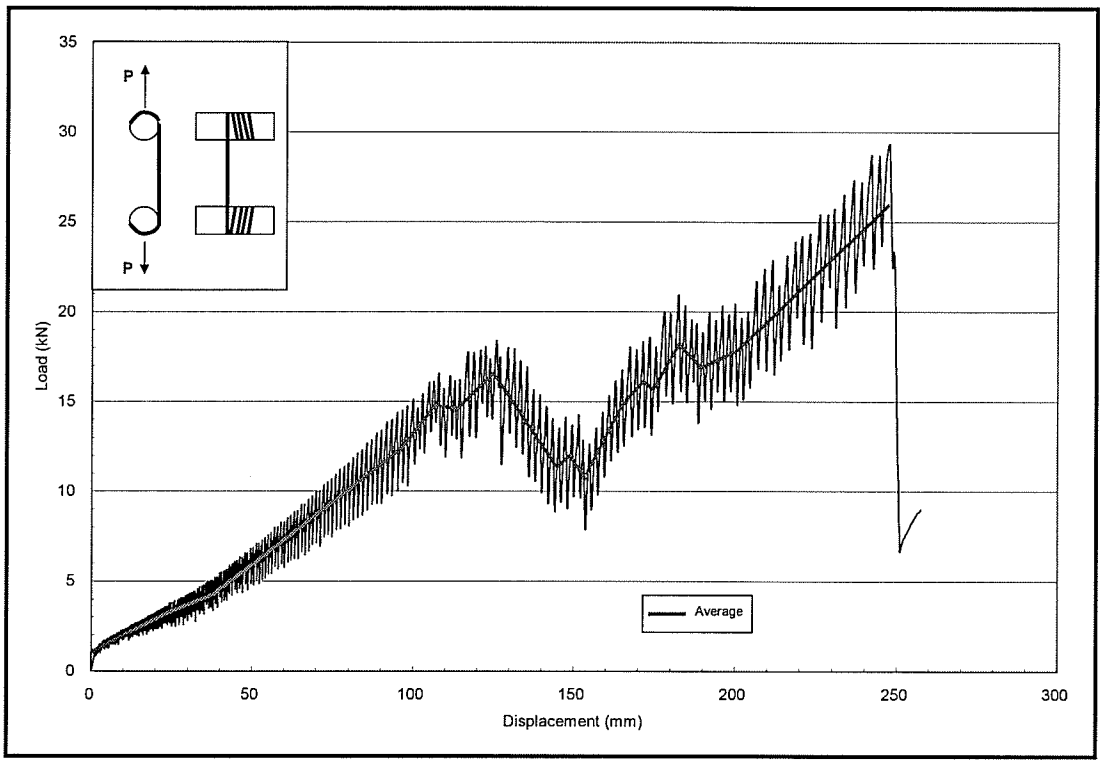


Figure 4.6: Typical Load-Displacement Response of Pure Tension Test (Specimen PT1)

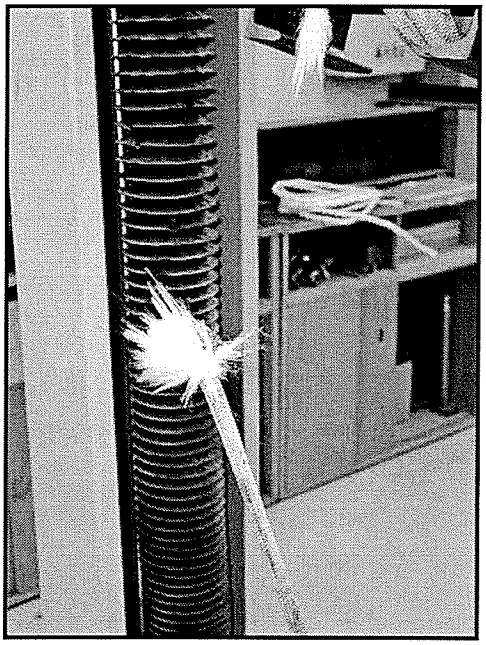


Figure 4.7: Tension Failure of Cable

Table 4.2: Test Result Summary for Anchor test with Uncoated Ends

SPECIMEN	ULT. STRESS, MPA	MOE, MPA
AT1	49	754
AT2	51	823
AT3	54	794
AVERAGE	51	790

Table 4.3: Test Result Summary for Anchor test with Epoxy-coated Ends

SPECIMEN	ULT. STRESS, MPA	MOE, MPA
ATR1	202	1,836
ATR2	231	1,958
ATR3	215	1,972
AVERAGE	216	1,922

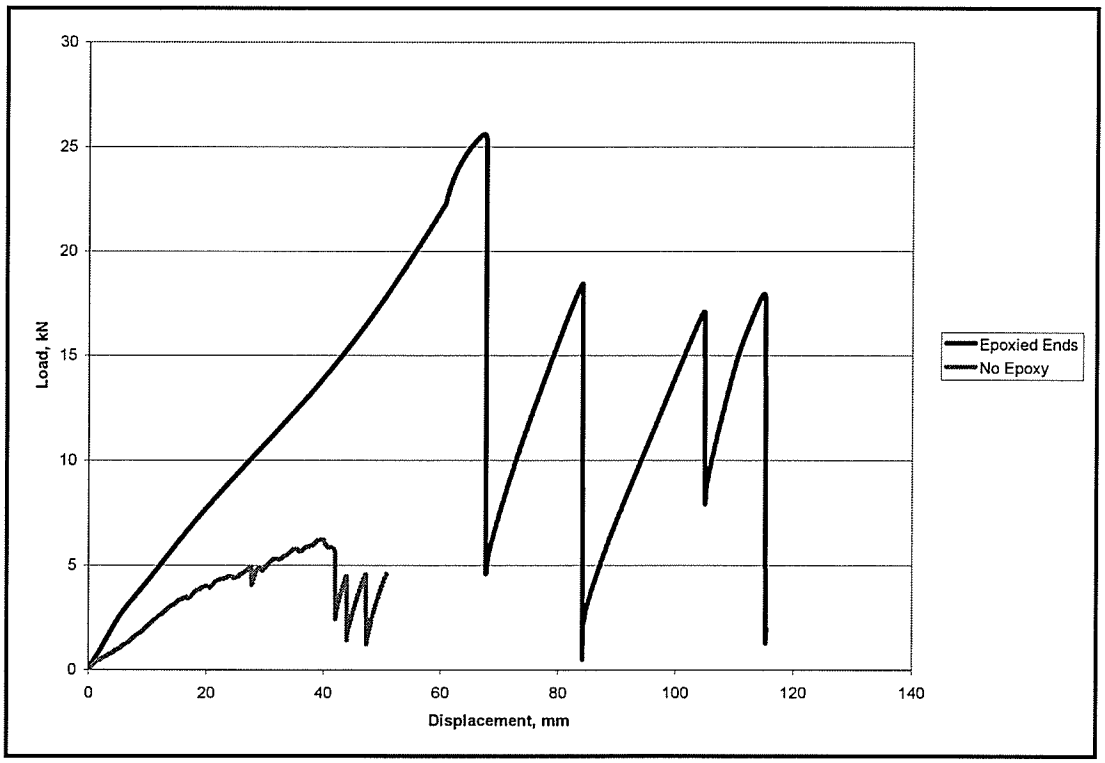


Figure 4.8: Typical Load-Deflection Response for Anchor Test (Specimens AT1, ATR1)

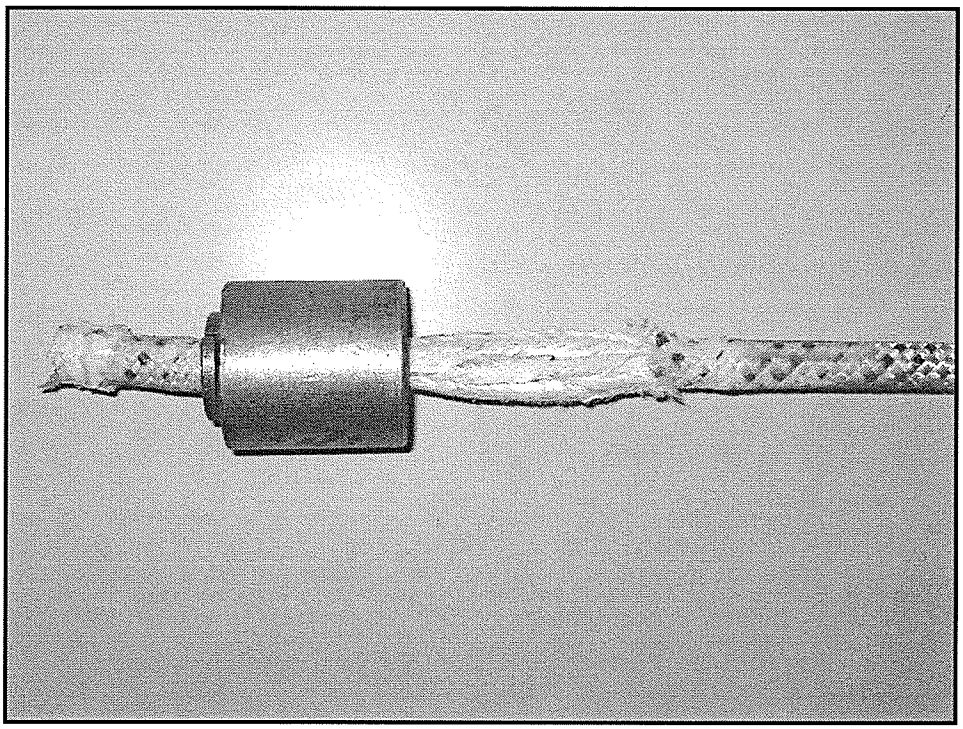


Figure 4.9: Typical Failure of Cable Under Tension Directly Below the Anchor

4.5. INTERFACE SHEAR BETWEEN PANELS

The relationship between the applied torque and the resulting tension in the rod obtained experimentally is given in Figure 4.10. This relationship was used to control the magnitude of the tension in the rod for each shear test. The relationship between ultimate interface shear and the post-tensioning force for both panel-to-channel and panel-to-panel tests is shown in Figure 4.11 and Figure 4.12, respectively. Ultimate shear is defined here as the force required to overcome the friction between the panels or between the panel and the pultruded section. The actual shear strength is half of the experimentally obtained load, since both specimens were subjected to double shear. The coefficient of friction was obtained by conducting linear regression through the four experimental

results for each test. Results show that the coefficient of friction between the panel and the pultruded channel was about 1.04/2 = 0.52 and for the panel-to-panel interface it was 1.11/2 = 0.56.

In terms of design, knowing the coefficient of friction between panels or between a panel and a pultruded section, the force required to overcome this friction can be calculated with respect to the applied post-tensioning force of the cable. This overcoming force will be considered as an ultimate state for the strength along a joint between two Ambiente panels. For a design post-tensioning force of 13.3kN, the resulting joint shear strength between panels is 0.56x13.3=7.4kN per cable. For vertical panels with three cables running through each panel, the total joint shear strength is 7.4x3=22.2kN, while for roof panels having four lines of cables, the total joint shear strength is 7.4x4=29.6kN.

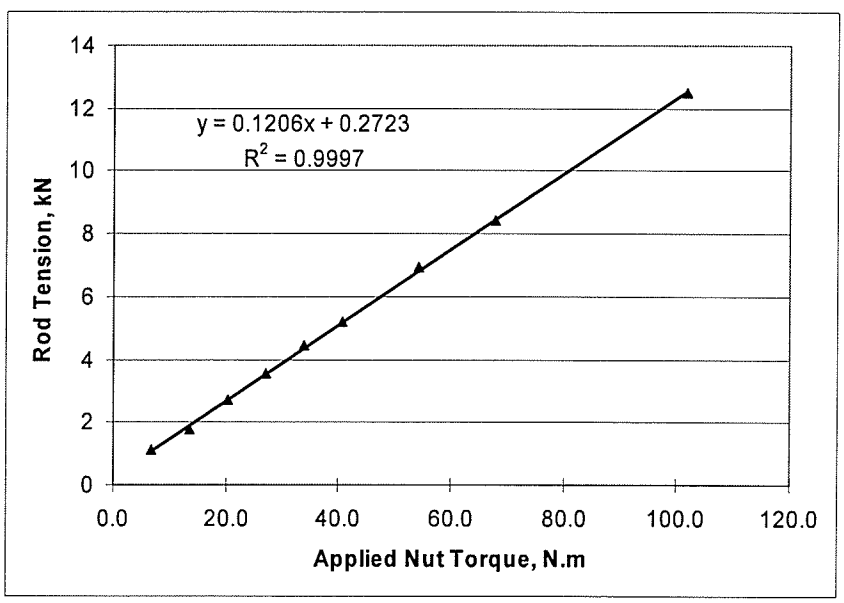


Figure 4.10: Relationship between Applied torque and Resulting Tension in Rod

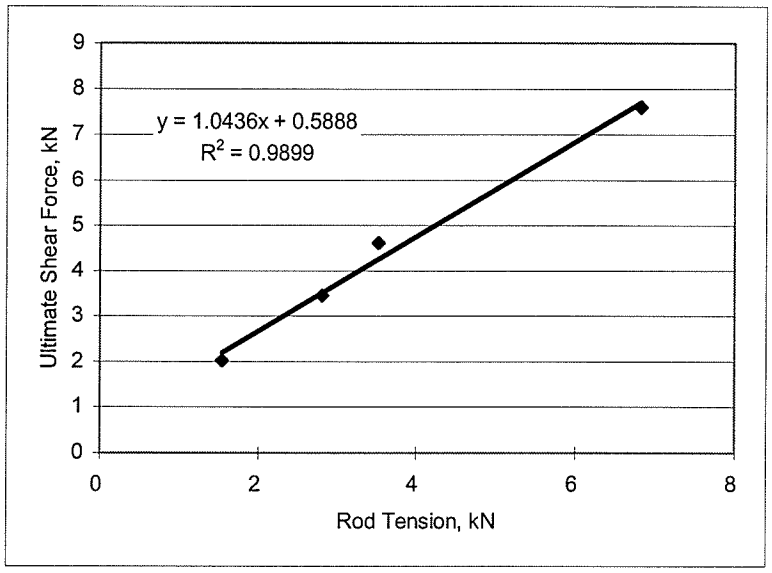


Figure 4.11: Ultimate Panel-to-Channel Shear vs. Rod Tension

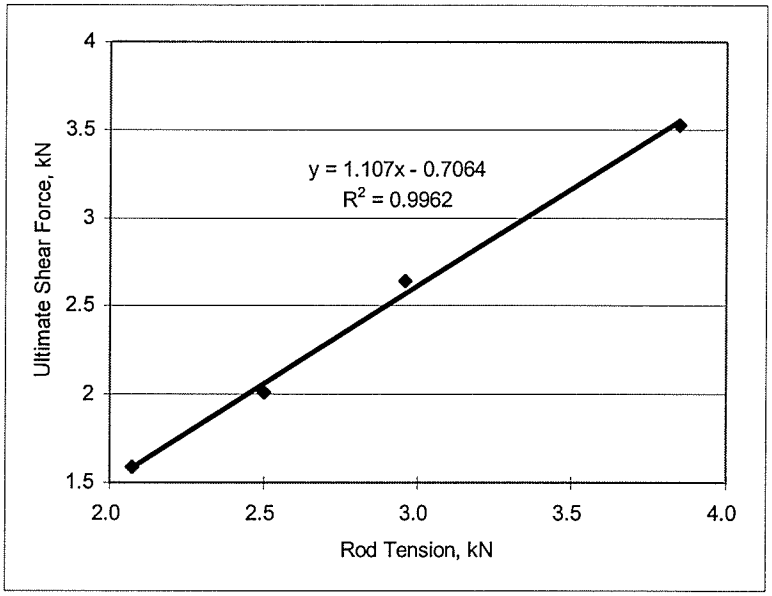


Figure 4.12: Ultimate Panel-to-Panel Shear vs. Rod Tension

4.6. IN-PLANE PANEL BENDING

Figure 4.13 shows the load-deflection behaviour for both unreinforced and reinforced panels. As discussed in Section 3.6, one panel was reinforced along the top and bottom with composite channels, while the other was only reinforced at the bearing surfaces (loading locations and supports). For the unreinforced case, the beam failed in flexure (Figure 4.14) with an ultimate load of 67.5kN, at a deflection of 13.8mm. The second specimen reinforced with channel sections failed prematurely at the support (Figure 4.15). This did not occur in the first case because the thickness of the bearing channels provided at the supports was almost three times larger than the thickness of the channel used to reinforce the second wall.

