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Flexural creep behavior of structural insulated timber panels

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FLEXURAL CREEP BEHAVIOR OF STRUCTURAL INSULATED TIMBER PANELS

by

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ABSTRACT

A Structural Insulated Panel (SIP) is a panel composed of insulation core laminated between two oriented-strand boards (OSB). SIPs deliver building efficiencies by replacing several components of traditional residential and commercial construction, including: (i) studs; (ii) insulation; (iii) vapour barrier; and (iv) air barrier. A SIP-based structure offers superior insulation, exceptional strength, and fast installation. Besides those benefits, the total construction costs are less with SIPs compared to wood-framed homes, especially when considering speed of construction, less expensive HVAC equipment required, reduced site waste, reduction construction financing costs, more favourable energy-efficient mortgages available, and the lower cost of owning a home built with SIPs. This thesis presents the experimental testing on selected SIP sizes to investigate their short- and long-term creep behavior under sustained loading. The experiment study performed in a manner to comply with applicable test methods and, Canadian Codes. Short-term creep test results showed the structural adequacy of the tested panels, while the long-term creep test results established the increase in panel total deflection with time. The ultimate load test results showed that the structural qualification of SIPs is "as good as" the structural capacity of the conventional wood-frame buildings.

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NOTATIONS

x, y, z	Rectangular coordinates.
D	Flexural Rigidity
b	Width of Panel
t	Thickness of panel
l	Span length
L	Live load
E_f, E_c	Moduli of face and core along axis
c	core thickness
Δ	Deflection of panel
τ_{zx}, τ_{xy}, τ_{yz}	Shear Stress
Δ_B	Mid span sandwich deflection due to bending
Δ_S	Mid span sandwich deflection due to shear
Δ_{Total}	Immediate deflection under dead load and long term portion of live loads
K	Constant to calibrate the long term effects of dead load and live load
$\Delta_{short\ term}$	Deflections under short term portions of design load

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CHAPTER I

INTRODUCTION

1.1 General

A Structural Insulated Panel (SIP) is a panel composed of insulation core laminated between two face sheathings. SIPs have been used for many different framing applications, such as walls, roofs, floors, foundations over the recent past decades. SIPs are an innovative wood framing offering a variety of benefits over the conventional framing (lumber framing). Figure 1.1 shows the application of the Structural Insulated Panel in a single family dwelling. A perspective view of SIP foundation walls supporting the SIP floor frame above is shown in Figure 1.2.

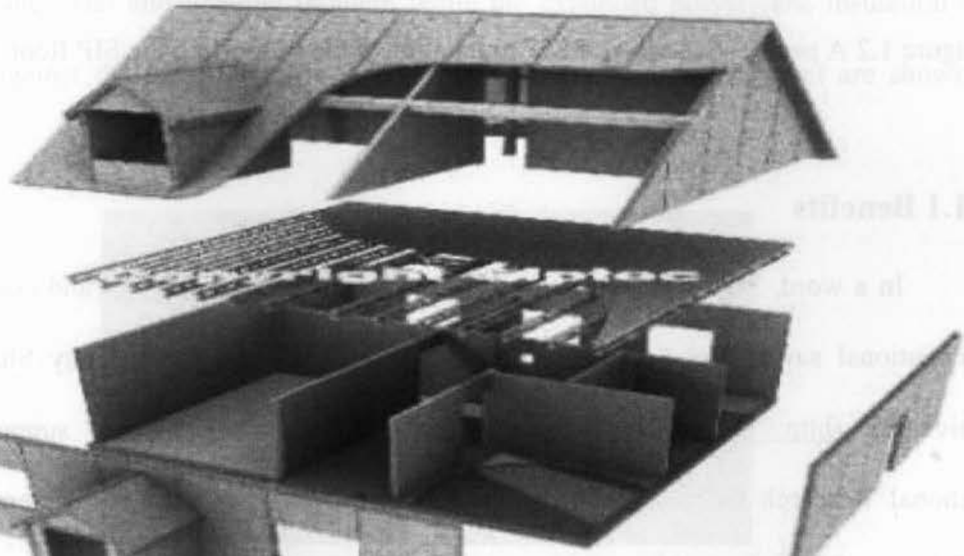

Figure 1.1 Aapplication of the Structural Insulated Panel in a single family dwelling.

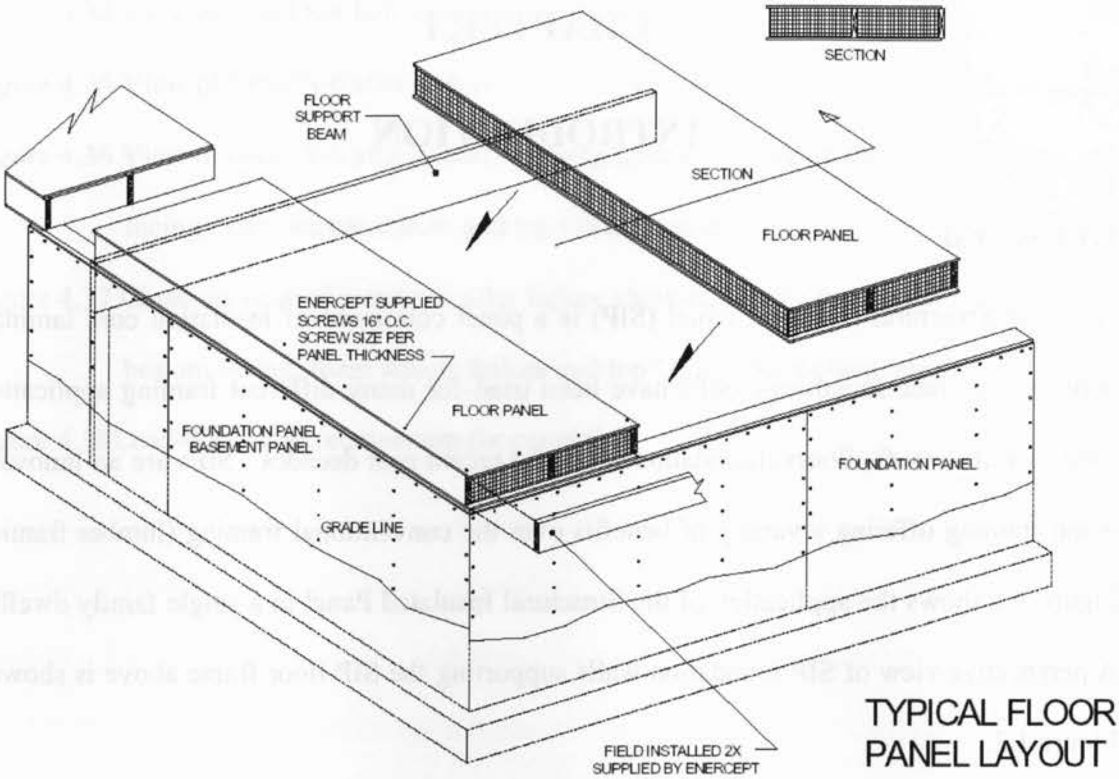

Figure 1.2 A perspective view of SIP foundation walls supporting the SIP floor frame above

1.1.1 Benefits

In a word, SIPs frame homes consumes less energy for heating and cooling than the conventional sawn lumber frame ones. The recent studies by Dr. Tony Shaw of Brock University (http://www.thermapan.com/cases/brock.html), which was supported by the National Research Council of Canada (NRC), showed tremendous difference in energy efficiency of SIPs framed house vs. conventionally framed house constructed with studs and batt insulation. The energy efficiency of wood framed homes is the most important factor for consumers these days with sky rocketing natural gas prices. In fact, in his study, shaw concluded that on a semi-detached home, a SIP-based home consumes about 65% less energy than a conventionally constructed fibreglass home of the same size, leading to long-term investment for home owners.

The SIPs provide building efficiencies by replacing several components of traditional construction including studs, insulation, vapor barrier and air barrier. That means saving time, man hour and material which make this new product cost effective. The cross section of a SIP is similar to an I-beam as shown in Fig. 1.3. The expanded polystyrene insulation core represents a web and the laminated oriented strand boards (OSB) are similar to the I-beam's flanges. In flexure, the laminated OSBs are in tension and compression, while the polystyrene insulation core sustains the shear force. The SIPs under applied axial load creating bending and compression is similar to I section Beam-Column. The laminated oriented strand boards (OSB) take the axial and bending moment while the expanded polystyrene insulation core withstands against the local buckling. Views of a structural insulated panel are shown on Figure 1.4.

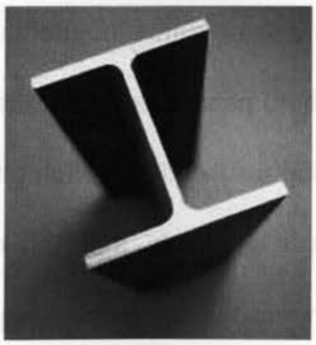

Figure 1.3 Similarities between Structural Insulated Panel and I-Beam Cross-section

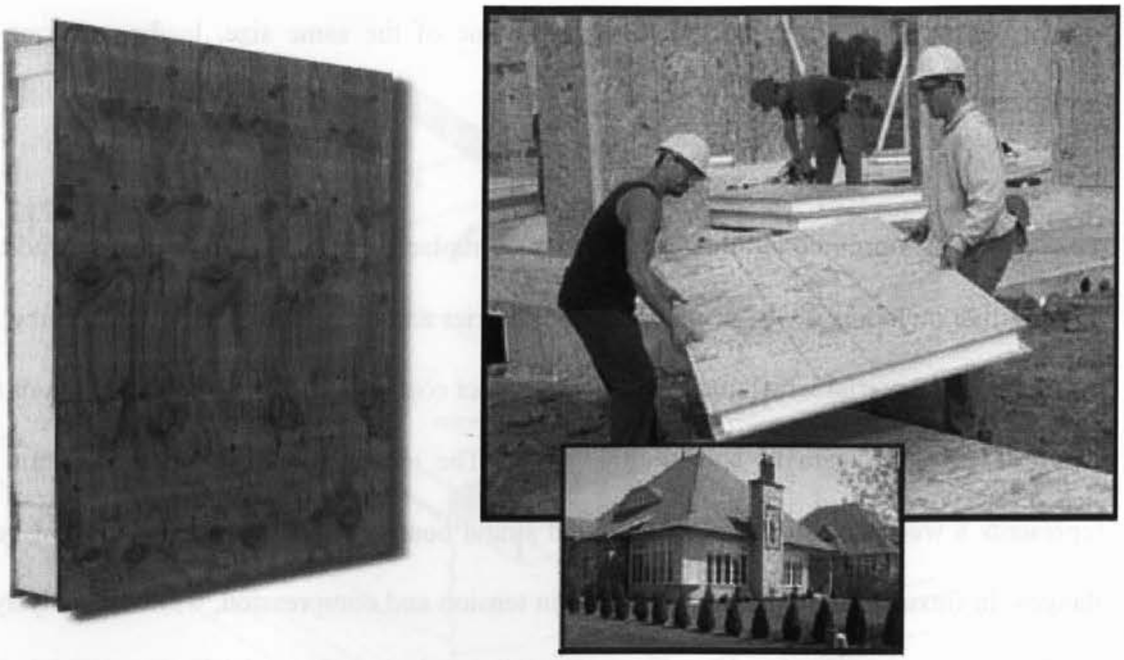

Figure 1.4: Views of a Structural Insulated Panel

1.2 The Problem

Clause 8.6 of Canadian Standard for Engineering Design of Wood, CAN/CSA-O86-01 (2005) outlined the effective stiffness, bending resistance and shear resistance of stressed skin panels. These stressed skin panels have continuous or splice longitudinal web members and continuous or spliced panel flanges on one or both panel faces, with the flanges glued to the web members. However, the developed structural insulated sandwich timber panels comprise insulated foam glued between two OSB boards. As such, it is felt necessary to conduct experimental testing on these panels to determine the structural adequacy of the level of adhesion between the foam and the OSB boards and the level of composite action between them when subjected to flexural creep loading. To address the need for testing of these developed panels, Canadian Construction Materials Commission (CCMC) and National Research Council Canada (NRC) developed technical guide for stressed skin panels (with lumber 1200 mm O.C. and EPS core) for walls and roof. This guide formed the basis for the experimental testing conducted in this thesis for flexure and creep, with the ultimate goal of providing enough technical data for strength and serviceability of such panels.

1.3 The Objectives

The main objectives of this research work can be stated as follow:

1. To contribute to the efficient design of structural insulated sandwich timber panels by developing experimentally calibrated models capable of predicting accurately their response when subjected to flexural loading.
2. To develop creep constant for serviceability limit state design.
3. To investigate the structural qualification of the studied panels for ultimate and serviceability limit state design requirements.

1.4 The Scope

The scope of this study includes:

1. A literature review on previous research work and codes of practice related to the structural behavior of sandwich timber panels when subject to transverse loading.
2. Perform experiments up-to-collapse on 2 actual-size timber panels according to ASTM standards to determine their ultimate load carrying capacity, deflection at service load level and flexure-creep performance.
3. Correlate the experimental findings with code requirements at ultimate and serviceability limit states requirements for possible qualification for building construction.
4. Draw conclusion with respect to the structural adequacy of the tested sandwich panels for possible use in residential construction.

1.5 The Contents and the Arrangement of the Thesis

Chapter II of the thesis presents a literature review of previous research on all type of sandwich panels. While Chapter III discusses the experimental program conducted on selected panel sizes, including panel sizes and material properties, and the ASTM Standard test procedure for flexure and creep. Chapter 4 summarizes the experimental findings and their correlations with theoretical results. Chapter 5 presents the conclusion of this research work and recommendations for future research. Finaly Appendix A summarizes the experimental results for each panel.

CHAPTER II
LITERATURE REVIEW

2.1 General

In the past, a significant amount of research was conducted to predict the behavior of sandwich panels. However, only very few researchers have undertaken experimental studies to investigate the accuracy of design of timber sandwich panels. Building panels come in many configurations, known variously as foam-core panels, stressed-skin panels, nail-base panels, sandwich panels, and curtain-wall panels, among others. Many of these building panels are nonstructural, while some have no insulation. And the term "panelized construction" can also include prefabricated stud walls and other configurations associated with the modular industry.

The literature review conducted is presented in the following manner:

1. History of SIPs
2. Types of Structural insulated sandwich panels
3. Structural analysis and design of Sandwich panels
4. Experimental studies

2.2 History of SIPs

The concept of a structural insulated panel began in 1935 at the Forest Products Laboratory (FPL) in Madison, Wisconsin. FPL engineers speculated that plywood and hardboard sheathing could take a portion of the structural load in wall applications. Their prototype structural insulated panels (SIPs) were constructed using framing members within the panel combined with structural sheathing and insulation. The panels were used to construct test homes that were continually monitored for over thirty years, then disassembled and reexamined. During this time, FPL engineers continued to experiment with new designs and materials. Famed architect Frank Lloyd Wright used structural insulated panels in some of his affordable Usonian houses built throughout the 1930's and 1940's. SIPs took a major leap in technology when one of Wright's students, Alden B. Dow, son of the founder of Dow Chemical Company, created the first foam core SIP in 1952. By the 1960's rigid foam insulating products became readily available resulted in the production of structural insulated panels as we know them today. The Structural Insulated Panel Association (SIPA) in U.S.A. was founded in 1990 to provide support and visibility for those manufacturing and building with this emerging building technology. In the 1990's, forming the SIPs was positively affected by the development of advanced computer aided manufacturing (CAM) technology. Using these systems, computerized architectural drawings (CAD drawings) can be converted to the necessary code to allow automated cutting machines to fabricated SIPs to the specific design of a building. CAD/CAM technology has streamlined the SIP manufacturing process, bringing further labor savings to builders and produces SIPs with amazing accuracy to deliver flat, straight, and true walls. In response to the need for the industry to develop product documentation SIPA has cooperated with the American Society for testing Materials (ASTM) task group to define a standard test method to determine structural capacities of Insulated panels (ASTM, 1996). The ASTM standard defines a testing protocol to be followed by all manufacturers to document the strength and stiffness properties of their product to code agencies for product certification. The ASTM standard tests include test methods for the following load applications: (1) transverse Loads; (2) axial Loads; (3) racking and diaphragm Loads; (4) uplift Loads; (5) creep; (6) combined Loading; (7) impact loading; and (8) concentrated Loading.

2.3 Types of Common Structural Sandwich panels

2.3.1. Light Weight Steel Frame Panels

Mild steel panels tend to be of the open type. Locating insulation on the external side of the frame overcomes the risk of cold bridging. Protection against corrosion is provided by galvanizing.

2.3.2. Fiber Cement Faced Structural Panels

Cementitious SIPs are typically manufactured of cellulose reinforced cement boards, for inside and outside skins. The material can be taped and finished on the interior surface. The fire-resistive cement board eliminates the need for gypsum drywall. The exterior surface can be painted or coated with a vinyl or synthetic stucco permanent finish. If siding or brick veneer is to be used, oriented strand board (OSB) can be applied on the exterior to accept nailing of siding or brick wall ties. It is not necessary to have both OSB and fiber-cement board on one side for brick and stucco applications. OSB can be used instead of fiber-cement board for such an application. However, there may be some difficulty in finding a manufacturer that produces this type of SIPs. Cementitious SIPs can be used for below and above grade applications. They can be used to construct foundation or basement walls, floors spanning up to 4.90 m between supports, load-bearing walls up to four stories and roof panels up to 6.10 m spans. Cementitious SIPs are fastened together with power-driven screws through the inner and outer skins into either cement board or wood splines. Because of the strength of the panels, no headers are needed over standard size doors and windows. Connection details are similar to those of OSB-sheathed panels. Cementitious SIPs are light weight, and panels can be erected by as few as two workers, with minimal equipment. They are as energy efficient as OSB SIPs. Consumers often think that R-value is of primary importance, but effective air sealing is also significant. For the best energy performance, a continuous air barrier and uniform insulation coverage, with as few gaps as possible, are needed. Every air leak and every thermal bridge adds to heating and cooling bills. Cementitious SIP panels are air-tight and fully insulated. Buildings constructed with Cementitious SIPs typically last longer and require less maintenance than other types of SIPs panels. Fiber-Cement Board used as skins will not rot, burn, or corrode. It has a higher fire rating than OSB faced SIPs, and in most residential applications no drywall would be necessary. Cementitious boards will not support black mold growth, and they have a high resistance to moisture absorption. They are rot and vermin resistant, and are not significantly affected by water vapor. Fiber-cement panels can have different finished looks, such as a wood grain, stucco, or smooth. With the smooth finish, stucco, vinyl siding, brick or stone can be installed.

2.3.3. Concrete Sandwich Panel

Concrete panels have been in use for 50 years, and the science and engineering of durable concrete has made great progress since the precast concrete non-traditional housing of the 1960s and 1970s. Brick clad concrete panels should have a service life greater than 60 years. Externally light weight insulating materials such as foamed concrete, plastic and glass provide good insulation, but they have low resistance to handling and service loads. A protective or load bearing structural concrete shell must be provided over one or both sides of these materials. These shells also provide a convenient means of imparting architectural treatment to the wall. Attractive surfaces may be obtained by many methods, such as exposed aggregate or patterns obtained from three dimensional forms. The face shells of sandwich panels must not only provide protection to the insulation and meet the immediate demands of handling and imposed loads, but must continue to give satisfactory performance under long time service. Exposure conditions cause temperature and moisture differentials in sandwich construction and these conditions may have a more pronounced effect on the satisfactory long time structural behavior than do the imposed loads. The light weight aggregate used in the concrete shells was expanded shale produced in a rotary kiln with the raw material pre- sized prior to burning. The particles are generally rounded and sealed. The structural concrete shells of the sandwich panels were reinforced with welded wire fabric confirming to ASTM A82-62T, "Cold Drawn Steel wire for concrete reinforcement." The insulating materials were commercially available rigid board stock or batting: one foamed polyurethane plastic, two foamed polystyrene plastics, one glass fiber, one foamed glass and one autoclaved cellular concrete.

2.3.4. Plywood Sandwich Panels

Plywood serves as an ideal material for the facings of sandwich panels. It is strong, light in weight, easily finished, dimensionally stable, and easily repaired if damaged. A variety of core materials may be used with plywood to complete the panel. Among these are polystyrene foams, and paper honey combs. Besides resistance to shearing forces, for some applications such as exterior wall panels and roof panels the core should posses high resistance to heat and vapor transmission. The designer should consider the stability of the core material to his application. Factors to consider include resistance to degradation by heat, age, and moisture; compatibility with glues; etc.

2.4 Structural Analysis and Design of Sandwich Panels

Sandwich construction is commonly used in structures where strength, stiffness, and weight efficiency are required. Sandwich Panel is composed of "weak" core material with "strong and stiff" faces bonded on the upper and lower side. The facings provide practically all of the over-all bending and in plane extensional rigidity to the sandwich. In principle, the basic concept of a sandwich panel is that the faceplates carry the bending stresses where as the core carries the shear stresses. The core plays a role which is analogous to that of the I-beam web while the sandwich facings perform a function very much like that of the I-beam flanges. The sandwich is an attractive structural design concept since, by the proper choice of materials and geometry, constructions having high ratios of stiffness to-weight can be achieved. Since rigidity is required to prevent structural instability, the sandwich is particularly well suited to applications where the loading conditions are conducive to buckling. The Sandwich Panel can be used in different approaches as: (a) honeycomb material; (b) corrugated material; (c) wood; (d) expanded plastics (foam); and (e) mineral wool. Also, the faces can be made of different materials, such as: (a) thin metal plates; (b) profiled plates; (c) thick fiber reinforced composite materials like glass fiber, carbon fiber, and aramid fiber..etc. The components of the sandwich material must also be bonded together, using either adhesives or mechanical fastenings, such that they can act as a composite load-bearing unit.

Sandwich materials generally exhibit the following properties:

• High load bearing capacity at low weight.

• Surface finished faceplates provide good resistance against aggressive environments.

• Excellent thermal insulation.

• Long life at low maintenance cost.

• Good water and vapor barrier.

• Excellent acoustic damping properties.

Naturally, the less favorable properties of sandwich materials can be identified as follows:

• Creep under sustained load with rigid foam cores

• Low thermal capacity

• Poor fire resistance with rigid plastic foam cores.

• Deformation when one side of faceplate is exposed to intense heat.

The correct design of the details of sandwich construction is at least as important as the analysis of deflections, stresses and backing loads. These details include nature of the edge members, splices and joints in the cores and faces, stiffeners and inserts to distributed concentrated load, type of adhesive, method of fabrication and so fourth. If the temperatures of the two faces differ, or if the moisture contents differ (as they may in asbestos cement or hardboard, for example) the differential expansion of the faces may lead to substantial transverse deflections. In building panels, especially problems arising concerning acoustic insulation, vapors transmission and fire resistance (but not usually heat insulation). All of the factors mentioned can be very important design considerations but they are beyond the scope of this topic.

2.4.1 Historical Development of Sandwich Theory

Very few papers have been published which deal with the bending and buckling of sandwich panels with cores which are rigid enough to make a significant contribution to the bending stiffness of the panel, yet flexible enough to permit significant shear deformations.

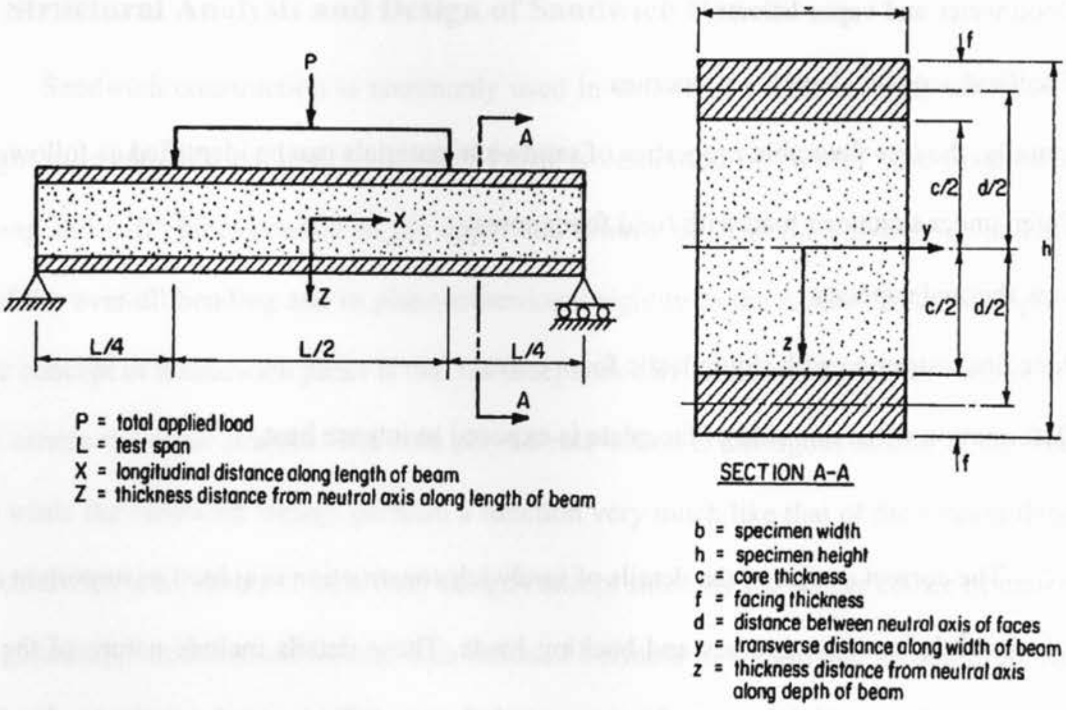

Fig. 2.1 Dimensions of Sandwich Panel

Figure 2.1 shows a typical longitudinal and cross-section in a sandwich beam made of a foam core and two facings (i.e. OSB boards). There remains the considerable problem of the sandwich panel with an anti-plane core, one which posses no stiffness in X-Y plane and in which the shear stresses τ_{zx}, τ_{yz} are constant throughout the depth (i.e. they are independent of Z). Such panels differ from ordinary homogeneous plates in that the bending deformations may be enhanced by the existence of non-zero shear strains (ε_{zx}, ε_{yz}) in the core and of direct strains ε_z in the core, perpendicular to the faces. The shear strain and the direct strain in the core are also directly associated with the possibility of short wavelength instability of the faces (wrinkling). This problem has been the subject of two main methods of analysis, which may be referred to for convenience as the general and the selective methods. In the general methods equations are setup to define the equilibrium of the separate faces and of the core and to prescribe the necessary continuity between the faces and the core. The result is a set of differential equations which may be solved in particular cases for the transverse deformation of the panel, the flattening of the core and other equations of interest. In the selective method, which has been the basis of this being named (again for convenience) as the bending problem and the wrinkling problem. In the bending problem, it is convenient to assume that the core is not only anti-plane, but also indefinitely stiff in the Z- direction. This excludes the flattening of the core and wrinkling instability, but it does permit the assessment of the effect of core shear deformation on the deflections and stresses in the panel. In the wrinkling problem the true elastic properties of the core are taken into account but the task is simplified by permitting the middle planes of the faces to deflect in the Z- direction only, not in their own planes.

2.4.2 The general Method

The general method was investigated by Reissner (1948) in relation to isotropic panels with very thin faces. Although his analysis is not simple, it is possible for Reissner to conclude that the effect of core flexibility in the Z- direction is after all less important than effect of core shear deformation in the transverse planes. Wrinkling instability as such is not discussed. It is only by neglecting the effect of direct transverse core strains that Reissner is able to derive a relatively simple differential equation for the transverse displacement, w. A much more recent analysis by Heath (1960) also includes a very similar equation, but for a sandwich with an orthotropic core. Heath's analysis work based on earlier work by Hemp (1948) and is apparently independent of the work of Reissner. Raville (1955) applies the general method to the simply-supported rectangular panel with the uniform transverse load and with thin faces. The three displacements of points in the orthotropic anti plane core are expresses as polynomials in z, but the complexity of the analysis again makes it necessary to revert to the simplifying assumption of infinite core stiffness in the z- direction. For practical purposes, the general method is the interact-table when applied to sandwich panels, but more success has been achieved in relation to sandwich struts and beams. The early works of Williams et al. (1941) and of Cox and Riddell (1945) fall into this category. The first of these deals with a sandwich struts with thick faces and an isotropic core (with an extension for an orthotropic cores) and the analysis used to form a link between the extreme cases of wrinkling instability (no longitudinal displacements of the faces during buckling) and of overall Euler-type instability, modified for shear deformations in the core (no direct core strains in the z-direction). A very thorough analysis of the behavior of the struts with isotropic faces and cores has been outlined by Goodier (1946) and completed by Goodier and Neou (1951). In the latter paper the works of Williams and of Cox are verified to a high degree of accuracy.

2.4.3 The Selective Method

Most of the published work on sandwich panels refers to the selective method and, in particular, to the bending problem, in which core strains in the z-direction is neglected.

