New Shear Connector for Open Web Steel Joist with Metal Deck and Concrete Slab Floor Systems

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New Shear Connector for Open Web Steel Joist with Composite Concrete Slab Flooring

By

Gregory Joseph Charles Merryfield

A Thesis
Submitted to the Faculty of Graduate Studies through the Department of Civil and Environmental Engineering in Partial Fulfillment of the Requirements for the Degree of Master of Applied Science at the University of Windsor

Windsor, Ontario, Canada

2015

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New Shear Connector for Open Web Steel Joist with Composite Concrete Slab Flooring

by

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ABSTRACT

Composite floor systems consist of a concrete slab poured on steel sheets, supported over Open-Web Steel Joists (OWSJ); it is widely used in commercial and industrial buildings. To achieve the desirable strength, shear connector (studs) have to be welded on the OWSJ to ensure composite action of the three components. Extending the application of this composite floor system into residential buildings, alternative shear connectors such as puddle-welds and Hilti-screws would greatly reduce the expense and accelerate construction. Yet the current design codes consider these alternates structurally inadequate due to lack of research. The objective of this research is to investigate the ability of puddle-welds and Hilti-screws to develop composite action. Experimental testing under different loading conditions had been carried out on small and large-scale composite floor prototypes to investigate the behaviour of the proposed shear connectors. Test results showed that significant composite action is developed using both shear connectors and their behaviour meets the code requirements for residential applications.

Keywords: Open Web Steel Joist, Shear Connectors, Shear Studs, Longitudinal Shear, Composite Construction, Composite Action
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CHAPTER 1: INTRODUCTION

1.1. Introduction

Over the last few decades, steel and concrete structures have been a very popular selection in construction; particularly composite floor systems consisting of a concrete deck poured on top of corrugated steel sheets, supported by Open-Web Steel Joists (OWSJ). This type of construction is called composite construction and is defined as one or more dissimilar materials rigidly connected to each other to perform as a single, unified structure. The purpose of this construction practice is to build upon the strengths of each material and to compensate for the weakness of each other simultaneously.

Composite construction is typically used in bridges and repeated floor buildings because the floor system can be replicated efficiently floor after floor. This type of construction is chosen because composite elements can be lighter, slenderer and more economical while resisting identical loads, when compared to a non-composite option. In comparison to non-composite elements, composite structures are stiffer, which can lead to less deflection and increased span lengths. Composite elements also have a larger moment capacity which allows for smaller section sizes; therefore, reducing height over the course of a soaring structure. The main challenge in composite construction is to ensure that forces are transmitted effectively and safely between the two materials and there is full strain compatibility at the interfaces.

Composite action is the term used to describe the behaviour of a composite structure and has a great effect on the stress and strain of beam and floor composite systems. To ensure full composite action and full strain continuity is achieved, stiff and
rigid shear connectors, commonly called shear studs, are used. However, partial shear connection – or partial composite action – is used when full shear connection is not necessary; this is typically in the range from 25-75% of the theoretical connection depending on the requirements of the design. (Canadian Standards Association, S-16-09). Fully composite elements will have a larger ultimate capacity and deflect less as compared to partial composite elements.

Shear studs are shear resisting dowels welded to the top flange of the steel sections and encased in concrete in order to transfer the longitudinal shear forces between the deck and the supporting beam. Figure 1.1-a illustrates the behaviour and strain profile of a composite to a non-composite beam. The composite section undergoes less deflection and no slip due to the addition of the shear studs. The shear studs are the primary horizontal shear resisting component in the composite element and are the only code approved method for achieving sufficient composite action. (Canadian Standards Association, S-16-09). Without composite action, there is no strain compatibility between the deck and the supporting girders, i.e. there are two separate neutral axis when the floor is subjected to bending under gravity loads, as displayed in Figure 1-b.

A composite cross section containing two neutral axis means that both the beam and deck are both acting in tension and compression; therefore, the structure is not fully exploiting the strengths of each material. For composite systems with full composite action, there is only one neutral axis and a continuous strain profile, a composite cross section is displayed in Figure 1.1-c. This allows the concrete to resist all the compression stress, while the steel beam undertakes all of the tension stress and this is an ideal design with respect to each individual material’s strengths. To achieve the desirable composite
action, shear connectors have to be welded on the girder. This method imposes extra cost, time and construction difficulties. The current design codes allow only the strength provided by shear studs as shear connectors to develop and secure composite action. This is mostly due to the lack of research allotted to this topic.

![Composite beam with shear connectors](image1)

a) Composite beam with shear connectors (right) vs. non-composite beam (left)

![Dis-continuous strain profile](image2)

b) Dis-continuous strain profile (non-composite section)

![Continuous strain profile](image3)
c) Continuous strain profile (composite section)

**Figure 1.1:** Composite vs. non-composite structures (Tata Steel Limited, 2015)

Composite floor decks consist of four main elements: Open Web Steel Joist (OWSJ), corrugated steel deck sheets, shear connectors and a thin concrete deck slab. The OWSJs act as secondary beams transferring loads to the main girders and are designed to resist the floor gravity loads. Joists are the preferred steel section in composite flooring because they possess an exceptional strength to weight ratio and also provide room for subsequent utilities such as duct work and electrical. The joists are composed of a top chord, a bottom chord and web elements; the chords are typically
manufactured from steel angles oriented with the legs back-to-back and the web members are typically steel round bar welded between the angles. A typical joist can be seen below in Figure 1.2.