Test results suggest that a 5mm-thick channel was able to provide sufficient bearing for a point load of 67.5/2=33.8kN. As calculated in section 5.4.4.4.1, the factored floor load produces a line load of 2.2kN/m on each joist. For a joist span of 3.81m, this corresponds to a bearing load of 2.2x8.31/2 = 4.2kN. Therefore, providing a 5mm-thick channel section along the pony wall can ensure adequate bearing support, as suggested by test results. In the case that the Ambiente panel would be used as simply supported pony wall, the maximum unfactored moment resistance would be (67.5/2)x0.864 = 29.2kN.m.

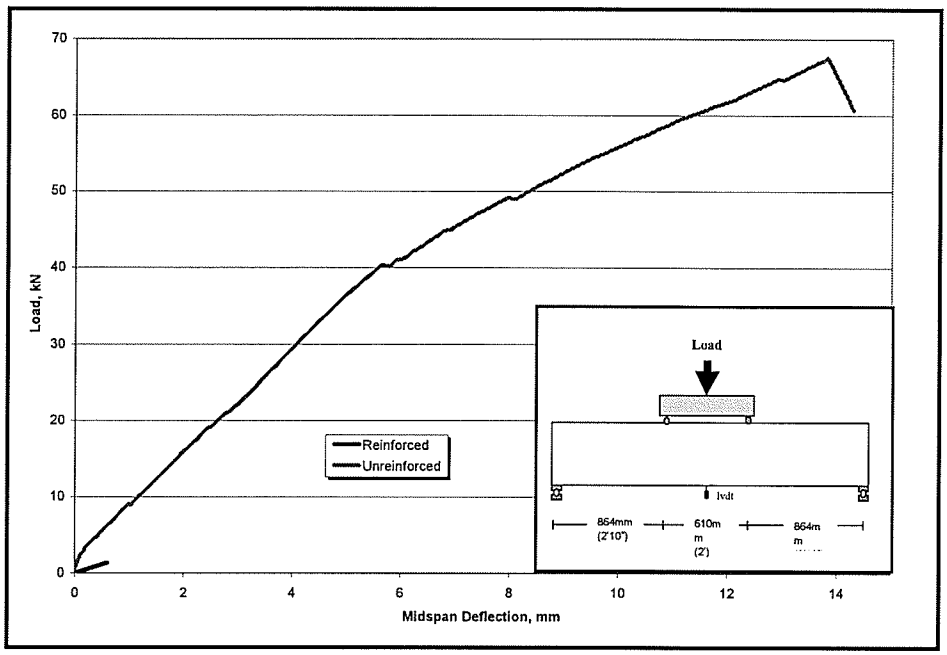


Figure 4.13: Load vs. Mispan Deflection for both Unreinforced and Reinforced Beams

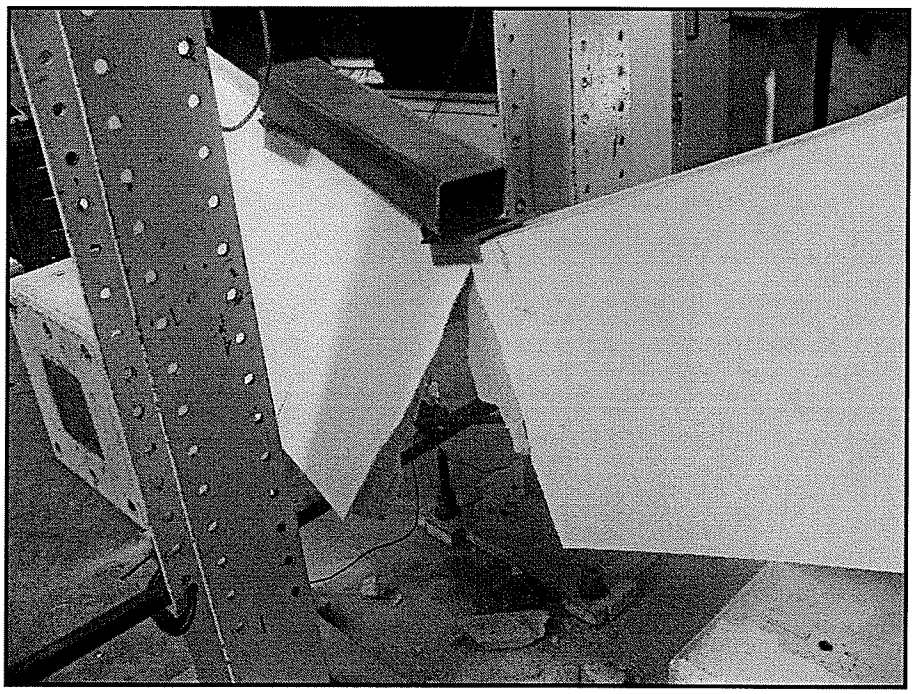


Figure 4.14: Bending Failure of Unreinforced Beam

Figure 4.15: Bearing Failure of Reinforced Beam

4.7. UV RADIATION AND CONDENSATION DURABILITY

After 192 hours of UV and 96 hours of condensation exposure, specimens E1 to E9 were visually inspected. Figure 4.16 shows the amount of discoloration and loss of gloss that occurred to an exposed specimen with respect an unexposed specimen. No other defects or irregularities were observed on the exposed specimens. The specimens were purposely left unprotected; however UV inhibitors, such as polyurethane or aluminum oxide can reduce discoloration (Hassan et. al, 1999). The ASTM G 53-91 standard specifies that no theoretical attempt should be made to correlate laboratory exposure hours with an actual weathering period, unless an extensive study is done in observing variations of

natural exposure such as daily sunlight and condensation variations throughout a year, as well as geographical and natural conditions.

Table 4.4 summarizes the results for the tensile strength test for both groups of specimens. Figure 4.17 shows the stress-strain relationship of three specimens for each group. Results indicate that mean tensile strength of the exposed specimens changed only within the range of the standard deviation, with respect to the unexposed specimens. Therefore, results suggest that 192 hours of UV and 96 hours of condensation exposure did not affect the tensile capacity of the skin material of the Ambiente panel.

Table 4.4: Summary of Ultimate Tensile Strengths for Specimens

SPECIMEN	ULTIMATE STRESS (MPA)	SPECIMEN	ULTIMATE STRESS (MPA)
U1	51.4	**E1**	50.4
U2	58.0	**E2**	54.5
U3	53.7	**E3**	51.7
U4	64.9	**E4**	64.4
U5	58.8	**E5**	56.9
U6	53.9	**E6**	55.4
U7	62.9	**E7**	49.5
U8	52.2	**E8**	55.2
U9	59.4	**E9**	66.3
MEAN	**57.2**	**MEAN**	**56.0**
ST.DEVIATION	**4.77**	**ST.DEVIATION**	**5.84**

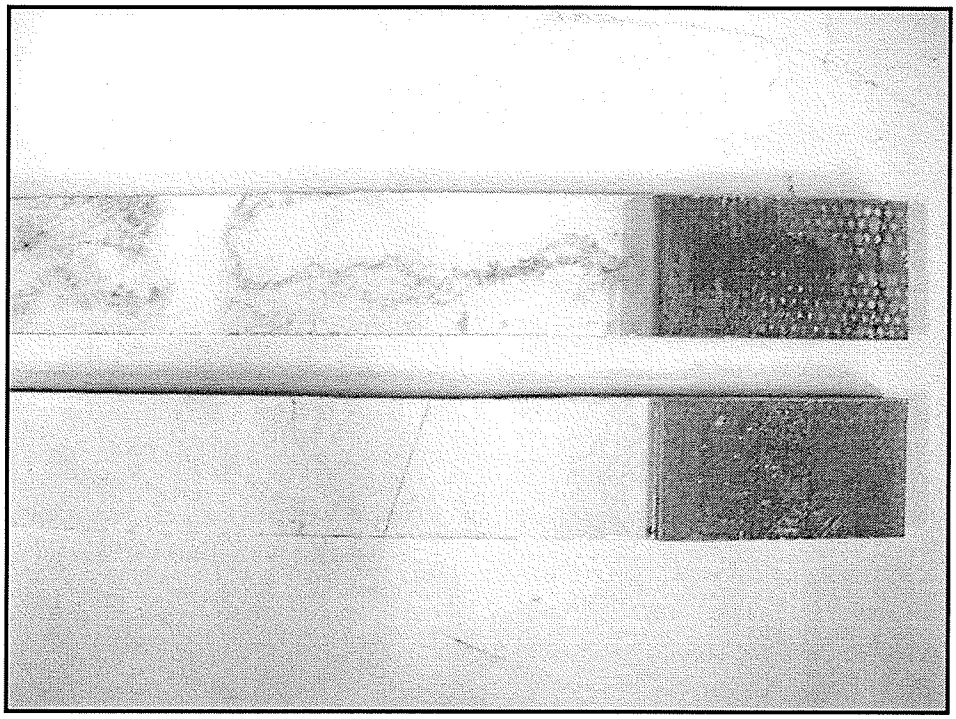


Figure 4.16: Exposed (Top) and Unexposed (Bottom) Specimens

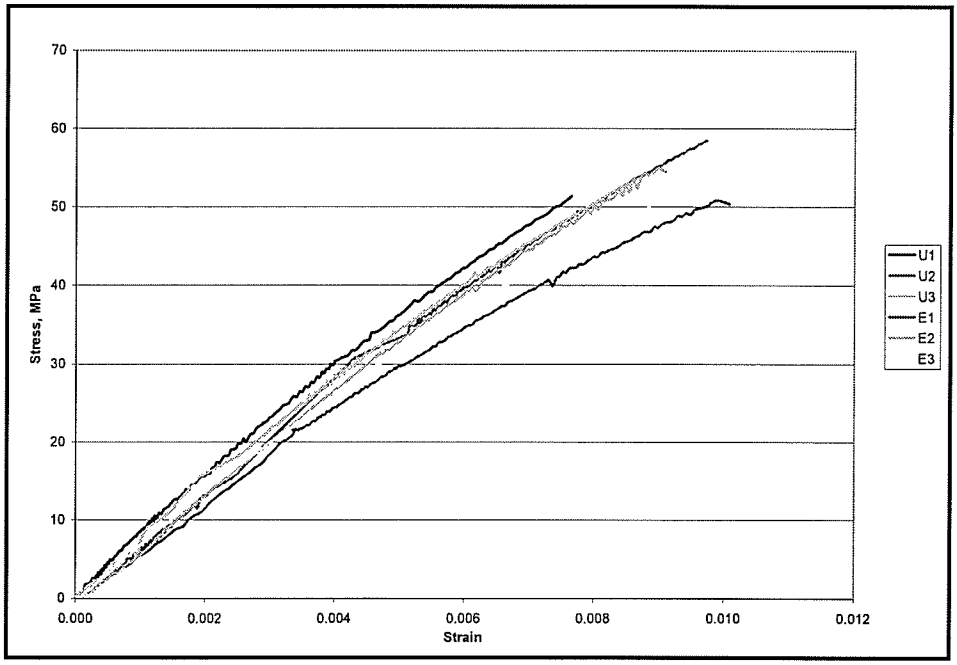


Figure 4.17: Stress-Strain Behaviour for Specimens U1-U3 and E1-E3

4.8. FREEZE-THAW DURABILITY

Tables 4.5 and 4.6 show a summary of the weight loss and compressive strength results for the two groups. The most weight loss was observed for the uncoated specimens at 30 cycles, at a mean value of 0.6g which corresponds to 0.33% of weight loss. For the coated specimens, the weight loss for all cycling ranges was insignificant. Figures 4.18 and 4.19 provide a comparison of the average group strength for the uncoated and coated specimens respectively. Both figures show that it cannot be statistically concluded that there is a difference in the compressive strength as the number of freeze-thaw cycles increases, since there exists a strength range (indicated by the hatched band) which lies within the standard deviation limits for each group.

Table 4.5: Summary of Freeze-thaw Durability Results for Uncoated Specimens

Cycles	Sample	Weight Loss, g	Mean Weight Loss, g	Mean % loss	Comp. Strength, MPa	Mean Comp. Strength	St. Dev.
0	1A	N/A	N/A	N/A	1.8	**1.4**	0.5
	1B	N/A			1.6		
	1C	N/A			1.5		
	1D	N/A			1.5		
	1E	N/A			0.6		
20	2A	0.50	0.52	0.29	1.9	**1.7**	0.4
	2B	0.53			1.7		
	2C	0.69			2.2		
	2D	0.45			1.7		
	2E	0.44			1.1		
25	3A	0.39	0.58	0.31	1.2	**1.3**	0.4
	3B	0.66			1.9		
	3C	0.52			0.9		
	3D	0.64			1.7		
	3E	0.71			1.0		
30	4A	0.66	0.60	0.33	0.9	**1.2**	0.2
	4B	0.72			1.4		
	4C	0.59			1.5		
	4D	0.40			1.2		
	4E	0.61			1.1		

Table 4.6: Summary of Freeze-thaw Durability Results for Epoxy-coated Specimens

Cycles	Sample	Weight Loss, g	Mean Weight Loss, g	Mean % loss	Comp. Strength, MPa	Mean Comp. Strength	St. Dev.
0	1A'	N/A	N/A	N/A	2.1	**1.8**	0.3
	1B'	N/A			2.1		
	1C'	N/A			1.7		
	1D'	N/A			1.4		
	1E'	N/A			1.7		
20	2A'	0.02	0.02	0.01	1.3	**1.3**	0.4
	2B'	0.02			1.6		
	2C'	0.02			1.7		
	2D'	0.03			1.2		
	2E'	0.03			0.8		
25	3A'	0.05	0.04	NILL	1.4	**1.4**	0.5
	3B'	0.03			1.0		
	3C'	0.04			0.9		
	3D'	0.04			1.5		
	3E'	0.03			2.1		
30	4A'	0.04	0.04	NILL	1.2	**1.5**	0.5
	4B'	0.05			1.3		
	4C'	0.05			1.7		
	4D'	0.05			2.1		
	4E'	0.03			1.0		