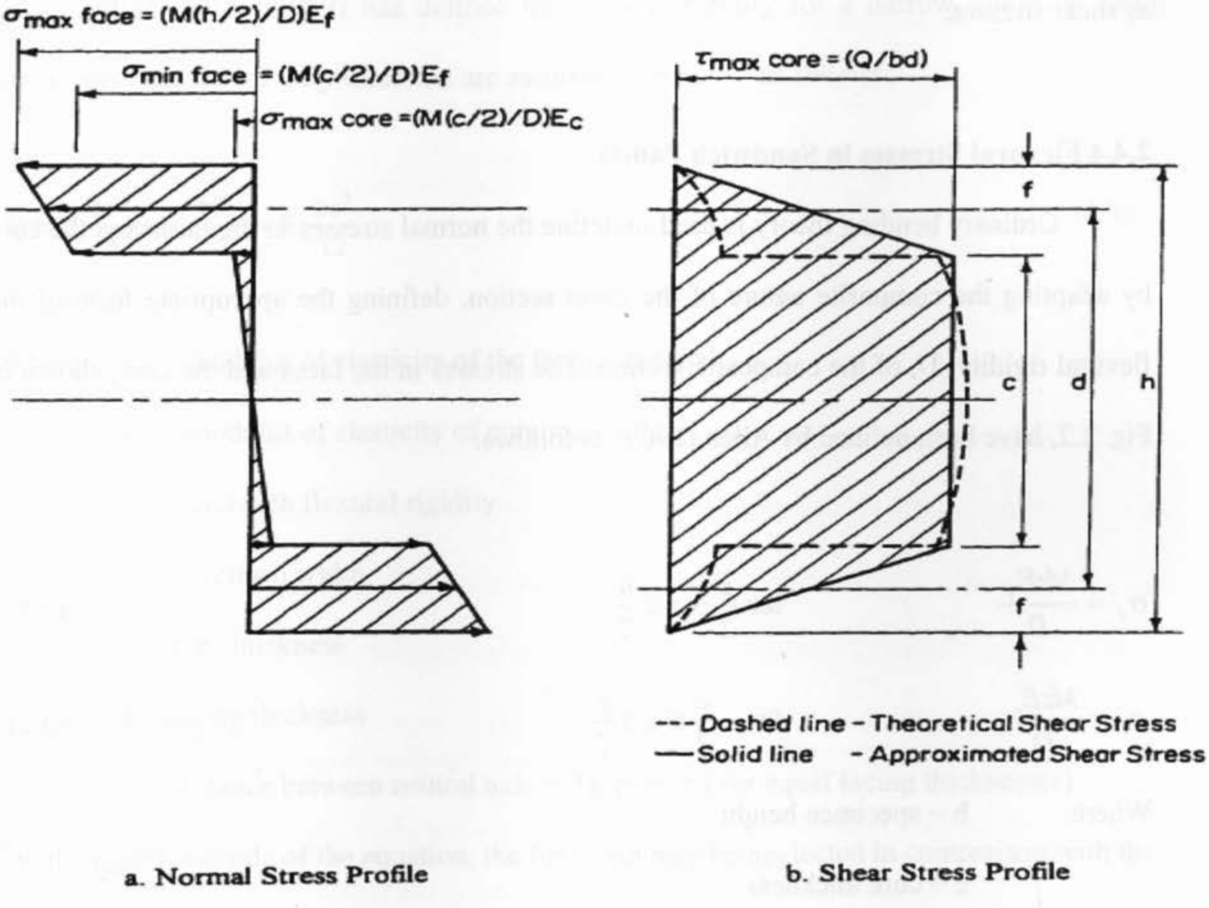

Fig. 2.2. Flexural Stress and Shear Stress Distribution across the Depth of the Sandwich Panel

The assumption that core is weak in the xy-plane leads in any case to the conclusions that the core makes no contribution to the flexural rigidity of the sandwich, that the core shear stresses τ_{zx} and τ_{yz} are independent of z and a straight line drawn in the unloaded core normal to the faces remains straight after deformation, but is no longer normal to the faces. These assumptions (core weak in x-y plane, stiff in z- direction) allow the displacements of the panel to be expressed in terms of only three variables, one of which is the transverse displacement w. Figure 2.2 shows a summary of this method of analysis in case of flexural stresses as well as shear stresses.

2.4.4 Flexural Stresses in Sandwich Panels

Ordinary bending theory is used to define the normal stresses in the faces and the core by adapting the composite nature of the cross section, defining the appropriate form of the flexural rigidity, D, of the composite section. The stresses in the faces and the core, shown in Fig. 2.2, have been defined by Allen (1969) as follows:

$$\sigma_f = \frac{MzE_f}{D} \qquad \text{for } \frac{c}{2} \leq z \geq \frac{h}{2} \tag{2.1}$$

$$\sigma_c = \frac{MzE_c}{D} \qquad \text{for } -\frac{c}{2} \leq z \geq \frac{c}{2} \tag{2.2}$$

Where: h = specimen height

c = core thickness

E_f = modulus of elasticity of the facing material

E_c = modulus of elasticity of the core material

D = sandwich flexural rigidity (Equation 2.3)

σ_c = normal core stress

σ_f = normal facing stress

M = bending moment

z = distance from the neutral axis of the sandwich

The flexural rigidity is commonly referred to as D and can be defined as the sum of the flexural rigidities of the faces and the core measured about the neutral axis of the sandwich cross-section. Allen (1969) has defined the flexural rigidity for a narrow sandwich beam (transverse stresses in the y direction are assumed to be zero) as follows.

$$D = E_f \frac{bf^3}{6} + E_f \frac{bfd^2}{2} + E_c \frac{bc^3}{12} \qquad (2.3)$$

Where: E_f = modulus of elasticity of the facing material

E_c = modulus of elasticity of core material

D = sandwich flexural rigidity

b = specimen width

c = core thickness

f = facing thickness

d = distance between neutral axis of faces (c + f for equal facing thicknesses)

On the right hand side of the equation, the first term may be neglected in comparison with the second if:

$$D / f > 5.77 \qquad (2.4)$$

If this condition is fulfilled, the local bending stiffness of the faces (bending about their own separate centroidal axes) makes a negligible contribution of the flexural rigidity of the sandwich.

The third term may be neglected in comparison with the second if

$$\frac{E_f f d^2}{E_c c^2} \phi\, 16.7 \qquad (2.5)$$

If this condition is fulfilled, the bending stiffness of the core is negligible.

2.4.5 Flexural and Shear Stresses in Sandwich Panels

The form of the shear stress (τ) for a point located at distance z from the neutral axis of a homogenous beam can be easily derived by ordinary bending theory and appears in many basics text books as follows.

$$\tau = \frac{QS}{Ib} \qquad (2.6)$$

Where Q = shear force at the section

I = second moment of area of the entire section about its centroid

b = width at given depth in section ($b = z_1$)

S = first moment of area of that part of the section where $z > z_1$

For a sandwich beam, the moduli of elasticity of the component parts are accounted for by representing the sum of the products of S and E in Equation (2.7); the profile of the shear stress through the depth is defined in Equation 2.8 (Allen, 1969).

$$\tau = \frac{QS}{Db}\sum(SE) \qquad (2.7)$$

$$\tau(z) = \frac{Q}{D}[E_f \frac{fd}{2} + \frac{E_c}{2}(\frac{c^2}{4} - z^2)] \qquad (2.8)$$

Allen (1969) shows that Equation 2.8 may be simplified if the sandwich has a relatively weak core and if the flexural rigidity of faces about axis of faces is small (Equation 2.4 satisfied). For sandwich cross-section with relatively stiff faces and weak core, it is common to assume the shear stress of the faces is negligible. Therefore, Equation 2.6, which defines the shear stress through the depth of the core, reduces to Equation 2.9

$$\tau = \frac{Q}{bd} \tag{2.9}$$

The normal and shear stress profiles of a sandwich beam are given in Fig. 2.2 where the maximum facing stress at the outer fiber is obtained by using z = h/2 in Equation 2.1, the minimum facing stress at the interface of the core is obtained by using z = c/2, and maximum shear stress in the core as given in Equation 2.9.

2.4.6 Elastic Deflection Analysis of Sandwich Panels

Equations defining the instantaneous elastic mid-span deflection of uniformly loaded simply-supported sandwich beams (with relatively thin, stiff faces and thick weak cores) are well known and widely cited. The plywood Design Specification Supplement, entitled "Design and Fabrication of plywood Sandwich Panels" (APA 1990) simplifies the total sandwich beam mid-span deflection (Δ_T) to the sum of bending and shear deflection as follows:

$$\Delta_T = \Delta_B + \Delta_S \tag{2.10}$$

Where: Δ_B = mid-span sandwich panel deflection due to bending

Δ_S = mid-span sandwich panel deflection due to shear

The form of the elastic bending deflection for a simply-supported homogeneous beam of uniform cross-section in quarter-point loading, see Fig. 2.1, is easily derived by ordinary bending theory and appears in many basic text books as follow:

$$\Delta_B = \frac{11PL^3}{384EI} \tag{2.11}$$

Where: P = total applied load

L = beam span

E = modulus of elasticity of the beam material

I = moment of inertia of the uniform cross-section

When defining the deflection of a sandwich beam, however, the flexural rigidity (EI) must be defined in terms of its component materials and their position in the cross-Section.

Allen (1969) also shows that for thin faces (local bending of faces is negligible), negligible core bending stiffness, constant shear stress throughout the core, and negligible shear stress in the skin material, the displacement (w_2) associated with the shear deformation of the core can be determined by integrating Equation 2.12.

$$\int \frac{dw_2}{dx} = \frac{Q}{AG} \tag{2.12}$$

Where A = bd^2 /c and AG is referred to as the shear stiffness

Q = shear force= P/2 for quarter-point loading

G = core shear modulus

X = distance from the reaction in shear zone of beam

w_2 = displacement at x

By applying the boundary conditions for the simply-supported quarter point load beam (w_2 = 0 at x = 0, the maximum shear deflection (at x = L/4) associated with the shear deformation of the sandwich loaded at quarter points is defined by Equation 2.13. Thus the total sandwich beam deflection reflecting the bending and shear component is defined in Equation 2.14.

$$\Delta_s = w_{2\max} = \frac{PL}{8AG} \qquad (2.13)$$

$$\Delta_s = \frac{11PL^3}{384D} + \frac{PL}{8AG} \qquad (2.14)$$

In 1996, ASTM included creep loading as an official protocol addressing SIP performance. At this point engineers and designers need validated techniques to define SIP creep performance to consumers and code officials. The National Design Specification for Wood, NDS, (NFPA, 1991) provided convenient method (equation 2.15) for calculating total deflections for structural wood products subject to long term loading:

$$\Delta_{\text{Total}} = K\,(\Delta_{\text{long term}}) + \Delta_{\text{short term}} \qquad (2.15)$$

Where $\Delta_{\text{long term}}$ = immediate deflection under dead load +long-term portion of live loads

K = constant to calibrate the long-term effects of dead load and live load

$\Delta_{\text{short term}}$ = deflections under short-term portions of design load

The long-term deflection constant, K, ranges in magnitude from 1.5 for seasoned lumber and glue laminate timbers, and; up to 2 for green lumber. There is a great need in the SIP industry to develop a similar relationship for long-term SIP behavior. This creep behavior can be defined by experimental testing. Figure 2.3 shows a schematic diagram of creep behavior of a typical material. The first region shows the instantaneous deflection-time relationship as the member reaches its immediate deflection. The next region defines primary creep where deflection increases at a decreasing rate. The secondary creep region shows the deflection increasing at a nearly constant rate and finally, the tertiary creep region ending in failure. Alternatively, if the structure is unloaded before the onset of the tertiary stage, the deflection is immediately reduced; the elastic deflection will be fully recovered for viscoelastic material and the structure continues to recover its creep deflection.

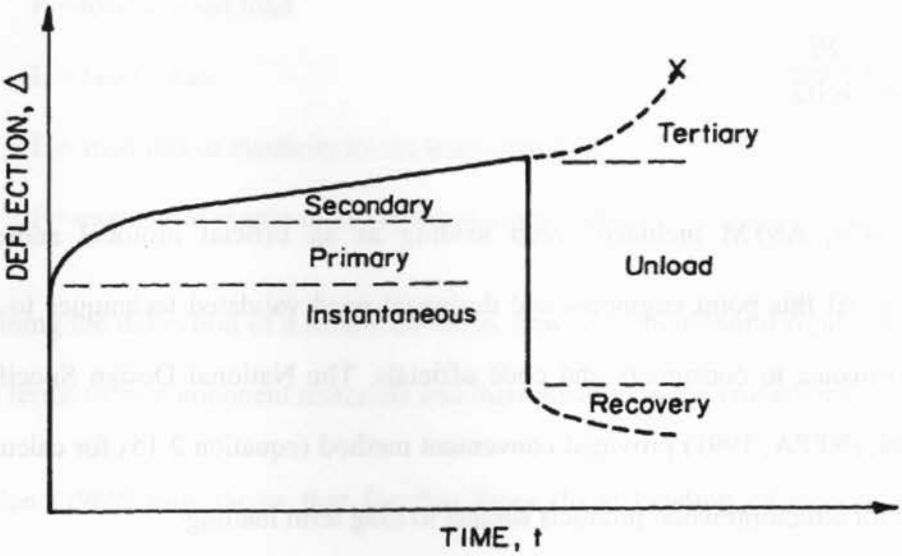

Figure 2.3 Creep Behavior Graph

Models of parabolic form have been employed with good success to describe the primary and secondary creep deflection of wood and rigid foam materials as well as metal skinned sandwich panels (e.g. Davies (1987), Huang and Gibson (1990. 1991), Gerhards (1985) and Hoyle et al. (1985)). Typically called a power model, the full form is presented in Equation 2.16

$$\Delta(t) = \Delta_0 + A_1 t^{(A2)} \quad (2.16)$$

where:

$\Delta(t)$ = Total time dependent deflection
Δ_0= Initial deflection
A_1, A_2= creep parameters

The creep behavior of wood on wood (OSB faced solid-sawn wood stud core) panels has been researched by Wong et al. (1988) for three months load duration. Davis (1987) summarized research predicting the influence of creep on urethane and EPS core metal faced panels for ten year load duration. Huang and Gibson (1990) reported results on the creep of metal faced urethane core panels. Other work by Huang and Gibson (1991) defined creep parameters for polyurethane foam cores from shear creep tests as recommended by ASTM-C273-61. Taylor (1996) conducted a series of creep testing on OSB/foam structural insulated panels to measure the three month mid-span creep deflections due to sustained loading at the quarter points. Four manufacturers were included in the experimental plan (two EPS core SIP manufacturers and two urethane core SIP manufacturers). The SPS designates the expanded polystyrene core type. The results suggested the use of a fractional deflection factor, K, for the calibration of long-term deflection as 1.5 for EPS core and 2.0 for urethane core for cumulative deflection duration up to three months in the NDS long-term equation.

Few authors conducted research work on the structural behavior related to sandwich panels. Among them, Liu and Zhao (2007) studied the effect of soft honeycomb core on the flexural vibration of sandwich panel using low order and high order shear deformation models. Aviles and Carlsson (2006) conducted experimental study of the in-plane compressive failure of sandwich panels consisting of glass/epoxy face sheets over a range of PVC foam cores, and a balsa wood core containing one or two circular or square interfacial debonds. In most specimens, failure occurred by local buckling of the debonded face sheet followed by rapid debond growth towards the panel edges, perpendicular to the applied load.

Meyer-Piening (2006) dealt with the linear static and buckling analysis of an asymmetric square sandwich plate with orthotropic stiffness properties in the face layers. Gupta and Woldesenbet (2005) and Gupta et al. (2002) studied experimentally and theoretically the behavior of sandwich-structured composites containing syntactic foam as core material under three-point bending loading conditions. They presented a method of analysis for syntactic foams and the sandwich structures containing syntactic foam as core material. Olsson (2002) suggested an engineering method to predict the impact response and damage of flat sandwich panels. The approach accounts for local core crushing, delamination and large face sheet deflections. Yoon el al. (2002) studied experimentally the non-linear behavior of sandwich panels made of thermoplastic foam core and carbon/epoxy fabric faces. The experimental data were compared with the predicted results from a proposed analytical method and the finite-element analysis. Tham et al. (1982) studied, using the finite-prism-strip modeling, the flexural and axial compressive behavior of the prefabricated architectural sandwich panels made of foam-in-place rigid urethane cores and light-gauge cold-formed metal facing. A similar study was recently conducted elsewhere but with plain concrete core (Hossain and Wright, 2004).

CHAPTER III

EXPERIMENTAL STUDY

3.1 General

Thermapan Inc. is Canadian-based Company that developed Structural Insulated Timber Panels (SIPs). These panels are composed of thick layer of expanded polystyrene insulation (EPS) board laminated between two sheets of oriented strand board (OSB). Thermapan SIPs can be used for many different applications, such as interior and exterior walls, roofs, floors, foundations, timber frame, log homes, additions, and renovations. Figure 3.1 shows structural insulated panel definition sketch.

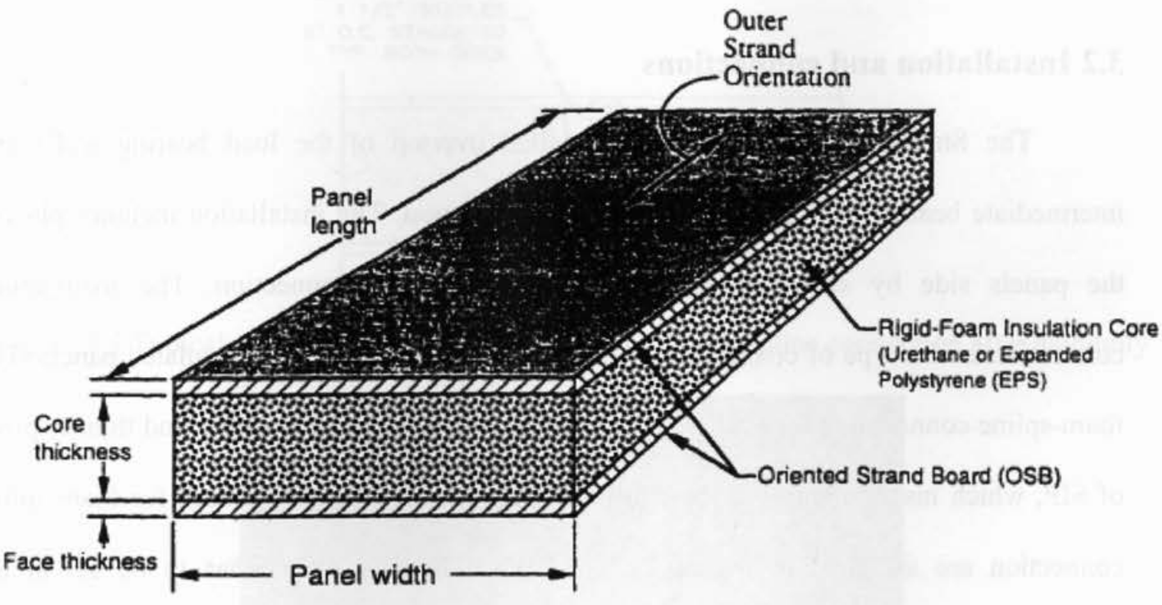

Figure 3.1 Sketch of Structural Insulated Panel, SIP

Thermapan SIPs are available in the following standard sizes of 1.2 m wide and lengths of 2.43, 2.72, 3.05, 3.66, 4.27 and 4.90 m. The used facing of these developed panels is made of 11 mm (7/16") Oriented Strand Board (OSB) on both sides of the foam core for floor and wall construction. Some panels developed for extra resistance to environmental conditions are made of 11 mm (7/16”) Oriented Strand Board (OSB) on one facing and 28.6 mm (1-1/8’’) thick timber sheathing on the outer facing. These SIPs provide exceptional resistance to fire and it meets building code for many commercial applications based on the R-Value shown in Table 3.1 (Thermapan, 2007).

Table 3.1 Thermapan SIPs Properties

SIP Thickness (Timber)	4.5”	6.5”	8.25”	10.25”	12.25”
EPS Core Thickness	3-5/8"	5-5/8"	7-3/8"	9-3/8"	11-3/8"
Dimensional Lumber	2x4	2x6	2x8	2x10	2x12
Weight (lbs/sq.ft.)	3.13	3.32	3.48	3.66	3.84
R-Value	19.147	29.147	37.897	47.897	57.897

3.2 Installation and connections

The Structural Insulated panels installed overtop of the load bearing walls and intermediate beams for roof and floor frame construction. The installation includes placing the panels side by side and installation of longitudinal connection. The foam-spline connection is one type of connection installing between the Structural Insulated panels. The foam-spline connection piece called Insul-spline and consisted of a narrow and thinner piece of SIP, which inserted between the two panels. The detailed cross sections for foam-spline connection are sketched in Figures 3.2 and 3.3. The insul-spline has to be set in the longitudinal edges of the panels connecting to each other. The recommended caulking and foam sealant must be applied as specified. The complete assembly view for foam-spline connection is shown in Figure 3.4.

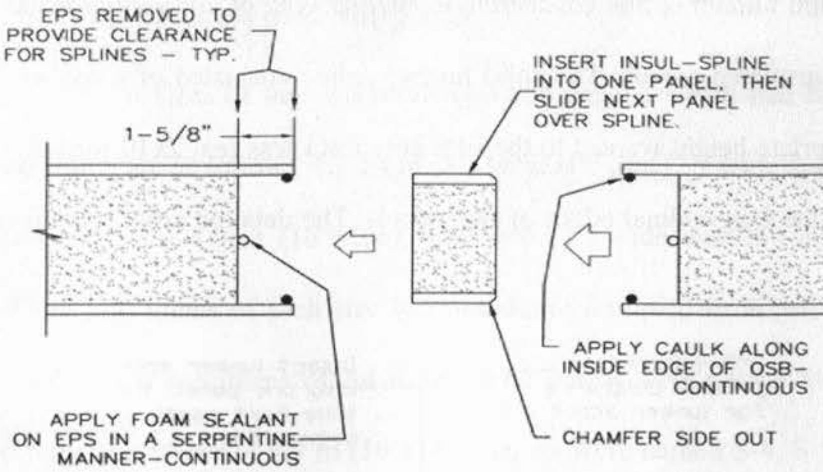

Figure 3.2 Typical cross section for SIP floor and roof foam-spline connection before assembly

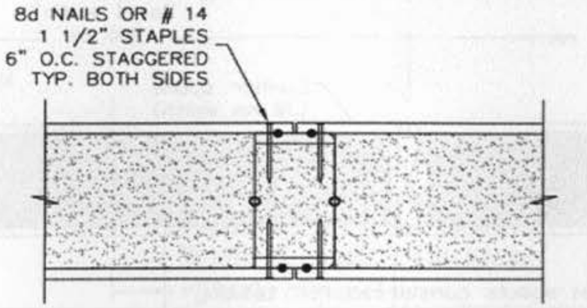

Figure 3.3 Typical cross section for SIP floor and roof foam-spline connection after assembly

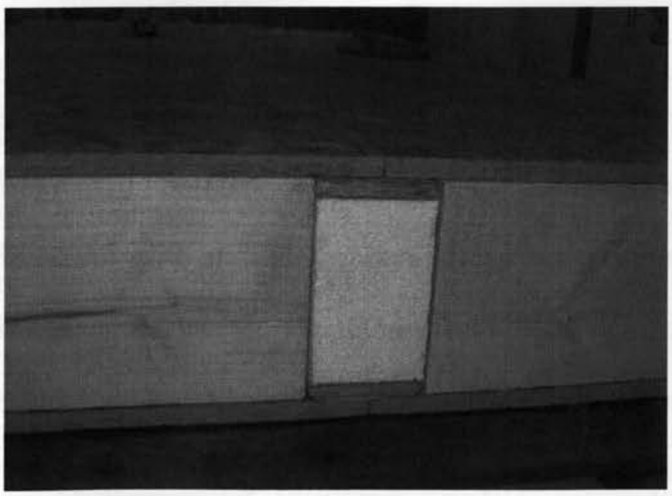

Figure 3.4 View of a foam-spline connection

The solid lumber spline connection is another type of connection installing between the structural insulated panels. The solid lumber spline consisted of a conventional lumber with an appropriate height respect to the EPS core thickness (ex. 2x10 for 9-3/8" thick core) setting within the longitudinal edges of two panels. The detailed cross section is sketched in Figures 3.5.

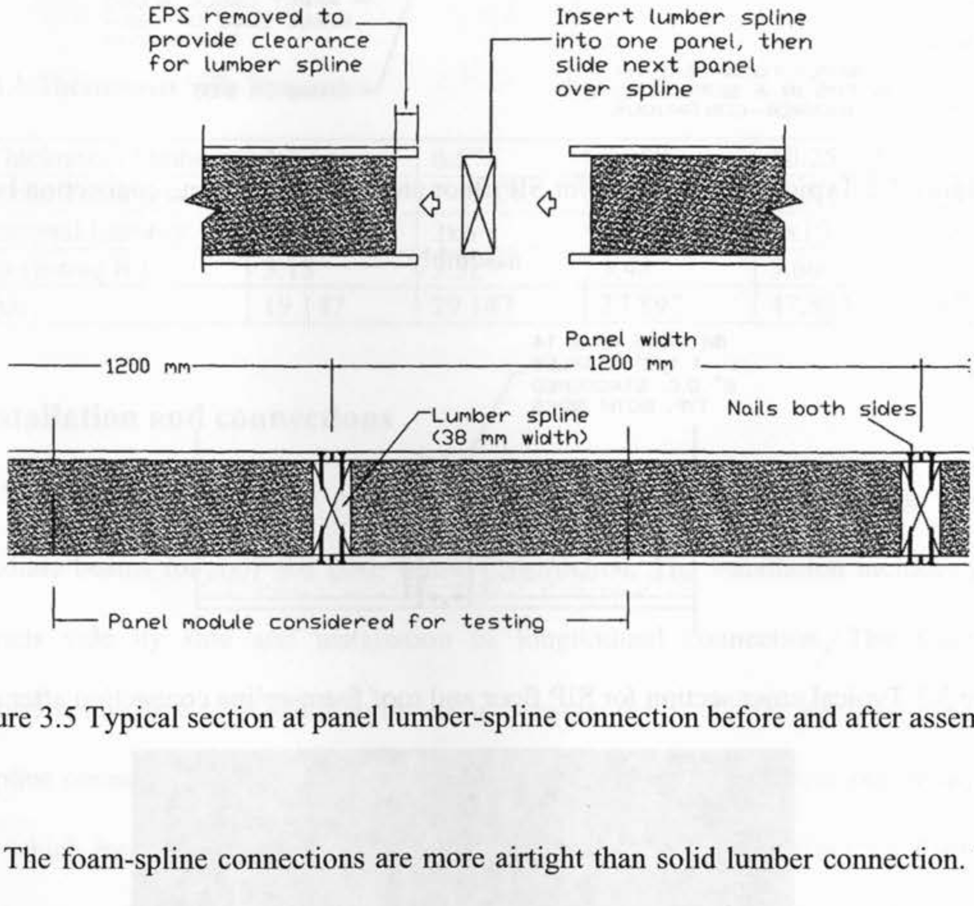

Figure 3.5 Typical section at panel lumber-spline connection before and after assembly

The foam-spline connections are more airtight than solid lumber connection. That is foam-spline connections are recommended for roof and wall frame since the most heat loss is occurred from the roof space and the exterior walls. In order to assemble the air tight foam-spline connection the provided details in Figures 3.2 and 3.3 have to be followed. The solid lumber spline connection is recommended for heated area (living space) within the dwelling where the heat loss is not an important factor.

3.3 Description of Panels and Supports

Two different sizes of the structural insulated panels, which had been used for the experimental study, are as follows: (i) 2.4 m (8') long, 600 mm (2') wide and 165 mm (6 ½") total thickness; and (ii) 4.9 m (16') long, 600 mm (2') wide and 260 mm (10 ¼") total thickness. Three specimens of each size were randomly sampled from full 4' (1.2 m) wide panels. The first three specimens of 2.4 m (8') long panels were named S-1, S-2, and S-3. The second three specimens of 4.9 m (16') long panels were named S-4, S-5 and S-6. Table 3.1 presented the geometric description of these panels along with the type of test considered in this study. Three types of structural qualification tests were considered in this study, namely: (i) short-term creep test; (ii) long-term creep test; and (iii) flexural failure test.