![Steel Frame with OWSJ](image1) ![Typical OWSJ in Composite Flooring](image2)

**Figure 1.2:** Open web steel joists (OWSJ)

The corrugated steel decking is fastened on top of the OWSJ and span the entire area (Figure 1.3) of the composite floor in all directions; the flutes (valleys) are placed perpendicular to the joists. The decking not only acts as cast-in-place formwork but also as a thin layer of tensile reinforcement for the concrete slab. The decking is strong enough to support labourers during construction which allows for mobility and stress-free installation. Most corrugated steel decking used in composite flooring systems contain embossments, or patterned dimples within the profile of the deck. These indentations generate composite action by creating a superior bond to the concrete deck.
a) Corrugated steel deck sheets (SMD Stockyards Limited, 2013)  
b) Embossments in steel decking

**Figure 1.3:** Corrugated metal deck sheets

The shear connectors are welded to the top of the joists, typically through the steel decking, and are the most critical component in generating composite action. The connectors are designed to resist longitudinal shear which is produced when the floor is subjected to bending and the concrete deck is forcefully attempting to slip against the steel joist(s). In addition, they are the primary stress/strain transferring mechanism within the composite element and ensure the deck remains rigidly connected to the steel joists. The shear connectors also resist uplift of the concrete slab during bending because the headed studs are fully encased in the concrete and secured to the joist. A typical shear stud, or Nelson Stud, measures from 50-250 mm tall and 13 mm diameter at the shank with the head usually 1.5-3 times the shank diameter. Shear stud details can be seen in Figure 1.4.
Lastly, once the studs and decking are installed on the OWSJ, a thin concrete slab is cast on top and is designed to resist compressive stresses. This slab is only strengthened with light, top reinforcement (welded wire mesh) to eliminate/control shrinkage cracks.

**Figure 1.4:** Shear stud connectors (SMD Stockyards, 2013)

Due to the modular nature of composite flooring with steel joists, several floors can be rapidly constructed and easily repeated so it is an ideal design for buildings that
require the same floor after floor construction. This construction method saves a tremendous amount of time and materials needed under typical flooring construction, which greatly reduces the cost and accelerates construction.

1.2. Scope of Thesis

Currently, the Canadian Institute of Steel Construction (Canadian Standards Association, 2009) mandates the use of shear studs in composite floors as the only means to develop sufficient composite action and strength. However, this practice imposes construction difficulties and hazards as well as the shear studs add extra material costs. Shear stud connectors also require more time to install as the work must be completed by licensed professionals.

![Shear stud connectors](image)

a) Puddle weld (SECB, 2013)  
b) Hilti-screws or steel deck screws (Hilti, 2015)

**Figure 1.6:** Alternate shear connectors

Composite flooring is typically used in industrial settings; however, residential applications require lighter live loads and shorter spans. Composite designs with shear studs are also difficult and designers often chose simpler, non-composite elements. In order to keep the composite floor system as cost and time effective as possible, designers can opt for non-composite options and use deeper cross sections for girders or shorter
spans with lighter loads; therefore, this is less economical. This motivates the investigation into analyzing smaller, simpler shear connectors. However, during construction, arc spot weld (commonly called puddle welds) or screw pins are used in order to temporarily fasten and secure the metal deck sheets in place on top of the supporting members. Figure 1.6 displays two alternate fasteners.

To date, all design codes do not account for any level of composite action that puddle welds or screw pins might provide, i.e. no composite action is exploited in a puddle weld or Hilti-screw connector design. This is mostly due to the lack of research allotted to this topic. The objective of this research is to determine if partial or full-composite action can be achieved when puddle welds and Hilti-screw pins are used as an alternate shear connector for shear studs. By eliminating the exorbitant shear stud, a simple steel deck fastener can streamline composite floor construction, with safer installation and construction procedures and could have vast effects on the steel and concrete flooring market.

1.3. Methodology

To be able to achieve the research objectives, an extensive experimental program was designed. The experimental program is composed of two phases: (I) Small-scale Composite T-beams and (II) Full-scale Composite Floor Decks with material properties testing. In each of the two phases, the key variables are the shear connector and loading patterns.
1.4. **Thesis Organization**

Chapter I presents a brief overview of the topic, along with the motivation for the research, the objectives and the methodology.

In Chapter II, a literature review is introduced, where a brief history of composite construction is reviewed.

Chapter III presents the experimental program. It shows a detailed description of the test specimens, their construction and dimensions, as well as the loading scheme and the instrumentation.

In Chapter IV, an analysis of the experimental results is performed. The influence of the parameters and loading program is evaluated. In addition, a comparison with Steel Construction code predictions such as CAN/CSA-S16-09 and results are related to the National Building Code of Canada 2010.

Chapter V presents the conclusions that can be drawn from this research.
Advancements in composite flooring are extremely important to designers today because it is the epitome of sustainable, structural engineering. Engineers and designers are always developing new and improved solutions to the same challenge; gravity loads. With soaring steel and material costs, it is ever more important to address and tackle structural engineering with a clear, concise and economical approach, while responsibly maintaining public safety.