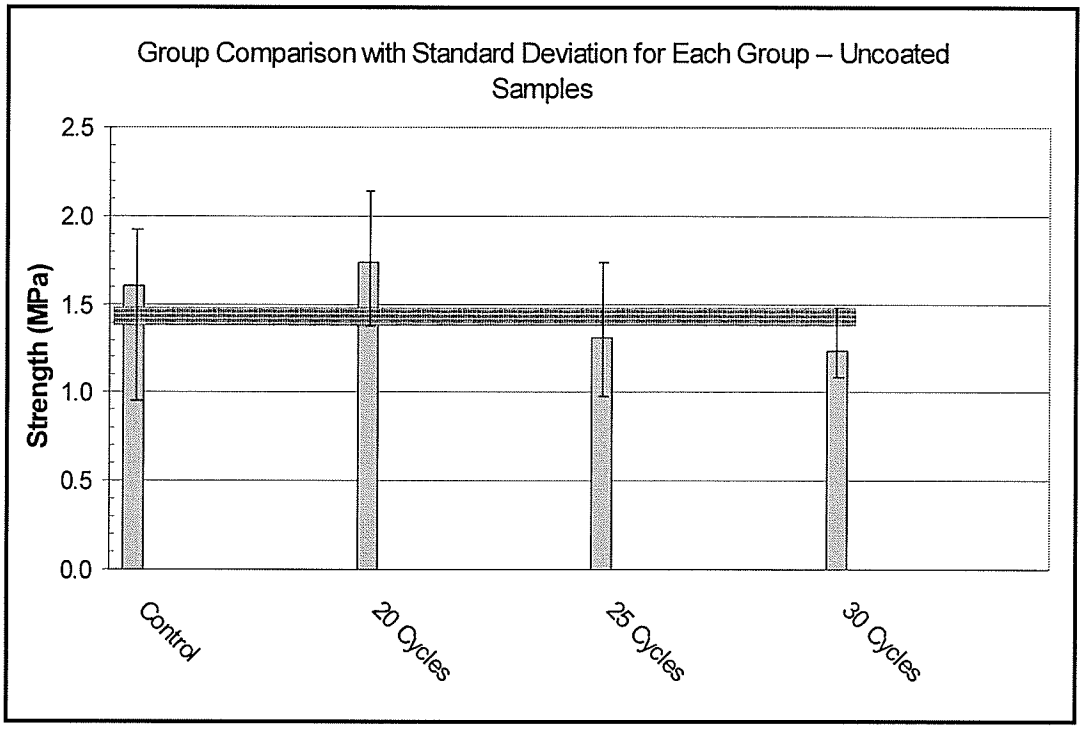


Figure 4.18: Strength Comparison of Uncoated Freeze-Thaw Specimens

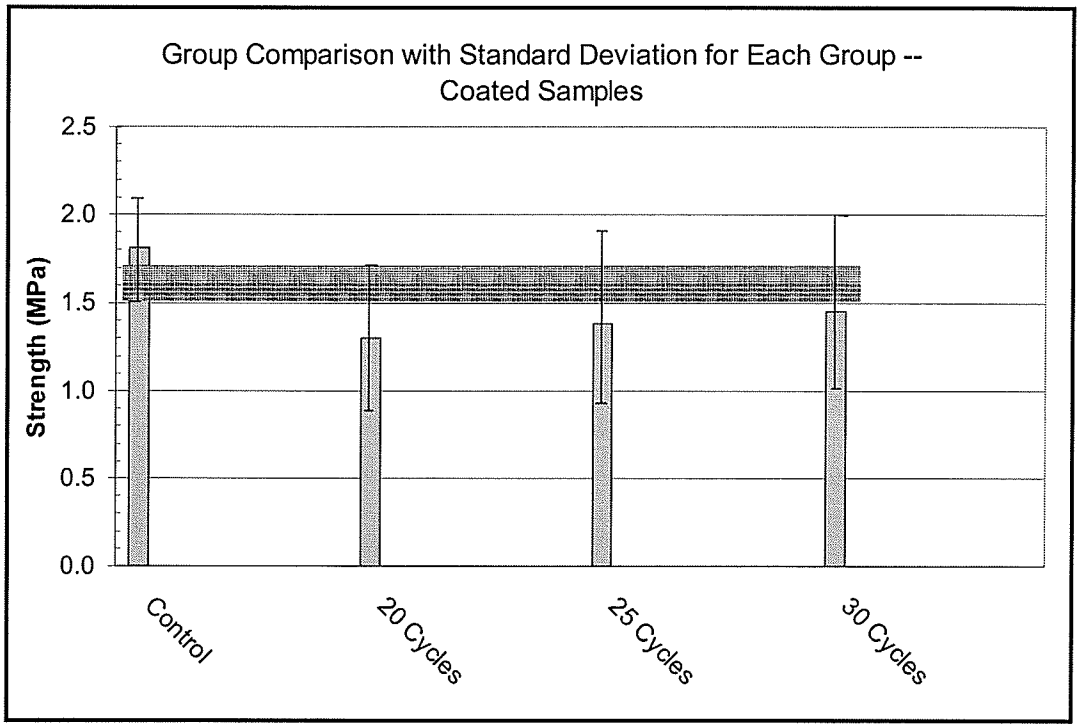


Figure 4.19: Strength Comparison of Coated Freeze-Thaw Specimens

5. DESIGN OF A FULL-SCALE TEST HOUSE

5.1. GENERAL

The second phase in the evaluation of the Ambiente Housing system is the construction and monitoring of a full-scale test house on the grounds of the Smartpark, a research and technology park owned and managed by the University of Manitoba. This unit will be called the Smart House to reflect the innovations it will feature in the areas of construction and energy efficiency. The evaluation process in Phase II is a long term one and is outside the scope of the present study. However, the design of the Smart House is based on research findings discussed in the previous four chapters and is included here as part of the objectives set forth at the beginning of the investigation. The construction of the Smart House will be part of a training session for local construction crews that will be hired to build Ambiente homes in the northern communities. Further thermal tests, similar to the ones performed on the small-scale unit, will also be performed at a larger scale and duration. Mechanical and heating systems will also be evaluated in this unit. Other monitoring will include motion sensors on structural joints as well as foundation settlements. Phase II is expected to begin in the fall of 2004 and to be completed in three years.

Many features of the Smart House, such as heating and ventilation, as well as architectural finishes, have been designed to accommodate the needs and the lifestyle of the people in the north. The heating system will allow the owner to switch between a woodstove and a conventional heating system depending on the seasonal availability of heating fuel sources. The floor plan was designed to accommodate a large family, including an extra utility room referred to as the mud room, which can serve as a storage or work place. The furnace and the water tank will also be located in this room. The architectural finish of the roof will have a shingles look, typical of many homes in the north. An architectural rendering of the Smart House along with detailed drawings are provided in Appendix A. The architectural planning has been carried out by Ambiente while the structural drawings have been prepared by Wardrop Engineering of Winnipeg in consultation with Ambiente and the University of Manitoba.

5.2. FLOOR PLAN

The floor plan for the smart House was designed to accommodate a family of 5 to 6 people, with a plan area of approximately 1100 square feet. A front and a back porch, made of composite materials, are included to reduce heat losses during entering or exiting the house, as well as to capture dirt and mud before entering the house. The mud room is located next to the back porch which also serves as a utility room. Appendix A includes a detailed drawing of the floor plan including dimensions and descriptions of each room. The open area between the kitchen and the living room imposes a discontinuation of the internal wall, unlike

the continuous internal wall of the initial designs of Ambiente houses. This was expected to create a need to reinforce the ridge beam that supports the roof panels. This is discussed in Section 5.4.4.3.

5.3. FLOOR AND FOUNDATION

The foundation for the Smart House will be an elevated type, incorporating a fiberglass floor joist system, as described in section 2.2.1. A detailed drawing of the foundation is also provided in Appendix A

5.4. STRUCTURAL DESIGN

5.4.1. General

This section provides an overview of the structural design associated with the Smart House. As described in section 5.3, the Smart House will be constructed with an elevated floor on a strip footing foundation. This type of foundation will be used for homes built on permafrost, which is commonly the case for buildings in Northern regions of Canada.

The calculations of the various loading conditions, discussed in the following sections, are based on Part 4 of the 1995 National Building Code of Canada (NBCC).

5.4.2. DEAD LOADS

The density of the Ambiente panel was obtained by a 1219mm by 1219mm sample to be 465kg/m^3. This value was assumed to be uniform in all panels in order to obtain self-weight loads. The weight of the pultruded sections was considered to be negligible.

5.4.2.1. Roof Panel Self-Weight

Given that the average thickness of the roof is 152mm(6in), the vertical pressure due to the self-weight of the roof can be calculated as:

$$P_w = 465 kg/m^3 \times 0.152m \times 9.81 m/s^2 = 693.4 N/m^2 \approx 0.7 kPa \quad \text{(Eq. 5.1)}$$

No additional dead-weight is imposed since the panel itself includes the shingles, the insulation and the interior finish.

5.4.2.2. Self-Weight of Wall Panels

The line load due to the weight a wall panel of height h can be calculated in kN/m as:

$$w(kN/m) = 465 \times 9.81 m/s^2 / 1000 \times h = 4.56h \quad \text{(Eq. 5.2)}$$

The height of the exterior wall is 2438mm, with an additional 914mm extending to the foundation giving a total wall height of 3352mm. The height of the main interior wall is 3759mm. For the gable end walls an average height of 3099mm with an additional 914mm extending to the foundation was used. Therefore,

ignoring window and door openings, the line load due to self weight of the exterior, interior and gable end walls are calculated as:

$w_{SWe} = 4.56 \times 3.352 = 15.3kN/m$ (Eq. 5.3),

$w_{SWi} = 4.56 \times 3.759 = 16.9kN/m$ (Eq. 5.4),

and,

$w_{SWg} = 4.56 \times 4.013 = 18.3kN/m$ (Eq. 5.5).

5.4.2.3. Floor Dead Load

Based on the weight of the assembly tested, as described in section 3.2, the load due to the self-weight of the floor joists was found to be 0.1 kN/m.

Furthermore, an additional 0.5 kPa will be assumed to estimate the weight of the flooring and the mechanical systems mounted on the joists. For a 559mm (1ft-10.125□) joist spacing this will impose an additional load, which combined with the self-weight of the joists will give a dead load of:

$w_{SWj} = 0.1kN/m + 0.5kN/m^2 \times 0.559m = 0.4kN/m$ (Eq. 5.6)

5.4.2.4. Post-Tension Cables

Results obtained by tension tests on the anchored cables with epoxy-coated ends (section 4.4) showed an ultimate tensile strength of 29.3kN (6587lbs). A maximum tensile force of 13.3kN (3000lbs) is recommended, in order to allow for additional tensile forces due to live load action and temperature effects. The factor of safety in this case is sufficient, since the force in the cable is not expected to increase after the post-tensioning force has been applied.

5.4.3. LIVE LOADS

Live loads, including snow, wind, and occupancy, are calculated according to part 4 of the NBCC. The Smart House was designed taking into account some of the weather conditions in the province of Manitoba. Seismic loading is not considered, since these forces are specified to be zero for Manitoba.

5.4.3.1. Floor Live Load

According to Clause 4.1.6.1 of the NBCC the specified floor live load for use and occupancy is given as w_{LF} = 1.9kPa. For the utility room, a load considered characteristic of a service room, specified at w_{LS} = 3.6kPa, is used.

5.4.3.2. Snow Load

According to clause 4.1.7.1 of the NBCC, the specified load S due to snow accumulation on a roof is given as:

$S = S_s(C_b C_w C_s C_a) + S_r$ (Eq. 5.7)

The worst case for ground snow load, S_s, in Manitoba applies to Churchill and is given in Appendix C of the NBCC, to be S_s = 2.6kPa. The associated rain load is specified as S_r = 0.2kPa across Manitoba.

The basic roof snow load factor will be taken as C_b = 0.8, which accounts for the fact that the amount of snow on roofs is generally less than the ground snow load.

The wind exposure factor C_w can be reduced to 0.75 for exposed areas beyond the tree line; however, the worst case will be considered where the building is not fully exposed to wind. Therefore, $C_w = 1.0$

The slope factor, C_s, depends on the roof slope and the roughness of the roof. The roof panels for the specific house will have a shingles finish and will not be considered as slippery, a situation which is conservative, since the Ambiente panel surface is such that will allow snow to slide off the roof. Furthermore, the roof slope is 1:3, which implies an angle of 18.4^0. According to clause 4.1.7.1(4) of the NBCC, since this angle is less than 30^0 the slope factor shall be taken as $C_s = 1.0$

Finally, the accumulation factor, C_a, will be equal to 1.0 for case I loading as described in Commentary H of NBCC. Case II loading, where half of the snow load is applied on one side, cannot possibly produce a worse case since the roof is rigidly supported in the middle by the interior wall and the horizontal pultruded beam; as a result, Case II loading will not be considered. Therefore the specified snow load can be calculated from Equation 5.7 as:

$S = 2.6 \times (0.8 \times 1.0 \times 1.0 \times 1.0) + 0.2 = 2.3kPa$ (Eq. 5.8)

5.4.3.3. Wind Load

The specified internal pressure or suction due to wind is given by clause 4.1.8.1 of the NBCC to be:

$p = qC_eC_gC_p$ (Eq. 5.9)

For internal pressure, a similar relationship is specified as:

$p_i = qC_eC_gC_{pi}$ (Eq. 5.10)

The reference velocity pressure, q, is typically taken as the hourly 10-year wind for secondary members and as the 30-year wind for primary members. The worst 30-year and 10-year hourly wind that can occur in Manitoba is 0.59 and 0.48kPa respectively which corresponds to conditions in Churchill, Manitoba. (Appendix C, NBCC). Since the panels are an integral part of the load carrying mechanism of the structure, they will be considered as a primary member.

5.4.3.3.1. Internal Pressure

The exposure factor, C_g can be obtained by Table 4.1.8.1 of NBCC. Considering the average mid-height of the building to be 1.5m, the specified value for the exposure factor is C_g=0.9.

Assuming a Category 2 building under the provisions of section 37, commentary B of the NBCC, the internal pressure coefficient, C_{pi}, can take the values -0.7 to -.0.7, while the gust factor, C_g, is specified to be 1.0. Therefore, the internal pressure can be calculated as:

$p_i = 0.59 \times 0.9 \times 1.0 \times (\pm 0.7) = \pm 0.37 kPa$ (Eq. 5.11)

In the above equation, the positive value indicates pressure, while the negative indicates suction.

5.4.3.3.2. Roof Panels

The average height of the roof is roughly 3m. Therefore, according to clause 4.1.8.1(5) of NBCC, the exposure factor C_e shall be taken as 0.9.

The gust effect factor, C_g, and the coefficient of external pressure C_p, are provided as a combined value in Figure B-10, commentary B of NBCC. The tributary area will be considered as the area of one roof panel, which is $A=4.7m^2$. The coefficient product, C_eC_g, is given for different areas of the roof, as shown in Figure 5.1. Based on these coefficients and the 30-year reference pressure of 0.58kPa, the exterior pressure acting on various locations of the roof panels was calculated and summarised in Table 5.1

The total effect of wind due to internal and external pressure can be obtained by combining internal suction with external pressure and internal pressure with external suction. Combinations, for which the internal pressure is counter-acting to the external pressure, shall not be considered.

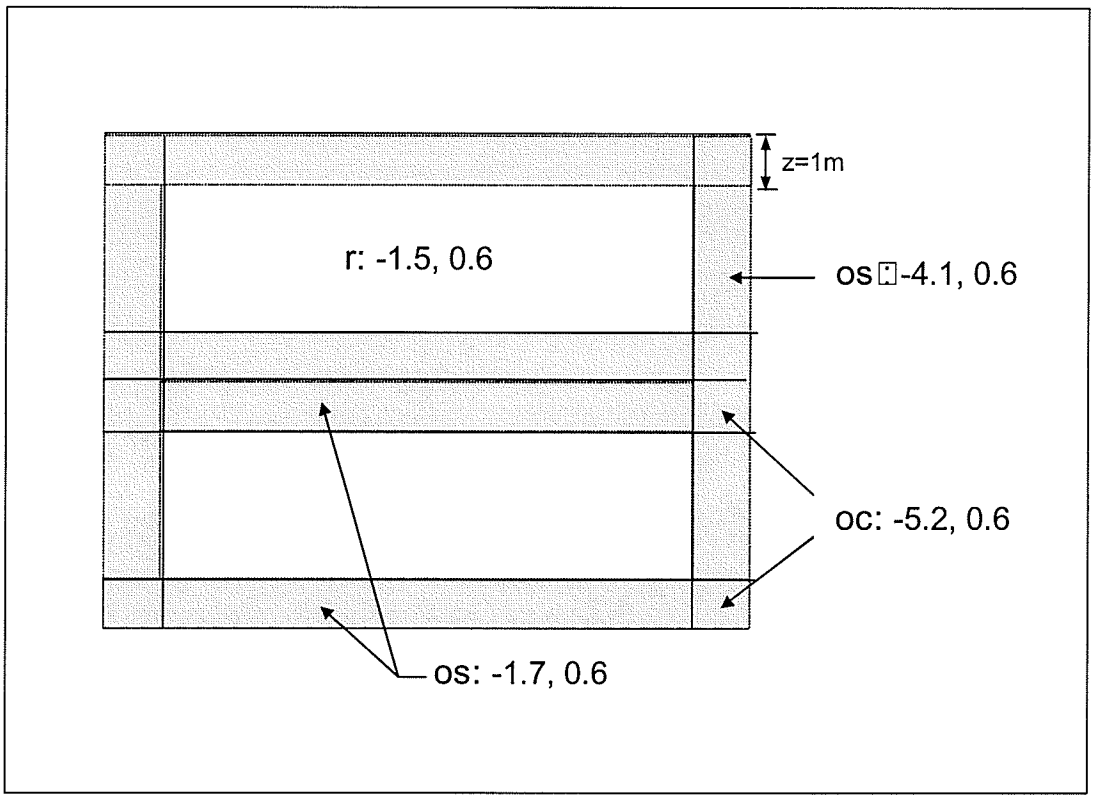


Figure 5.1: External Peak Pressure Coefficients, C_pC_g, for roof with overhangs.

Table 5.1: Pressure on Roof Panels due to External Wind Effects

Location	Positive, kPa	Uplift, kPa
Internal (r)	0.32	-0.80
Eaves, long side (os)	0.32	-0.90
Eaves, short side (os)	0.32	-2.18
Corners (oc)	0.32	-2.76

Table 5.2: Pressure on Roof Panels due to Internal and External Wind Effects

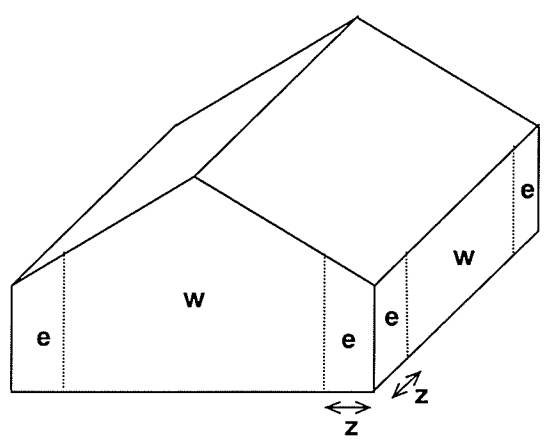
Location	Positive, kPa	Uplift, kPa
Internal (r)	0.69	-1.17
Eaves, long side (os)	0.69*	-1.27*
Eaves, short side (os)	0.69*	-2.55*
Corners (oc)	0.69*	-3.13*

* These values are conservative since internal pressure does not apply on the 0.3m overhang portion of each panel

5.4.3.3.3. Wall Panels

The exposure factor C_e shall be taken as 0.9, for a corresponding reference height of 3m, according to clause 4.1.8.1(5) of NBCC.