Table 3.2 Tested panels description

Groups	Test No.	Test type	Panel size Width ×Length ×Thick. of panel	(OSB) Thickness	Specimen Name
A	1	Short Term Creep	2×8×6 ½"	7/16"	S-1
	2	Short Term Creep	2×8×6 ½"	7/16"	S-2
	3	Short Term Creep	2×8×6 ½"	7/16"	S-3
B	4	Short Term Creep	2×16×10 ¼ "	7/16"	S-4
	5	Short Term Creep	2×16×10 ¼ "	7/16"	S-5
	6	Short Term Creep	2×16×10 ¼ "	7/16"	S-6
A	7	Long Term Creep	2×8×6 ½"	7/16"	S-1
	8	Long Term Creep	2×8×6 ½"	7/16"	S-2
	9	Long Term Creep	2×8×6 ½"	7/16"	S-3
B	10	Long Term Creep	2×16×10 ¼ "	7/16"	S-4
	11	Long Term Creep	2×16×10 ¼ "	7/16"	S-5
	12	Long Term Creep	2×16×10 ¼ "	7/16"	S-6
A	13	Flexural Failure	2×8×6 ½"	7/16"	S-1
	14	Flexural Failure	2×8×6 ½"	7/16"	S-2
	15	Flexural Failure	2×8×6 ½"	7/16"	S-3
B	16	Flexural Failure	2×16×10 ¼ "	7/16"	S-4
	17	Flexural Failure	2×16×10 ¼ "	7/16"	S-5
	18	Flexural Failure	2×16×10 ¼ "	7/16"	S-6

3.4 Material Properties

The Thermapan SIPs as manufactured by Thermapan Industries Inc. consist of three elements, factory crafted in a computer assisted lamination assembly line. The exterior faces are oriented strand board (OSB) manufactured and grade stamped as per APA (1990). The OSB board used to fabricate the panels had 1R24/EF16/W24 panel mark with 10.5 mm thickness construction sheathing. The minimum material properties for OBS boards, as supplied by the SIP manufacturer are specified as follows:

Modulus of rupture: 28.955 MPa (4200 psi) in the span direction

12.409 MPa (1800 psi) in the direction normal to the span direction

Modulus of elasticity: 5515 MPa (800,000 psi) in the span direction

1551 MPa (225,000 psi) in the direction normal to the span direction

However, material characteristics as specified in the OSB Design Manual (2004) for the 1R24/EF16/W24 panel are listed below.

Bending resistance	M_r= 228 N.mm/mm
Axial tensile resistance	T_r= 57 N.mm/mm
Axial compressive resistance,	P_r = 67 N.mm/mm
Shear through thickness resistance,	V_r = 44 N/mm
Bending stiffness,	EI= 730,000 N.mm^2/mm
Axial stiffness,	EA= 38,000 N/mm
Shear through thickness rigidity,	G = 11,000 N/mm

Expanded Polystyrene is a polymer impregnated with a foaming agent, when exposed to steam, creates a uniform closed cell structures highly resistant to heat flow and moisture penetration. This in-plant expansion process is fused into blocks which are cured for dimensional stability and cut into boards. The expanded polystyrene (EPS) core used to fabricate the panels was Type 1. Thermapan Industries Inc. specifies a priority density that demonstrates a load failure of 25 psi when tested as per ASTM C297. EPS core material must meet the minimum standard CAN/ULC-S701 and demonstrate the following minimum strength characteristics:

Nominal density	$16\ kg/m^3$ ($1.0\ lbs/ft^3$)
Shear strength:	83 kPa (12 psi)
Shear modulus:	2758 kPa (400 psi)
Flexural strength:	172 kPa (25 psi)
Tensile strength:	103 kPa (15 psi)
Compressive strength:	70 kPa (10 psi)

The urethane adhesive must meet the following standards:

ASTM D-2294: 7 Day High Temperature Creep Test

ASTM D-905: Block Shear Test Using Plywood

ASTM C-297: Tension Test of Flat Sandwich Construction in a Flatwise Plane

ASTM D-1877: Resistance of Adhesive to Cyclic Laboratory Aging Conditions

ASTM D-1002: Strength Properties of Adhesive Bonds in Shear by Tension Loading

3.5 Experimental Test Methods

In 2007, the National Research Council Canada (NRC) prepared a technical guide (IRC, 2007) that describe the technical requirements and performance criteria for the assessment of stressed skin panels (with lumber 1200 mm o.c. and EPS core) for walls and roofs for the purpose of obtaining a CCMC (Canadian Construction Materials Commission) evaluation report. The requirements and criteria referenced in this guide were developed to evaluate the performance of stressed skin panels for walls and roofs with respect to their performance as an alternative solution established with respect to Part 4, Structural Design, and Part 9, Housing and Small Buildings, of the National Building Code of Canada (NBCC, 2005). The Technical Guide focuses on the structural qualification of stressed skin composite panels as being "as good as" the structural capacity of the conventional wood-frame buildings. A successful evaluation conforming to this Technical Guide will result in a published CCMC Evaluation Report that is applicable only to products bearing the proper identification number of CCMC's evaluation number. This NRC/IRC/CCMC Technical Guide specifies test methods for SIPs similar to those specified in ASTM E72-02, *Standard Test Methods for Conducting Strength Tests of Panels for Building Construction*, (ASTM, 2002) as well as ICC AC04, *Acceptance Criteria for Sandwich Panels*, (2004). It should be noted that ICC AC04 acceptance criteria is based on ASTM E72 standard test methods. As such, bending qualification tests on the panels were conducted in accordance with the method described in the ASTM E72-02, *Transverse Load Test*. ASTM E72-02 specifies at least three identical specimens for each test. This condition is reflected in the tested panel groups shown in Table 3.2.

The structural behavior of structural insulated panels was examined under specified and ultimate loads at structures laboratory of Ryerson University. Short-term creep, long-term creep and flexural failure tests were successfully conducted on panel configurations listed in Table 3.2. The following subsections describe the test procedure and the structural qualification criteria for each test.

3.5.1 Short-Term Creep Test

3.5.1.1 Test method for SIP Panels for Short-Term Creep and Recovery Performance

In case of test method for short-term creep of SIPs, the 2007 NRC/CCMC Technical Guide specified a creep and recovery test for roof panels in Appendix II. It should be noted that the Guide specifies at least three panels to be tested to evaluate the design. Also, the Guide specifies a 0.5 kPa dead load simulating the weight of superimposed finished roofing and ceiling materials. The live load is expected to be the anticipated snow and rain loads for the anticipated geographical areas. In this study, a roof live load is assumed to be 1.9 kPa similar to the floor live load in residential construction. The test procedure for short-term creep and recovery of performance of SIP's is as follows, while Figure 3.6 shows a schematic diagram of the loading schedule for creep and recovery test.

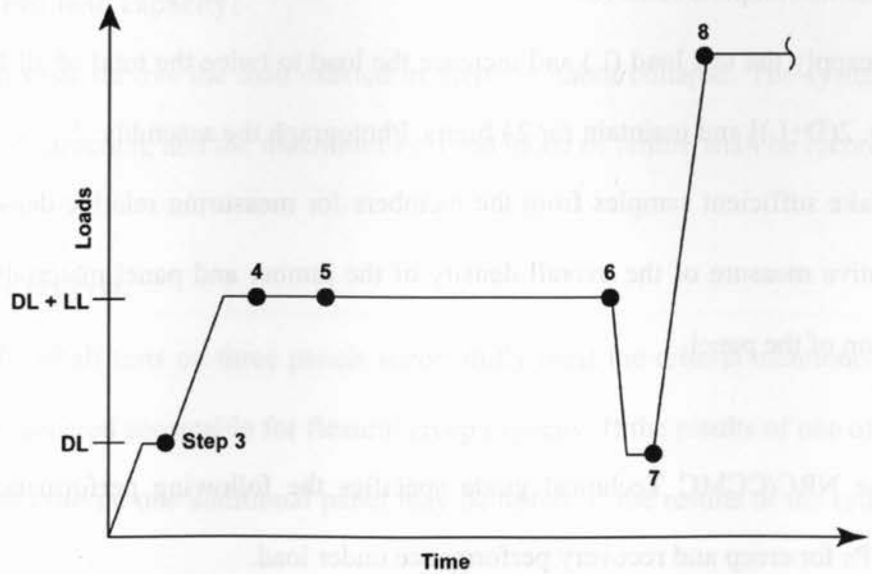

Figure 3.6 Schematic diagram of the loading steps for creep and recovery test

Step 1: Measure the moisture content of wood members at a sufficient number of points to give a representative picture of the overall moisture condition of the members at the time of test. If members are not accessible, moisture contents may be obtained shortly after the test.

Step 2: Take zero deflection readings before applying any load.

Step 3: Apply test load (D), representing the superimposed dead load, at the uniform rate without shock to the system. At the conclusion, following a full five minutes (300 s) for the deflection to stabilize, take the deflection readings. Photograph the assembly.

Step 4: Apply test load (L), representing the superimposed live load, at the uniform rate without shock to the system. At the conclusion, following a full five minutes (300 s) for the deflection to stabilize, take the deflection readings. Photograph the assembly.

Step 5: Measure the deflections at one hour from the beginning of loading (Step 3).

Step 6: Maintain these loads (D+L) for an additional full 23 hours and take deflection readings again.

Step 7: Remove test load (L) [Design Live Load] and take deflection readings five minutes (300 s) after its complete removal.

Step 8: Reapply the test load (L) and increase the load to twice the total of all loads applied before [i.e. 2(D+L)] and maintain for 24 hours. Photograph the assembly.

Step 9: Take sufficient samples from the members for measuring relative density to give a representative measure of the overall density of the lumber and panel materials used in the construction of the panel.

The NRC/CCMC Technical guide specifies the following performance criteria to qualify SIPs for creep and recovery performance under load.

(i) Deflection under the action of live loads:

The maximum difference in the deflection measured in Step 4 o (dead load plus live load) and Step 3 (dead load only) shall not exceed 1/360 of the span.

(ii) Creep deflection criterion:

The difference in deflection at any one point as measured between Step 6 and Step 4, shall not exceed 25% of that measured in Step 4 (attributable to the creep produced by the dead load plus live load in place for approximately 24 hours).

(iii) Recovery from creep criterion:

The lack of recovery determined by the maximum difference in the deflections measured in Step 3 (dead load only) and that measured in Step 7 (on removal of the live load) shall not exceed 1/1440 of the span.

(iv) Sustained load capacity:

The system shall survive the load exerted in Step 6 without collapse. The system shall then be taken to destruction, and the maximum load and mode of failure shall be recorded.

Acceptance criteria:

If the results of all tests on three panels successfully meat the criteria mentioned above, the design is considered acceptable for flexural creep capacity. If the results of one of the tests do not meet the criteria, one additional panel may be tested. If the results of the retest or If two of the original panels do not meet the criteria, the design is deemed unacceptable and design values must be adjusted.

3.5.1.2 Description of Test Specimens and Applied Loads

Two panel sizes were considered in this testing with 8' and 16' long, respectively. Three identical panels of 2.4 m (8') long, S-1, S-2 and S-3, were considered for short-term creep test. Other set of identical panels of 4.9 m (16') long, S-4, S-5 and S-6, was also tested. Figure 3.6 shows a schematic diagram of the test setup per ASTM E72-02. The panel was supported over two steel rollers of 25.4 mm diameter and 600 m length, with a 600×150×12 mm steel plate between each supporting roller and the specimen. Also, similar steel plates were inserted between the steel rollers and 150 x150 x 13 HSS steel box beam that is in turn supported over two concrete cylinder of 150 mm diameter and 300 mm length. The assembled support system is shown in Figure 3.8. While view of the unloaded specimen S-3 is shown in Figure 3.9.

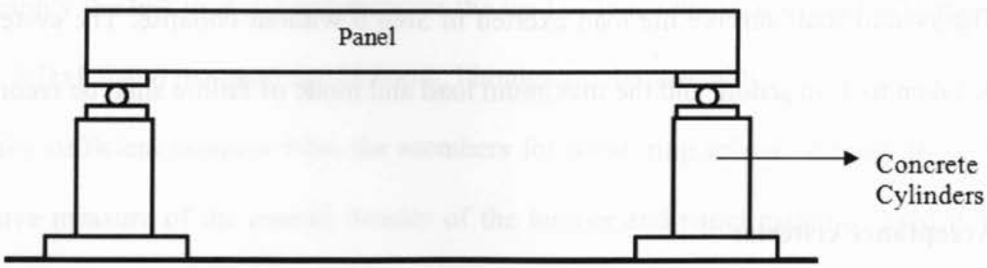

Figure 3.7 Schematic view of the test setup for creep testing per ASTM E72-02

Figure 3.8 Typical support system for creep test

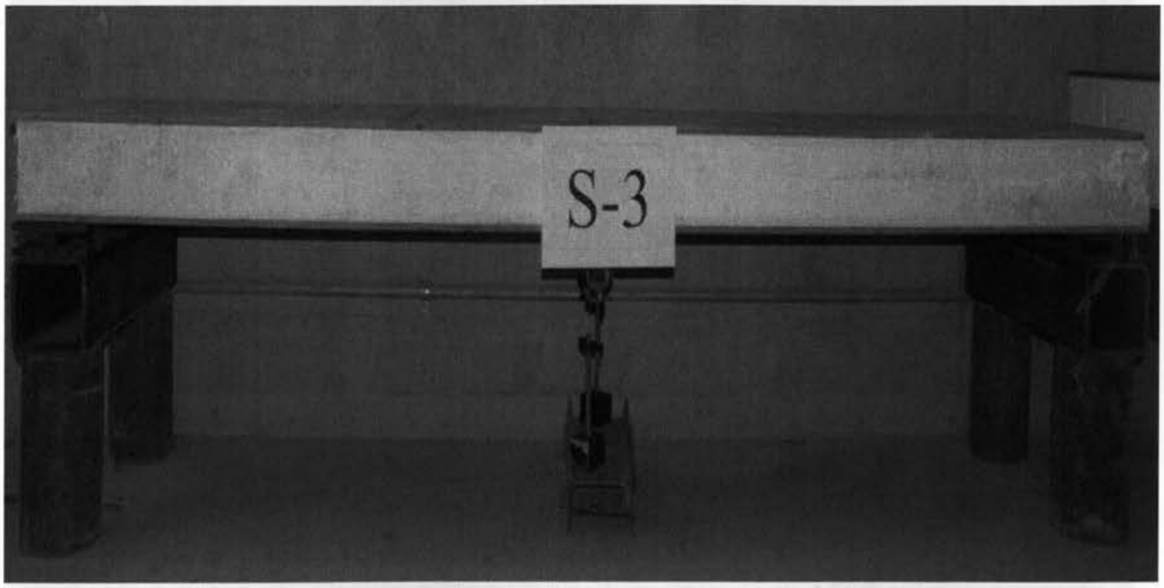

Figure 3.9 View of the specimen S-3 before loading

In case of the 8’ long panels, two cement bags of 40 kg weight were used to load the panels with simulated dead load. This provides an equivalent dead load, D, of 0.563 kPa in contrast to design dead load of 0.5 kPa. Also, 10 cement bags were used to simulate the dead and live load on the panels leading to an equivalent D+L load of 2.816 in contrast to a design value of 2.4 kPa. In case of the 16' long panels, four cement bags of 40 kg weight were used to load the panels with simulated dead load. This provides an equivalent dead load of 0.545 kPa in contrast to design dead load of 0.5 kPa. Also, 20 cement bags were used to simulate the dead and live load on the panels leading to an equivalent D+L load of 2.725 in contrast to a design value of 2.4 kPa. Figures 3.9 to 3.26 show views of each specimen when subjected to simulated dead load, dead and live load, and double the dead and live load, respectively.