Composite construction has been increasing in popularity over the past few decades but the lack of research allotted to this topic may be hindering the potential economical and structural benefits. There was not much research on this topic before the 1960`s but in the years since, there has been continual progress in composite design. (Canadian Standards Association, 2009). In the late 1970`s puddle-welds and shear studs were tested for shear connectors in composite flooring applications. Both showed good promise moving forward. Over the years leading to the new millennia, improvements were made with regards to the embossments on the corrugated steel sheets and reduced slab depths – Figure 2.1.
Figure 2.1: Advancements in corrugated metal decking (Lakshmikandhan, 2013)

One of the key aspects of structural design is the ability to control the failure mode of the structure. With steel and concrete composite flooring there are three main failure modes including: yielding of the bottom chord, buckling/yielding of the top chord and shear failure in the concrete slab but can also include shear/tensile failure of the shear connector. Rigid (steel) and flexible (plastic) shear connectors were tested to determine if the failure mode could be influenced by one or the other.

It was found that flexible shear connectors failed at an earlier level but in a slow ductile manner as opposed to the rigid connectors that failed in a sudden, brittle manner. Several factors affecting the shear capacity of a connector were proposed including:
shape and dimension of the connector, quality of the concrete, type of loading, connector spacing among others (Rankovic and Drenic, 2002).

![Diagram of failure mode of headed stud](image)

**Figure 2.2:** Failure mode of headed stud (Rankovic and Drenic, 2002)

The experimental program was divided into two categories based on the type of shear connector: rigid and flexible. The rigid connectors resist forces through the front side by shearing and experience little deformation. This connector produces a much stiffer element but also causes it to fail in concrete because the stiff connections produce great amounts of stress in the deck around them; Figure 2.2. This resulted in a concrete crushing or weld failure mode. The flexible connectors resist forces through bending, shear and tension and the connection point with the beam. Shearing strength is maintained even with large amounts of plastic deformation with flexible connectors. As the flexible connector is stretched, the tensile forces in the connector pull the concrete close to the steel section and these increased the shear resistance. ie. Friction increases at the interface, between the concrete and steel, and this helps resist horizontal shear.
In 2003, scholars researched the induced slip between the concrete deck and supporting members which merited a reassessment of the equations that predict deflections in composite structures. Slip effects are very important for composite beams and floors because composite elements are typically used in long-span applications. The reduced sections, resulting from composite action, are subject to larger deflections; therefore, composite designs are typically controlled by deflection. “At service loads, the actual stiffness of beams with a full composite design is about 85-90% of the calculated stiffness where slip is ignored (Nie and Cai. 2003).” Equations were proposed through experimental testing and were compared alongside the current codes (American Association of State Highway and Transportation Officials, American Institute of Steel Construction and Eurocode 4, 2003). Nie and Cai (2003) determined that “even for full composite beams, slip effects may result in a stiffness reduction of up to 17% for short span beams.” Short span beams are more likely to fail in shear as opposed to flexure, which validates Nie and Cai’s experimentation and supports the requirement to calculate accurate slip effects in composite beam design.

**Figure 2.3:** Flexile vs. rigid shear connector (Rankovic and Drenic, 2002)
Lam and El-Lobody (2005) sought out to develop a new finite element model for composite beams that would better predict the capacity and behavior of the shear stud connection. “Present knowledge of load-slip behavior and the shear capacity of the shear stud in composite beams are limited to data obtained from push-off tests. Therefore, an effective numerical model using the finite element method to simulate the push-off test was proposed.” (Lam and El-Lobody, 2005). Three-dimensional elements used to model the shear studs and concrete are shown in Figure 2.X.

![C3D8 element](image1)
![C3D15 element](image2)
![C3D20 element](image3)

**Figure 2.4**: Three-dimensional solid elements in FEM (Lam and El-Lobody, 2005)

Lam and El-Lobody validated a model against original test results and also alongside the current code practices in British Standard 5950, EuroCode 4 and American Institute of Steel Construction. It was found that “the strength of the connector and the concrete strength are the main factors affecting the behavior of shear connectors.” (Lam and El-Lobody, 2005) From a parametric study, it was found that the formulae given in EuroCode 4 had a good correlation with the experimental results and original finite element solutions while it would appear that the British Standard 5950 and American Institute for Steel Construction standards may have overestimated the shear capacity of the headed stud.
a) Comparison between push-off test and FE model for 35 MPa concrete

b) Codes comparison of shear capacity for headed stud in various concrete strength

Figure 2.5: FEM and experimental comparison to design codes (Lam and El-Lobody, 2005)
Lastly, it was concluded that using the developed finite element model, expensive push-off tests can be eluded and computer simulations can be opted for. Full-scale composite tests and push-off tests alike remain a very costly and time consuming process; therefore, it is becoming ever more important to develop economical and efficient computer models to replace full scale testing.

Over the past 10 years, further refined equations have been developed for predicting longitudinal shear stress which prompted further testing in composite flooring systems. Hedaoo et al. (2012) presented the structural behavior of composite concrete slabs with profiled steel decking through an experimental and analytical study. Corrugated steel sheets with embossments were used to increase the composite interaction between the concrete and to improve their shear bond characteristics. Eighteen specimens divided into six sets, three specimens each, were tested with different shear span lengths under static and cyclic loadings over simply supported condition. For each set, one specimen was tested under monotonic loading, and the other two specimens were tested under cyclic loading. Experimental results were compared to two design methods established by Eurocode 4 – Part 1.1; namely, the m-k method (shear bond method) and the Partial shear connection (PSC) method.