The gust effect factor, C_g, and the coefficient of external pressure C_p, are provided as a combined value in Figure B-8 of the Commentary B of NBCC. Figure 5.2 shows the coefficient product, C_gC_p, based on the tributary area of the side wall panels which is $A=2.9m^2$. The area of the gable-end panels is, on average, 5% larger which makes no significant difference in obtaining C_gC_p. For a 30-year reference pressure of 0.58kPa, the combined effect of internal and external pressure on the two specified zones for the wall panels is given in Table 5.3



Zone	CpCg
w	1.7, - 1.7
e	1.7, -1.9

*z = 1m

Figure 5.2: Exteral Peak Pressure Coefficients, C_pC_g, for wall panels.

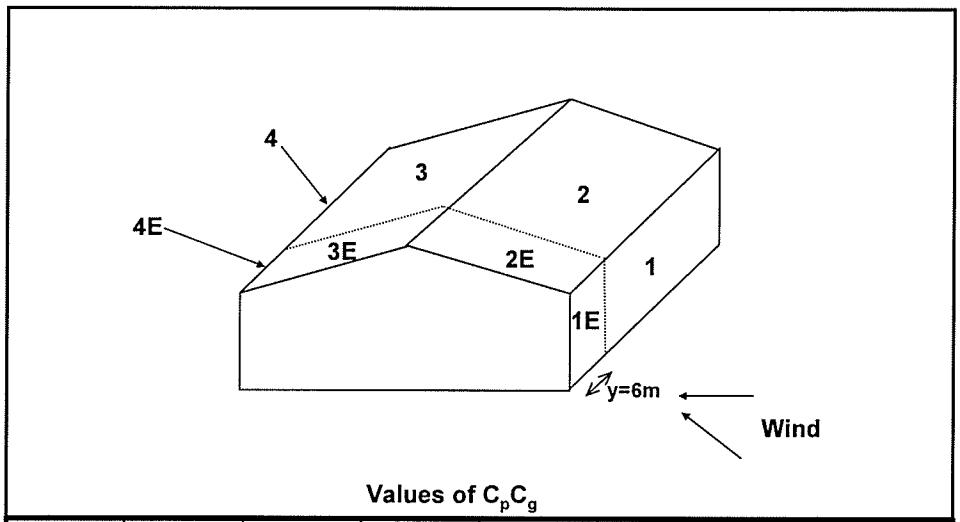
Table 5.3: Pressure on Wall Panels due to Internal and External Wind Effects

	Interior Pressure, kPa		Exterior Pressure, kPa		Combined Effect, kPa	
Location	Positive	Negative	Positive	Negative	Positive	Negative
w	0.37	-0.37	0.89	-0.89	1.26	-1.26
e	0.37	-0.37	0.89	-0.99	1.26	-1.36

Note: positive pressure is acting against the external face, while negative is acting away from it

5.4.3.3.4. Overall Stability

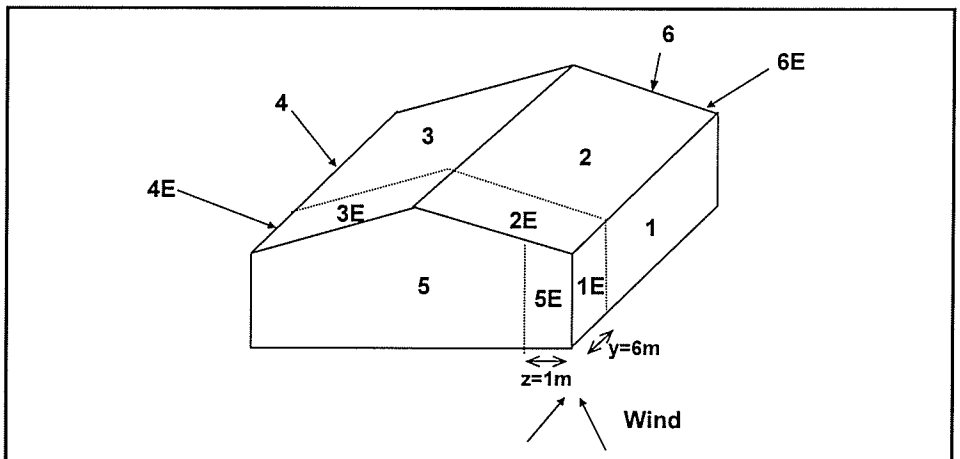
Figure B-7 provides the peak pressure coefficients to be considered when designing the building for overall stability. The applicable coefficients are shown in Figure 5.3 for a wind generally perpendicular to the 12.5m-side and Figure 5.4 shows the coefficients applicable for a wind generally perpendicular to the 7.8m-side. The z-end zone is 1m, yet the y-end zone is the greater of 6m or 2z, which implies a y-end zone of 6m from each corner. For a conservative design, the y-zone will be considered to cover the whole 12.5m-side of the building. Therefore, the zones 1, 2, 3 and 4 of Figure B-7 are assumed to be subjected to the higher pressures associated with zones 1E, 2E, 3E and 4E. Based on these coefficients, the applied wind pressures in different zones of the roof and the walls, when considering overall stability, is shown in Table 5.4.



Values of C_pC_g

1	1E	2	2E	3	3E	4	4E
n/a	1.5	n/a	-2.0	n/a	-1.3	n/a	-1.2

Figure 5.3: External Peak Pressure Coefficients (Wind Case A)



Values of C_pC_g

1	1E	2	2E	3	3E	4	4E	5	5E	6	6E
n/a	-0.9	n/a	-2.0	n/a	-1.0	n/a	-0.9	0.75	1.15	-0.55	-0.8

Figure 5.4: External Peak wind Coefficients (Wind Case B)

Table 5.4: Summary of Pressure on Building Faces for Overall Stability Design

Wind Case	Building Face	Interior Zone, kPa	End Zone, kPa
A	Windward Side	n/a	0.80
	Leeward Side	n/a	-0.64
	Windward Roof	n/a	-1.06
	Leeward Roof	n/a	-0.69
B	Windward Side	n/a	-0.48
	Leeward Side	n/a	-0.48
	Windward Gable	0.40	0.61
	Leeward Gable	-0.29	-0.42
	Windward Roof	n/a	-1.06
	Leeward Roof	n/a	-0.53

5.4.4. STRENGTH AND SERVICEABILITY CHECKS

The concept of limit states design will be adopted in checking the resistance of the structure, except of the foundation, and its components subjected to dead and live loads, as calculated in sections 5.4.2 and 5.4.3. In order to assure structural adequacy and stability, the effect of factored loads and their combined action shall be greater than the factored resistance of the component being checked. (Clause 4.1.3.2 of NBCC). Thus,

$\varphi R \geq a_D D + \gamma\psi[a_L L + a_W W + a_T T]$ (Eq. 5.12)

where D is the total acting dead loads, L the total acting live loads due to static and inertia forces including occupancy and snow loads, W is the live load due to wind and T the load caused due to temperature or moisture changes, creep or movement due to differential settlement. The load factors are specified to have the values a_D=1.25, a_L=1.5, a_W=1.5 and a_T=1.25. The importance factor, γ, shall be taken as unity, a typical value for homes and other non post-disaster

buildings. The combination factor, ψ, can take the values 1.0, 0.7 and 0.60 if one, two or all of the loads L, W and T are considered respectively.

The load factors shall not be considered when checking for serviceability limits, as indicated in Clause 4.1.3.3.

When checking against overturning, reversal or uplift, the dead load factor should take the value 0.85

A material resistance factor of 0.6 will be used for the Ambiente wall panels, and 0.7 for the tensile strength of the cables. A factor of 0.6 will also be used for the floor joist, as this factor is typical when designing fiberglass reinforcement in structures. A factor of 0.8 will be used for the post-tensioning force in the cables when resisting loads and 1.2 when being considered as an applied load, as specified by the NBCC.

5.4.4.1. Roof Panels

5.4.4.1.1. Uplift Resistance

The worst-case uplift force due to wind was calculated in section 5.4.3.3.2 to be -2.55kPa acting within 1m from the gable end, as well as a 1m by 1m area at the corners which can experience a -3.13kPa uplift. Each end panel is 1.2 m wide, yet it can be conservatively assumed that the uplift force is being acted on the whole panel, minus the 1m by 1m zone. Given that each panel is 4.3m by 1.2 m, the worst case factored uplift force, U_f, for the gable-end panels can be calculated as:

$$U_f = 1.5 \times [2.55kPa \times (4.3m \times 1.2m - 1m \times 1m) + 3.13kPa \times (1 \times 1)] = 20.1kN \quad \text{(Eq. 5.13)}$$

The factored resisting force due to the self-weight of the panel alone is:

$SW_f = 0.85 \times [0.7kPA \times (4.3m \times 1.2m)] = 3.1kN$ (Eq. 5.14)

where 0.7kPa equals the dead load due to the self weight of a roof panel, as calculated in section 5.4.2.1.

The remaining 20.1-3.1=17.0kN must be resisted by the vertical cables that tie down the roof panels. According to the test results in section 4.3, the ultimate load capacity of the cable is 29.3kN. Vertical cables exist along each vertical wall panel. Therefore, the uplift action on each roof panel is resisted by two vertical cables, with a total factored resistance of 0.7x(2x29.3kN)=40.6kN, which well-exceeds the net uplift of 17.0kN.

5.4.4.1.2. Bending Resistance

The worst case bending condition would apply to the internal roof panels, since they are only supported at two ends. According to section 5.4.3.3.2, the maximum pressure due to wind is 0.69 for positive pressure and -1.17 and -1.27 kPa for uplift pressure at the internal zone and the eaves respectively. The maximum pressure due to snow load was calculated in section 5.4.3.2 to be 2.30kPa. When considering only snow load, the total factored load, $q_{f(D+L)}$, on the roof can be calculated based on Equation 5.12 as:

$q_{f(D+L)} = 1.25 \times 0.7kPa + 1.0 \times (1.5 \times 2.3kPa) = 4.3kPa$ (Eq. 5.15)

When considering both snow and wind loading, the factored load becomes:

$q_{f(D+L+W)} = 1.25 \times 0.7kPa + 0.7 \times (1.5 \times 2.3kPa + 1.5 \times 0.69kPa) = 4.0kPa$ (Eq. 5.16)

In the case of uplift forces acting alone, assuming conservatively that the eave pressure is acting along the whole panel, the factored load can be calculated as:

$$q_{f(D+W)} = 0.85 \times 0.7kPa + 1.0 \times [1.5 \times (-1.27kPa)] = -1.3kPa \quad \text{(Eq. 5.17)}$$

A comparison of the above cases shows that the governing one is when the snow load is acting alone producing a load $q_{f(D+L)}$=4.3kPa. For a nominal width of b=305mm (1ft) the equivalent bending load is:

$$w_{f(D+L)} = q \times b = 4.3kPa \times 0.305m = 1.3kN/m \quad \text{(Eq. 5.18)}$$

Assuming the roof panels are simply supported, for a clear roof panel span of L=3.7m the maximum bending moment can then be calculated as:

$$M_{f(D+L)} = \frac{w_{f(D+L)}L^2}{8} = \frac{1.3kN/m \times (3.7m)^2}{8} = 2.2kN \cdot m \quad \text{(Eq. 5.19)}$$

According to results from a series of 4-point bending tests conducted on 1ft-wide roof panels by the University of Puerto Rico (Lopez, 2000), the average ultimate bending capacity of the roof panel was calculated to be 12.2kNm. The factored moment resistance can be calculated as:

$$M_{r(Roof)} = 0.6 \times 12.2 = 7.3kN \cdot m > M_{f(D+L)} = 2.2kN \cdot m \quad \text{(Eq. 5.20)}$$

Therefore, the roof panels are structurally adequate.

5.4.4.1.3. Deflection

The unfactored pressure acting on a roof panel is:

$$q = 0.7 + 2.3 = 3.0kPa \quad \text{(Eq. 5.21)}$$

Combining equations 6.18, 6.19 and 6.21, the unfactored moment can be found as:

$$M_{(D+L)} = \frac{(3.0kPa \times 0.305m) \times (3.7m)^2}{8} = 1.6kN \cdot m \quad \text{(Eq. 5.22)}$$

According to results from the 4-point bending tests (Lopez, 2000), a moment of 1.6kNm would cause a deflection of about 4mm. The suggested deflection limit (Commentary A, NBCC) for roof members not supporting plaster is L/180, where L, the unsupported span, in particular:

$$\Delta_{\max(Roof)} = 3700mm/180 = 20mm > 4mm \quad \text{(Eq. 5.23)}$$

Therefore, the roof panels meet deflection criteria against the specified loads.

5.4.4.2. Wall Panels

5.4.4.2.1. Compression

According to calculations in Section 5.4.4.1.2, the worst-case factored roof load is given by Equation 5.15 to be 4.3kPa. Given that the area of the roof panel is A=4.7m^2, the total factored roof force, $N_{f(D+L)}$, supported by the exterior and interior wall is:

$$N_{f(D+L)} = 4.3kN/m^2 \times 4.7m^2 = 20.2kN \quad \text{(Eq. 5.24)}$$

Assuming that this force will be equally shared by the interior and exterior wall panels, the total factored compressive force on each wall panel is $C_{f(D+L)}$=20.2/2=10.1kPa. The assumed tensile force P=13.3kN in the post-tensioning cables, will be treated as a dead load, as provided in clause 4.1.5 of the NBCC. Therefore, the total factored compressive force in the wall panel is:

$C_{f(D+L+P)} = 10.1kN + 1.25 \times 13.3kN = 26.7kN$ (Eq. 5.25)

Results from tests performed by the University of Puerto Rico (Lopez, 2000) on 2.4m-panels, yielded an average compressive strength of 58.6kN. Thus, the factored resistance of the wall panel is:

$C_r = 0.6 \times 58.6kN = 35.2kN > 26.7kN$ (Eq. 5.26)

The above calculation does not take into consideration any resistance contributions from the 2in x 2in pultruded section that exists along every vertical panel joint. The mode of failure for the tested panels was local buckling of the panel skin and not lateral buckling due to slenderness effects. Thus, it is anticipated that the interior wall panel, which has a length of 3.7m, will not produce a slenderness effect great enough to reduce the compressive resistance below 26.7kN. Therefore, it can be concluded that the wall panels are adequate against the factored compressive loads.

5.4.4.2.2. Out-of-Plane Bending

The worst-case lateral bending will occur at zone w (Figure 5.2), since the end panels are vertically supported by the pultruded corner connections. Table 5.3 shows that the specified wind loading is 1.26kPa, both for positive and negative pressure. The factored lateral pressure is computed as:

$q_{f(W)} = 1.5 \times 1.26 = 1.9kPa$ (Eq. 5.27)

The critical areas will be considered where a door or window opening exists. In this case, the adjacent panels are expected to carry the lateral loads. Therefore,

a tributary area consisting of a whole panel plus half of a panel with a door or window opening was considered. The pultruded section between the panels was not considered in these calculations. Thus, for a tributary width of 1.2+1.2/2=1.8m, the factored bending load is:

$w_{f(W)} = 1.8m \times 1.9kN/m^2 = 3.4kN/m$ (Eq. 5.28)

For a panel height of 2.4m, this corresponds to a factored bending moment of:

$M_{f(W)} = \frac{3.4kN/m \times (2.4m)^2}{8} = 2.5kN \cdot m$ (Eq. 5.29)

Using results from 4-point bending tests on wall panels (Lopez, 2000) the average bending moment resistance was calculated to be 14.22kNm, which yields a factored resistance of:

$M_{r(Wall)} = 0.6 \times 14.2 = 8.5kN \cdot m > M_{f(W)} = 2.5kN \cdot m$ (Eq. 5.30)

Therefore, the wall panels have an adequate bending resistance under the lateral wind loads.

5.4.4.2.3. Deflection of Panels

For an unfactored lateral wind pressure of 1.26kPa, the resulting bending moment can be computed as:

$M_{(W)} = \frac{(1.26kPa \times 1.8m) \times (2.4m)^2}{8} = 1.6kN \cdot m$ (Eq. 5.31)

Test results (Lopez, 2000) suggest that the above moment will yield a lateral deflection of less than 3mm. The suggested deflection limit (Commentary A, NBCC) for wall members is L/360, which yields:

$\Delta_{\max(Wall)} = 2400mm/360 = 7mm > 3mm$ (Eq. 5.32)

Therefore, the wall panels meet the serviceability deflection criterion.