Fig. 3.10 View of dead load over panel S-1

Fig. 3.11 View of dead and live load over panel S-1

Fig. 3.12 View of double dead and live load over panel S-1

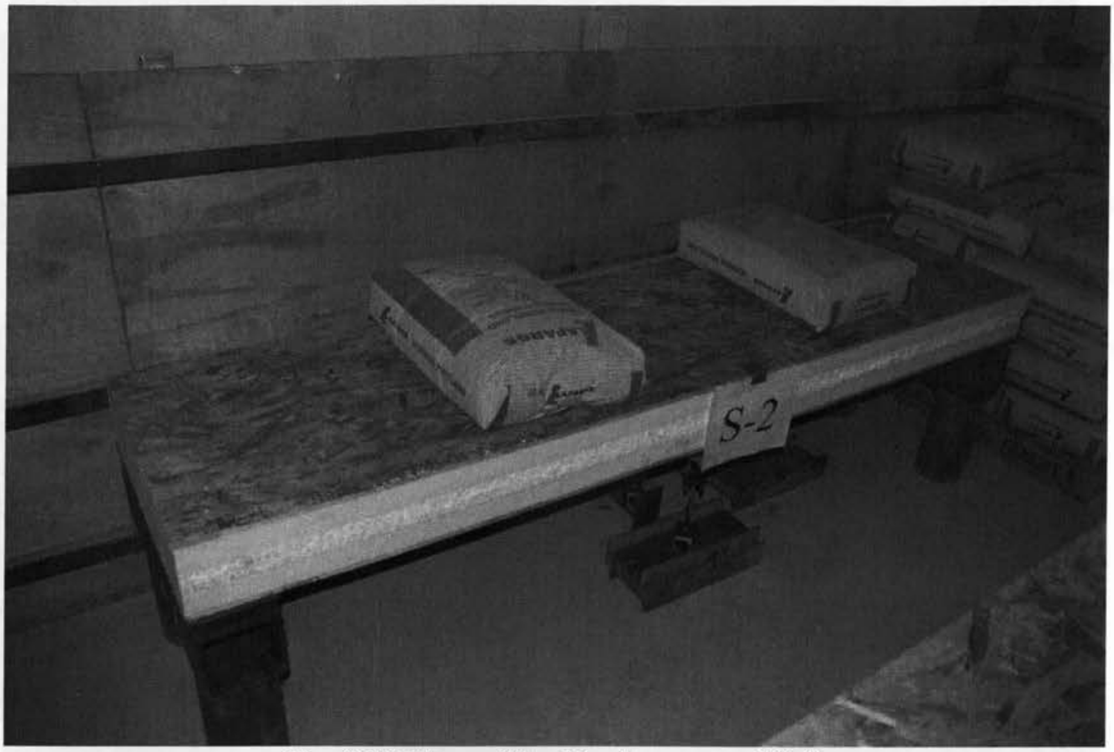

Fig. 3.13 View of dead load over panel S-2

Fig. 3.14 View of dead and live load over panel S-2

Fig. 3.15 View of double dead and live load over panel S-2

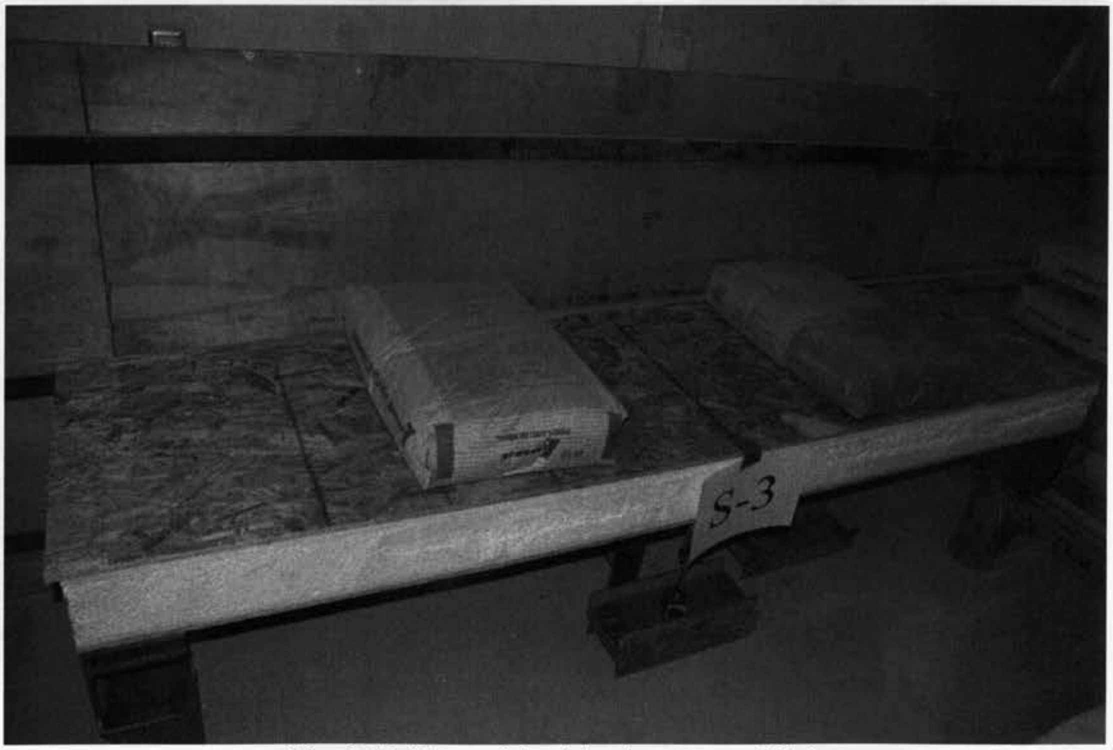

Fig. 3.16 View of dead load over panel S-3

Fig. 3.17 View of dead and live load over panel S-3

Fig. 3.18 View of double dead and live load over panel S-3

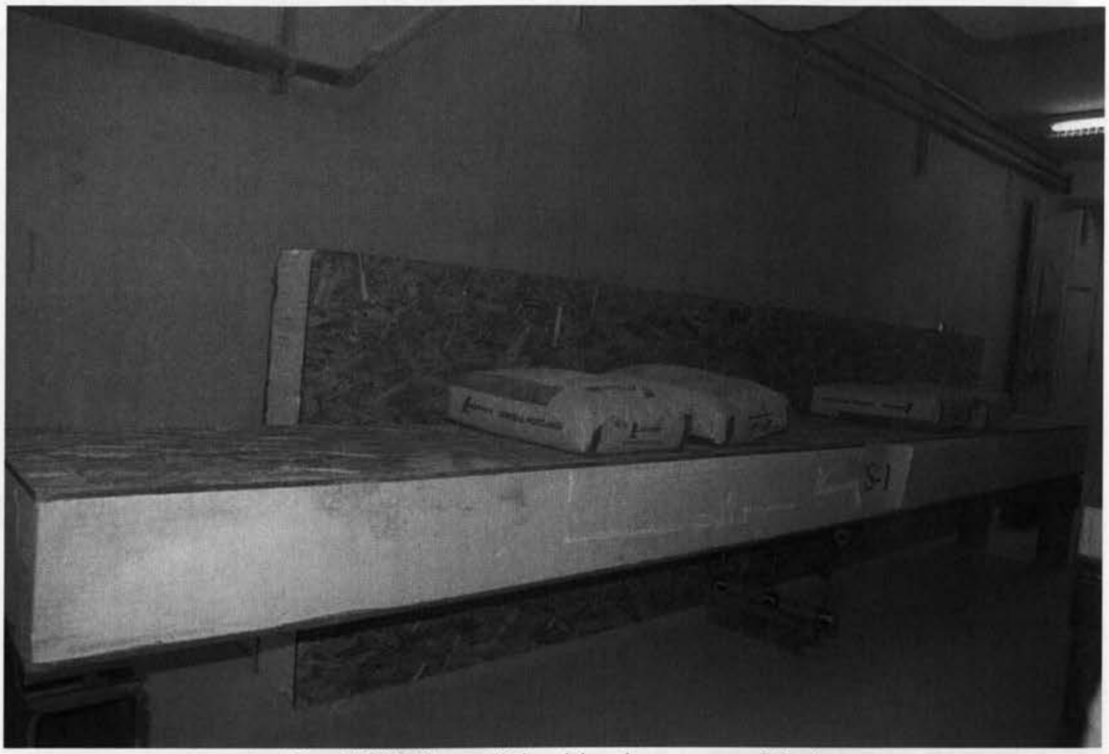

Fig. 3.19 View of dead load over panel S-4

Fig. 3.20 View of dead and live load over panel S-4

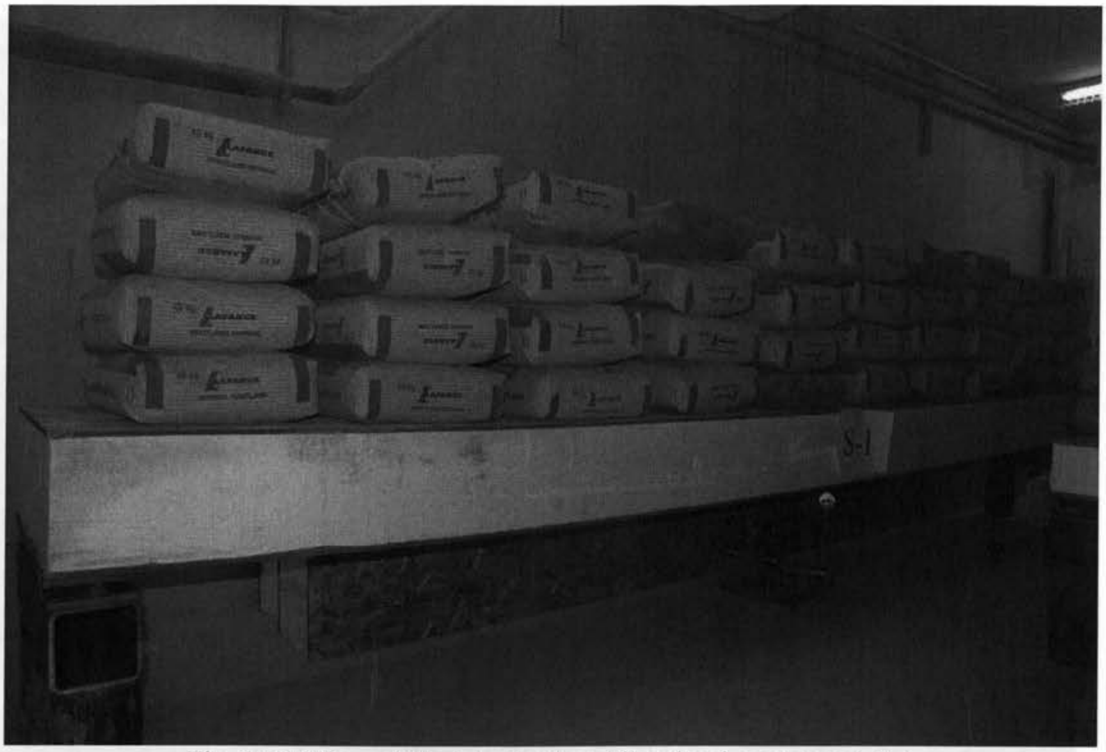

Fig. 3.21 View of double dead and live load over panel S-4

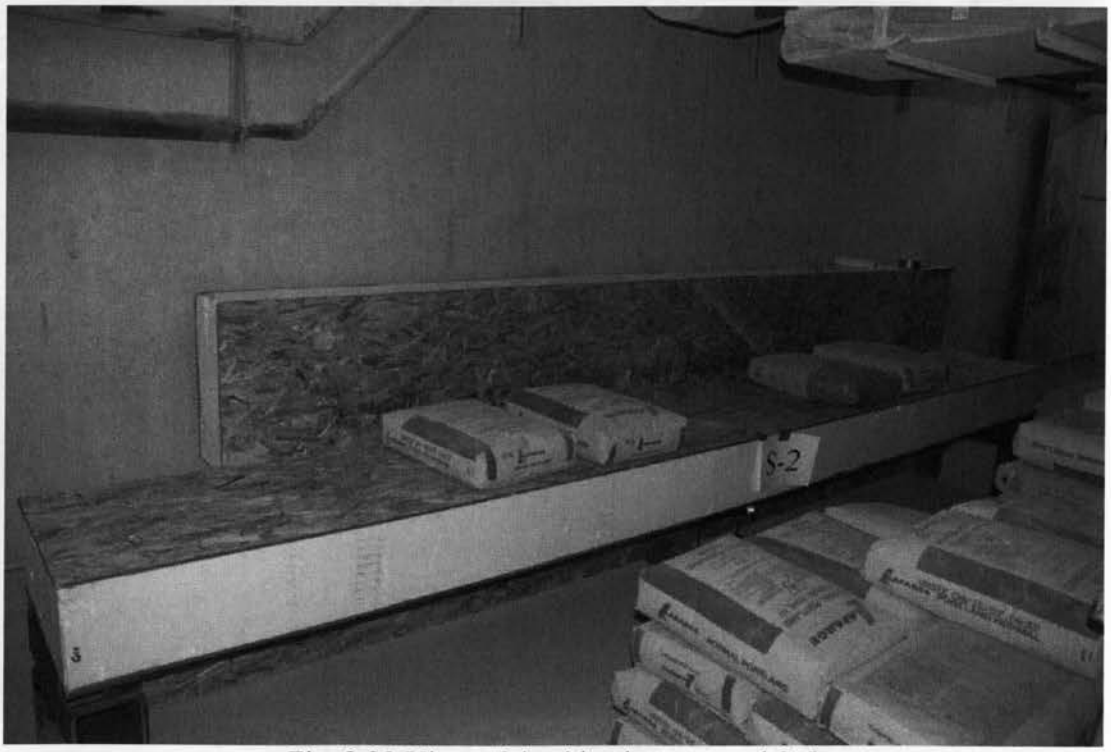

Fig. 3.22 View of dead load over panel S-5

Fig. 3.23 View of dead and live load over panel S-5

Fig. 3.24 View of double dead and live load over panel S-5

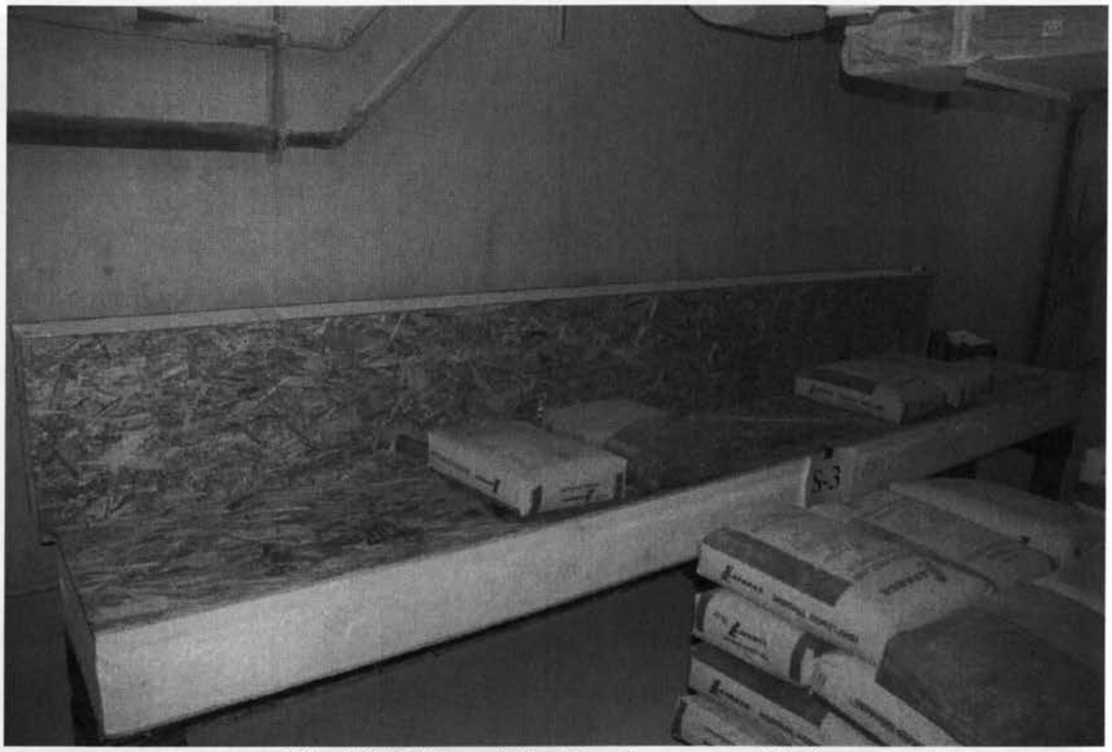

Fig. 3.25 View of dead load over panel S-6

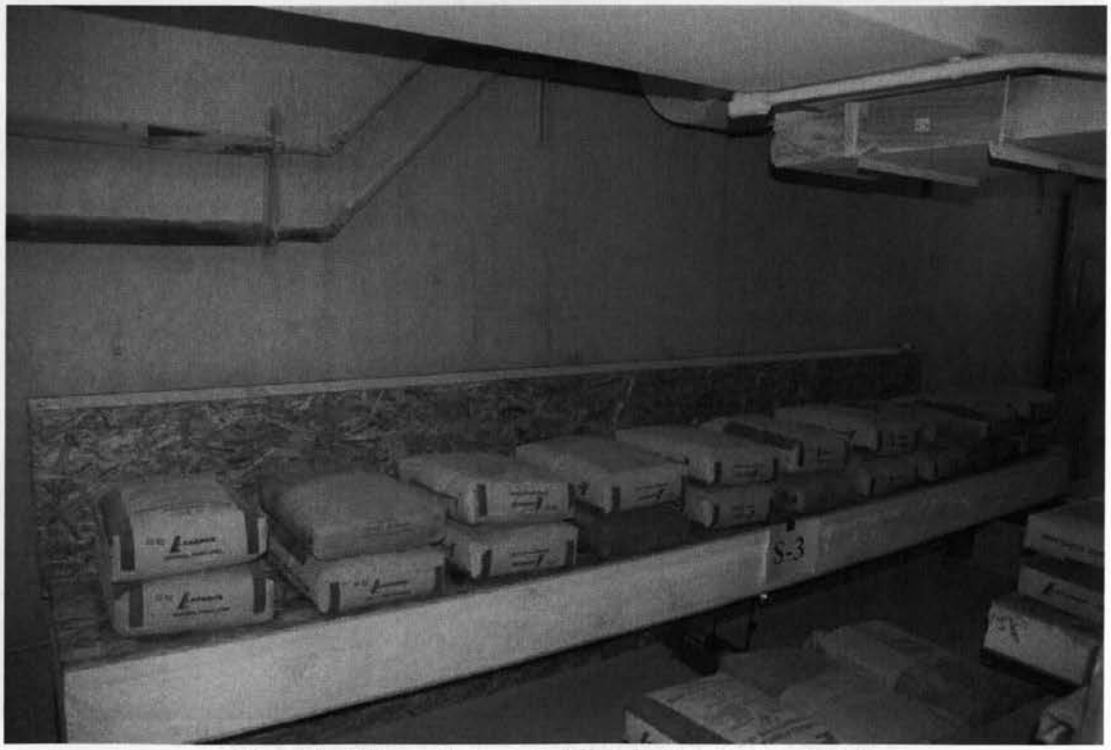

Fig. 3.26 View of dead and live load over panel S-6

Fig. 3.27 View of double dead and live load over panel S-6

3.5.2 Long Term Creep Test

3.5.2.1 Test method for SIP Panels for Long-Term Creep and Recovery Performance

Most recently, Sennah and Butt (2009) and Butt (2008) conducted flexural creep testing on to predict the behavior of structural insulated panels before up to 47 days. The developed SIPs in their study were three panels, S-43, S-44 and S-45, of standard width of 1.2 m but with lengths of 3.0, 3.6, and 4.2, respectively. It should be noted that panels S-43 and S-44 were of 165 mm thickness, while panel S-45 had a thickness of 260 mm. The facing of these developed panels was made of 11 mm Oriented Strand Board (OSB) on both sides of the foam core. The tested panels exhibited an increase in total deflection by an average of 70% after 47 days of flexural creep testing, simulating the snow load sustained over the roof.

The panels were then successfully passed the flexural load testing after being subjected to creep testing. In order to get accurate result for the long-term creep, it was decided to conduct long-term creep testing over a period of 9 months.

Flexure-creep is defined as deflection under constant load over period of time beyond the initial deformation due to the application of the load. ASTM C 480-62, *Standard Test Methods of for Flexural Creep of Sandwich Construction*, (1988) covers the determination of the creep rate of sandwich panels under constant flexural load. A typical setup for this test consists of a simply-supported panel loaded by uniformly distributed loads along the panel. Thus, the SIPs are subjected to one-dimensional flexure thereby minimizing the influence of transverse stiffness on the study results. Specified flexure-creep load is applied and mid-span instantaneous deflection is recorded using dial gauges. The averaged measured mid-span deflection readings can then be used to develop the average deflection-time history for each tested specimen. Figure 3.27 schematically defines the immediate deflection (Δ_0) as the deflection at time t = 0 immediately after the application of the load. The final deflection Δ_f is defined as the deflection immediately before the removal of the load at the end of the test period. Figure 3.27 shows the definitions of critical points in deflection-time relationship for a typical creep curve as follows:

Δ_0 = immediate deflection after application of full load

Δ_i = Deflection at time t_i

Δ_f = Final deflection before removal of the sustained load

Δ_u = Deflection immediately after removal of load (unload)

Δ_{u24} = Deflection 24hours after removal of load

Δ_{u48} = Deflection 48 hours after removal of load

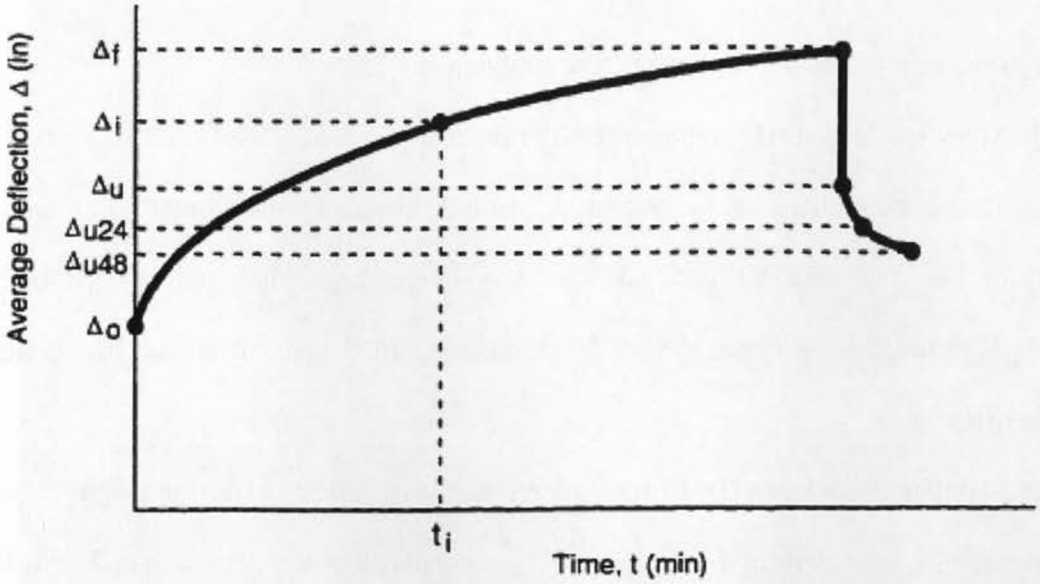

Figure 3.28 Typical deflection-time history for long-term flexural creep test

In case of test method for Long-term creep of SIPs, the 2007 NRC/CCMC Technical Guide specified a creep and recovery test for roof panels in Appendix II. It should be noted that the Guide specifies at least three panels to be tested to evaluate the design. Also, the Guide specifies a 0.5 kPa dead load simulating the weight of superimposed finished roofing and ceiling materials. The live load is expected to be the anticipated snow and rain loads for the anticipated geographical areas. In this study, a roof live load is assumed to be 1.9 kPa similar to the floor live load in residential construction. The test procedure conducted for Long-term creep and recovery performance of SIP's are as follows.

Step 1: Measure the moisture content of wood members at a sufficient number of points to give a representative picture of the overall moisture condition of the members at the time of the test. If members are not accessible, moisture contents may be obtained shortly after the test.

Step 2: Take zero deflection readings before applying any load.

Step 3: Apply test load (D+L), representing the superimposed dead load and live load, at the uniform rate without shock to the system. At the conclusion, following a full five minutes (300 s) for the deflection to stabilize, take the deflection readings. Photograph the assembly.

Step 4: Measure the deflections every 30 minutes up to 6 hours from the beginning of loading (Step 3).

Step 6: Maintain these loads (D+L) for 9 month and take deflection readings every day for the first month, take deflection readings every other day for the second month and take deflection readings every week for the rest (7 month). Photograph the assembly.

Step 7: Remove test load (D+L) and take deflection readings five minutes (300 s) after its complete removal.

Step 8: Measure the deflections every 30 minutes up to 6 hours from the beginning of unloading loading (Step 7) and take deflection readings every day for 3 weeks. Photograph the assembly.

3.5.2.2 Description of Test Specimens and Applied Loads

Panels tested earlier for short-term creep were used in this long-term creep testing as listed in Table 3.1. The support system was identical to that used for short-term creep testing. Also, the applied D+L loading and test setup was exactly the same as those for the short term creep. Figures 3.28 to 3.31 show views of the tested panels under sustained D+L loading for long-term creep.

Figure 3.29 View of panels S-1 during long-term creep test

Figure 3.30 View of panels S-2 during long-term creep test

Figure 3.31View of panels S-3 during long-term creep test

3.32 Views of panels S-4, S-5 and S-6 during long-term creep test

3.5.3 Flexural Failure Test

Following the long-term creep testing, the six panels were then tested to-complete-collapse under flexural loading per ASTM E72-02 test method to determine their structural behavior and ultimate load carrying capacities.

3.5.3.1 Test method for SIP Panels in Flexural Failure

Figure 3.32 shows view of the test setup for flexural loading test per ASTM E72-02 test method. The panel was supported over two steel rollers of 25.4 mm diameter and 600 mm length, with a 600×150×12 mm steel plate between each supporting roller and the specimens. Also, similar steel plates were inserted between the steel rollers and the supporting steel pedestal resting on the laboratory strong floor. The loading was applied at the quarter points from the jack using a 152 ×152×12.7 HSS beam of 2.6 m length. The spread beam was resting on a supporting assembly that rested over the panel at the quarter points. This supporting assemble consisted of a 25.4 mm steel roller, and a 150×150×12 mm steel base plate and a 102 ×102×6.4 HSS of 600 mm length. The weight of this loading system is 2.0 kN. Figure 3.33 shows view of the test setup for panel S-1, while Figure 3.34 shows enlarged view of the roller support system used in this testing.

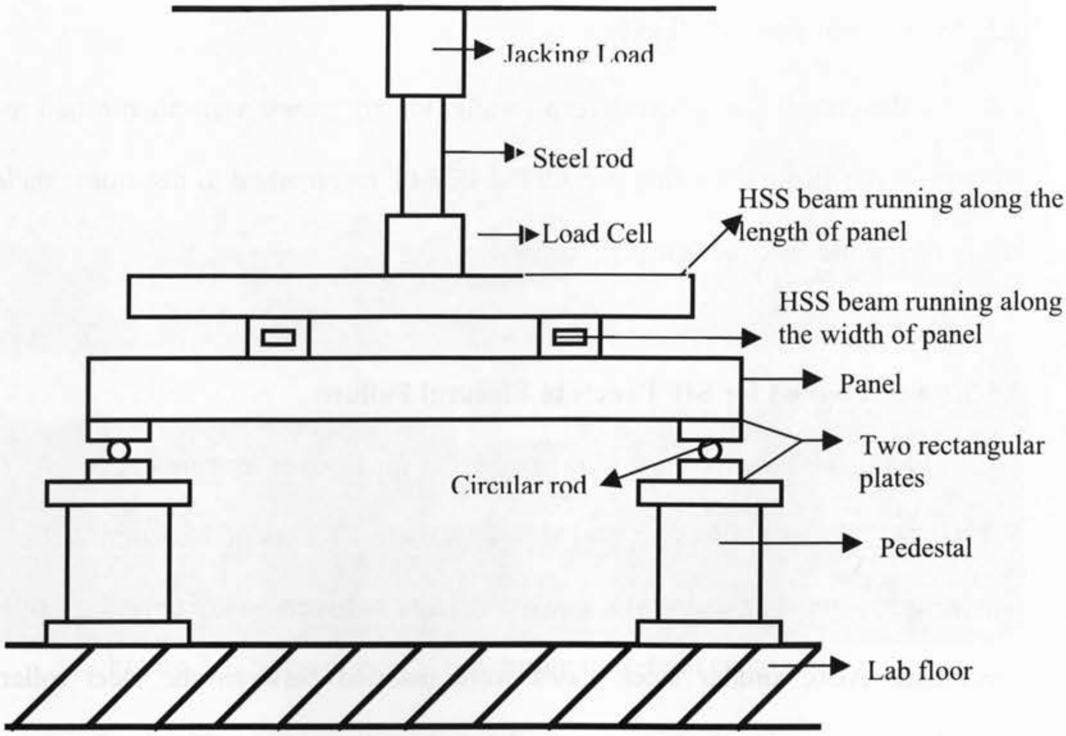

Figure 3.33 Schematic view of the test setup for flexural failure testing

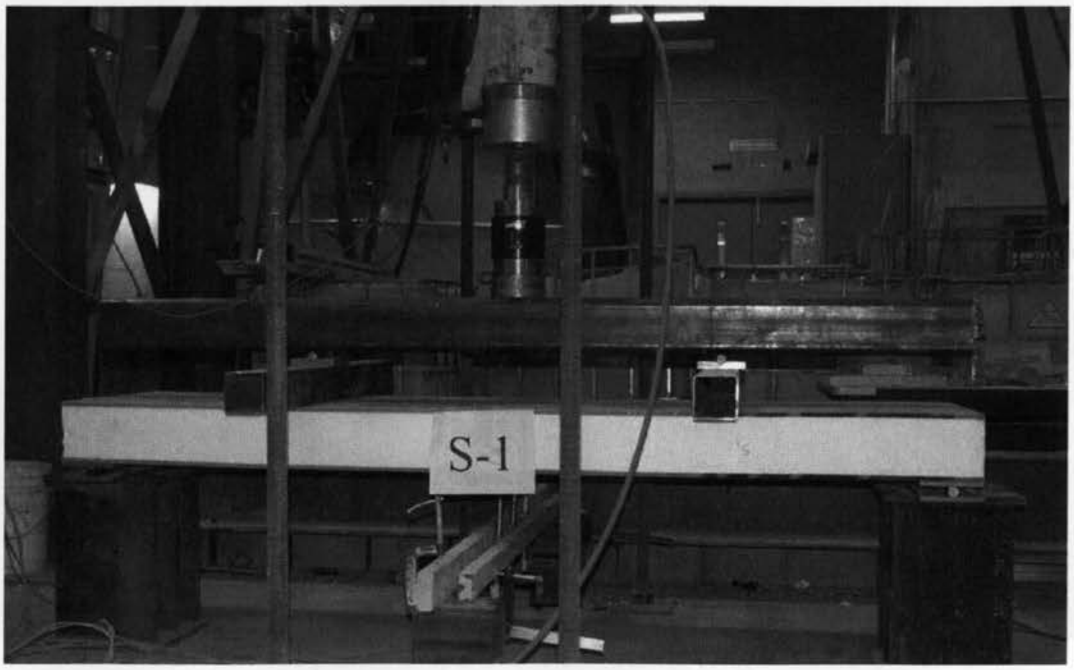

Figure 3.34 View of specimen S-1 before flexural load testing

Figure 3.35 Enlarged view if the roller support of the tested panels

3.5.3.2 Instrumentation for Flexure Failure Test

Linear Variable Displacement Transducers, (LVDTs) of 9.84 mm electrical stroke was used to measure deflection in all panels at the mid-span location and at the 2" distance from the two panel edges. Figure 3.35 shows view of the LVDTs installed under the panel at mid-span location. A universal flat load cell of 222 kN (50,000 Ib) capacity was used to measure the jacking loads applied on all the panels. During testing, the data from the LVDTs, strain sensors and load cell were captured by a test control software (TCS) using a SYSTEM 5000 data acquisition unit. The test control software, TCS, is a powerful tool developed specially for acquiring, reducing and analyzing the dynamic analog data captured using the SYSTEM 5000. It simplifies the process for collecting and converting data captured by strain gauges and LVDT's. The TCS was adjusted to sample the data at rate of one reading per second during the loading test.

Figure 3.36 View of the LVDTs under a tested panel

3.5.3.3 Flexure Failure Test Procedure

A test set-up was prepared for each specimen at the structures laboratory of Ryerson University. Linear Variable Displacement Transducers (LVDTs) were installed at the mid-span location under the panel to record deflection. The available MTS system-5000 data-acquisition system was used to capture reading from sensors. For each panel, jacking load was applied continuously in a slow rate till failure. Visual inspection was conducted at each load increment during the test to record any change in structural integrity of the sandwich panel. The test was terminated after specimen failure. The test stopped when the jacking load was not increasing while panel deflection was increasing by continuous pressing the pump handle. Test data were then used to draw the load-deflection relationships for each panel. Mode of failure of each panel was also recorded.

CHAPTER IV

EXPERIMENTAL RESULTS

4.1 General

This chapter presents the experimental results of the tested panels for (i) short-term creep, (ii) long-term creep, and (iii) flexural behavior and ultimate load carrying capacity. Structural qualification criteria for tested panels as set by test methods, codes and standards are discussed. Conclusions regarding structural qualification of the tested panels of being as good as the structural capacities of the conventional joist/stud building system are then drawn.

4.2 Short Term Creep Result

4.2.1 Results for the 8' long panels

Three identical panels of 8' long, named as S-1, S-2 and S-3, were tested for short-term creep recovery performance. Three dial gauges were installed at the mid-span location of each panel, one at the centre and two at the free edges of the panel cross-section. The loading steps shown in Figure 3.8 were followed as shown in Figures 3.10 to 3.12. Tables 4.1, 4.2 and 4.3 present a summary of deflection values recorded from the three dial gauges located at the right edge, the central point and the left edge of the mid-span section of each panel for specimens S-1, S-2 and S-3, respectively. An average value of deflection is shown in the last column of the table and is then used in Tables 4.7 through 4.9 to examine performance criteria for each panel for creep and recovery under flexural loading. It can be observed that all panels passed the performance criteria stated before in this report for deflection under the action of live load since the maximum difference in the deflection measured in Step 4 o (dead load plus live load) and Step 3 (dead load only) does not exceed 1/360 of the span. The average deflection for the three identical panels under the action of live load was recorded as 3.32 mm while the permissible live load deflection is 6.35 mm.

To investigate creep deflection criterion, stated in the CCMC Technical Guide, the difference in deflection at the mid-span as measured between Step 6 and Step 4, as well as 25% of that measured in Step 4 (attributable to the creep produced by the dead load plus live load in place for approximately 24 hours) were calculated as shown in Table 4.8. Since the former was 0.373 mm while the later was 1.113 mm, it can be concluded that panels S-1 to S-3 meet the creep deflection requirements. To investigate recovery from creep criterion, the lack of recovery determined by the maximum difference in the deflections measured in Step 3 (dead load only) and that measured in Step 7 (on removal of the live load) shall not exceed 1/1440 of the span. It can be observed that the average lack of recovery from creep for panels S-1 to S-3 was calculated in Table 4.9 as 0.59 mm while the permissible value is 1.59 mm. As such, the tested panels meet the recovery from creep criterion.

4.2.2 Results for the 16' long panels

Three identical panels of 16' long, named as S-4, S-5 and S-6, were tested for short-term creep recovery performance. Three dial gauges were installed at the mid-span location of each panel, one at the centre and two at the free edges of the panel cross-section. The loading steps shown in Fig. 3.8 were followed as shown in Figures 3.13 through 3.15. While Tables 4.4, 4.5 and 4.6 present a summary of deflection values recorded from the three dial gauges located at the right edge, the central point and the left edge of the mid-span section of each panel for specimens S-4, S-5 and S-6, respectively. An average value of deflection is shown in the last column of the table and is then used in Tables 4.7 through 4.9 to examine performance criteria for each panel for creep and recovery under flexural loading. CCMC Technical Guide states that panels pass the performance criterion for the deflection under the action of live load if the maximum difference in the deflection measured in Step 4 o (dead load plus live load) and Step 3 (dead load only) does not exceed 1/360 of the span. However, the average deflection for the three identical panels under the action of live load was recorded as 14.75 mm and the permissible live load deflection calculated as 13.12 mm. One may observe that the live load deflection in column 3 of Table 4.7 was recorded for a simulated uniform live load 2.207 kPa rather than 1.9 kPa. As such, the live load deflection due to 1.9 kPa live load is 0.861 times the recorded deflection (i.e. 12.62 mm for S-4, 12.78 mm for S-5 and 12.69 mm for S-6). This makes the actual deflection for the sake of comparison with the permissible deflection as 12.70 mm. As such, the 16' long panels are qualified for live load deflection criterion.

To investigate creep deflection criterion, stated in the CCMC Technical Guide, the difference in deflection at the mid-span as measured between Step 6 and Step 4, as well as 25% of that measured in Step 4 (attributable to the creep produced by the dead load plus live load in place for approximately 24 hours) were calculated as shown in Table 4.8. Since the former was 2.23 mm while the later was 4.83 mm, it can be concluded that the 16' long panels meet the creep deflection requirements.

To investigate recovery from creep criterion, the lack of recovery determined by the maximum difference in the deflections measured in Step 3 (dead load only) and that measured in Step 7 (on removal of the live load) shall not exceed 1/1440 of the span. It can be observed that the average lack of recovery from creep for panels S-4 to S-6 was calculated in Table 4.9 as 1.97 mm while the permissible value is 3.28 mm. As such, the tested panels meet the recovery from creep criterion.

Table 4.1: Recorded test data for panel S-1

Creep test- 8' Panel (S-1)						
Date	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
February 08,2008	4:50 PM	**0**	0	0	0	**0**
February 08,2008	5:00 PM	**DL**	1.13	1.13	1.16	**1.14**
February 08,2008	5:10 PM	**DL+LL**	4.56	4.49	4.31	**4.45**
February 08,2008	6:00 PM	**DL+LL**	4.69	4.6	4.4	**4.56**
February 09,2008	5:00 PM	**DL+LL**	5.03	4.91	4.72	**4.89**
February 09,2008	5:10 PM	**DL**	2.02	1.89	1.7	**1.87**
February 09,2008	5:20 PM	**2(DL+LL)**	9.24	9.54	9.4	**9.39**
February 10,2008	4:50 PM	**2(DL+LL)**	9.75	10.07	9.93	**9.92**
February 10,2008	5:00 PM	**0**	1.39	1.34	1.3	**1.34**

Table 4.2: Recorded test data for panel S-2

Creep test- 8' Panel (S-2)							
Step	**Date**	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
1	February 15,2008	4:00 PM	**0**	0	0	0	**0**
2	February 15,2008	4:10 PM	**DL**	1.12	1.13	1.19	**1.15**
3	February 15,2008	4:20 PM	**DL+LL**	4.35	4.46	4.54	**4.45**
4	February 15,2008	5:10 PM	**DL+LL**	4.48	4.58	4.66	**4.57**
5	February 16,2008	4:10 PM	**DL+LL**	4.75	4.84	4.91	**4.83**
6	February 16,2008	4:20 PM	**DL**	1.74	1.73	1.76	**1.74**
7	February 16,2008	4:30 PM	**2(DL+LL)**	8.87	9.1	9.27	**9.08**
8	February 17,2008	4:30 PM	**2(DL+LL)**	9.39	9.63	9.82	**9.61**
9	February 17,2008	4:40 PM	**0**	1.06	1.04	1.07	**1.06**

Table 4.3: Recorded test data for panel S-3

Creep test- 8' Panel (S-3)						
Date	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
February 22,2008	2:50 PM	**0**	0	0	0	**0**
February 22,2008	3:00 PM	**DL**	1.15	1.1	1.11	**1.12**
February 22,2008	3:10 PM	**DL+LL**	4.49	4.5	4.41	**4.47**
February 22,2008	4:00 PM	**DL+LL**	4.62	4.6	4.49	**4.57**
February 23,2008	3:00 PM	**DL+LL**	4.81	4.8	4.71	**4.77**
February 23,2008	3:10 PM	**DL**	1.57	1.58	1.54	**1.56**
February 23,2008	3:20 PM	**2(DL+LL)**	9.09	9.33	9.42	**9.28**
February 24,2008	3:20 PM	**2(DL+LL)**	9.89	10.19	10.31	**10.13**
February 24,2008	3:30 PM	**0**	1.09	1.22	1.25	**1.19**

Table 4.4: Recorded test data for panel S-4

Creep test- 16' Panel S-4						
Date	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
February 08,2008	5:05 PM	**0**	0	0	0	**0**
February 08,2008	5:15 PM	**DL**	4.78	4.76	4.77	**4.77**
February 08,2008	5:25 PM	**DL+LL**	19.43	19.38	19.49	**19.43**
February 08,2008	6:15 PM	**DL+LL**	19.75	19.69	19.79	**19.74**
February 09,2008	5:15 PM	**DL+LL**	21.05	20.97	21.04	**21.02**
February 09,2008	5:25 PM	**DL**	7.09	6.97	6.9	**6.99**
February 09,2008	5:35 PM	**2(DL+LL)**	35.75	36.2	36.44	**36.13**
February 10,2008	5:35 PM	**2(DL+LL)**	40.38	41.02	41.44	**40.95**
February 10,2008	5:45 PM	**0**	2.46	3.05	3.29	**2.93**

Table 4.5: Recorded test data for panel S-5

Creep test- 16' Panel (S-5)						
Date	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
February 15,2008	4:30 PM	**0**	0	0	0	**0**
February 15,2008	4:40 PM	**DL**	4.46	4.43	4.45	**4.45**
February 15,2008	4:50 PM	**DL+LL**	19.36	19.28	19.22	**19.29**
February 15,2008	5:40 PM	**DL+LL**	19.8	19.72	19.65	**19.72**
February 16,2008	4:40 PM	**DL+LL**	20.95	20.86	20.77	**20.86**
February 16,2008	4:50 PM	**DL**	6.43	6.39	6.26	**6.36**
February 16,2008	5:00 PM	**2(DL+LL)**	40.53	40.89	41.29	**40.90**
February 17,2008	5:00 PM	**2(DL+LL)**	43.94	44.48	44.54	**44.32**
February 17,2008	5:10 PM	**0**	4.72	4.71	4.57	**4.67**

Table 4.6: Recorded test data for panel S-6

Creep test- 16' Panel (S-6)							
Step	**Date**	**Time**	**Load**	**Dial 1 (mm)**	**Dial 2 (mm)**	**Dial 3 (mm)**	**Average**
1	February 22,2008	3:20 PM	**0**	0	0	0	**0.00**
2	February 22,2008	3:30 PM	**DL**	4.56	4.56	4.53	**4.55**
3	February 22,2008	3:40 PM	**DL+LL**	19.23	19.22	19.43	**19.29**
4	February 22,2008	4:30 PM	**DL+LL**	19.64	19.63	19.86	**19.71**
5	February 23,2008	3:30 PM	**DL+LL**	20.48	20.49	20.8	**20.59**
6	February 23,2008	3:40 PM	**DL**	6.32	6.35	6.35	**6.34**
7	February 23,2008	3:50 PM	**2(DL+LL)**	40.48	41.18	42.23	**41.30**
8	February 24,2008	3:50 PM	**2(DL+LL)**	43.49	44.3	45.09	**44.29**
9	February 24,2008	4:00 PM	**0**	5.01	5.1	4.98	**5.03**

Table 4.7: SIP qualification for deflection under the action of live load

Panel #	Panel length (mm)	Step 4 – Step 3 (D+L) – (D)	Deflection limit = span/360	Test results
S-1	2438.4-152.4 = 2287	3.31	6.35	Test passed
S-2	2438.4-152.4 = 2287	3.30	6.35	Test passed
S-3	2438.4-152.4 = 2287	3.35	6.35	Test passed
S-4	4876.8-152.4 = 4724.4	14.66	13.12	Test passed*
S-5	4876.8-152.4 = 4724.4	14.84	13.12	Test passed*
S-6	4876.8-152.4 = 4724.4	14.74	13.12	Test passed*

* the live load deflection in column 3 is given for a simulated uniform live load using cement bags of 2.207 kPa rather than 1.9 kPa. As such, the live load deflection due to 1.9 kPa live load is 0.861 times the recorded deflection (i.e. 12.62 mm for S-4, 12.78 mm for S-5 and 12.69 mm for S-6.