A comparison of experimental and partial shear connection analysis of the load-carrying capacity revealed that as the shear span length increased, the longitudinal shear stress of slab decreased but also slipped at lower load levels; see Figure 2.6. The legend corresponds to the respective shear span.
Figure 2.6: Load-end slip curves for slab specimens (Hedaoo et al. 2012)

The m-k method for predicting longitudinal shear was found to be more conservative than the PSC method and therefore the PSC method would give optimum design. “As the shear span increased, the longitudinal shear stress of the slab decreased. The design longitudinal shear stress values of slabs resulting from line loads obtained by the m-k method is slightly higher than that of the PSC method. It can be concluded that the m-k method has better longitudinal shear strength” (Hedaoo et al. 2012) – see Figure 2.7.
a) Longitudinal shear stress to shear span under flexural loading

b) Failure/design load to shear span under flexural loading

**Figure 2.7:** Experimental comparison of shear stress for m-k method to PSC method (Hedaoo et al, 2012)

Lakshmikandhan et al. (2013) investigated the longitudinal shear transfer mechanism at the interface between steel and concrete. The test specimens were divided into three series where the main variable is the shear connection. Composite floors with
headed shear stud, shear rods or no connection were investigatively tested. There shear connector configurations are displayed in Figure 2.8. The floor decks constructed with stud bolts and shear rods showed different behaviors when compared to that of the composite slab without shear connectors. The insertion of shear connector modifies the brittle behavior of the composite slab into ductile.

The composite deck without shear connectors slipped and failed at the earlier load level because the concrete deck and supporting steel section are not rigidly connected. Multiple neutral axis develop, leaving the thin slab subjected to compression and tension stresses in addition to high shear stress. Load vs. deflection results were compared to a traditional reinforced concrete slab and a composite slab without shear connectors, see Figure 2.9.

The insertion of shear connector generates composite action and modifies the brittle behavior of the composite slab system to ductile; allowing for the concrete to resist compression exclusively and the steel section to resist tension stresses. This phenomenon (composite action) is possible through strain compatibility across the concrete deck and OWSJ connection and is the most important aspect of the composite floor deck system.
a) Scheme1 mechanical shear connector

b) Scheme2 mechanical shear connector

c) Scheme3 mechanical shear connector

**Figure 2.8:** Shear connector systems with schematic view of metal decks (Lakshmikandhan, 2013)
Figure 2.9: Load vs. deflection comparison for different types of slabs (Lakshmikandhan, 2013)
CHAPTER 3: EXPERIMENTAL PROGRAM

This research includes the design, construction, testing and analysis of a series of composite floors to investigate the behaviour of different shear connection for composite flooring for residential buildings. Smaller, shallower joists are also being considered to reduce overall story height and accommodate for additional floors in high-rise buildings.

The experimental program was divided up into two phases. Phase I addressed a multitude of shear connector alternatives for the purpose of narrowing down the most viable alternative connector. Four small-scale composite T-beams were constructed utilizing four (4) separate shear connectors, one of which being a control beam with traditional shear studs. Mechanical properties of the materials and the capacity of different shear connectors in direct shear tests were also conducted.

Phase II was designed according to the findings in phase I. Accordingly, two full-scale, composite floor decks were constructed utilizing the proposed shear connector alternatives; further details are provided in section 3.1.

3.1. Details of Test Specimens

3.1.1. Phase I: Four Small-scale Composite T-beams

The phase I experimental approach consisted of multiple, smaller scale, composite T-beams. Four composite beams were constructed and tested in order to narrow down a viable shear stud alternative, moving into phase II. A single, non-composite joist was also tested in phase I to compare against the variety of composite beams.
The four small-scale test specimens were constructed identical to one another with the exception of the shear connector. Each test specimens was constructed from a single open web steel joist, 254 mm deep and 4.57 m long. The top and bottom chords consisted of 2 L32x32x3.2 mm back-to-back angles and 13mm round bars for the web members. In addition to the joist, a 400 mm wide x 65 mm thick clear concrete flange was cast on top of the 1 mm corrugated steel sheets, fastened to the top cord of each joist. Figure 3.1 shows the details of the test specimens. Table 3.1 lists details regarding to the shear connectors used in phase I.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Concrete Top Flange</th>
<th>Shear Connector Type</th>
<th>Shear Connector Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>COW-S</td>
<td>400 mm wide x 65 mm thick</td>
<td>Shear stud, 13 mm Dia.</td>
<td>300 mm longitudinally</td>
</tr>
<tr>
<td>COW-W</td>
<td>400 mm wide x 65 mm thick</td>
<td>Puddle Weld, 19 mm Dia.</td>
<td></td>
</tr>
<tr>
<td>COW-P1</td>
<td>400 mm wide x 65 mm thick</td>
<td>Small Hilti screw, 4.14 mm Dia.</td>
<td></td>
</tr>
<tr>
<td>COW-P2</td>
<td>400 mm wide x 65 mm thick</td>
<td>Larger Hilti screw, 6.35 mm Dia.</td>
<td></td>
</tr>
<tr>
<td>COW-00</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

For test specimen COW-S, traditional shear studs, 13 mm in diameter and 100 mm deep were used as shear connectors. The studs were welded to the top cord through the steel deck at the middle of each flute in the deck sheets, 300 mm spacing in the longitudinal direction. Puddle-welds were used as shear connectors in test specimen COW-W. Puddle-welds of 19 mm diameter were welded to the top cord and metal deck at the middle of each flute in the metal deck sheets, 300 mm spacing in the longitudinal direction. Hilti screw pins were used as shear connectors in test specimens COW-P1 and COW-P2. Two different Hilti-screws of 4.14 mm and 6.35 mm diameters were used in COW-P1 and COW-P2, respectively. The screws were fastened to the top cord and metal
deck at the middle of each flute in the metal deck sheets, 300 mm spacing in the longitudinal direction.