5.4.4.3. Ridge Beam

The Ambiente house, typically, has an internal partition wall which supports the roof ridge beam. The proposed Smart House plan includes an open area between the living room and the kitchen, with no internal wall in the middle. This leaves a length of 6.1m for which the roof panels are supported only by the pultruded ridge beam (Figure 2.13). The total unfactored load required to be supported by the ridge beam is 12.3kN/m, which includes both the worst case roof load and the dead load of the roof panels. The deflection, Δ, for a simply supported beam of length l, under a continuous load w can be calculated by the following expression:

$\Delta = \frac{5wl^4}{384EI}$ (Eq. 5.33)

where, E and I are the modulus of elasticity and moment of inertia of the pultruded beam respectively.

The modulus of elasticity for pultruded sections manufactured by Bedford Reinforced Plastics is 19.3Gpa (Bedford, 2003). For a maximum allowable deflection limit of L/180 or 6100/180=34mm, the required moment of inertia, I_{req}, can be calculated as:

$I_{req} = \frac{5wl^4}{384E\Delta_{\max}} = \frac{5 \times 12.3kN/m \times (6.1m)^4}{384 \times 19.3 \times 10^6 \times 0.034m} = 3.38 \times 10^4 m^4$ (Eq. 5.34)

The ridge beam currently used has a moment of inertia of $0.36 \times 10^4 m^4$, which is not sufficient. Figure 5.5 shows a proposed beam, which shall be continuously bonded to the bottom of the ridge beam in order to provide additional stiffness. The total moment of inertia of the combined beams is $5.97 \times 10^4 m^4$, which satisfies maximum defection criteria.

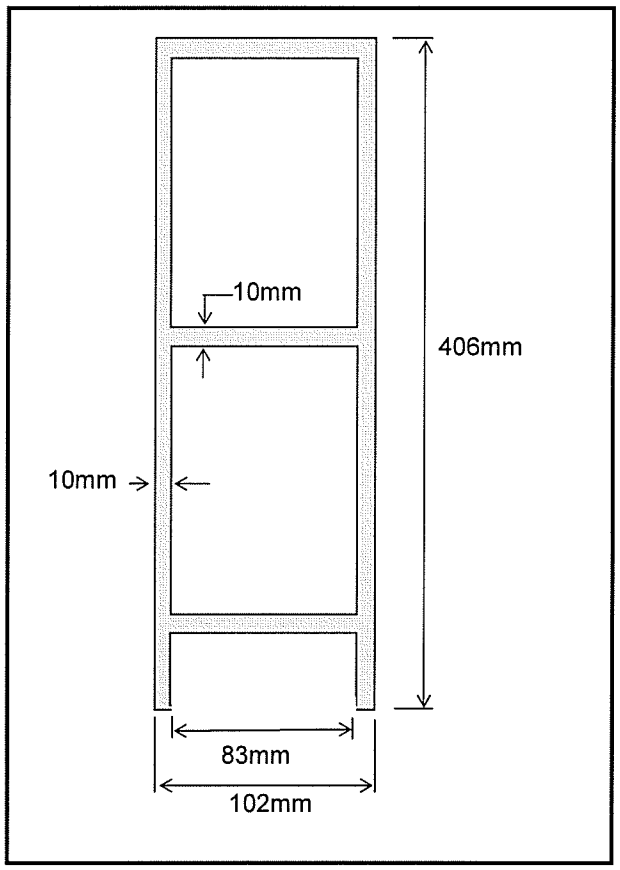


Figure 5.5: Proposed Ridge Supporting Beam

5.4.4.4. Floor

5.4.4.4.1. Bending

The dead load due to the weight of the floor joists and the mechanical systems was calculated in Section 5.4.2.3 to be 0.4kN/m. According to NBCC,

Clause 4.1.6.1, the worst-case floor load for residential areas is 4.8kPa which corresponds to storage areas and will be considered applicable to the utility room of the house. The specified live load requirement for the rest of the areas in the house is 1.9kPa. For a floor joist spacing of 610mm, the specified bending load for areas excluding storage becomes:

$w_{L(floor)} = 1.9kN/m^2 \times 0.559m = 1.1kN/m$ (Eq. 5.35)

According to Equation 5.12, the total factored load can then be computed as:

$w_{f(floor)} = 1.25 \times 0.4 + 1.5 \times 1.1 = 2.2kPa$ (Eq. 5.36)

For a floor joist span of 3.81m(12.5 feet) the above bending load will cause a maximum bending moment of:

$M_{f(floor)} = \frac{2.2kPa \times (3.81)^2}{8} = 4.0kN \cdot m$ (Eq. 5.37)

Results from the floor joist bending test (Section 4.1) indicate that the bending moment capacity for a single joist is 10.8kN.m, which yields a factored resistance of:

$M_{r(floor)} = 0.6 \times 10.8kN \cdot m = 6.5kN \cdot m > M_{f(floor)} = 4.0kN \cdot m$ (Eq. 5.38)

Therefore, a joist spacing of 559mm is structurally adequate for all areas excluding the utility room.

The specified live load for the utility room is 4.8kPa, which yields a bending load of:

$w_{L(storage)} = 4.8kN/m^2 \times 0.559m = 2.7kN/m$ (Eq. 5.39)

According to Equation 5.12, the total factored load can then be computed as:

$w_{f(storage)} = 1.25 \times 0.4 + 1.5 \times 2.7 = 4.6kPa$ (Eq. 5.40)

For a floor joist span of 3.81m(12.5 feet) the above bending load will cause a maximum bending moment of:

$$M_{f(storage)} = \frac{4.6kPa \times (3.81)^2}{8} = 8.3kN \cdot m > M_{r(floor)} = 6.5kN \cdot m \text{ (Eq. 5.41)}$$

Therefore the 559mm joist spacing is not sufficient. In order to meet strength criteria, it is suggested to refine the joist spacing under the utility room. A spacing of 280mm will increase the factored resistance to 13.0kN.m which will provide a structurally adequate design.

5.4.4.4.2. Deflection

The unfactored specified load for the floor is w_{floor}=0.4+1.1=1.5kN/m, which corresponds to a maximum bending moment of:

$$M_{(floor)} = \frac{1.5 \times (3.81)^2}{8} = 2.7kN \cdot m \text{ (Eq. 5.42)}$$

Using Figure 4.1, for 2.7kN.m on each joist would produce a total moment of 5.4kN.m, which would produce a deflection of less than 6mm. According to Commentary A of the NBCC, the maximum allowable deflection for a span of 3.81m would be:

$$\Delta_{\max(floor)} = L/360 = 3810/360 = 10.6mm > 6mm \text{ (Eq. 5.43)}$$

The unfactored specified load for the utility room, assuming a joist spacing of 280mm is $w_{storage}$=(0.1+4.8)x0.28 =1.4 kN/m, which will also produce a deflection that is less than the maximum allowable. Therefore, a joist spacing of 280mm

under the utility room and 559mm for the rest of the areas will provide a design that meets deflection criteria.

5.4.4.5. Roof Diaphragm

The worst case loading for the roof diaphragm (Figure 5.6) will occur when the wind is acting perpendicular to the longer side, identified as Case A in Section 5.4.3.3.4 (Figure 5.3). As Table 5.4 suggests, the total effect of wind action on the windward and leeward sides will produce a uniform pressure of 0.80-(-0.64) =1.44kPa. Assuming that half of this pressure will be resisted by the roof diaphragm, for a wall height of 2.4m, the factored uniformly distributed load can be calculated as:

$$w_{rd} = \frac{2.4m}{2} \times 1.44kPa = 1.7kN/m \Rightarrow w_{f(rd)} = 1.5 \times 1.7 = 2.6kN/m \quad \text{(Eq. 5.44)}$$

Assuming that the roof behaves like a deep beam of length L= 12.5m which is restrained at the ends by the gable ends (Figure 5.6), the total shear force acting on each gable can be calculated as:

$$U_f = \frac{w_{f(rd)} \times L}{2} = \frac{2.6kN/m \times 12.5m}{2} = 16.3kN \quad \text{(Eq. 5.45)}$$

Since the roof of the specific design has 10 panels on either side of the ridge, there is a total of eight shear interfaces between the roof panels. Therefore, each shear interface will need to resist on average a load of u_f=16.3/8=2.0kN.

Results from tests done on determining the shear capacity between panel interfaces with respect to cable tension (Section 3.5), showed that the ratio of ultimate shear to cable tension is equal to 0.56. There are a total of 4 lines of

cable running parallel to the long side of the building, with a recommended tension value of 13.3kN in each. Therefore a total force of 13.3x4=53kN will yield an ultimate factored shear resistance of:

$u_r = 0.6 \times 53kN \times 0.56 = 17.8kN > u_f = 2.0kN$ (Eq. 5.46)

Assuming that the roof is acting as a beam under in-plane bending, the factored moment can be calculated as:

$M_f = \frac{w_{f(rd)}L^2}{8} = \frac{2.6 \times 12.5^2}{8} = 50.8kN \cdot m$ (Eq. 5.47)

The neutral axis of bending is forced to be along the ridge since the panels on either side end at the ridge. Assuming a linear strain distribution in the cables, using similar triangles (Figure 5.6) the following can be derived:

$\frac{T_1}{4.2} = \frac{T_2}{8.4} \Rightarrow T_2 = 2T_1$ (Eq. 5.48)

Taking moments about the centroid of the compression zone yields:

$M_f = T_1 d_1 + T_2 d_2$ (Eq. 5.49)

where d_1 and d_2 are distances from the each cable to the centroid of the compression zone, which is located at a distance of 1/3 from the top. Combining Equations 5.48 and 5.49, T_2 can be calculated as:

$M_f = 2T_2 d_1 + T_2 d_2 \Rightarrow 50.8 = 2T_2 \times 3.6 + T_2 \times 4.8 \Rightarrow T_2 = 4.2kN$ (Eq. 5.50)

The net factored load on the cable includes the post-tensioning force, therefore:

$T_{f(net)} = T_2 + T_{f(p)} = 4.2kN + 1.2 \times 13.3kN = 20.2kN$ (Eq. 5.51)

Test results (Section 4.4) indicate a tensile strength of 29.3kN, therefore the factored resistance of a single cable is:

$T_r = 0.7 \times 29.3 = 20.5kN \geq T_{f(net)} = 20.2kN$ (Eq. 5.52)

It can be concluded that the roof panels tied with a post-tension force of 13.3kN per cable can provide a structurally adequate diaphragm in resisting applied wind loading.

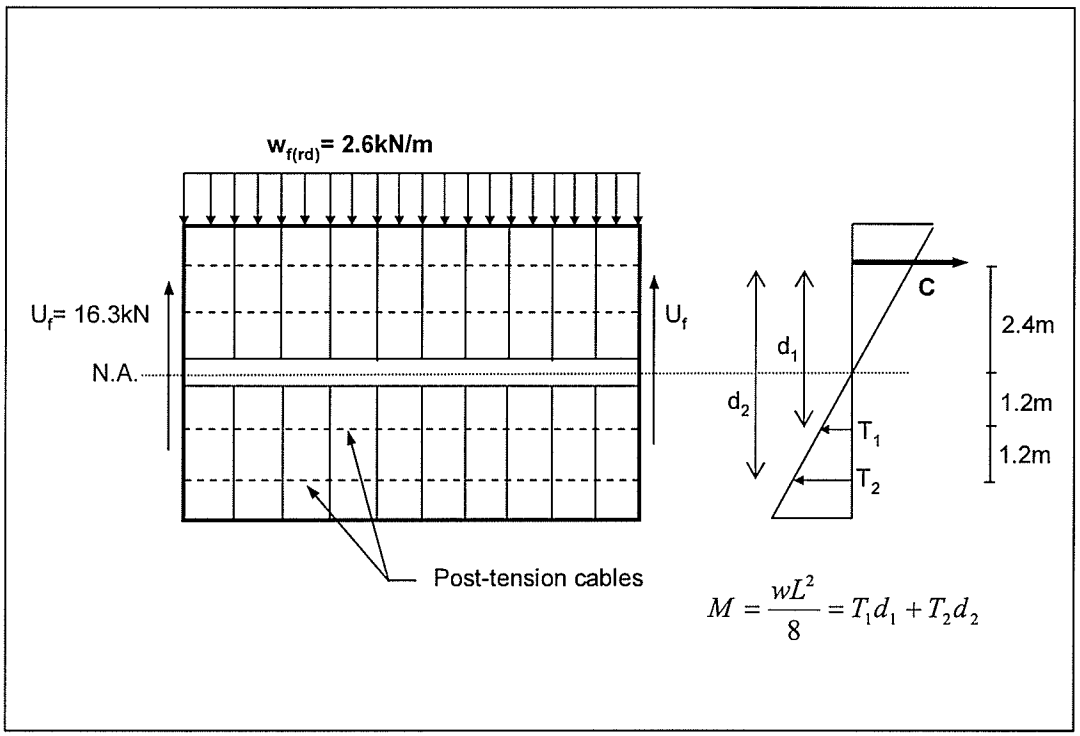


Figure 5.6: Beam Action Assumption for Roof Diaphragm

5.4.4.6. Wall Diaphragm

As discussed in the previous section, the worst-case force will act on the gable ends. The total factored load that will be resisted by the wall diaphragm was calculated by Equation 5.45 to be U_f=16.3kN. Test results in Section 4.3 indicate an ultimate shear wall resistance of 39.5kN. Since failure initiated due to

local buckling, it cannot be assumed that a wall of more than two panels will have a greater ultimate resistance. The factored resistance can be calculated as:

$U_r = 0.8 \times 39.5kN = 31.6kN > U_f = 16.3kN$ (Eq. 5.53)

The unfactored wind load of w_{rd}=1.7kN/m along the roof diaphragm will produce a load that will be resisted by the gable end walls with a value of:

$U = \frac{1.7kN/m \times 12.5m}{2} = 10.6kN$ (Eq. 5.54)

Test results obtained from the assembly of two panels indicate that a lateral force of 10.6kN will produce a lateral distortion of about 7mm (Figure 4.3). The maximum allowable deflection, as recommended by Commentary A of the NBCC, is h/180. For an average gable end height of 4013mm, this corresponds to a maximum allowable deflection of 22mm. Since the gable end walls actually consist of 6 panels, it can be concluded that the minimum deflection criteria are well-satisfied.

5.4.4.7. Foundation

Foundations in Canada are traditionally being designed using the concept of working stress design. Under this concept the resistance of the foundation against bearing failure is checked under unfactored loads taking into consideration an acceptable factor of safety. The Canadian Foundation Engineering Manual (CFEM) suggests using a factor of safety of 3.0 (CFEM, 1985). Details of the foundation for the Smart House are shown in Appendix A.

5.4.4.7.1. End Footings

The dead load due to the weight of the panels was calculated in Equation 5.3 to be 15.3kN/m. The total worst-case unfactored roof load is 3.0kPa, or 12.3kN/m for a roof panel width of 4.115m. Assuming that half of the roof load is equally shared by the interior and exterior footings, the total load being transferred through the end walls is:

$$l_w = 15.3 + \frac{12.3}{2} = 21.5kN/m \quad \text{(Eq. 5.55)}$$

The total unfactored load acting from the floor is 1.5kPa over a half-span of 1.9m or:

$$l_f = 1.5kPa \times 1.9m = 2.9kN/m \quad \text{(Eq. 5.56)}$$

The loads l_w and l_f act eccentrically at distances of 76mm and 44mm respectively form the center of he footing. The maximum pressure below the footing under an eccentric load is shown in Figure 5.7. For a footing width of 0.61m, the maximum pressure per meter of footing due to l_w and l_f respectively, are:

$$q_{lw} = \frac{21.5}{0.61 \times 1}(1 \pm \frac{6 \times 0.076}{1}) = 51.3 and 19.2kPa \quad \text{(Eq. 5.57)}$$

and

$$q_{lf} = \frac{2.9}{0.61 \times 1}(1 \pm \frac{6 \times 0.044}{1}) = 6.0 and 3.5kPa \quad \text{(Eq. 5.58)}$$

Superimposing both loads will give the pressure distribution shown in Figure 5.8. Conservatively, the maximum pressure of 54.7kN/m will be assumed to act uniformly across the whole footing. Assuming a concrete density of 23.5kN/m^3,

the weight of the footing will be 3.6kN/m giving a total design load of 58.3kN/m. The soil strength can be conservatively calculated using Skempton's theory of bearing capacity (Craig, 1997) as follows:

$q_{net\,safe} = c_u N_c + \gamma D$ (Eq. 5.59)

where c_u is the undrained shear strength of soil, N_c is a factor that depends on the shape of the footing and γ is the unit weight of the soil displaced at a depth D. For a strip footing at a depth of 0.25m and width of 0.61m, N_c has a value of 5.7 (Craig, 1997). Assuming a soil unit weight of 21kN/m^3 and an undrained strength of c_u=50kPa, which is the lowest value that can be obtained for firm clays (NBCC, Commentary L), the allowable pressure can be calculated as:

$q_{net\,safe} = 50 \times 5.7 + 21 \times 0.254 = 290.3kPa$ (Eq. 5.60)

Therefore, the factor of safety can be computed as:

$F.S. = 290.3/58.3 = 5.0$ (Eq. 5.61)

which is acceptable.