Table 4.8: SIP qualification for creep deflection under the action of live load

Panel #	Panel length	Step 6	Step 4	Step 6 – Step 4 [(D+L) at t=24hrs] – [(D+L) at t =0]	Deflection limit = 0.25% [(D+L) at t=0]	Test results
S-1	2287	4.89	4.45	0.44	1.11	Test passed
S-2	2287	4.83	4.45	0.38	1.11	Test passed
S-3	2287	4.77	4.47	0.30	1.12	Test passed
S-4	4724.4	21.02	19.43	1.59	4.86	Test passed
S-5	4724.4	20.86	19.29	1.57	4.82	Test passed
S-6	4724.4	20.59	19.29	1.30	4.82	Test passed

Table 4.9: SIP qualification for recovery from creep deflection

Panel #	Panel length (mm)	Step 3 – Step 7 [(D) at t=0] – [(D) at t=24 hrs]	Deflection limit = span/1440	Test results
S-1	2287	0.73	1.59	Test passed
S-2	2287	0.59	1.59	Test passed
S-3	2287	0.44	1.59	Test passed
S-4	4724.4	2.22	3.28	Test passed
S-5	4724.4	1.91	3.28	Test passed
S-6	4724.4	1.79	3.28	Test passed

4.3 Long-Term Creep Result

4.3.1 Results for the 8' long panels

Three identical panels, S-1, S-2 and S-3, or 8' long were tested for long-term creep performance. Chapter III discussed the test setup and test procedure. Figures 4.1, 4.2 and 4.3 show the mid-span deflection-time history for specimens S-1, S-2 and S-3, respectively. It can be observed that the initial deflection for specimen S-1 was 4.94 mm, while the deflection after 271 days was 7.3 mm, an increase of about 48%. It can also be observed the creep recovery was 2.01 mm after 21 days of removing the sustained loading, a decrease in attained deflection of about 72.6%. For specimen S-2, the initial deflection was recorded as 4.31 mm, while the deflection after 271 days was 6.62 mm, an increase of about 53.6%. It can also be observed the creep recovery was 1.89 mm after 21 days of removing the sustained loading, a decrease in attained deflection of about 71.5%. In case of specimens S-3, the creep deflection was 51.52% more that the instantaneous deflection, while the creep recovery was 64.9% less that the creep deflection after 21 days of removing the sustained loading. Equation 2.15 provided the total deflection of the panel under dead and live load including creep effect. This equation is as follows:

$$\Delta_{Total} = K(\Delta_{long\ term}) + \Delta_{short\ term} \quad (4.1)$$

Where $\Delta_{long\ term}$ = immediate deflection under dead load +long-term portion of live loads; K = constant to calibrate the long-term effects of dead load and live load; $\Delta_{short\ term}$ = deflections under short-term portions of design load. For the tested panel $\Delta_{long\ term} = \Delta_{short\ term}$ since it is assumed that the full portion of live load (or snow load) is sustained over the test period. As such, creep constant, K, can be considered the smallest percentage increase in deflection after 9 months of sustained load duration (i.e. K = 0.51).

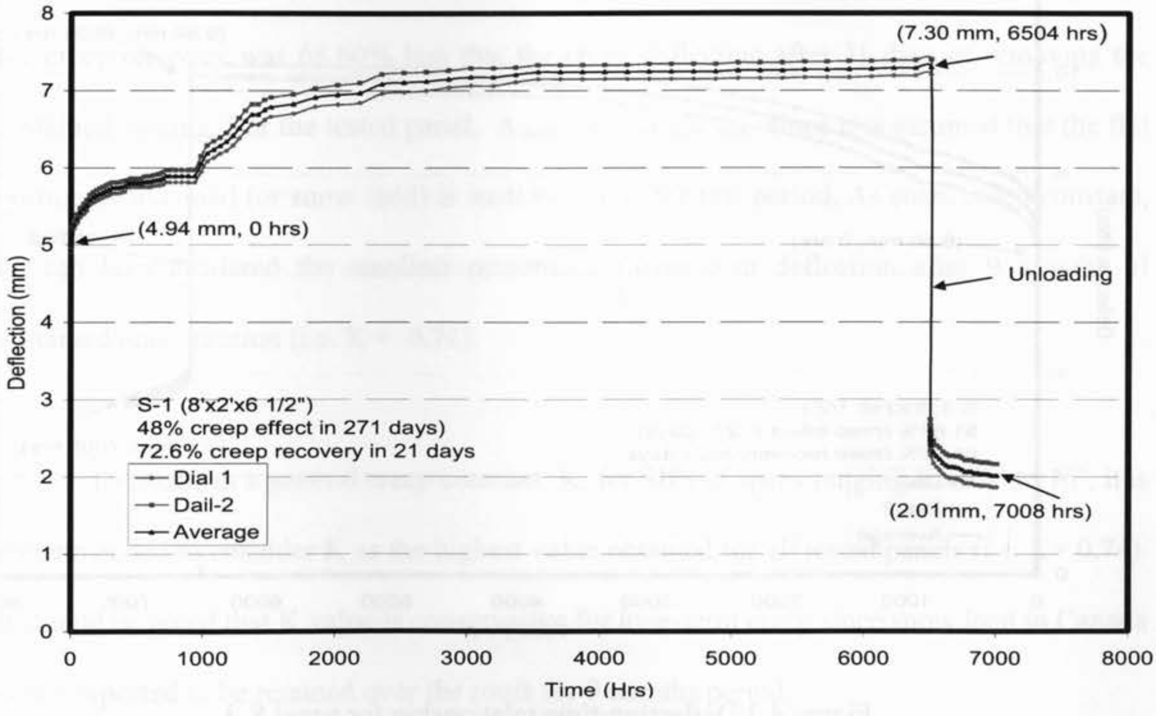

Figure 4.1 Deflection-time relationship for panel S-1

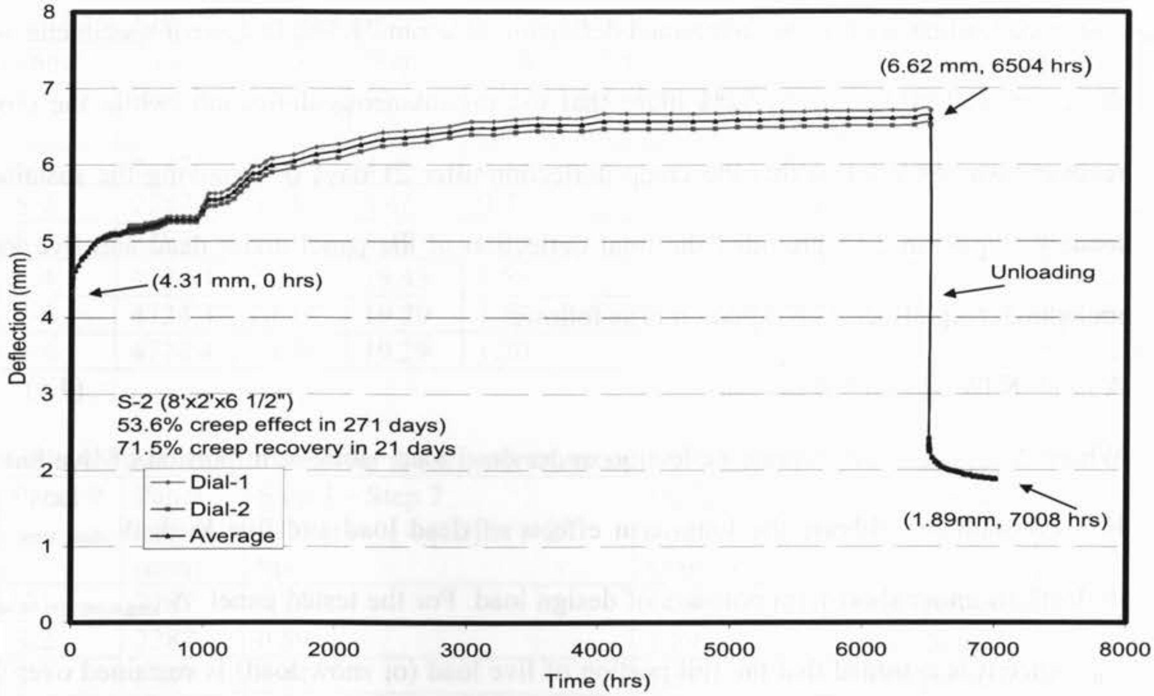

Figure 4.2 Deflection-time relationship for panel S-2

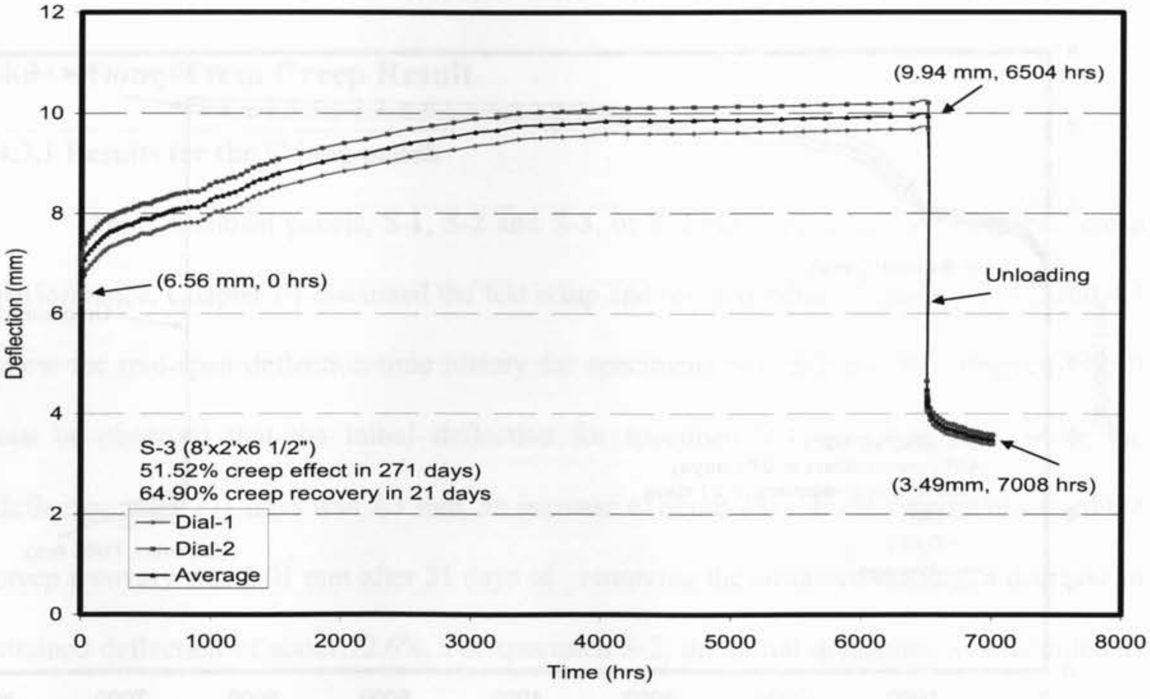

Figure 4.3 Deflection-time relationship for panel S-3

4.3.2 Results for the 16' long panels

Three identical panels, S-1, S-2 and S-3, or 16' long were tested for long-term creep performance. Chapter III discussed the test setup and test procedure. Figures 4.4, 4.5 and 4.6 show the mid-span deflection-time history for specimens S-4, S-5 and S-6, respectively. It can be observed that the initial deflection for specimen S-4 was 19.13 mm, while the deflection after 271 days was 33.23 mm, an increase of about 73.7%. It can also be observed the creep recovery was 11.69 mm after 21 days of removing the sustained loading, a decrease in attained deflection of about 64.8%. For specimen S-5, the initial deflection was recorded as 19.99 mm, while the deflection after 271 days was 34.00 mm, an increase of about 70.10%. It can also be observed the creep recovery was 11.06 mm after 21 days of removing the sustained loading, a decrease in attained deflection of about 67.50%. In case of specimens S-6, the creep deflection was68.10% more that the instantaneous deflection, while the creep recovery was 68.60% less that the creep deflection after 21 days of removing the sustained loading. For the tested panel, $\Delta_{\text{long term}} = \Delta_{\text{short term}}$ since it is assumed that the full portion of live load (or snow load) is sustained over the test period. As such, creep constant, K, can be considered the smallest percentage increase in deflection after 9 months of sustained load duration (i.e. K = 0.71).

To establish a general creep constant, K, for SIPs of spans ranging from 8' to 16', it is recommended to consider K as the highest value obtained for all tested panels (i.e. K= 0.74). It should be noted that K value is conservative for long-term creep since snow load in Canada is not expected to be retained over the roofs for 9 months period.

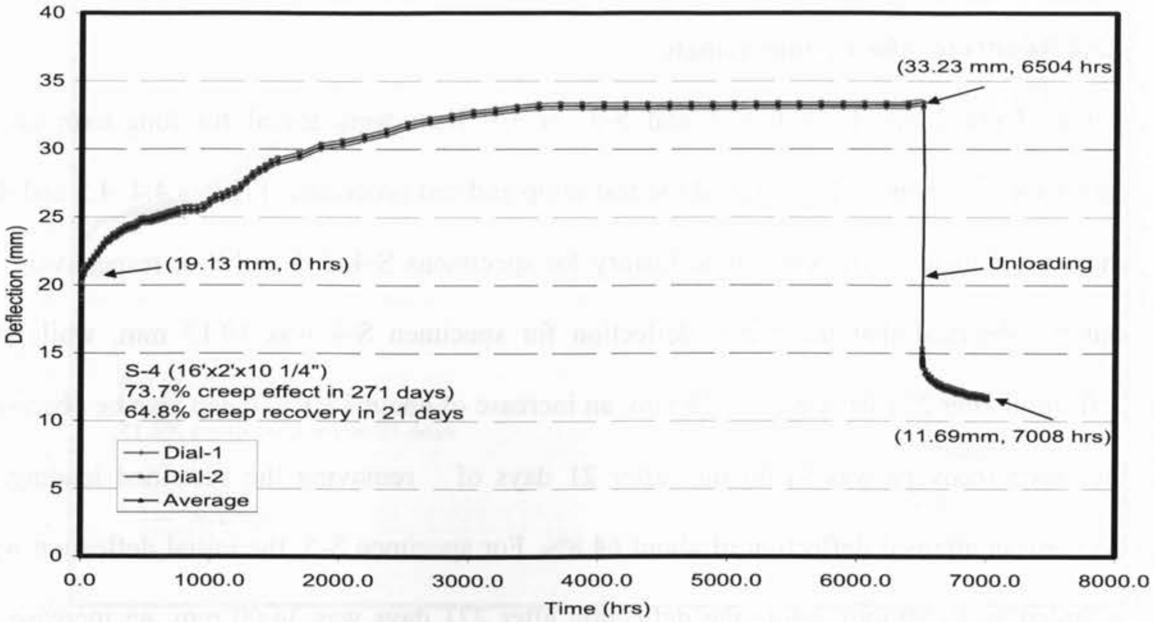

Figure 4.4 Deflection-time relationship for panel S-4

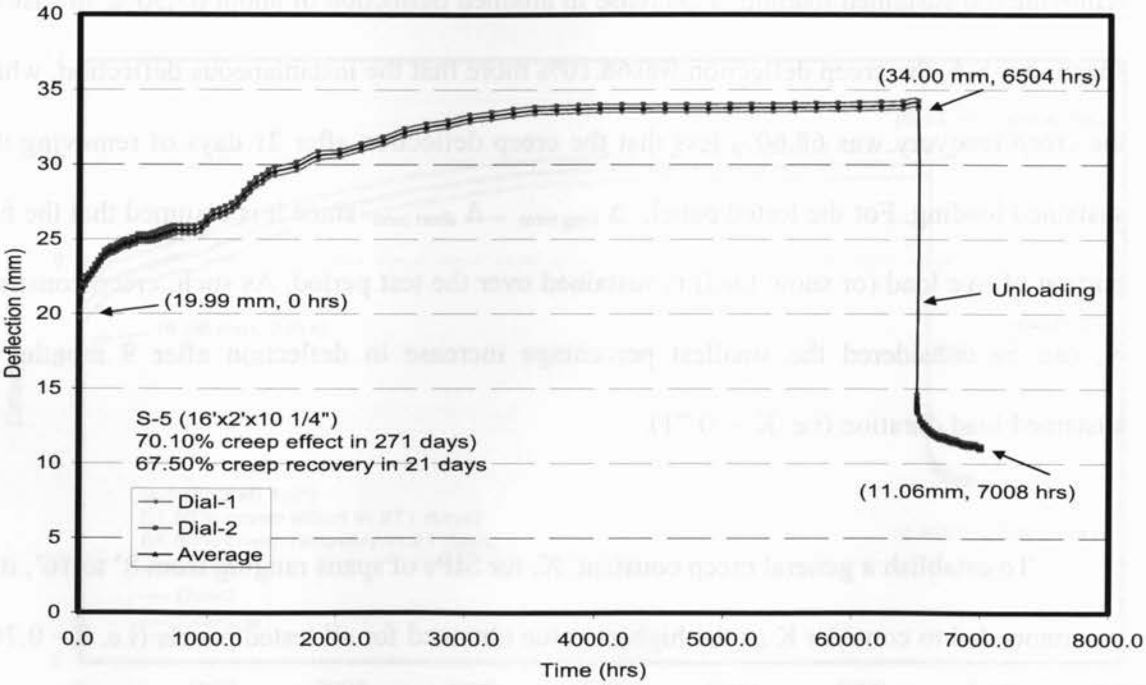

Figure 4.5 Deflection-time relationship for panel S-5

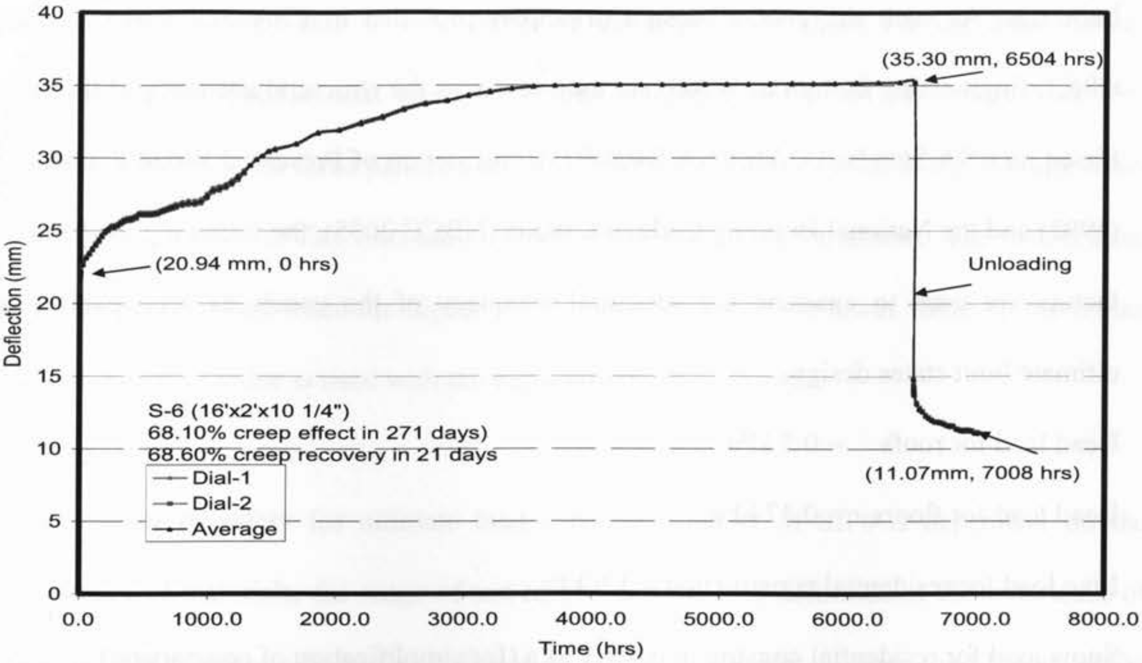

Figure 4.6 Deflection-time relationship for panel S-6

4.4 Flexural Failure Result

4.4.1 General

Following the completion of the long-term creep tests, the specimens were loaded to-failure under increasing flexural loading. Discussion of the experimental results with respect to the structural adequacy of structural insulated panels are then presented, with emphasis on code requirements and available evaluation criteria for ultimate and serviceability limit states design of SIPs.

4.4.2 Code Requirements for the Structural Qualification of the SIPs

The Structural qualifications of the SIPs as specified in NRC/CCMC Technical Guide focuses on them as being "as good as" the structural capacity of conventional wood-frame buildings. As such the general design principles provided in CSA Standard CAN/CSA-O86.1, Engineering Design of Wood, are used to assess the structural adequacy of the panels. Based on CSA Standard CAN/CSA-S406-92, Construction of Preserved Wood Foundations, (1992) and the National Building Code of Canada (NBCC 2005), the following load and load factors are used to examine the structural adequacy of the panels as serviceability and ultimate limit states design.

Dead load for roofs = 0.5 kPa

Dead load for floors = 0.47 kPa

Live load for residential construction = 1.9 kPa

Snow load for residential construction = 1.9 kPa (for simplification of comparison)

Dead load factor = 1.25

Live load factor =1.50

Deflection limit for serviceability = span / 360

The deflection limit of span/360 is intended to limit floor vibration and to avoid damage to structural elements or attached nonstructural elements. This condition may be waived in case of industrial buildings, with span/180 as live load deflection limit when no roof ceiling is provided and with span/240 when ceilings other than plaster or gypsum are used (NBCC Part 9, 2005). It should be noted the for wood construction the deflections limit for the total loads, including dead and live load, is span/180.

The Acceptance criteria for SIPs set forth in ICC-ES AC04 (2004) includes testing three identical panels from each panel size. The average deflection and ultimate load carrying capacity of this panel size will be basically the average of those for the three panels. However, AC04 specifies that when the results of each tested panel vary more than 15% from the average of the three panels, either the lowest test value is used or the average result based on a minimum of five tests may be used regardless of the variations. The results from two tests may be used when the higher value does not exceed the lower value by more than 5% and the lower value is used with the required factors of safety.

Factor of safety for ultimate load carrying capacity of SIPs is dependant on the consistency of materials, the range of test results and the load deformation characteristics of the panel. AC04 generally applies a factor of safety of 3 to the ultimate load based on the average of three tests. However, AC04 provides the following factors of safety applicable to uniform transverse loads, when is the case of the tested panels in this research.

F.S. = 2 for ultimate load determined by bending (facing buckling) failure for allowable live loads up to 958 Pa (20 Psf).

F.S. = 2.5 for ultimate load determined by bending (facing buckling) failure for allowable snow loads.

F.S. = 2.5 for ultimate reaction at failure for all loading conditions

F.S. = 3.0 for ultimate load at shear failure for all loading conditions.

4.4.3 Results for the 8' long panels

In this group, three identical panels were tested to complete collapse. Each panel was of 165 mm (6.5") thickness, 600 mm (2') width and 2.43 m (8') length, with foam-spline connection. Figure 4.7 shows view of panel S-1 before testing, while Figure 4.8 shows view of the permanent deformed shape of the panel after failure. It was observed that the failure mode of the panel was due to diagonal shear failure in the foam core at the support location as depicted in Figures 4.9 and 4.11 at the two fee edges at the same support line. The diagonal crack extended between the top surface of the foam and the adhesive over a panel length between the support and the quarter-point line, causing delamination (debonding) between the top foam-OSB interface and the foam core over the support. Figures 4.9 and 4.10 shows close-up view of this type of failure. It should be noted that noise was heard when approaching failure load and the shear failure was abrupt causing a sudden drop in the applied jacking load as depicted in the load-deflection history shown in Figure 4.12.

Similar failure mode was observed in panel S-2. Figures 4.13 and 4.14 show view of panel S-2 before and after failure, respectively. While Figures 4.15 to 4.17 show views of the failure mode similar to that for panel S-1. Figure 4.18 show the load-deflection relationship for panel S-2. In contract to panels S-1 and S-2, the failure mode of panel S-3 was due to horizontal shear between the top OSB facing and the foam core between the support line and the quarter point load location. Figures 4.19 to 4.21 show views of this panel before and after failure, while Figure 4.22 depicts the jacking load-deflection relationship for the panel.

Figure 4.12 depicts the jacking load-deflection history of panel S-1. The can be observed that the readings for all LVDTs at central and edge points of the mid-span location were almost identical. The design dead load, live load, the sum of dead and live load, and the factored dead and live load values were plotted in the figure to get the sense of the structural adequacy of the panel. It can be observed that linear elastic behavior was maintained at the live load level (i.e. serviceability limit state) and even at the design factored load level (i.e. ultimate limit state). Similar behavior was observed for panels S-2 and S-3 as shown in Figures 4.18 and 4.22, respectively. One may observe that the live load deflection for panels S-1, S-2, and S-4 were 2.25, 2.18 and 2.33 mm, respectively. These values were normalized to the span length and then averaged to be span/1014 which is far below the deflection limit of span/360. One may observe that the total dead and live load deflection for panels S-1, S-2, and S-4 were 2.89, 2.90 and 2.98 mm, respectively. These values were normalized to the span length and then averaged to be span/790 which is far below the deflection limit of span/180. It should be noted that the panel span length considered herein is the panel length of 2.43 m minus 76 mm (3") from each side of the support, which is typical in all panels. It can be observed herein that the deflection of each panel at the live load live is within 15% difference with the average deflection of the three panels. One may consider the total deflection based on equation 4.1 as follows:

$$\Delta_{\text{total}} = \Delta_{\text{short term}} + K\,\Delta_{\text{long term}} = 2.92 + 0.74\text{x}2.92 = 5.08 \text{ mm} \qquad (4.2)$$

This would make the total deflections as span/450 which is less that the deflection limit for total deflection of L/180 for short term loading and L/360 for sustained loading per CAN/CSA-O86-01.

With respect to the ultimate load carrying capacity of the tested panels, it can be observed that the ultimate jacking load was 21.66, 21.22 and 11.44 kN for panels S-1, S-2 and S-3, respectively. It is worth mentioning that the ultimate jacking load for panel S-3 is more that 15% difference with the average jacking load of the three panels. Therefore, its jacking load would be considered as the experimental ultimate load for the panel group.