![Diagram of small-scale composite T-beam](image1)

a) Cross section of small-scale composite T-beam

![Profile view with shear connectors highlighted](image2)

b) Profile view with shear connectors highlighted

![Auxiliary view](image3)

c) Auxiliary view

**Figure 3.1: Phase I small-scale composite T-beams**

The construction process began with leveling the supports, joists and decking, then fastening the deck to the joists with the respective shear connection, ie. Hilti-screw, puddle weld or shear stud. The flutes, or valleys, in the profile allow for the shearing connections between the slab and joists. The steel decking measures 2.4 m transversally across the deck, 915 mm in the longitudinal direction and has a gauge thickness of 1 mm;
the corrugation profile can be seen in Figure 3.3. The deck sections were cut into 400 mm widths for phase I. Only the concrete above the peaks in the deck is considered to resist compression and this is referred to as the clear cover. A welded wire fabric (WWF) was placed at mid height in the concrete cover to mitigate shrinkage and cracking.

The shear connections are spaced 300 mm longitudinally and made on one leg of the top chord only to mimic the worst scenario in field conditions. The steel deck sheets overlap each other and two shear connectors are placed at deck seams, this is illustrated in Figure 3.1. Next, wooden forms are manufactured and the concrete deck was cast on top of the steel decking. In the case of phase I, the four composite beams were constructed and cast at the same time; dividing forms were used to separate the beams. The welded wire mesh was used to control cracking in the slab. After casting, a moist environment is created using wet burlap and plastic sheathing to ensure the concrete does not dry out and crack; the curing process lasted 7 days. The process is sequentially laid out in Figure 3.2-a-f.
a) OWSJ clamped to supports

b) Steel decking placed on OWSJ

c) Shear connectors with WWF and forms

d) Composite deck during curing process

e) Full-scale composite deck after curing

f) Small-scale composite beams after curing

**Figure 3.2:** Construction of composite structures
3.1.2. Phase II: Two Full-scale Composite Floor Decks

In phase II, two full-scale structures were studied to gain a better understanding of composite structures and to see if composite action can be obtained without the use of the traditional shear studs. The composite structures built in phase II of the experimental program were full-scale, steel and concrete composite floor decks. Each of the composite decks was configured identical to one another, with the exception of the shear connectors and secondary reinforcement in the slab; this is discussed further in section 3.1.2. The dimensions were based on current construction practices where joist are spaced 1.2 m (4 feet) and spans typically range from 5-10 m.

The first composite deck utilized a 19 mm puddle weld to make the deck-joist shear connection (CD-W); whereas, the second composite deck employed a 6.35 mm steel deck screw or Hilti-screw for the same purpose (CD-S2). Further details regarding the phase II shear connectors are listed in Table 3.2.

**Figure 3.3:** Corrugated steel decking profile (One sheet) (Canadian Joist and Deck, 2007)
Table 3.2: Phase II Shear Connectors

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Shear Connector Type</th>
<th>Connector Spacing</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>CD-W</td>
<td>19 mm Puddle weld</td>
<td>300 mm Longitudinally</td>
<td></td>
</tr>
<tr>
<td>CD-S2</td>
<td>6.35 mm Hilti-screw</td>
<td>300 mm Longitudinally</td>
<td></td>
</tr>
</tbody>
</table>

Each floor deck in phase II measured 6.4 m long and 2.4 m wide, with a 65 mm clear concrete cover cast on top of the 1 mm corrugated steel deck sheets. Two, 254 mm deep, 6.4 m long open web steel joist are spaced 1.2 m transversally under the steel decking; this leaves a 600 mm cantilever on each side of the floor. The working length of the joists was 6.32 m and the floor details can be seen below in Figure 3.4.

The joist’s top and bottom chords are comprised of two L44x44x3.2 and two L51x51x4.8 back-to-back angles, respectively. The web members are 11/16’ round bar or about 17.5 mm in diameter. A steel deck sheet measures 2.4 m transversally across the deck, 915 mm in the longitudinal direction and has a gauge thickness of 1 mm; the corrugation profile can be seen in Figure 3.2.

The 6.35 mm Hilti-screw was chosen for the second shear connector in phase II because of positive results from phase I as well as its ease of installation and economical cost. CD-S2 also utilized a Fiber Reinforce Concrete (FRC) slab in lieu of a Welded Wire Fabric mesh. Small glass fibers were added to the concrete mix just prior to pouring and this replaced any secondary reinforcement. Mixing ratios are based off applications and manufacture recommendations; most suppliers possess a specific glass fiber mix for
composite floor applications. The dimensions of the phase II test prototypes can be seen in Figure 3.4.

![Figure 3.4: Phase II full-scale composite floor deck](image)

**Figure 3.4:** Phase II full-scale composite floor deck

### 3.2 Test Setup

The four composite beams in phase I were simply supported over their 4.57 m span. Two line loads were placed at the 1/3 points on the beam where a stiff spreader beam was used to apply the load until failure in successive, gradually increasing monotonic load cycles.
Each of the two composite floor decks in phase II were simply supported over their 6.4 m longitudinal length and loaded monotonically as a successive cycle, loading procedure. Two line loads were placed at the 1/3 span points on the deck and span the entire 2.4 m width, refer to Figure 3.4. The actuator is oriented directly in the centre of the deck so an ingenious loading setup was needed to ensure an even distribution of load across the two line loads. The load setup can be seen in Figure 3.4.