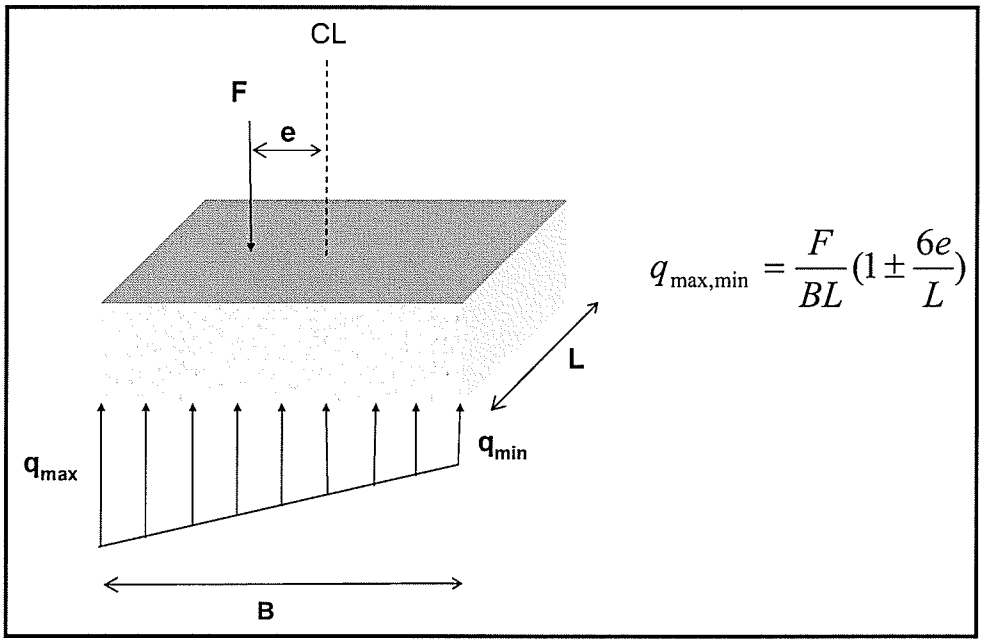


Figure 5.7: Eccentric Loading on Footings (Craig, 1997)

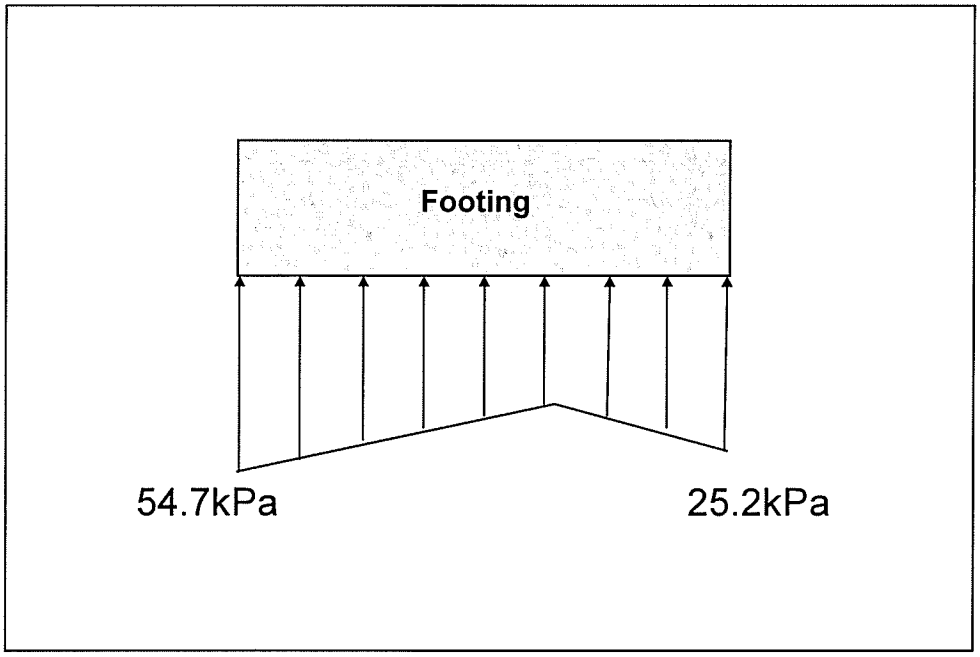


Figure 5.8: Pressure Distribution Under End Footing

5.4.4.7.2. Internal Footing

The internal footing, running along the middle of the house in the longitudinal direction, supports loads from the floor on either side, as well as the internal wall. The dead load of the internal wall was found to be 16.9kN/m (Equation 5.4). Therefore the total unfactored load transferred through the wall is:

$l_w = 16.9 + 12.3 = 19.2kN/m$ (Eq. 5.62)

The total unfactored floor load acting from both sides is:

$l_f = 2 \times (1.5kPa \times 1.9m) = 4.6kN/m$ (Eq. 5.63)

Since all loads are acting symmetrically, the pressure distribution under the footing will be uniform. The total pressure applied over a footing width of 1.22m is:

$q_a = (19.2 + 4.6)/1.22 = 19.5kPa/m$ (Eq. 5.64)

The dead weight of the footing is 7.2kN/m, giving a total design load of 26.7kN/m.

Using Skempton's equation, given that N_c is 5.4 for a 1.22m strip footing in a depth of 0.25m (Craig, 1997), the allowable pressure can be calculated from Equation 5.59 as follows:

$q_{net\,safe} = 50 \times 5.4 + 21 \times 0.254 = 275.3kPa$ (Eq. 5.65)

which yields a factor of safety of:

$F.S = 275.3/26.7 = 10.3$ (Eq. 5.66)

which is also well-acceptable.

5.4.4.7.3. Bearing Resistance of Footing

According to the Clause 10.8 of CSA A23.3-94, the factored bearing resistance of concrete, in MPa, shall be calculated as:

$f_b = 0.85 \times \varphi_c f_c^{'} A_1$ (Eq. 5.67)

where $f_c^{'}$ is the concrete strength and Φ_c the resistance factor for concrete to be taken as 0.6. A_1 in this case would be the bearing area of the wall. For a nominal length of 1m, the bearing area of the wall is 0.152m. Using a specified concrete strength of 35MPa, the bearing resistance can be calculated as:

$f_b = 0.85 \times 0.6 \times 35 \times 0.152 = 2.71 MPa$ (Eq. 5.68)

The worst-case wall load corresponds to the internal wall, having a total of 23.8kN/m. For a nominal width of 1m, this corresponds to a stress of 23.8/0.152=156.6kPa which is well-below the bearing resistance.

6. SUMMARY, CONLUSIONS AND RECOMMENDATIONS

Housing infrastructure is facing tough challenges in many Northern communities of Canada. Overcrowding and rapid deterioration has resulted into poor indoor air quality and poor thermal performance. The Ambiente system has shown a great potential to improve indoor air quality by not supporting mould growth and to provide a structure with higher life expectancy and thermal performance than current construction techniques. The purpose of this project was to evaluate the structural performance of the Ambiente system for use in Canada, especially for the Northern communities. Structural evaluations, based on tests conducted at the University of Puerto Rico (Lopez, 2000) and at the University of Manitoba, were used to design a full scale house with respect to structural requirements of the National Building Code of Canada. Based on test results, observations, and the design of the full scale house, the following conclusions and recommendations have been derived:

- Test results from the fiberglass floor joist manufactured by Rhino Composites showed that such joists are structurally adequate and satisfy current deflection limits.

- Test observations from the panel bending tests indicate that a minimum thickness of 5mm pultruded channel section is required between the floor joists and the pony wall that supports the floor in order to prevent premature bearing failure on the supporting panel.

- Tension tests revealed that when using steel post-tensioning anchors, the cables failed prematurely. Experimental results showed that applying a thin layer of epoxy along the anchorage area can increase the load capacity of the cables to almost the pure tensile strength of the cable. In order to achieve a suggested post-tensioning force of 13.3kN, the epoxy coating is necessary.

- A Smart House was designed on the basis of testing performed at the University of Puerto Rico (Lopez, 2000) and at the University of Manitoba. Based on existing evidence and calculations performed in Section 5.4, it can be concluded that the proposed design satisfies structural requirements of the National Building Code of Canada. In most cases the design was governed by deflection limits and not strength.

- Loads due to temperature effects were not considered in the structural design. It is not expected for temperature to produce significant stresses within the panels, since joints exist at regular intervals. However, the force in the post-tensioning cables may be significantly affected due to a

cumulative thermal expansion or contraction over the length of the wall and the roof. Therefore it is recommended for future research to investigate whether the effect of temperature variations on the post-tensioning force is significant or not.

- A post-tensioning force of 13.3kN is recommended. Changes to this value should be accomplished by design revisions to the uplift resistance of the roof due to wind suction, the compression of the wall panels due to roof loads and the diaphragm resistance of the roof and the walls due to wind action. During construction, a calibrated post-tensioning device should be used that can accurately apply the specified tensile force in order to ensure that the house meets design specifications.

- UV-condensation cycling tests indicated that after a total test time of 288 hours, the tensile strength of the fiberglass skin did not decrease, yet slight discoloration and loss of gloss was observed. The latter phenomenon is not expected to be visible in an actual house, since the exterior would be typically finished with a stucco effect. According to the standard specification for UV-Condensation cycling (ASTM G 53-91), there is no direct way of correlating test time with an equivalent period of natural weathering, unless an extensive study is done in observing variations of natural exposure such as daily sunlight and condensation variations throughout a year, as well as geographical and natural

conditions. Discolouration can be reduced by using commercially available UV inhibitors that can be added in the epoxy during manufacturing of the skin material. A study has shown that a coating of either polyurethane or aluminum oxide can also reduce deterioration (Hassan et. al., 1999).

- Results from freeze-thaw testing have shown that 30 cycles of exposure on the uncoated specimens yielded a weight loss of only 0.33%. In the absence of a classification system for this new material, specifications applying to concrete blocks of earth retaining walls exposed to cold weather were reviewed in order to give a basis of comparison. A special report by the US Army Corps of Engineers (Korhonen et. al., 1997) uses a maximum weight loss of 1.0% as a limit. Given that this limit was imposed for extreme weathering conditions, it can be concluded that the experimentally obtained deterioration of the uncoated specimens is insignificant. Furthermore, figures 4.18 and 4.19 show that it cannot be statistically concluded that the specimen strength deteriorated with an increase in the number of cycles, since a strength range exists (hatched region) which falls within the standard deviation of all groups.

- Evidences from the available experimental results indicate that the Ambiente housing system can provide a structure that satisfies the National Building Code of Canada for any location in Manitoba with respect to structural performance, yet the effect of thermal expansion and

contraction on the post-tensioning force is needed to be investigated. It is also recommended to closely monitor the effect of sunlight, specifically for discoloration on the panel skin. It is also recommended to repeat the tests conducted on the floor, the cables, and the panels in order to gain a higher statistical confidence.

- Taking into consideration other benefits of using the Ambiente system such as reduced skilled labour and construction time, an anticipated long maintenance-free lifespan of the structure, a promising thermal performance, as well as a lower risk of indoor air contamination due to the fact that the panel skin does not support mould growth, it can be concluded that the Ambiente system can provide a promising relief to the housing crisis in the Northern communities of Canada.

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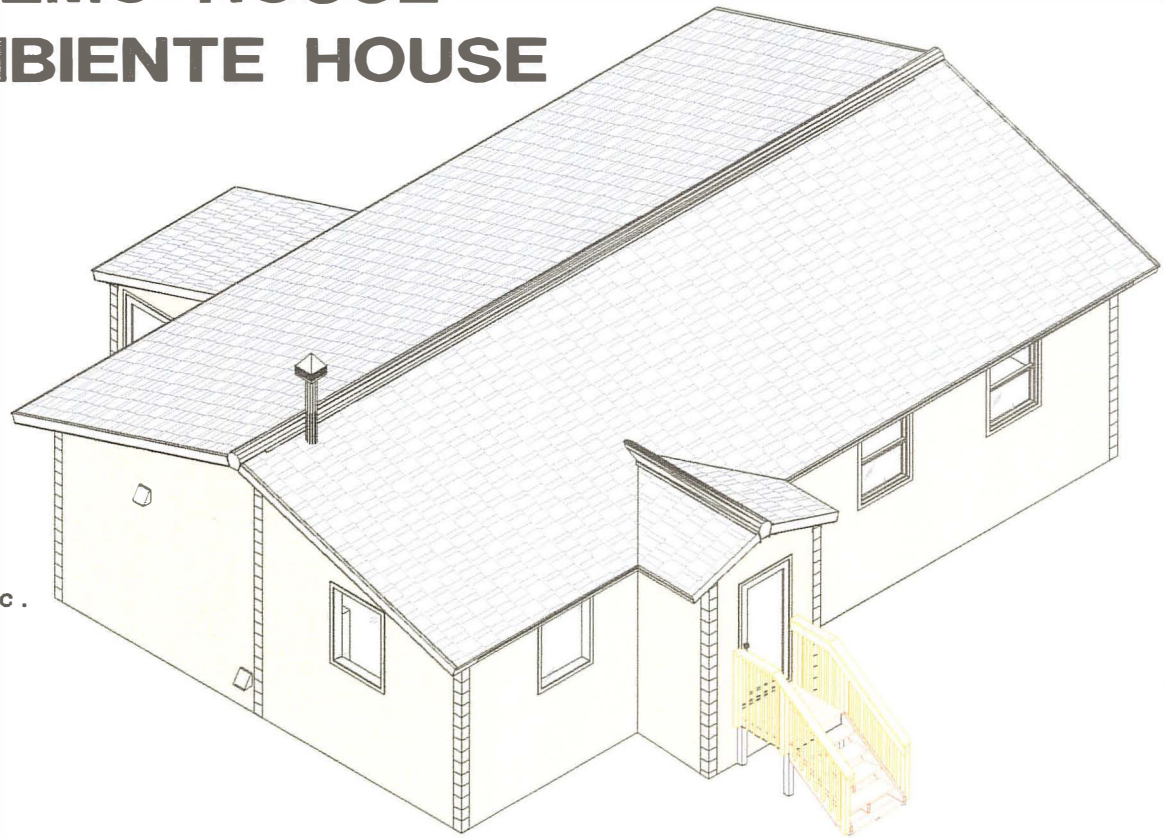
APPENDIX A:

Smart House Drawings (Courtesy Wardrop Engineering Inc.)

SMART PARK DEMO HOUSE
4 BEDROOM AMBIENTE HOUSE



WARDROP | Engineering Inc.