Thus, ratio of the ultimate jacking load to the design factored load of 1.25D+1.5L is 2.4 for panel group A, which is less than the allowable factor of safety of 3 for shear failure. It should be noted that this ratio was calculated using the jacking load rather without including the weight of the loading system of 2 kN. If the weight of the test loading assembly, this makes the ratio 2.82. In this case, the specified live or snow load on this roof panel must be less than 1.9 kPa. This makes the limiting specified snow load to be 1.76 kPa to maintain a safety factor of 3.

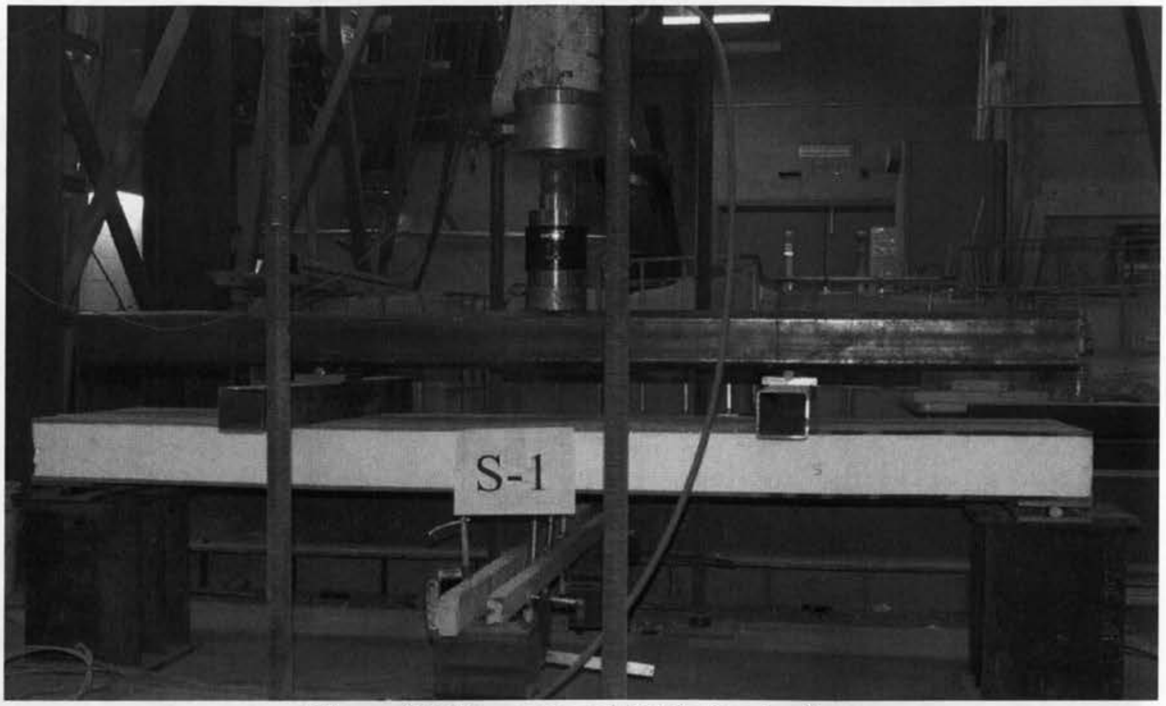

Figure 4.7 View of panel S-1 before loading

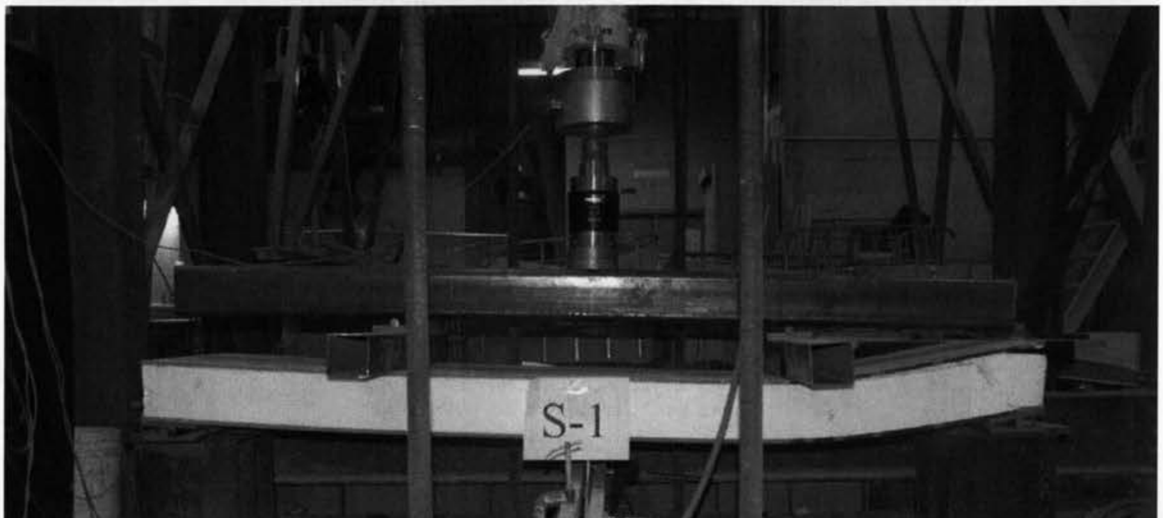

Figure 4.8 View of panel S-1 after failure

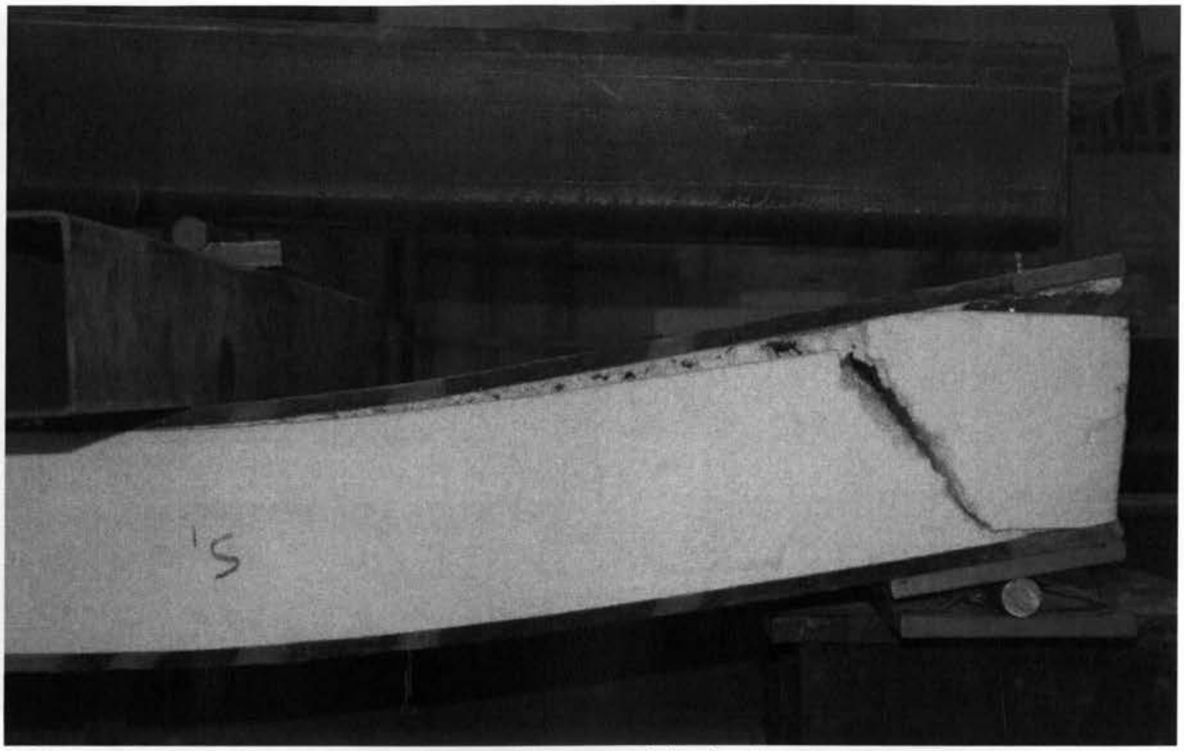

Figure 4.9 Close-up view of the diagonal shear crack in the foam at support of specimen S-1

Figure 4.10 Close-up view of foam-OSB splitting over the support of specimen S-1 at failure

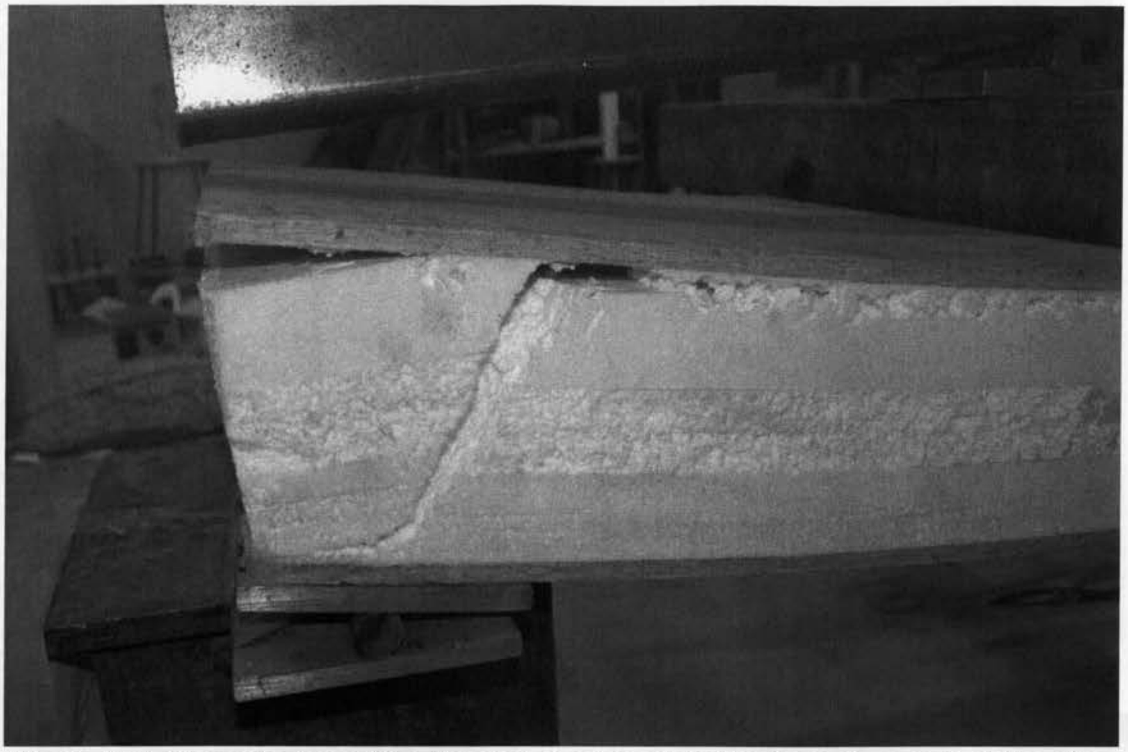

Figure 4.11 Close-up view of the diagonal shear crack in the foam at support at the other free edge of specimen S-1

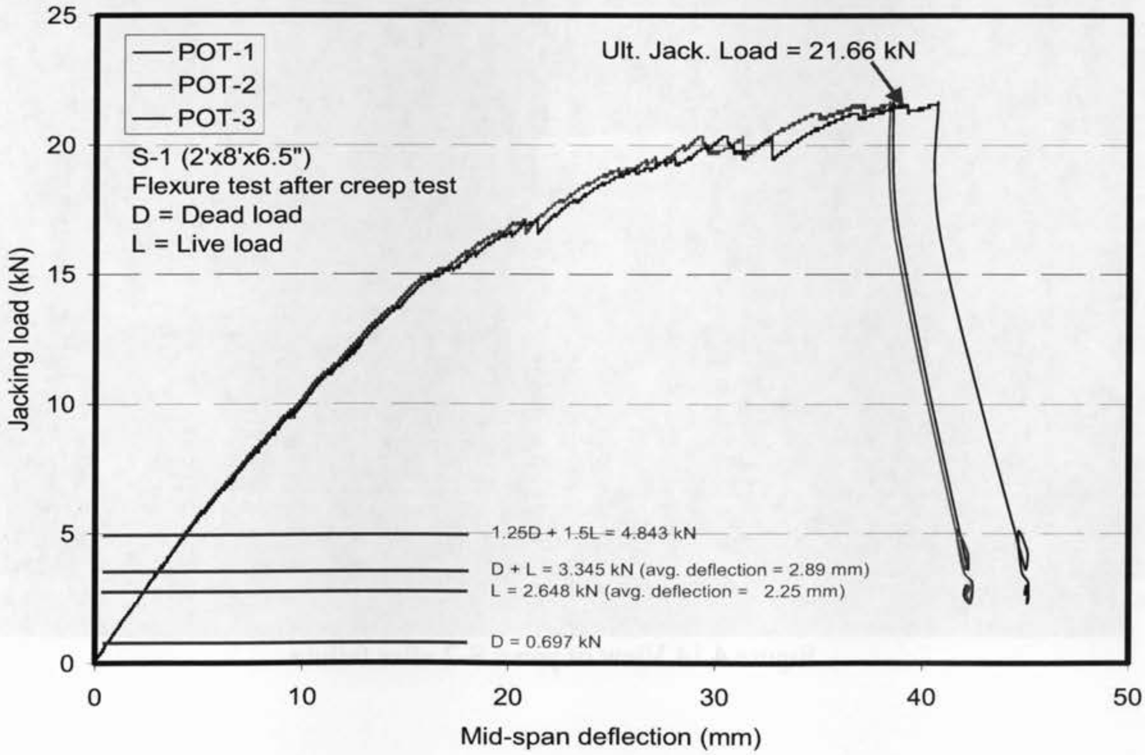

Figure 4.12 Load-deflection relationship for panel S-1

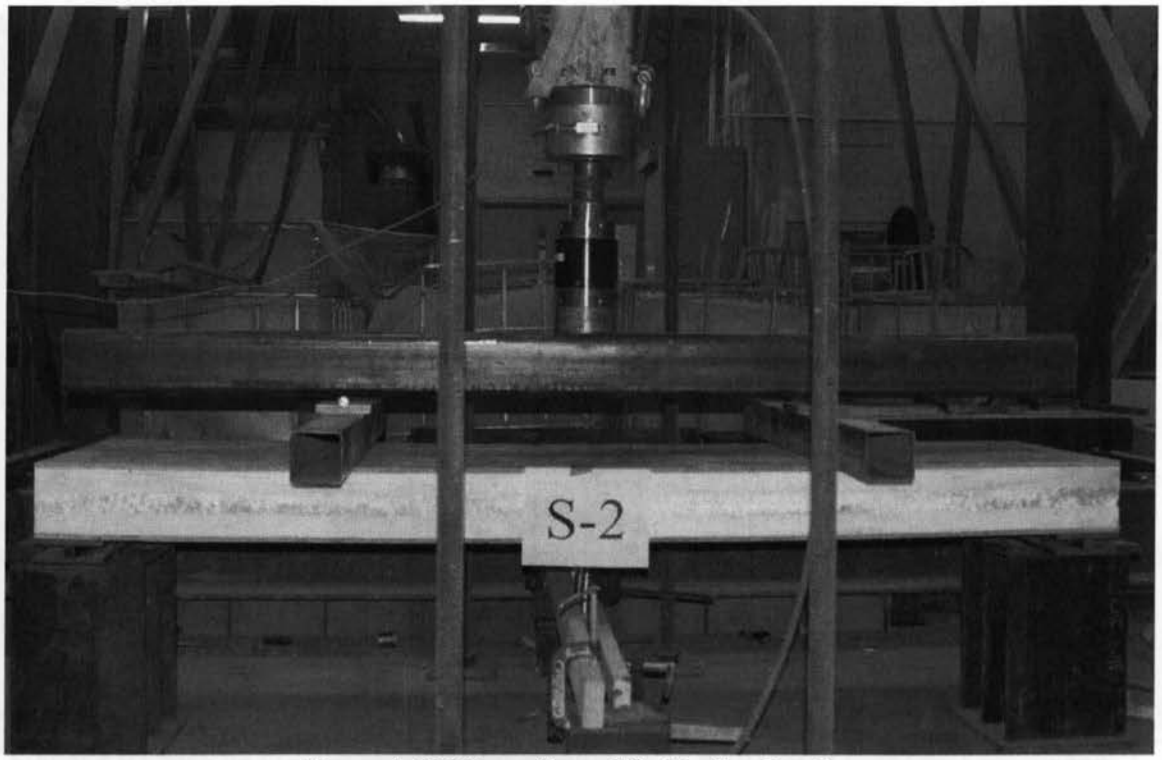

Figure 4.13 View of panel S-2 before loading

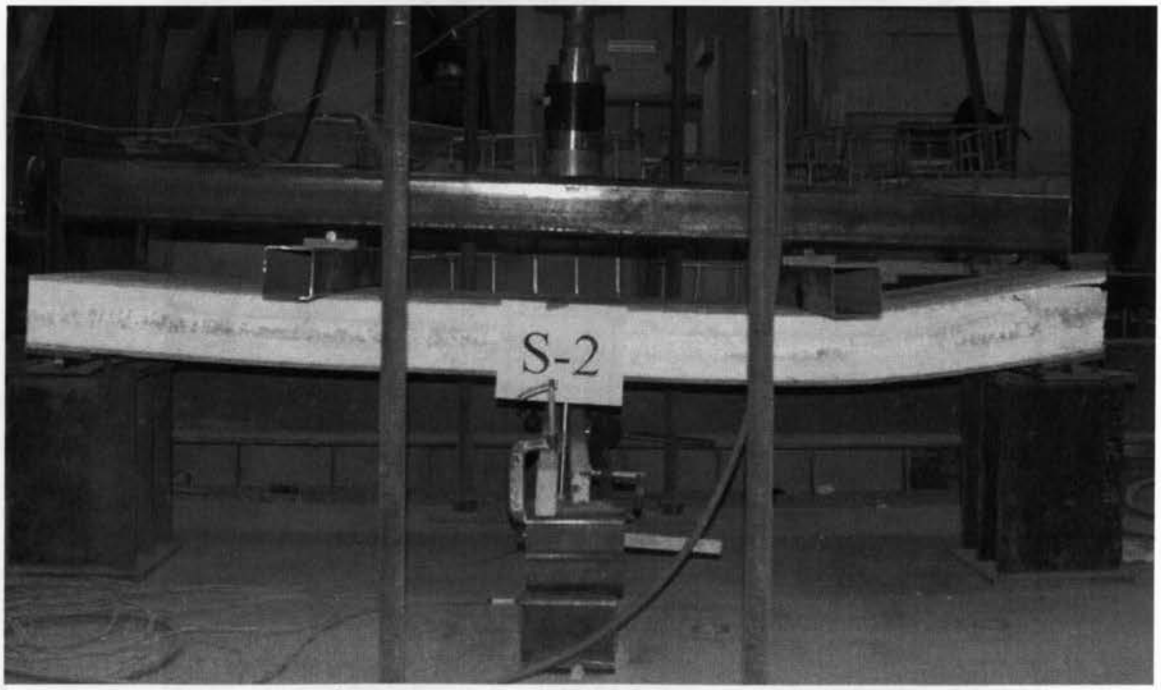

Figure 4.14 View of panel S-2 after failure

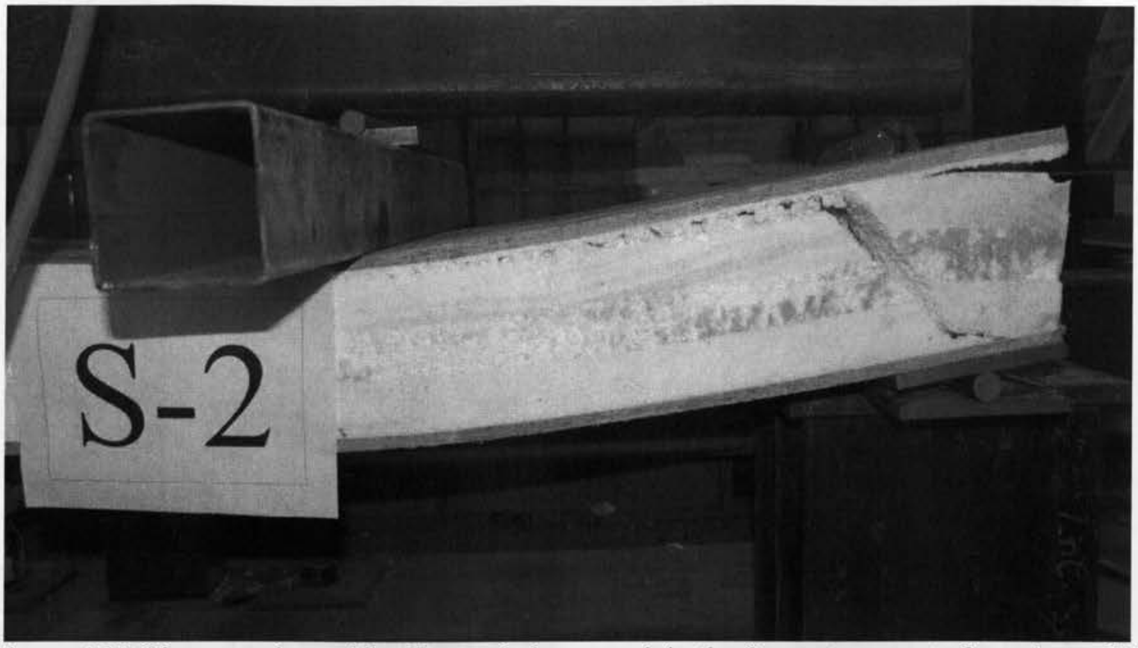

Figure 4.15 Close-up view of the diagonal shear crack in the foam at support of specimen S-2


Figure 4.16 Close-up view of foam-OSB splitting over the support of specimen S-2 at failure

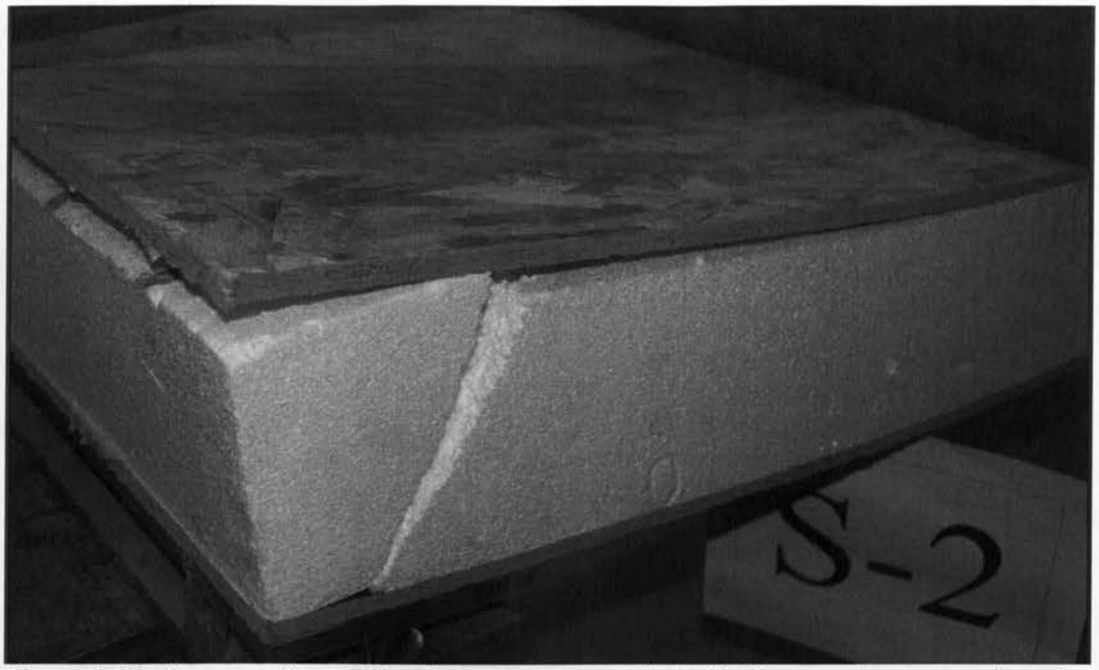

Figure 4.17 Close-up view of the diagonal shear crack in the foam at support at the other free edge of specimen S-2

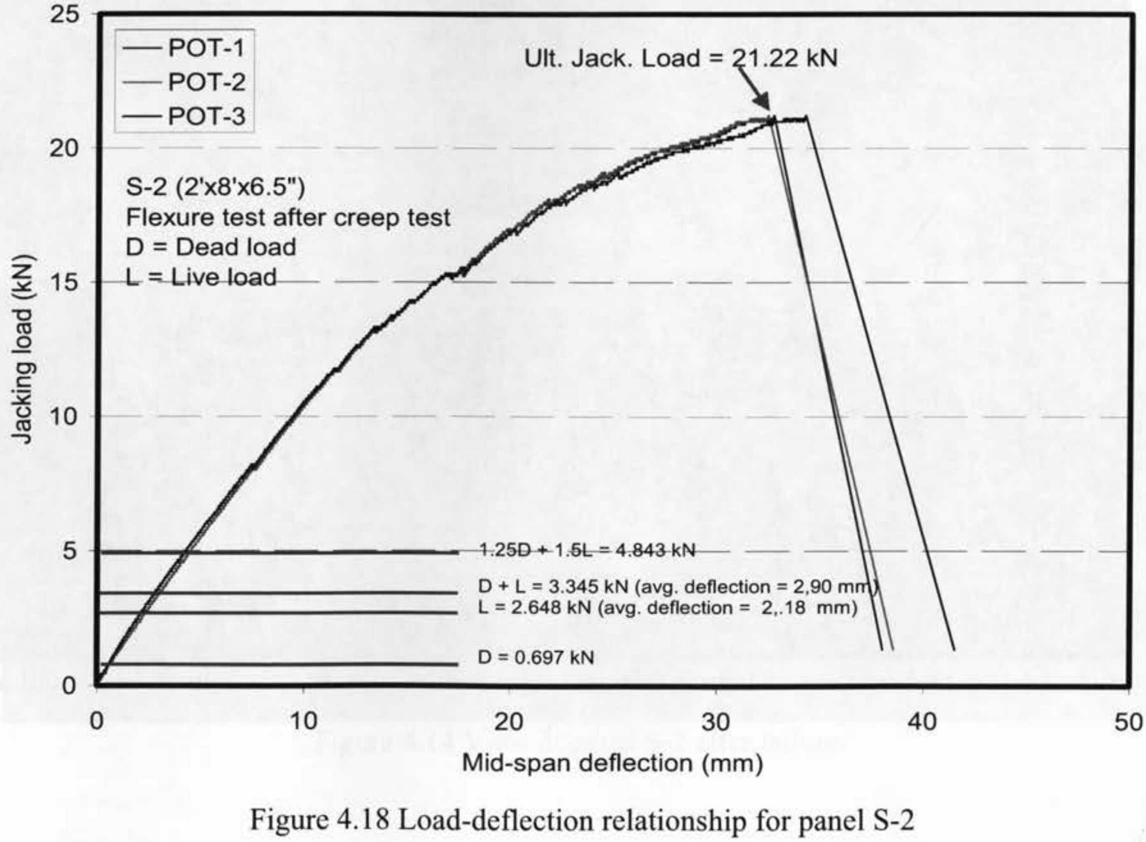

Figure 4.18 Load-deflection relationship for panel S-2

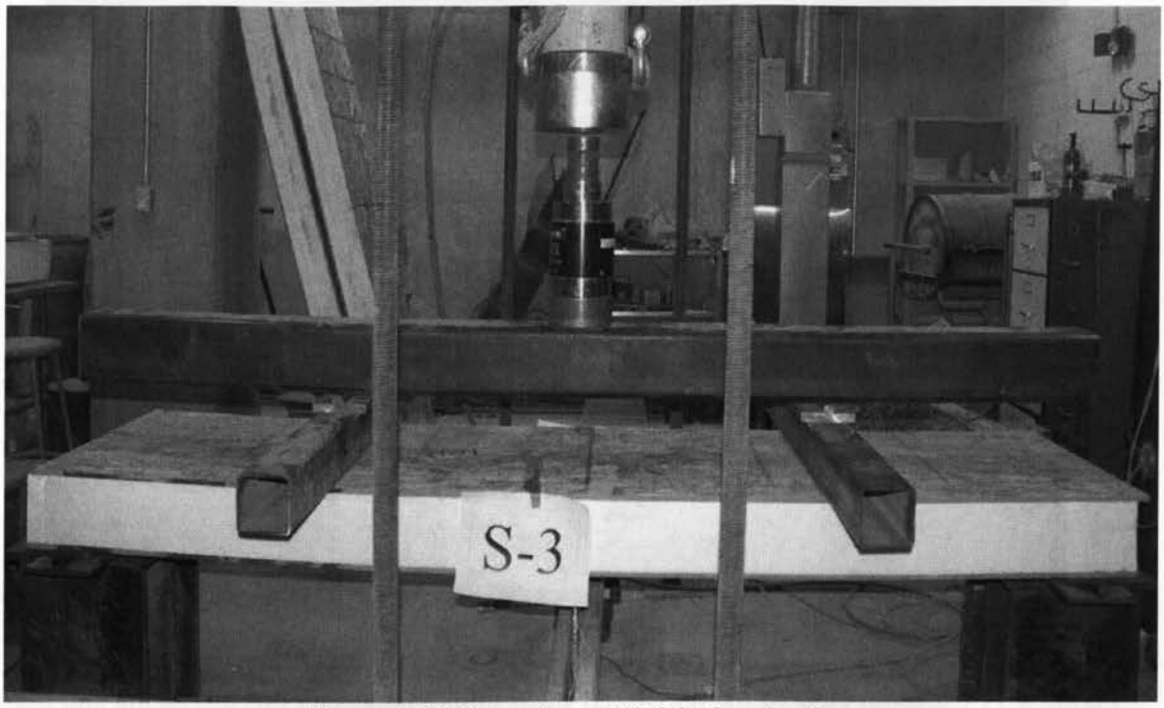

Figure 4.19View of panel S-3 before loading

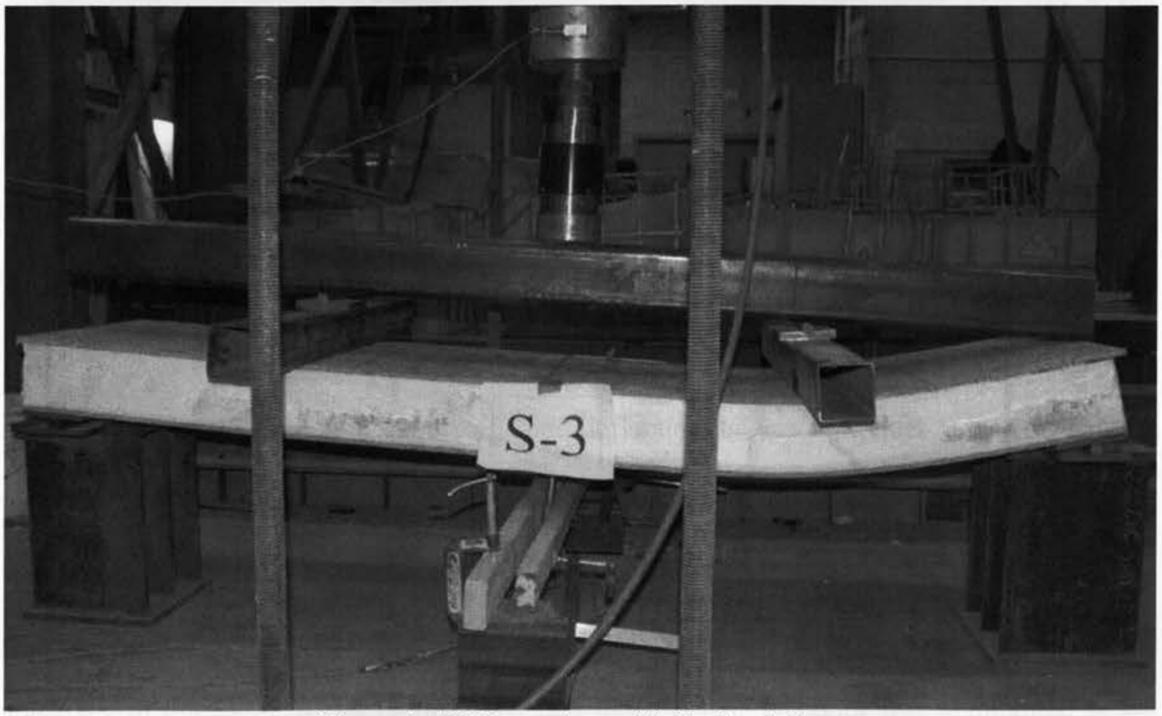

Figure 4.20 View of panel S-3 after failure

Figure 4.21 Close-up view of the horizontal shear failure between OSB top facing and foam core between support and quarter point of specimen S-3