3.3. Loading Procedures

3.3.1. Phase I Loading Procedures

The structures were all loaded in 10 kN incremental cycles. i.e. 10 kN, 20 kN, 30 kN, 40 kN and load till failure. Figure 3.5 displays the successive load cycles in phase I. The monotonic load was applied through the use of the same hydraulic actuator as phase II and the same precautions were taken with respect to even load distribution.
The monotonic load was applied through the use of a 500 kN hydraulic actuator with a +/- 1250 mm stroke. Successive, 10 kN incremental load cycles were applied to each beam up until failure; the loading scheme is presented in Figure 3.5.

### Phase II: CD-W Loading Procedures

A procedure according to Canadian Standards Association S-16-1980, was followed which included monotonically loading and unloading the structure to predetermined loads based on estimates before testing. The test procedure specifies 4-point bending, in three successive cycles: load up to 25% of the predicted ultimate load, load up to 60% of the predicted ultimate load and lastly, load up to ultimate failure of the structure. Figure 3.6 is shown below to depict the successive loading cycles for CD-W; the theoretical capacity of 133 kN is also displayed as the horizontal line. The load is presented in terms of a percentage of the theoretical capacity on the y-axis, across the load cycles on the x-axis.
Figure 3.6: Monotonic loading cycles for CD-W

Figure 3.7: 4-point bending load setup

3.3.3. Phase II: CD-S2 Load Procedures

The full-scale composite floor was tested in both monotonic loading as well as cyclical loading. CD-S2 underwent monotonic loading cycles as well as 50 000 cyclical
loading cycles at various loads. The structure was first subjected to monotonic loadings at service loads followed by cyclical testing. After every 10 000 cycles, a monotonic cycle was completed to track digression of the structure. The loading cycles are defined in detail below in this section.

Table 3.3: Phase II Loading Cycles

<table>
<thead>
<tr>
<th>Load Cycle</th>
<th>Cycle Name</th>
<th>Monotonic/Cyclical</th>
<th>Max Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1\textsuperscript{st} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>2\textsuperscript{nd} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>3</td>
<td>3\textsuperscript{rd} Cycle</td>
<td>M</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>4\textsuperscript{th} Cycle</td>
<td>M</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>5\textsuperscript{th} Cycle</td>
<td>M</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>6\textsuperscript{th} Cycle</td>
<td>M</td>
<td>70</td>
</tr>
<tr>
<td>7</td>
<td>5 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>8</td>
<td>6\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>9</td>
<td>10 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>10</td>
<td>7\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>11</td>
<td>20 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>12</td>
<td>8\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>13</td>
<td>30 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>14</td>
<td>9\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>15</td>
<td>40 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>16</td>
<td>10\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>17</td>
<td>11\textsuperscript{th} Cycle</td>
<td>M</td>
<td>70</td>
</tr>
<tr>
<td>18</td>
<td>50 000\textsuperscript{th} Cycle</td>
<td>C</td>
<td>40</td>
</tr>
<tr>
<td>19</td>
<td>12\textsuperscript{th} Cycle</td>
<td>M</td>
<td>40</td>
</tr>
<tr>
<td>20</td>
<td>13\textsuperscript{th} Cycle</td>
<td>M</td>
<td>70</td>
</tr>
<tr>
<td>21</td>
<td>14\textsuperscript{th} Cycle</td>
<td>M</td>
<td>90</td>
</tr>
<tr>
<td>22</td>
<td>15\textsuperscript{th} Cycle</td>
<td>M</td>
<td>110</td>
</tr>
<tr>
<td>23</td>
<td>16\textsuperscript{th} Cycle</td>
<td>M</td>
<td>Failure at 130</td>
</tr>
</tbody>
</table>

Over the course of 23 different monotonic load cycles, the deck was subjected to various loadings relating to SERVICE and ULTIMATE live loads for RESIDENTIAL and INDUSTRIAL settings; the cycle details are tabulated in Table 3.3, (National Research Council, 2010). In a monotonic load cycle, the load is applied at a rate of 10
kN/minute; once the max load was achieved, the load was removed. During the fatigue cycle loading, the max load of 40 kN is calibrated with the minimum load of 8 kN using displacement control with the actuator at, a frequency of 0.4 Hz, or 1 cycle every 2.5 seconds.

3.4. Instrumentation

3.4.1. Phase I Instrumentation

Several different types of sensors were used in order to retrieve pertinent information from the structure during testing. Both steel and concrete, 10 and 70 mm, electrical foil strain gauges, respectively, were strategically installed throughout each specimen just prior to testing. Table 3.4 lists all steel and concrete strain gauges and their locations with respect to depth from the top surface and also position across the front profile of the specimens; a schematic drawing is also provided in Figure 3.8.