W.E.I. 0328050100-DWG-S3001

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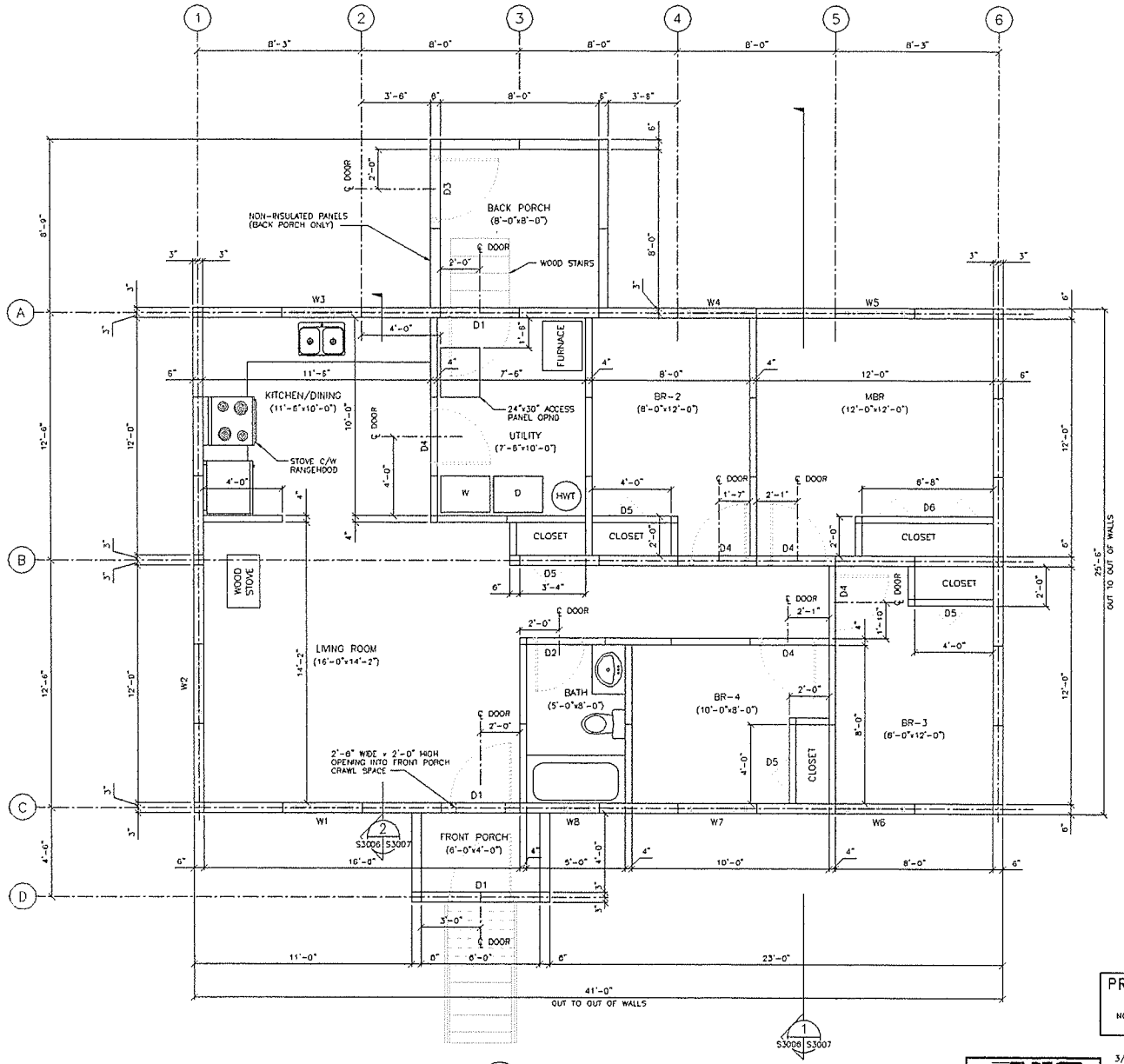

1 FLOOR PLAN
3/8" = 1'-0"

DOORS SCHEDULE						
Mk. No.	QTY	ROUGH OPENING		DOOR SIZES		DESCRIPTION
		WIDTH	HEIGHT	WIDTH	HEIGHT	
D1	3	3'-4"	6'-10"	3'-0"	6'-8"	EXTERIOR
D2	1	2'-10"	6'-10"	2'-8"	6'-8"	INTERIOR
D3	1	3'-4"	6'-10"	3'-0"	6'-8"	STORM
D4	5	3'-0"	6'-10"	2'-8"	6'-8"	INTERIOR
D5	4	2'-8"	6'-10"	2'-4"	6'-8"	BI-FOLD
D6	1	5'-2"	6'-10"	5'-0"	6'-8"	BI-FOLD

WINDOWS SCHEDULE				
Mk. No.	LOCATION	ROUGH OPENING		TYPE
		WIDTH	HEIGHT	
W1	LIVING ROOM	3'-0"	4'-0"	FIXED
W2	LIVING ROOM	3'-0"	4'-0"	FIXED
W3	KITCHEN	3'-0"	3'-0"	SASH
W4	BEDROOM 2	3'-0"	4'-0"	SASH
W5	MASTER BEDROOM	3'-0"	4'-0"	SASH
W6	BEDROOM 3	3'-0"	4'-0"	SASH
W7	BEDROOM 4	3'-0"	4'-0"	SASH
W8	BATH	1'-6"	1'-6"	FIXED

NOTE:
1. WINDOWS TO BE PVC TYPE MEETING THE REQUIREMENTS OF CSA A440. FIXED GLAZING UNITS TO HAVE AN ENERGY RATING (ER) EQUAL TO ZERO. FIXED UNITS WITH OPERABLE SASHES TO HAVE AN ER EQUAL TO -10.

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3/8" = 1'-0"

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NO.	DESCRIPTION	DATE	BY

REVISIONS/ISSUE

CLIENT

WARDROP | Engineering Inc.

PROJECT NAME
SMART PARK DEMO HOUSE

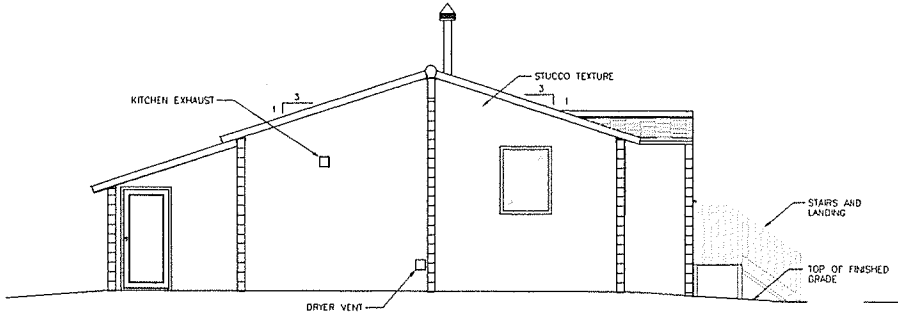
DWG. DESCRIPTION
4 BEDROOM AMBIENTE HOUSE
FLOOR PLAN (7-UNITS)

DESIGNED BY:	DRAWN BY:	CHECKED BY:
R.H.W.	C.I.	

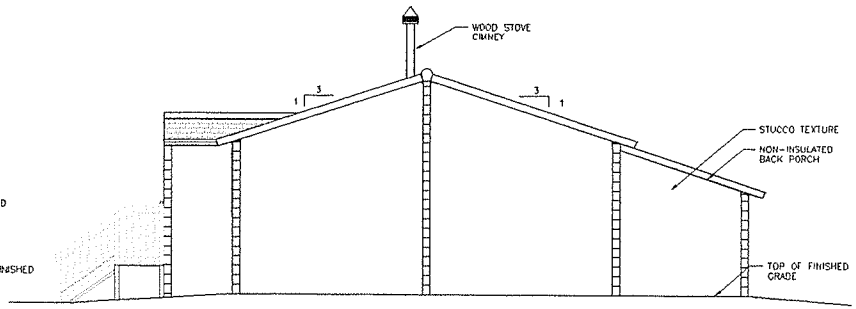
APPROVED BY:

SCALE:	DATE:
AS NOTED	04.01.23

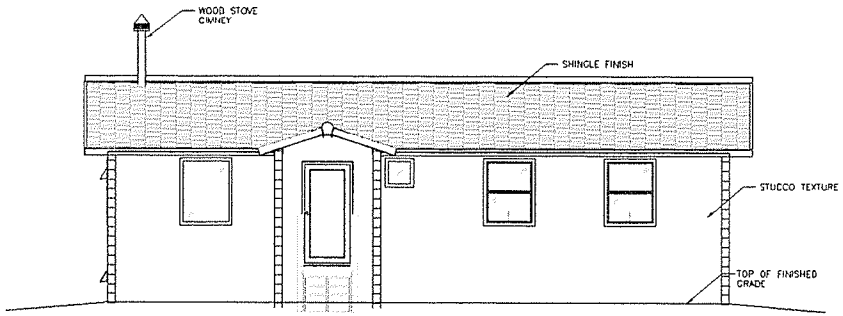
DRAWING NO	REV.
0328050100-DWG-S3006	AA



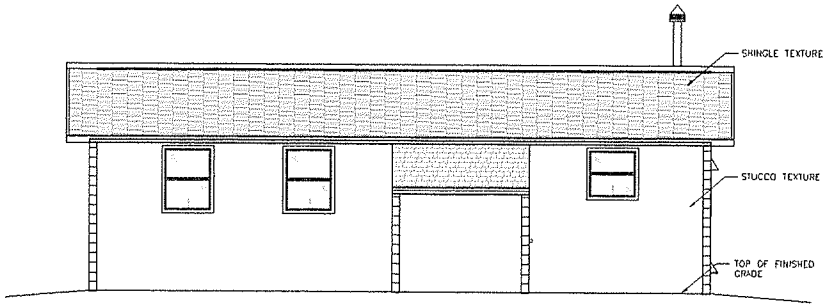
1 LEFT ELEVATION
1/4" = 1'-0"



2 RIGHT ELEVATION
1/4" = 1'-0"



3 FRONT ELEVATION
1/4" = 1'-0"



4 REAR ELEVATION
1/4" = 1'-0"

GENERAL NOTES:

1. ALL RELEVANT CSA STANDARDS, PROVINCIAL AND FEDERAL BUILDING CODES, WORKERS' COMPENSATION BOARD, AND LOCAL BYLAWS SHALL APPLY TO THIS PROJECT.
2. THE CONCRETE FOR ALL STRIP FOOTINGS SHALL BE 25 MPA AT 28 DAYS, TYPE 50 SULPHATE-RESISTANT CEMENT. MAXIMUM SLUMP 4", 1 1/2" MAXIMUM AGGREGATE, 3% TO 6% ENTRAINED AIR.
3. ALL REBAR SHALL BE NEW BILLET DEFORMED BARS GRADE 300 MPA FOR 10M, GRADE 400 MPA FOR 15M AND LARGER. ALL REBAR SHALL BE FREE OF RUST, MUD, OIL OR OTHER MATERIAL WHICH WOULD REDUCE BOND. ALL REBAR SHALL BE DETAILED AND PLACED IN ACCORDANCE WITH THE LATEST A.C.I. DETAILING MANUAL.
4. ALL DIMENSIONS AND ELEVATIONS NOTED ON THESE DRAWINGS SHALL BE VERIFIED BY THE CONTRACTOR/ OWNER PRIOR TO STARTING CONSTRUCTION. THE CONTRACTOR/ OWNER SHALL NOTIFY THE ENGINEER IMMEDIATELY UPON NOTING ANY DIMENSIONAL DISCREPANCIES.
5. HOT-DIPPED GALVANIZED NAILS SHALL BE INSTALLED IN PWF MATERIAL. STAINLESS STEEL ONLY IF STAPLES. ALL NAILING SHALL BE IN ACCORDANCE WITH TABLE NO. 7 OF THE CSA-S408 STANDARD.
6. THE DRAINAGE LAYER SHALL CONSIST OF CLEAN CRUSHED ROCK OR CLEAN GRAVEL MAXIMUM 1 1/2" DIAMETER WITH NOT MORE THAN 10% FINAL MATERIAL PASSING THROUGH A 0.12" SIEVE. EXTEND AGGREGATE MINIMUM 12" BEYOND THE EXTERIOR EDGE OF THE FOOTINGS. PLACE AGGREGATE WITH 45 MIL POLY MOISTURE BARRIER COVER OVER ENTIRE EXCAVATION.
7. UNLESS OTHERWISE NOTED ON THE CONTRACT DRAWINGS, ALL PWF STUDS SHALL BE SPEC 1 GRADE 2 OR BETTER TO THE SIZE AND SPACING AS NOTED ON THE DRAWINGS.
8. THE MATERIAL IN THE BACKFILL ZONE SHALL BE COMPRISED OF COARSE SAND OR GRAVEL INSTALLED IN MAXIMUM 12" LIFTS AND COMPACTED BY HAND TAMPING ONLY. THE TOP 12" OF BACKFILL SHALL BE COMPRISED OF IMPERVIOUS MATERIAL SLOPED AWAY FROM THE FOUNDATION WALL TO PROPERLY SHED MOISTURE AWAY FROM THE FOUNDATION SYSTEM. THE MATERIAL FOR BACKFILLING PLACED WITHIN 24" OF THE FOUNDATION SYSTEM. THE MATERIAL FOR BACKFILLING PLACED WITHIN 24" OF THE FOUNDATION WALLS SHALL BE FREE OF ALL DELETERIOUS DEBRIS, FROZEN CLUMPS, AND BOUNDERS LARGER THAN 6" IN DIAMETER. DO NOT BACKFILL UNTIL FLOOR JOISTS ARE SECURED TO WALLS.
9. HILTI NAIL OR CONCRETE NAIL PWF WALL BOTTOM PLATE TO FOOTING AT 32" O.C.
10. ALL LUMBER AND PLYWOOD THAT IS REQUIRED TO BE TREATED SHALL BE IDENTIFIED AS SUCH BY A CERTIFICATION MARK STAMPED ON THE MATERIAL THAT CONFIRMS THAT IT HAS BEEN TREATED IN CONFORMANCE WITH CSA STANDARD O80 NBC 4.2.3.2(2).

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1/4"=1'-0" 2 0 2 4 6FT.

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CLIENT

WARDROP | Engineering Inc.

PROJECT NAME
SMART PARK DEMO HOUSE

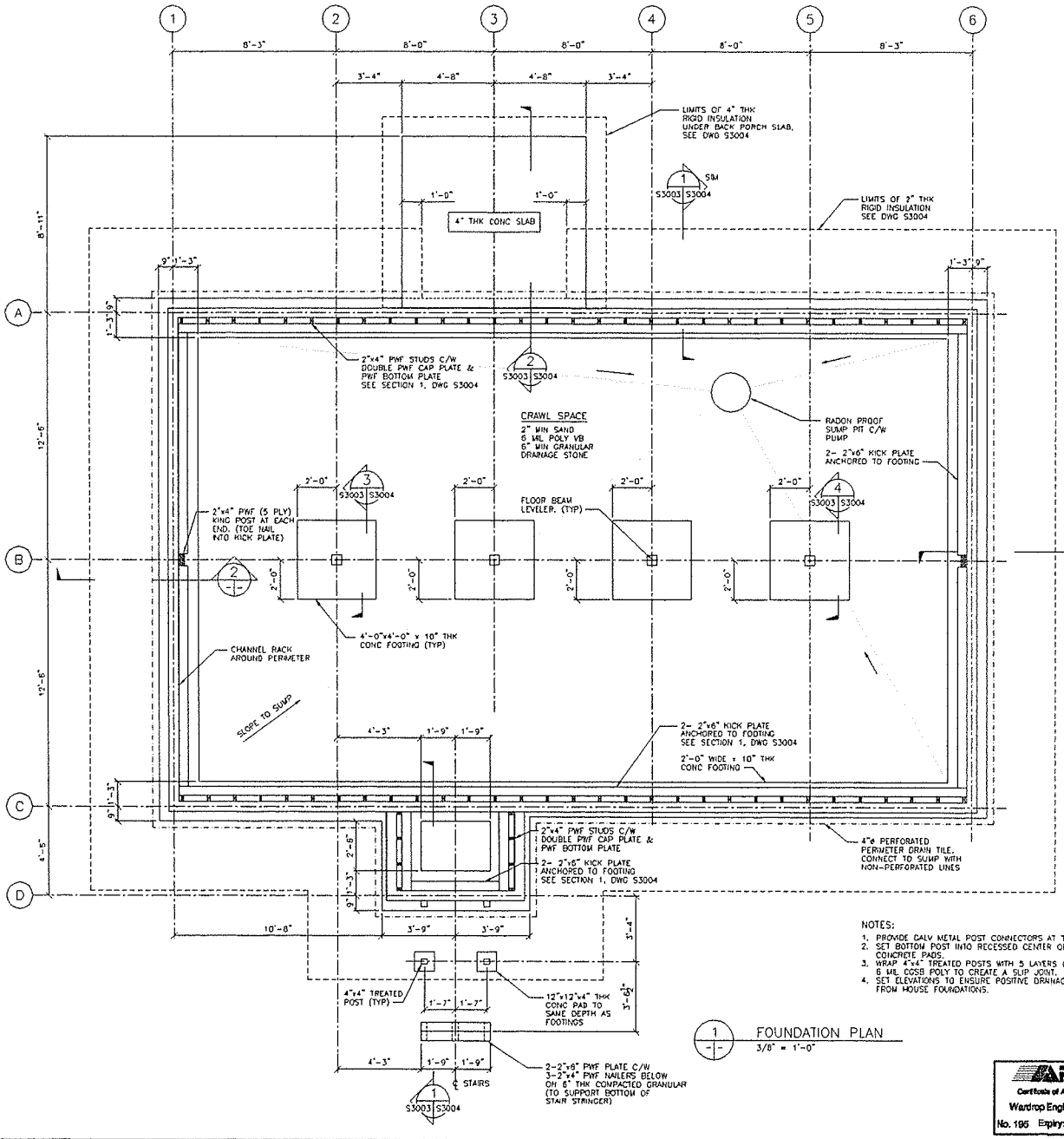
DWG. DESCRIPTION
4 BEDROOM AMBIENTE HOUSE
ELEVATIONS AND GENERAL NOTES

DESIGNED BY:	DRAWN BY:	CHECKED BY:
R.H.W.	G.I.	RHW

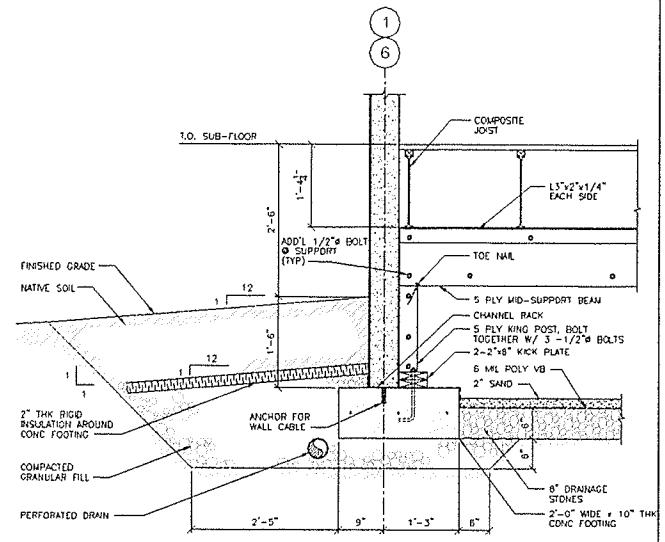
APPROVED BY:

SCALE: AS NOTED
DATE: 04.01.23

DRAWING NO: 0328050100-DWG-S3002
REV. AA



1 FOUNDATION PLAN
3/8" = 1'-0"



2 SECTION
1" = 1'-0"

NOTES:
1. PROVIDE GALV METAL POST CONNECTORS AT TOP.
2. SET BOTTOM POST INTO RECESSED CENTER OF CONCRETE PADS.
3. WRAP 4"x4" TREATED POSTS WITH 5 LAYERS OF 6 MIL CLEAR POLY TO CREATE A SLIP JOINT.
4. SET ELEVATIONS TO ENSURE POSITIVE DRAINAGE FROM HOUSE FOUNDATIONS.