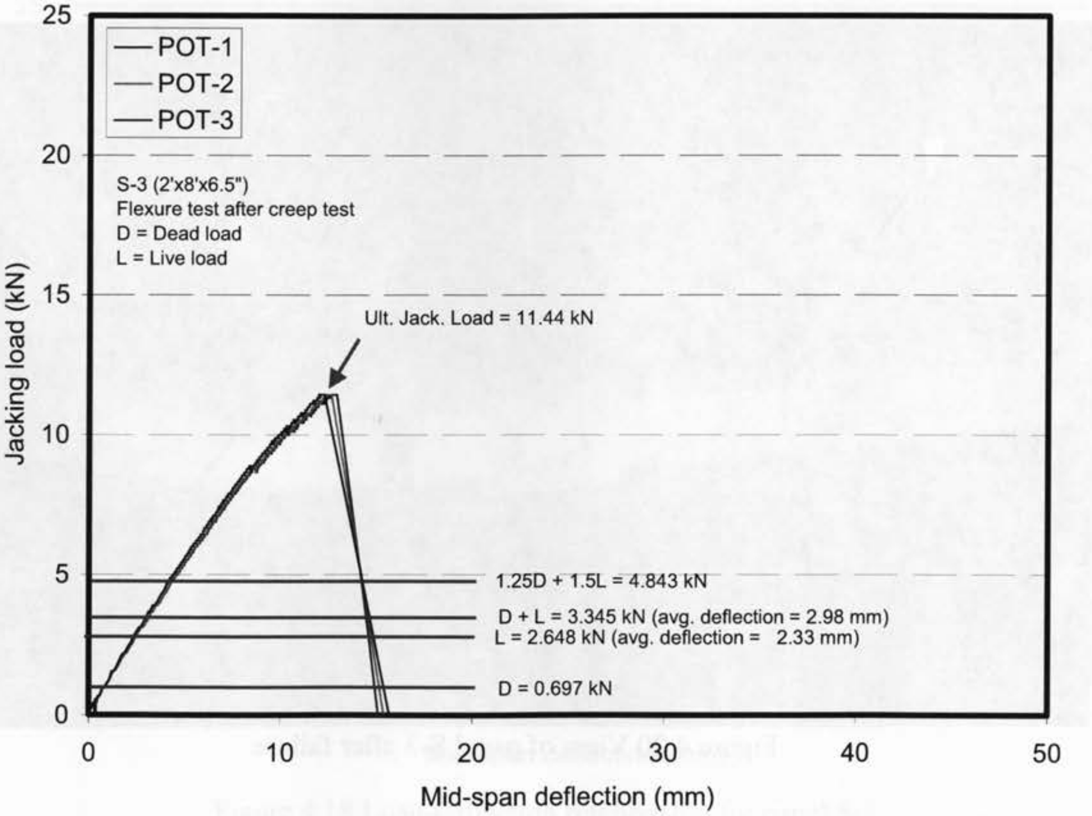

Figure 4.22 Load-deflection relationship for panel S-3

4.4.4 Results for the 16' long panels

In this group, three identical panels were tested to complete collapse. Each panel was of 260 mm (10 ¼") thickness, 600 mm (2') width and 4.9 m (16') length. Figure 4.23 shows view of panel S-4 before testing, while Figure 4.24 shows view of the permanent deformed shape of the panel after failure. It was observed that the failure mode of the panel was due to crushing of OSB top facing at the quarter point load location as shown in Figures 4.25 and 4.26. It should be noted that noise was heard when approaching failure load and the shear failure was abrupt causing a sudden drop in the applied jacking load as depicted in the load-deflection history shown in Figure 4.27. Similar failure mode was observed in panel S-5. Figures 4.28 and 4.29 show view of panel S-5 before and after failure, respectively. While Figures 4.30 to 4.32 show views of the failure mode similar to that for panel S-4. Figure 4.33 show the load-deflection relationship for panel S-5. In contract to panels S-4 and S-5, the failure mode of panel S-6 was due to tensile fracture of bottom OSB facing at a point between the mid-span and quarter point location as shown in Figures 4.35 to 4.37. Figures 4.34 and 4.35 show views of this panel before and after failure, while Figure 4.38 depicts the jacking load-deflection relationship for the panel.

Figure 4.27 depicts the jacking load-deflection history of panel S-4. The can be observed that the readings for all LVDTs at central and edge points of the mid-span location were almost identical. The design dead load, live load, the sum of dead and live load, and the factored dead and live load values were plotted in the figure to get the sense of the structural adequacy of the panel. It can be observed that linear elastic behavior was maintained at the live load level (i.e. serviceability limit state) and even at the design factored load level (i.e.

ultimate limit state). Similar behavior was observed for panels S-5 and S-6 as shown in Figures 4.33 and 4.38, respectively. One may observe that the live load deflection for panels S-4, S-5, and S-6 were 13.20, 13.50 and 13.10 mm, respectively. These values were normalized to the span length and then averaged to be span/356 which is almost equal to the deflection limit of span/360. One may observe that the total dead and live load deflection for panels S-4, S-5, and S-6 were 17.20, 17.10 and 16.80 mm, respectively. These values were normalized to the span length and then averaged to be span/277 which is far below the deflection limit of span/180. It should be noted that the panel span length considered herein is the panel length of 4.87 m minus 76 mm (3") from each side of the support, which is typical in all panels. It can be observed herein that the deflection of each panel at the live load live is within 15% difference with the average deflection of the three panels. One may consider the total deflection based on equation 4.1 as follows:

$$\Delta_{total} = \Delta_{short\ term} + K\,\Delta_{long\ term} = 17.03 + 0.74\text{x}17.03 = 29.63 \text{ mm} \qquad (4.2)$$

This would make the total deflections as span/159 which is more than the deflection limit for total deflection of L/180 for short term loading and L/360 for sustained loading per CAN/CSA-O86-01. As such this panel may be considered unqualified for long-term creep. To qualify this panel for long-term creep testing the live load or snow load must be limited to 1.63 kPa to produce a total deflection of L/180.

With respect to the ultimate load carrying capacity of the tested panels, it can be observed that the ultimate jacking load was 22.77, 21.55 and 22.33 kN for panels S-4, S-5 and S-6, respectively. It is worth mentioning that the ultimate jacking load for each panel was less than 15% difference with the average jacking load of the three panels. Therefore, the experimental ultimate load for the panel group is the average of the three vales. Thus, ratio of the ultimate jacking load to the design factored load of 1.25D+1.5L is 2.26 for panel group B, which is less than the allowable factor of safety of 3 for shear failure. It should be noted that this ratio was calculated using the jacking load rather without including the weight of the loading system of 2 kN. If the weight of the test loading assembly, this makes the ratio 2.46. In this case, the specified live or snow load on this roof panel must be less than 1.9 kPa. This makes the limiting specified snow load to be 1.32 kPa to maintain a safety factor of 3.

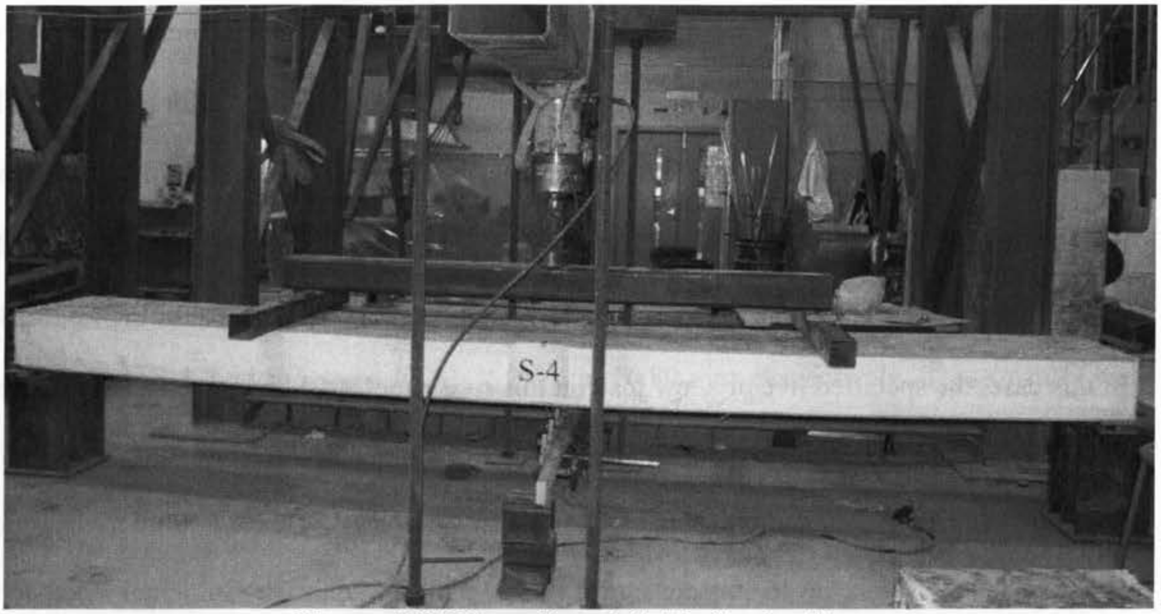

Figure 4.23 View of panel S-4 before loading

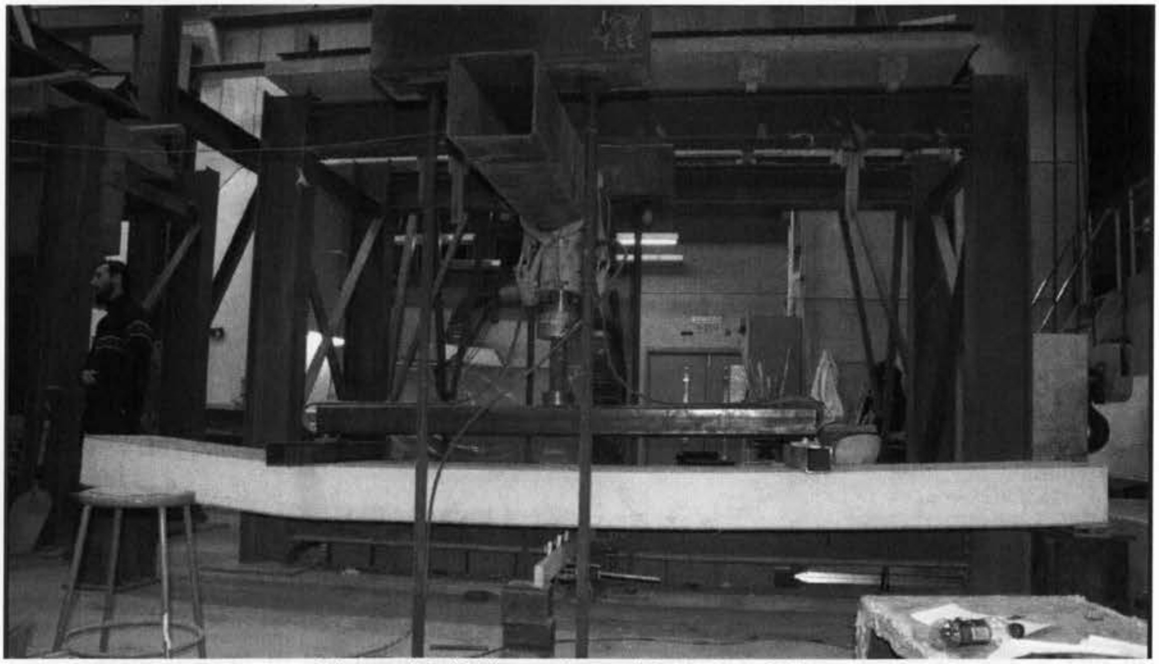

Figure 4.24 View of panel S-4 after failure

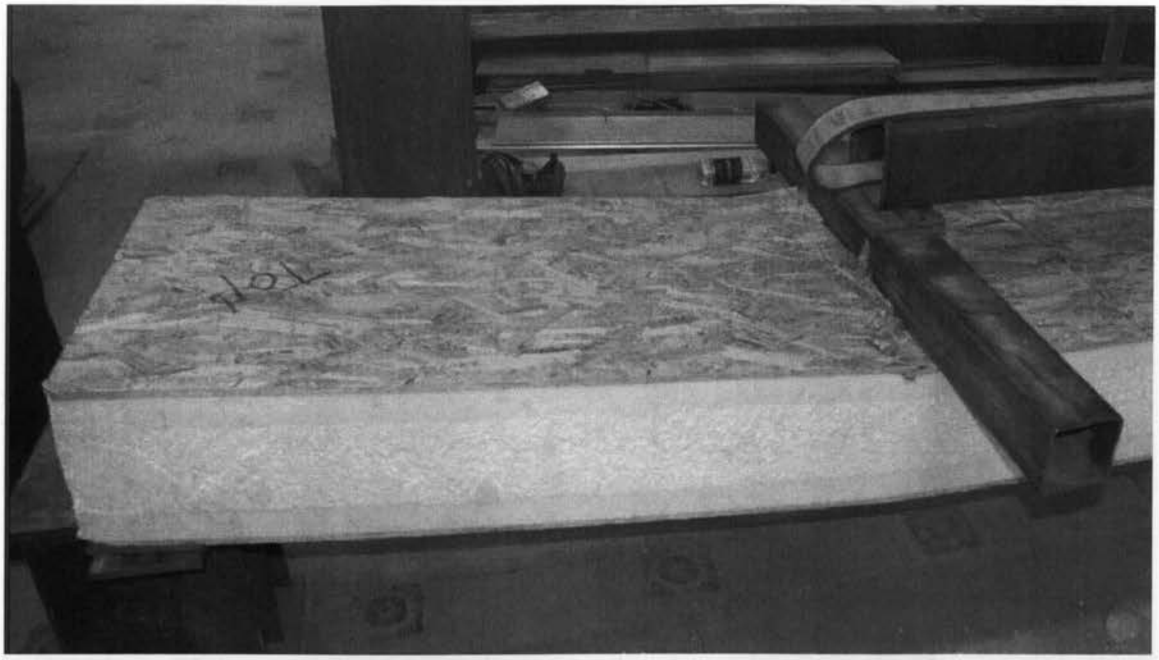

Figure 4.25 View of panel S-4 after failure showing OSB top facing crushing at the quarter point load location

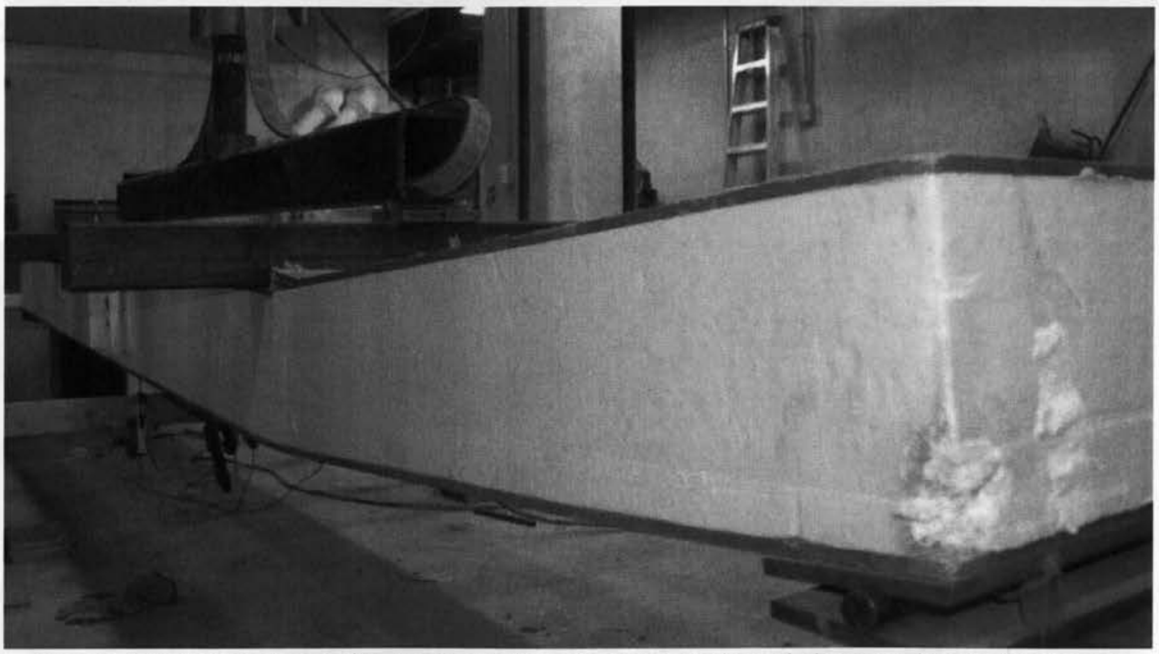

Figure 4.26 View of panel S-4 after failure showing deflected shape and OSB top facing crushing at the quarter point load location

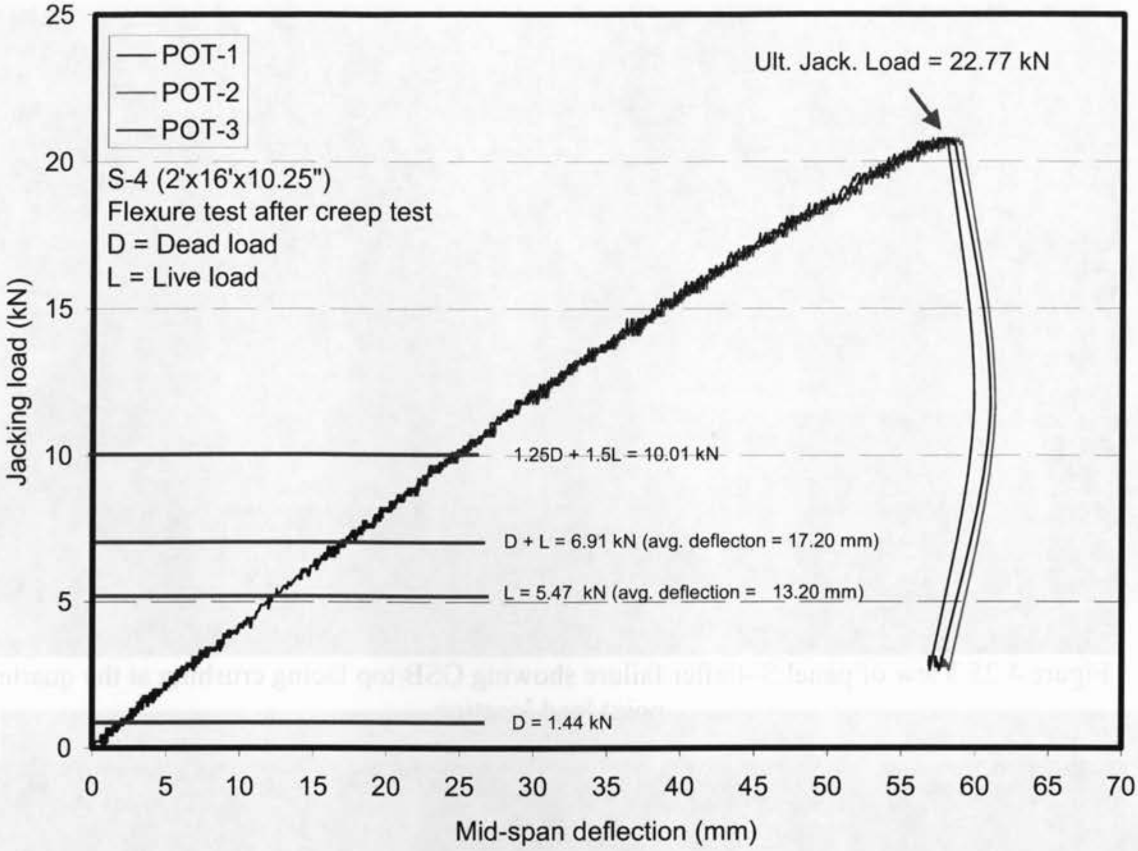

Figure 4.27 Load-deflection relationship for panel S-4

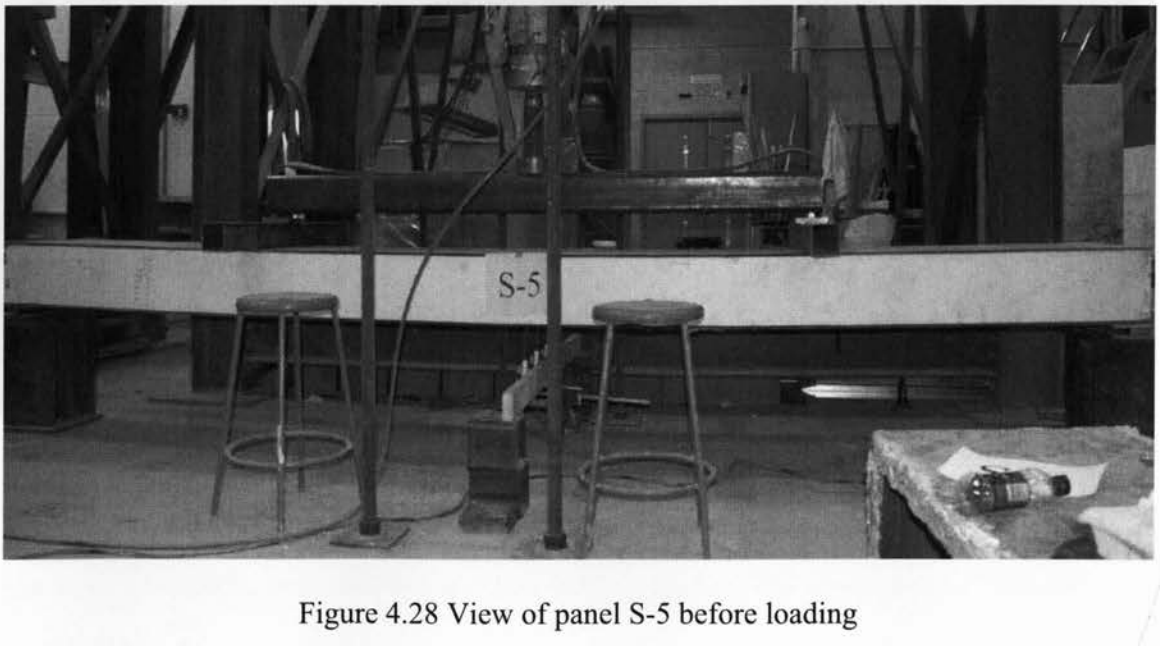

Figure 4.28 View of panel S-5 before loading

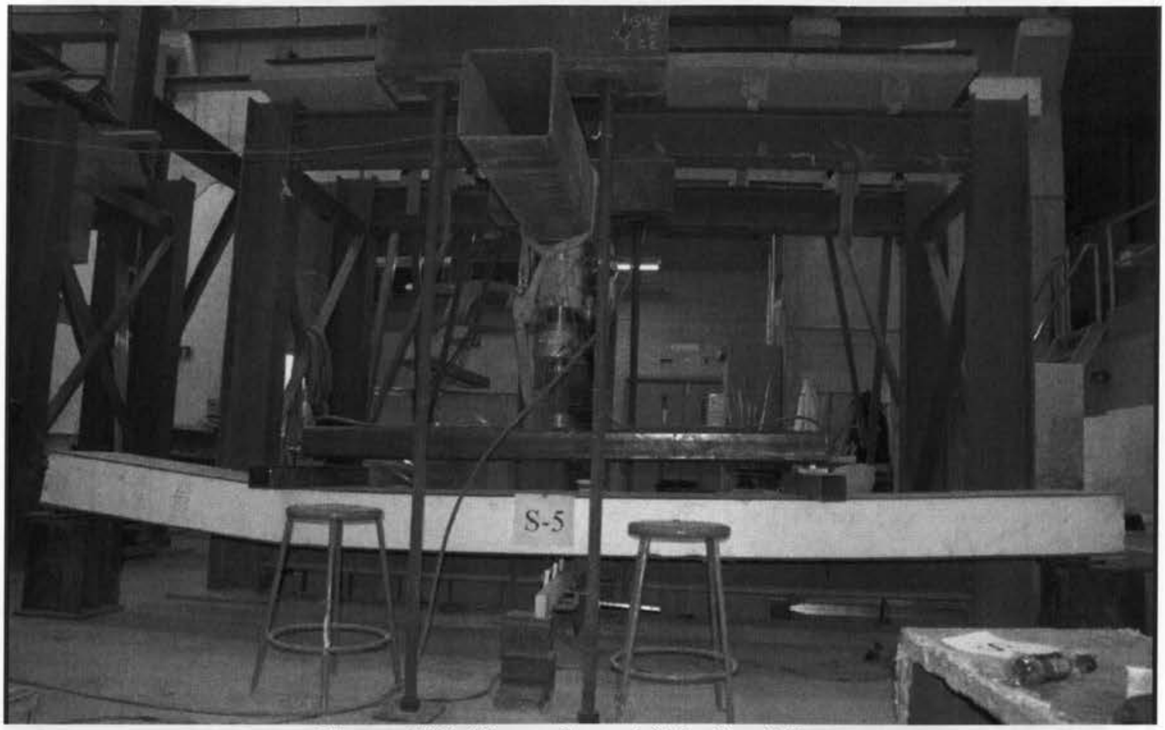

Figure 4.29 View of panel S-5 after failure

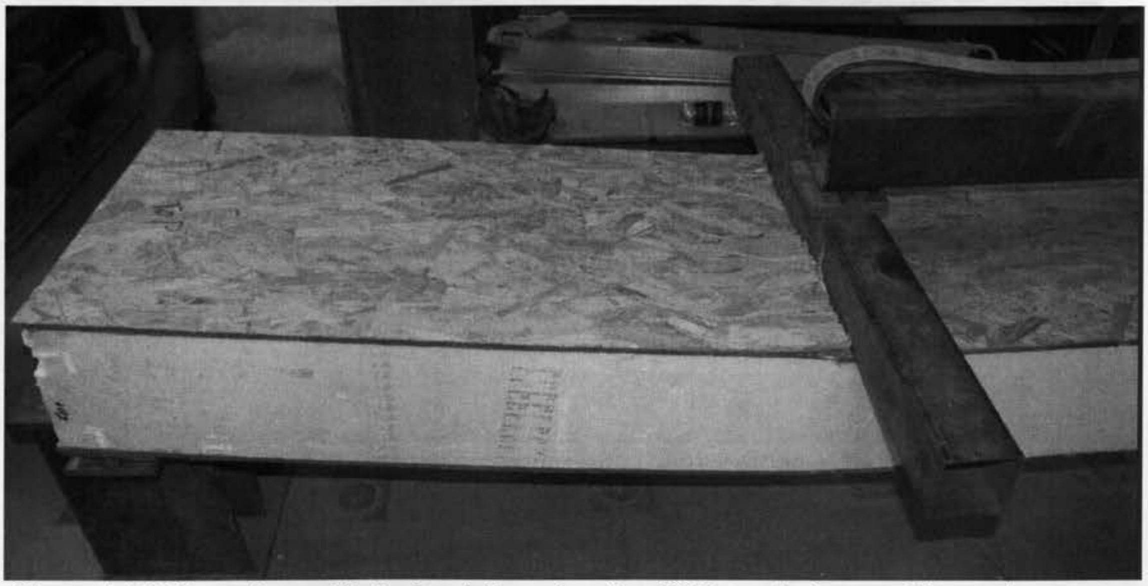

Figure 4.30 View of panel S-5 after failure showing OSB top facing crushing at the quarter point load location