Linear variable differential transformers (LVDT’s) were installed to measure deflection of key components of the composite deck during testing. All LVDT’s are listed in Table 3.5; all four specimens in phase I were setup identically. A load cell was also used to collect loading data to be paired against strain, deflection etc.
### Table 3.4: Strain Gauge Locations in Phase I

<table>
<thead>
<tr>
<th>Strain Gauge</th>
<th>Location on Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete 1</td>
<td>Placed on the top of concrete slab near the edge at 1/3 span, 0 mm from the top surface</td>
</tr>
<tr>
<td>Concrete 2</td>
<td>Placed on the front face of slab at 1/3 span, -20 mm from the top surface</td>
</tr>
<tr>
<td>Concrete 3</td>
<td>Placed on the slab front face of slab at 1/3 span, -45 mm from top surface</td>
</tr>
<tr>
<td>Steel 1</td>
<td>Placed on the underside of the Top Chord at 1/3 span, -127 mm from the top surface</td>
</tr>
<tr>
<td>Steel 2</td>
<td>Placed on the front of the Top Chord at 1/3 span (horizontal leg), -143 mm from the top surface</td>
</tr>
<tr>
<td>Steel 3</td>
<td>Placed on the front of the Bottom Chord at 1/3 span (horizontal leg), -360 mm from the top surface</td>
</tr>
<tr>
<td>Steel 4</td>
<td>Placed on the underside of the Bottom Chord at 1/3 span, -375 mm from the top surface</td>
</tr>
<tr>
<td>Steel 5</td>
<td>Placed on the underside of the Top Chord at 2/3 span, -127 mm from the top surface</td>
</tr>
<tr>
<td>Steel 6</td>
<td>Placed on the front of the Top Chord at 2/3 span (horizontal leg), -143 mm from the top surface</td>
</tr>
<tr>
<td>Steel 7</td>
<td>Placed on the front of the Bottom Chord at 2/3 span (horizontal leg), -360 mm from the top surface</td>
</tr>
<tr>
<td>Steel 8</td>
<td>Placed on the underside of the Bottom Chord at 2/3 span, -375 mm from the top surface</td>
</tr>
</tbody>
</table>

### Table 3.5: LVDT Locations in Phase I

<table>
<thead>
<tr>
<th>LVDT</th>
<th>Location on Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVDT 1</td>
<td>Placed at 1/3 span to measure deflection of the Concrete Deck</td>
</tr>
<tr>
<td>LVDT 2</td>
<td>Placed at 1/3 span to measure deflection of the Top Chord</td>
</tr>
<tr>
<td>LVDT 3</td>
<td>Placed at 1/3 span to measure the deflection of the Bottom Chord</td>
</tr>
</tbody>
</table>
3.4.2 Phase II Instrumentation

The same steel and concrete strain gauges were used but are located in different places across each of the full-scale composite decks. Table 3.6 lists all steel and concrete strain gauges and their locations with respect to depth from the top surface and also position across the front profile for CD-W; a schematic is provided in Figure 3.9.

The same LVDT’s were installed to measure deflection in phase II; the locations for phase II are listed in Table 3.7 below and are shown in Figure 3.9. Both specimens in phase II were setup identically. A load cell was also used to collect loading data to be paired against strain, deflection etc.
**Table 3.6**: Strain Gauge Locations in Phase II

<table>
<thead>
<tr>
<th>Strain Gauge</th>
<th>Location on Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete 1</td>
<td>Placed at centre on the top of concrete slab, 0 mm from top surface</td>
</tr>
<tr>
<td>Concrete 2</td>
<td>Placed above the truss on the top of concrete slab, 0 mm from top surface</td>
</tr>
<tr>
<td>Concrete 3</td>
<td>Placed near the edge of the top of the concrete slab, 0 mm from top surface</td>
</tr>
<tr>
<td>Concrete 4</td>
<td>Placed near the top edge on the side of the concrete slab, -13 mm from top surface</td>
</tr>
<tr>
<td>Concrete 5</td>
<td>Placed near the bottom edge on the side of the concrete slab, -33 mm from top surface</td>
</tr>
<tr>
<td>Steel 1</td>
<td>On the front truss, placed on the underside of the diagonal on the far left side</td>
</tr>
<tr>
<td>Steel 2</td>
<td>On the front truss, placed on the underside of the bottom chord left of centre near first diagonal, -369 mm from the top surface</td>
</tr>
<tr>
<td>Steel 3</td>
<td>On the front truss, placed on the underside of the top chord at centre, -120 mm from the top surface</td>
</tr>
<tr>
<td>Steel 4</td>
<td>On the front truss, placed on the front of the top chord at centre (horizontal leg), -137 mm from the top surface</td>
</tr>
<tr>
<td>Steel 5</td>
<td>On the front truss, placed on the underside of the bottom chord to the left of centre, -369 mm from the top surface</td>
</tr>
<tr>
<td>Steel 6</td>
<td>On the front truss, placed on the front of the bottom chord, right of centre (horizontal leg), -350 mm from the top surface</td>
</tr>
<tr>
<td>Steel 7</td>
<td>On the back truss, placed on the underside of the bottom chord to the right of centre, -369 mm from the top surface</td>
</tr>
<tr>
<td>Steel 8</td>
<td>On the back truss, placed on the underside of the bottom chord to the left of centre, -369 mm from the top surface</td>
</tr>
<tr>
<td>Steel 9</td>
<td>On the back truss, placed on the underside of the top chord at centre, -120 mm from the top surface</td>
</tr>
</tbody>
</table>

**Table 3.7**: LVDT Locations in Phase II

<table>
<thead>
<tr>
<th>LVDT</th>
<th>Location on Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVDT 1</td>
<td>Placed at centre to measure deflection of the Concrete Deck</td>
</tr>
<tr>
<td>LVDT 2</td>
<td>Placed at centre to measure deflection of the Top Chord on the front joist</td>
</tr>
<tr>
<td>LVDT 3</td>
<td>Placed at centre to measure the deflection of the Bottom Chord on the front joist</td>
</tr>
</tbody>
</table>
3.5. Material Properties

Material testing is conducted in order to verify the strength and behavior of the materials used, prior to testing the full scale structures. Concrete cylinder samples were collected from the concrete mix just prior to casting each of the two concrete deck slabs and set aside. Compression and split tests were conducted to verify the strength for the concrete and their results are listed in Appendix A. All cylinders were prepared and tested according to ASTM C39/39M; a compression cylinder sample is shown in Figure 3.10.