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1"=1'-0" 6 0 6 12 18in.
3/8"=1'-0" 1 0 1 2 3 4ft.

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NO.	DESCRIPTION	DATE	BY

REVISIONS/ISSUE
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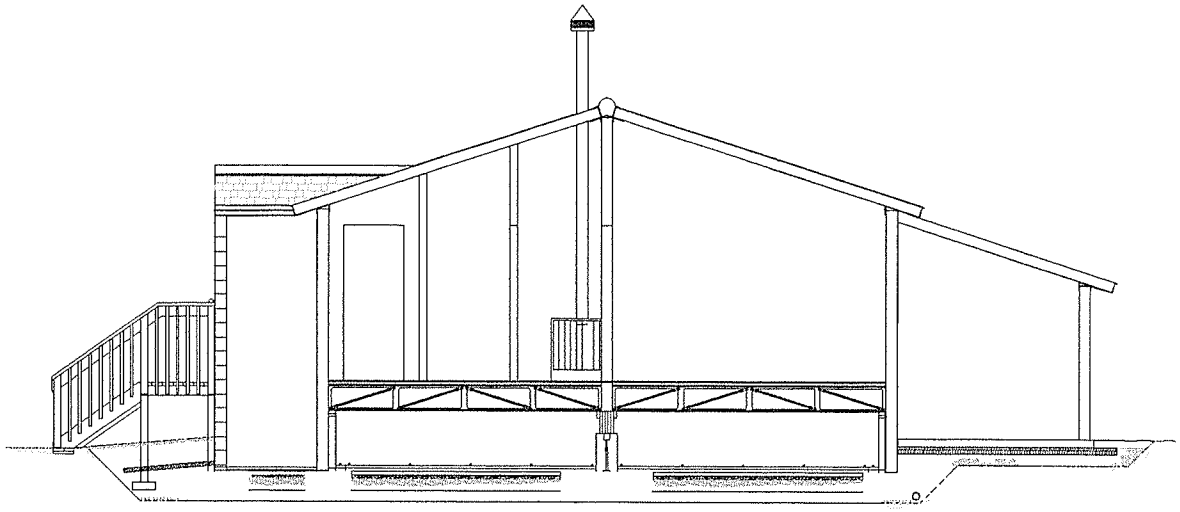
WARDROP | Engineering Inc.

PROJECT NAME
SMART PARK DEMO HOUSE

DWG. DESCRIPTION
4 BEDROOM AMBIENTE HOUSE
FOUNDATION PLAN

DESIGNED BY: R.H.W.	DRAWN BY: C.L.	CHECKED BY:
APPROVED BY:		
SCALE: AS NOTED		DATE: 04.01.23
DRAWING NO: 0328050100-DWG-S3003		REV. AA



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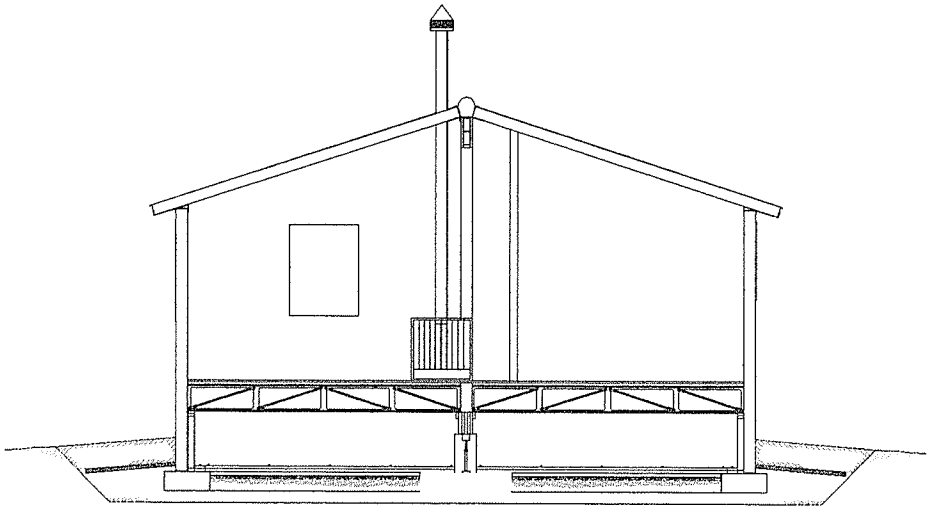
S3008 S3007 3/8" = 1'-0"



2 SECTION/ELEVATION
S3008 S3007 3/8" = 1'-0"

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3/8"=1'-0" 1 0 1 2 3 4FT.

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NO.	DESCRIPTION	DATE	BY

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CLIENT

WARDROP | Engineering Inc.

PROJECT NAME
SMART PARK DEMO HOUSE

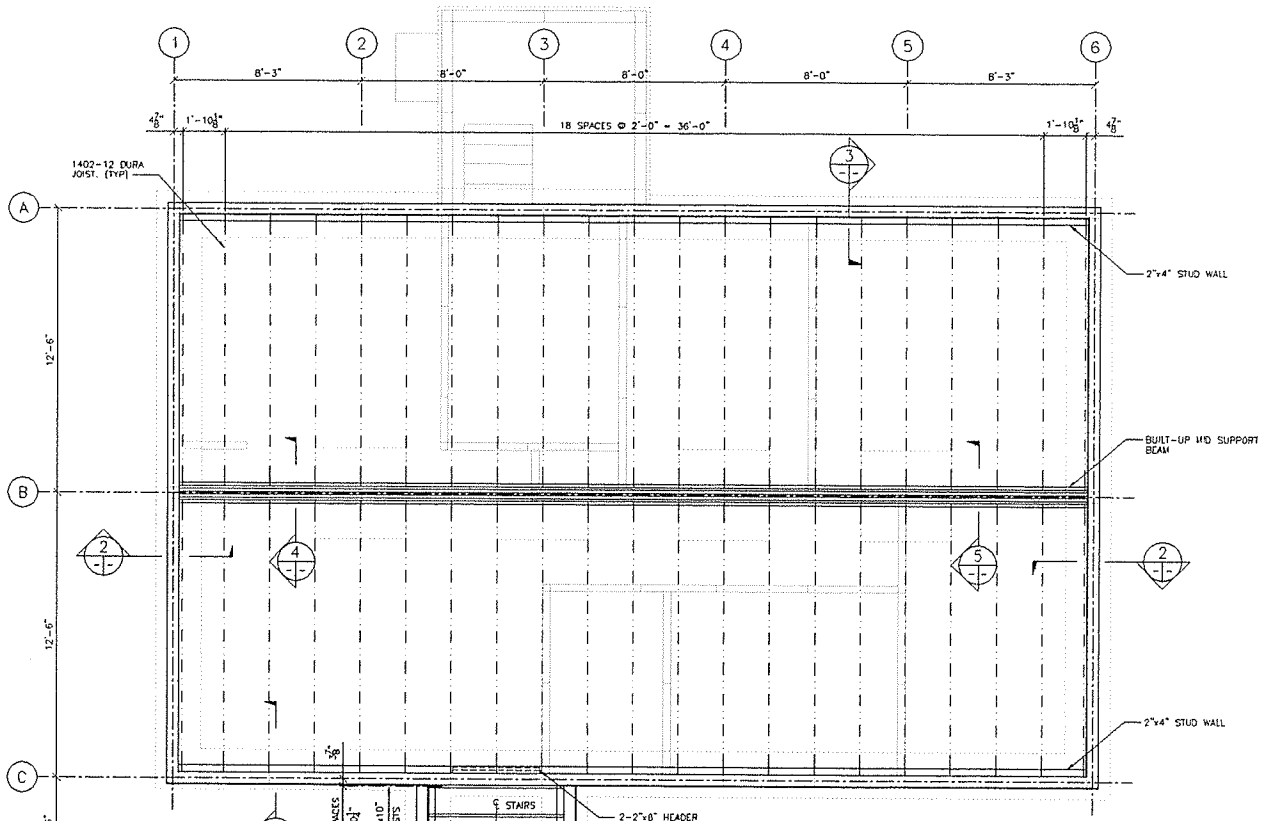
DWG. DESCRIPTION
4 BEDROOM AMBIENTE HOUSE
SECTIONS/ELEVATIONS

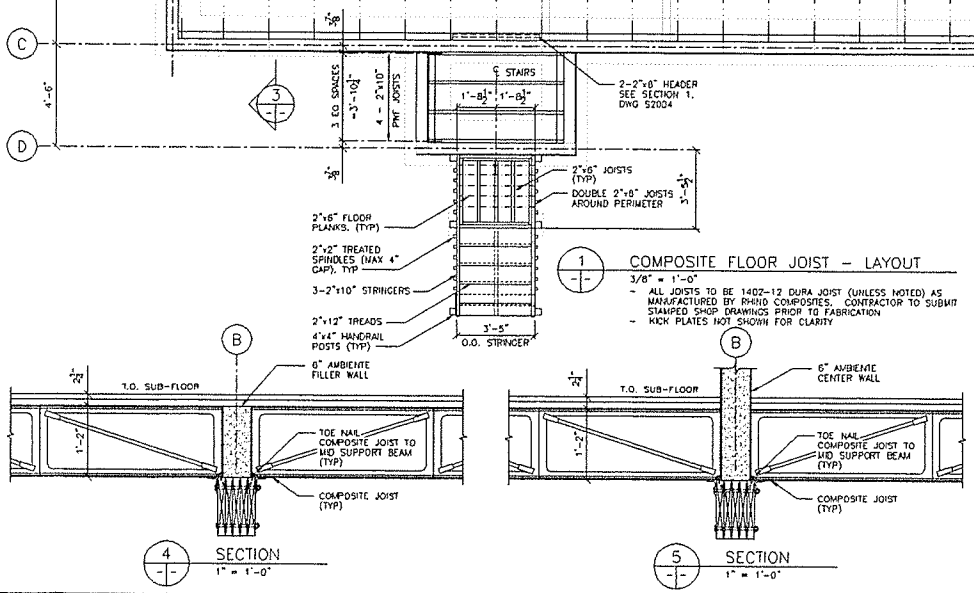
DESIGNED BY:	DRAWN BY:	CHECKED BY:
R.H.W.	C.L.	RHW

APPROVED BY:

SCALE:	DATE:
AS NOTED	04.01.23

DRAWING NO:	REV.
0328050100-DWG-S3007	AA





1 COMPOSITE FLOOR JOIST – LAYOUT

3/8" = 1'-0"

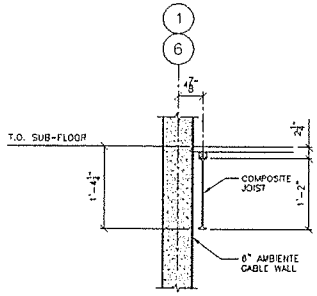
- ALL JOISTS TO BE 1402-12 DURA JOIST (UNLESS NOTED) AS MANUFACTURED BY RHINO COMPOSITES. CONTRACTOR TO SUBMIT STAMPED SHOP DRAWINGS PRIOR TO FABRICATION
- KICK PLATES NOT SHOWN FOR CLARITY

4 SECTION

1" = 1'-0"

5 SECTION

1" = 1'-0"



2 SECTION

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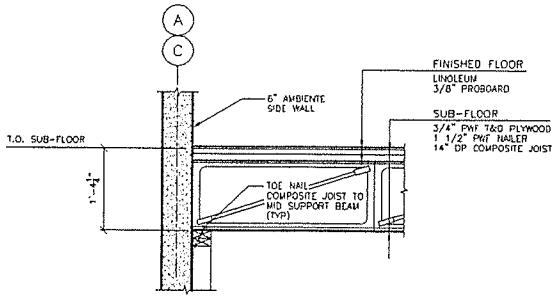

3 SECTION

1" = 1'-0"

- 3/4" PWF T&G PLYWOOD TO BE RESIN COATED

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NOT TO BE USED FOR CONSTRUCTION

(REMOVE WHEN DRAWING IS FINAL)

3" = 1'-0" 2 0 2 4 6in.

1" = 1'-0" 6 0 6 12 18in.

3/8" = 1'-0" 1 0 1 2 3 4ft.

APEGM Certificate of Authorization Wardrop Engineering Inc. No. 195 Expiry: April 30, 2004

NO.	DESCRIPTION	DATE	BY

REVISIONS/ISSUE

CLIENT

WARDROP | Engineering Inc.

PROJECT NAME: SMART PARK DEMO HOUSE

DWG. DESCRIPTION: 4 BEDROOM AMBIENTE HOUSE FLOOR JOIST LAYOUT AND SECTIONS

DESIGNED BY: R.H.W.	DRAWN BY: G.C.	CHECKED BY:
APPROVED BY:		
SCALE: AS NOTED	DATE: 04.07.23	
DRAWING NO: 0328050100-DWG-S3005		REV. AA

APPENDIX B:

Bedford Pultruded Sections Coupon Properties

TYPICAL COUPON PROPERTIES

Below are test results for typical coupon properties of Bedford Reinforced Plastics' structural fiberglass profiles (Standard, Fire Retardant, & Vinylester shapes). Properties are derived per the ASTM test method shown. Synthetic surfacing veil and ultraviolet inhibitors are standard.

MECHANICAL PROPERTIES	ASTM	UNITS	VALUE
Tensile Stress, LW	D-638	psi	30,000
Tensile Stress, CW	D-638	psi	7,000
Tensile Modulus, LW	D-638	10^6 psi	2.5
Tensile Modulus, CW	D-638	10^6 psi	.8
Compressive Stress, LW	D-695	psi	30,000
Compressive Stress, CW	D-695	psi	15,000
Compressive Modulus, LW	D-695	10^6 psi	2.5
Compressive Modulus,CW	D-695	10^6 psi	1.0
Flexural Stress, LW	D-790	psi	30,000
Flexural Stress, CW	D-790	psi	10,000
Flexural Modulus, LW	D-790	10^6 psi	1.8
Flexural Modulus, CW	D-790	10^6 psi	.8
Modulus of Elasticity, E	Full Section	10^6 psi	2.8
Shear Modulus	----	10^6 psi	0.450
Short Beam Shear	D-2344	psi	4,500
Punch Shear	D-732	psi	10,000
Notched Izod Impact, LW	D-256	ft.-lbs./in.	25
Notched Izod Impact, CW	D-256	ft.-lbs./in.	4

PHYSICAL PROPERTIES	ASTM	UNITS	VALUE
Barcol Hardness	D-2583	----	45
24 Hour Water Absorbtion	D-570	% max.	0.45
Density	D-792	lbs./in.^{3}	.062-.070
Coefficient of Thermal Expansion, LW	D-696	10^{-6} in./in./°C	8

ELECTRICAL PROPERTIES	ASTM	UNITS	VALUE
Arc Resistance, LW	D-495	seconds	120
Dielectric Strength, LW	D-149	kv./in.	35
Dielectric Strength, PF	D-149	volts/mil.	200
Dielectric Constant, PF	D-150	@60hz	5

Fire Retardant Polyester and Fire Retardant Vinylester Structural Profiles:

FLAMMABILITY PROPERTIES	ASTM	UNITS	VALUE
Tunnel Test	E-84	Flame Spread	25 max.
Flammability	D-635	----	Nonburning

LW = Lengthwise CW = Crosswise PF = Perpendicular to Laminate Face

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