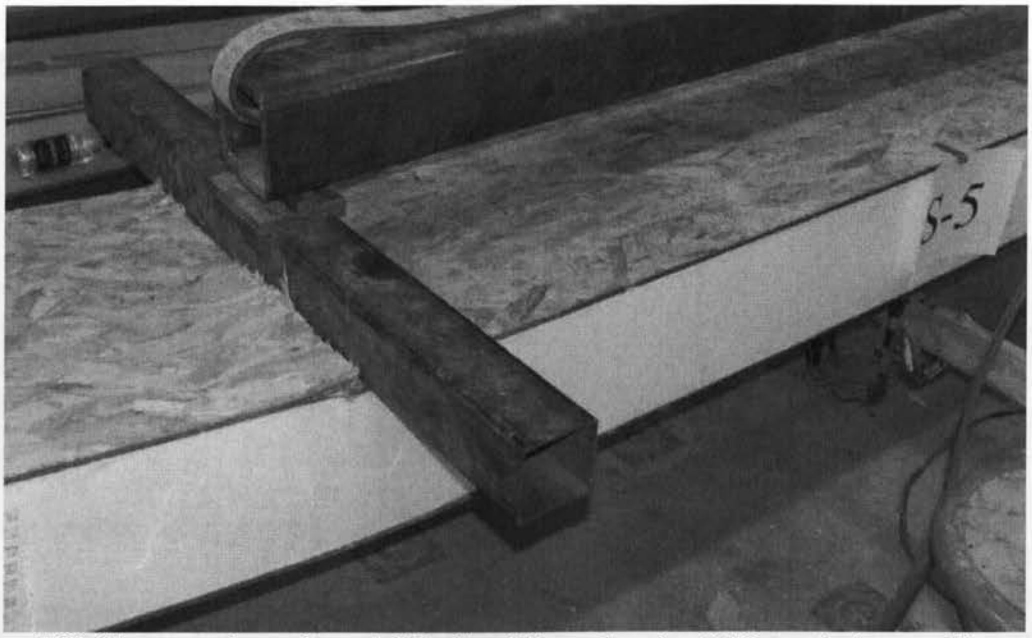

Figure 4.31 Close-up view of panel S-5 after failure showing OSB top facing crushing at the quarter point load location

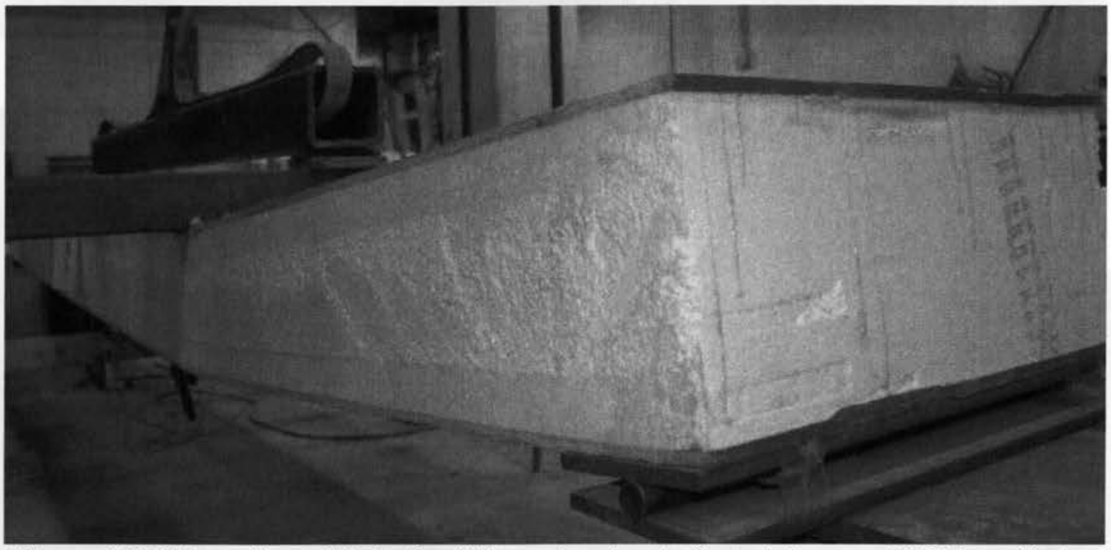
Figure 4.32 View of panel S-5 after failure showing deflected shape and OSB top facing crushing at the quarter point load location

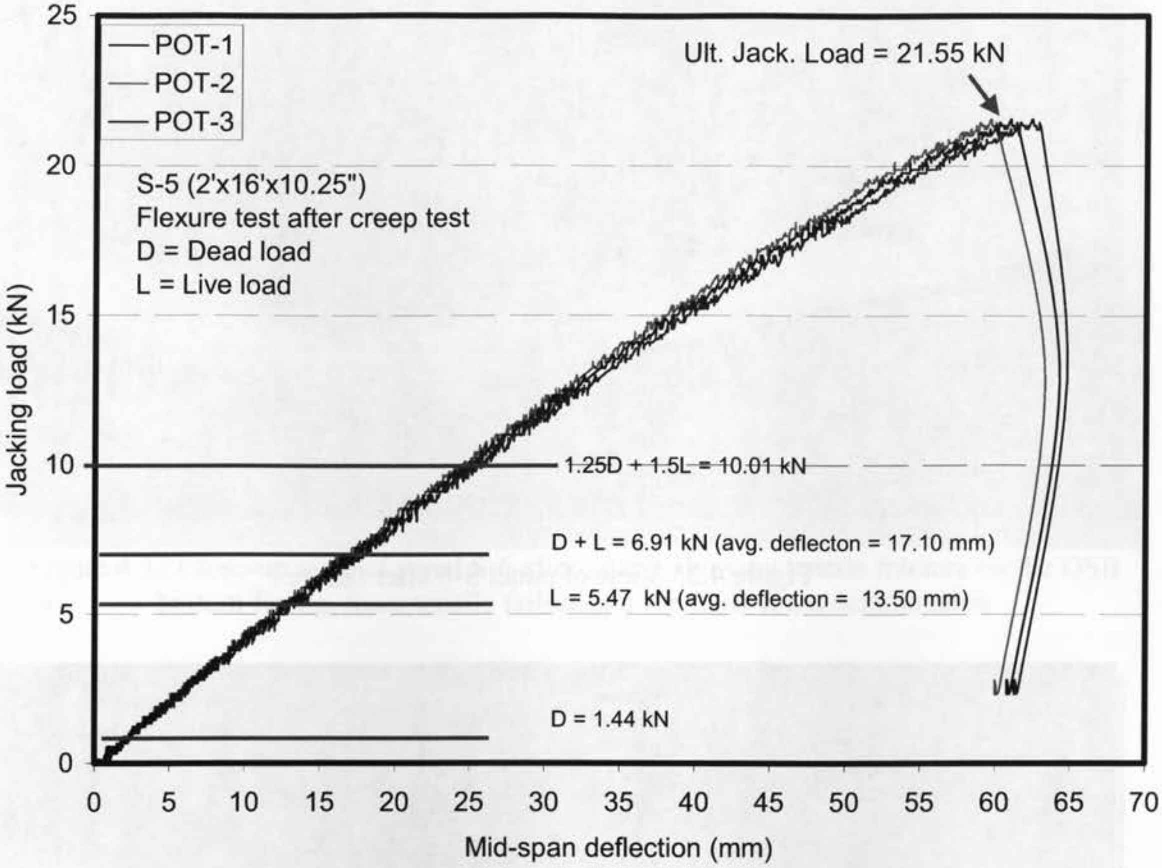

Figure 4.33 Load-deflection relationship for panel S-5

Figure 4.34 View of panel S-6 before loading

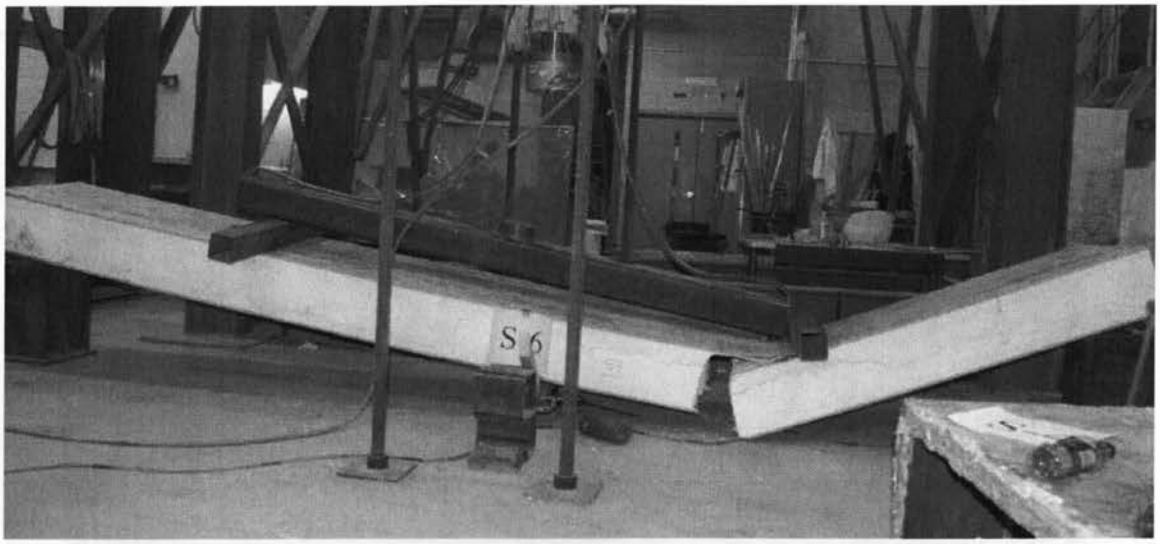

Figure 4.35 View of panel S-6 after failure

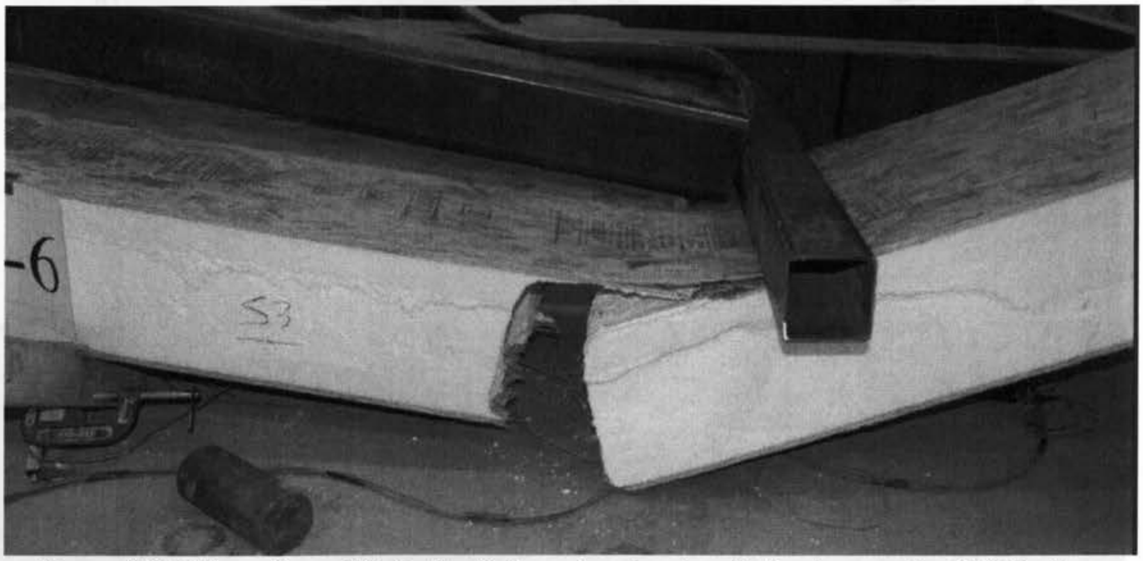

Figure 4.36 View of panel S-6 after failure showing tensile fracture on the OSB bottom facing, foam tensile failure and top OSB-foam delamination

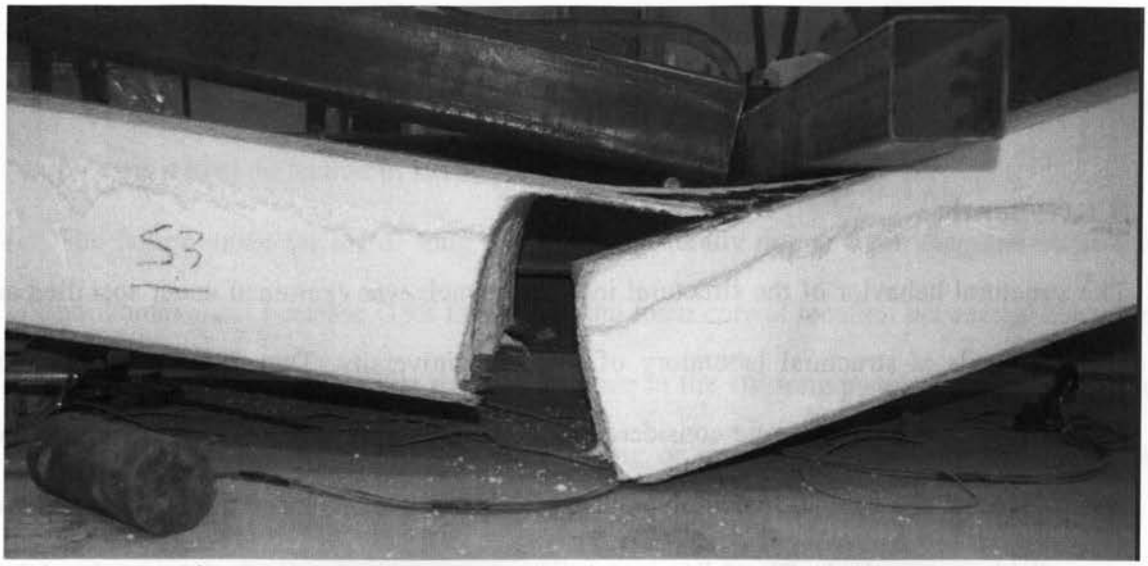

Figure 4.37 Close-up view of panel S-6 after failure showing tensile fracture on the OSB bottom facing, foam tensile failure and top OSB-foam delamination

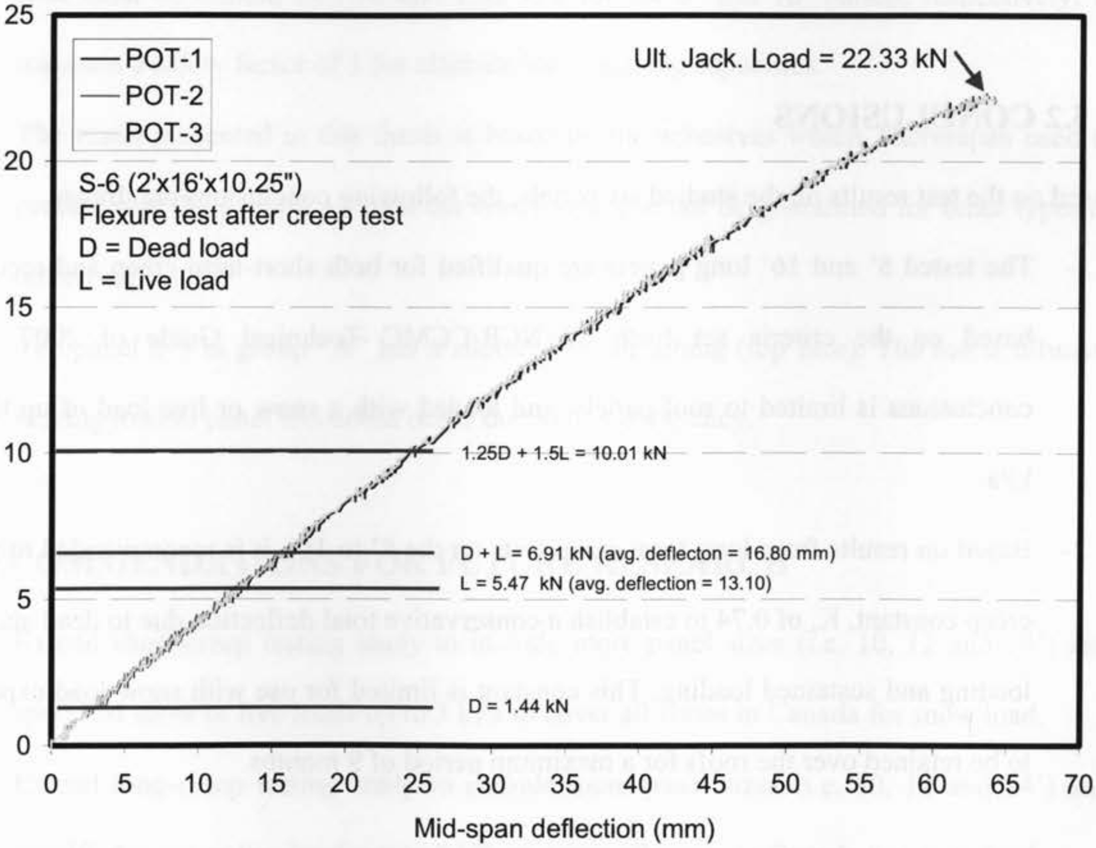

Figure 4.38 Load-deflection relationship for panel S-6

CHAPTER V

CONCLUSIONS

5.1 General

The structural behavior of the structural insulated panels was examined under specified and ultimate loads at structural laboratory of Ryerson University. Two different sizes of the structural insulated panels were considered in the experimental study. The experiment study was performed in a manner to comply with applicable Canadian Codes and Standards as well as available test methods. The following sections summarize the conclusions resulting from this research work as well as recommendations for future research.

5.2 CONCLUSIONS

Based on the test results on the studied six panels, the following conclusions are drawn:

1- The tested 8’ and 16’ long panels are qualified for both short-term creep and recovery based on the criteria set forth by NCR/CCMC Technical Guide of 2007. This conclusions is limited to roof panels, and loaded with a snow or live load of up to 1.9 kPa.

2- Based on results from long-term creep tests on the 8’ to 16’, it is recommended to use a creep constant, K, of 0.74 to establish a conservative total deflection due to dead and live loading and sustained loading. This constant is limited for use with snow load expected to be retained over the roofs for a maximum period of 9 months.

3- To qualify the tested panels for long-term creep, the live load or snow load on the panels must be limited to 1.9 kPa and 1.63 kPa for the 8' and 16' panels, respectively, to produce a total deflection of L/180 for sustained loading.

4- The failure mode for the 8' long panels was generally due to foam diagonal shear and horizontal shear between OSB facing and the foam core at location between the support and the quarter points. While the failure mode in the 16' long panels was generally due to tensile fracture of the bottom OSB facing or crushing of the OSB top facing in addition to foam tensile fracture.

5- To qualify the tested panels for ultimate limit state design, the specified snow or live load must be limited to 1.76 and 1.32 kPa for the 8' and 16' panels, respectively, to maintain a safety factor of 3 for ultimate load carrying capacities.

6- The result presented in this thesis is based on the adhesives which Thermapan used to prefabricate the panels. As such the conclusion can not be guarantied for other types of adhesives.

7- The panel S-3 in group "A" has a shorter face sheathing (top face). The lower ultimate jacking load in panel S-3 could occur due to this deficiency.

RECOMMENDATIONS FOR FUTURE RESEARCH

1- Extend short-creep testing study to include more panel sizes (i.e. 10, 12 and 14') and specified snow or live loads up to 3 kPa to cover all zones in Canada for snow load.

2- Extend long-creep testing study to include more panel sizes (i.e. 10, 12 and 14') and specified snow or live loads up to 3 kPa to cover all zones in Canada for snow load.

3- Study the impact of connection between panels as their short and long term creep behavior

4- Conduct short- and long-term creep tests on SIPs with lumber-spline connection to increase the live load capacity that was evident to be reduced in this research for panels with no connection at all.

5- Study the ultimate capacity and serviceability of SIPs under impact loading as well as under concentrated static load.

6- Conduct finite-element modeling of panels under flexural loading to developed empirical expressions for the ultimate load carrying capacity for various SIP sizes.

7- To study long term effect on adhesives to maintain bonding over the time, panels shoud be loaded for considerable time period (i.e. 5 years)

8- Study creep behavior of SIP walls under sustained loading.

REFERENCES

Allen, H. G. (1969). Analysis and Design of Structural Sandwich Panels. Pergaman Press.

APA. 1993. Design and Fabrication of Plywood Sandwich Panels. APA – The Engineering Wood Association, Tacoma, WA, USA.

ASTM. 2002. Standard Test Methods of Conducting Strength Tests of Panels for Building Construction, ASTM E72-02. American Society fro Testing Materials, Philadelphia, PA, USA.

ASTM. 1988. Standard Test Methods of for Flexural Creep of Sandwich Construction. ASTM C 480-62. American Society for Testing Materials, Philadelphia, PA, USA.

Aviles, F. and Carlsson, L. (2006). Experimental Study of Debonded Sandwich Panels under Compressive loading. Journal of Sandwich Structures and Materials. 8: 7-31.

Bau-Madsen, N.K., Svendsen, K.H. and Kildegaard, A. (1993). Large deflection of sandwich Plates– an experimental investigation. Composite Structures.

Canadian Standard Association. (1992). Construction of Preserved Wood Foundation, CAN/CSA-S406-92. Etobicoke, Ontario, Canada.

Canadian Standard Association (2001). Engineering Design of Wood. Etobicoke, Ontario, Canada.

Canadian Wood Council. (2005). Wood Design Manual 2005. Ottawa, Canada.

Daniel, I.M., Abot, J.L. and Wang, K.A. (1999). Test and analysis of composite sandwich Beams in Proceedings of ICCM 12. Paris.

Frostig, Y. and Baruch, M. (1990). Bending of sandwich beams with transversely flexible core.

Gdoutos, E.E., Daniel, I.M., Wang, K.A. and Abot, J.L. (1999). Nonlinear behavior of Composite sandwich beams in three-point. In: Proceedings of IX Int. Cong. on Experimental Mechanics. June 5–8, 2000, Orlando, FL.

Gupta, N., Kishore, Woldesenbet, E. and Sankaran, S. (2001). Studies on Compressive Failure, Features in Syntactic foam Material, J. Mater. Sci., 36(18).

Gupta, N. and Woldesenbet, E. (2005).Characterization of Flexural Properties of Syntactic Foam Core Sandwich Composites and Effect of Density Variation. Journal of Composite Materials, 39: 2197-2212.

Gupta, N., Woldesenbet, E., Hore, K. and Sankaran, S. (2002). Response of syntactic foam core sandwich structured composites to three point bending. Journal of Sandwich Structures and Materials, 4: 249-272.

Hossain, K. and Wright, H. (2004). Flexural and Shear Behavior of Profiled Double Skin Composite Elements. Journal of Steel and Composite Structures, 4(2): 1-20.

ICC AC04. (2004). Acceptance Criteria for Sandwich Panels. ICC Evaluation Service Inc., USA.

Institute for Research in Construction. 2005. National Building Code of Canada, NBCC-2005. National Research Council, Ottawa, Ontario, Canada.

Institute for Research in Construction. 2007. Technical Guide for Stressed Skin Panels (with lumber 1200 mm o.c. and EPS Core) for Walls and Roofs. National Research Council, Ottawa, Ontario, Canada.

Jones, Jr. and David, E. (1993). Flexural test methods and results for Dow thermoplastic Sandwich panels. In: Proceedings of 38th International SAMPE Symposium.

Karlsson, K.F. and Astrom, T.B. (1997). Composites

Liu, Q. and Zhao, Y. (2007). Effect of Soft Honeycomb Core on Flexural Vibration of Sandwich Panel using Low Order and High Order Shear Deformation Models. Journal of Sandwich Structures and Materials. 9: 95-108.

Meyer-Peining, H. (2006). Sandwich Plates: Stresses, Deflections, Buckling and Wrinkling Loads – A Case Study. Journal of Sandwich Structures and Materials. 8: 381-394.

Riber, H.J. (1997). Nonlinear analytical solution for laterally loaded sandwich plates. Composite Structures, 39(1–2).

Olsson, R. (2002). Engineering Method for Prediction of Impact Response and Damage in Sandwich panels. Journal of Sandwich Structures and Materials. 4: 3-29.

Structural Board Association. (2004). OSB Performance by Design Manual: Construction Sheathing and Design Rated Oriented Strand Board. Markham, Ontario, Canada.

Taylor, S. B. (1996). The Flexural Creep Behavior of Structural Insulated Panel (SIP) Sandwich Beam. Ph.D. Thesis. The Pennsylvania State University, University Park.

Tham, L., Chong, K., and Cheung, Y. (1982). Flexural Bending and Axial Compression of Architectural Sandwich Panels by Combined Finite-Prim-Strip Method. Journal of Reinforced Plastics and Composites, 1: 16-28.

Thermapan Structural Insulated Panels Inc. 2007. www.thermapan.com.

Vinson, J.R. (1999). The Behavior of Sandwich Structures of Isotropic and Composite Materials, Lancaster, PA: Technomic Publishing Co. Inc.

Vinson, J.R. (1999). The Behavior of Sandwich Structures of Isotropic and Composite Materials, Technomic Publishing, Lancaster, PA.

Wang, W. and Shenoi, R. (2004). Analytical Solutions to Predict Flexural Behavior of Curved Sandwich Beams.

Wennhage, P. and Zenkert, D. (1998). Testing of sandwich panels under uniform pressure .Journal of Testing and Evaluation, 26(2).

Yoon, K., Kim, C. and Park, H. (2002).Nonlinear Flexural Deflection of Thermoplastic Foam Core Sandwich Beam. Journal of Composite Materials, 36: 1529-1539.

Xanthos, M., Dhavalikar, S., Tan, V., Dey, S., and Yilmazer, U. (2001). Properties and Applications of Sandwich Panels Based on PET Foams.

Zenkert, D. (1995). An Introduction to Sandwich Construction. London: Engineering Materials Advisory Services.

Aleem Shahzad Butt. (2008). Experimental Study on the Flexural Behavior of Structural Insulated Sandwich Timber Panel. Civil Engineering Department, Ryerson University, Toronto, Ontario, Canada