Tension tests were conducted during phase I on the joist material. Extra angle material was supplied and cut into tension test specimens according to American
Standard for testing Materials – Test # E8/E8M. The tension tests were conducted in order to verify the manufacturer’s specifications and the results are listed in Appendix A.

Figure 3.10: Typical concrete cylinder

Double shear tests were also conducted in phase I. Six tests were conducted on two of the alternate shear connectors: the 19 mm puddle weld and the 6.35 mm Hilti-screw in order to verify the failure mode, strength and behavior of the deck joist connections. Three tests on each connector were conducted and the specimens consisted of two pieces of angle material, each sandwiched and fastened at the ends between two pieces of decking material. The specimens were attached to the universal testing machine at each end to apply an axial tensile force. Figure 3.11 shows the details of the tension tests and double shear test specimens; the results are listed in Appendix B.
A glass fiber reinforced concrete mix was used in the second composite deck in phase II. MasterFiber MAC 100 Plus fibers were used; they are a macro synthetic fiber. These are specifically designed for secondary reinforcement in composite metal decks and are produced by BASF – Admixture Solutions. The fibers came in a biodegradable bag that can be safely added to the concrete truck/mix, allowing for easy mixing. MAC 100 Plus fibers call for a mixing ratio of 3.0 kg/m$^3$ of concrete; this translates to about 5 kgs used in CD-S2. The fibers and biodegradable bag are shown in Figure 3.12-a-b. the manufactures product details can be found in Appendix D.

**Figure 3.11:** Phase I material and connection testing

**Figure 3.12:** MAC 100 Plus fibers for GFRC
CHAPTER 4: TEST RESULTS AND ANALYSIS

The results of the two phases in the experimental program are presented through a multitude of comparisons including: load vs. strain and load vs. deflection relationships at different load stages. Cross sectional strain profiles are also provided; these fundamentally quantify the performance of the composite structures and the effectiveness of composite action, while assisting with failure mode identification.

4.1. Phase I: Small-scale Composite T-Beams (COW-xx)

4.1.1. Deflection Behaviour

Figure 4.1 displays load vs. max deflection at \( \frac{1}{3} \) span, under the line load for each of the four composite beams at the 30 kN load; the max load in cycle 3. The 30 kN monotonic load is equivalent to an applied moment of 23 kN-m.

It can be noticed that specimen COW-S was the stiffest beam, deflecting less than 10 mm while COW-W, COW-P1 and COW-P2 deflected about 15.5 mm, 17 mm and 20 mm, respectively. When compared to the control beam, COW-W, COW-P1 and COW-P2 deflect an additional 63%, 111% and 79%, respectively, at the 30 kN monotonic load in cycle 3.
Figure 4.1: Phase I load vs. bottom chord deflection at 30 kN

Predominantly bi-linear behaviour can be seen here in each beam, all of which showed minimal residual deflections of 1.5 mm, 3 mm, 4 mm and 4.5 mm for COW-S, COW-P1, COW-W and COW-P2, respectively. A horizontal line is shown to represent the service live load for residential dwellings according to National Building Code of Canada, 2010 (National Research Council, 2010), which is 2.4 kPa or equivalent to a 13 kN monotonic load here. The deflections at this service load are 3.5 mm, 6 mm, 7.8 mm and 8.2 mm for COW-S, COW-W, COW-P1 and COW-P2, respectively. The allowable live load deflection for a simply supported composite beam of this size is about 15 mm, larger than the service deflections of each of the four composite beams in phase I.

The allowable live load deflection limit according to Canadian Standards Association S-16-09 is labeled as the black horizontal line at about 15 mm; this is equivalent to $L/300$ or $4572 \text{mm}/300$. 
Figure 4.2: Phase I max deflection in cycles 3 and 4

Figure 4.2, shows a comparison of the max deflection for each composite beam at the 30 kN load level and failure load (3rd of 4th load cycles). COW-S is seen as the stiffest beam followed by COW-W, COW-P2 and COW-P1. All of the composite beams in phase I, with the exception of COW-P1, lie within the allowable live load deflection, or about 15 mm, even at twice the service live load (at 30 kN).

Figure 4.3 displays the load vs. deflection plots for the two LVDT’s; one on top of the concrete deck and the second is located on the bottom chord at third span. An opening or gap between the LVDT measurements would indicate a physical separation of the concrete deck from the OWSJ.

Minimal separation can be seen for COW-W and COW-W but COW-W showed a slightly tighter connection with the concrete and OWSJ series overlapping each other during cycle 3; Figure 4.3-a. Lastly, there is minimal residual deflection between each
series while standing alone, indicating as the beam was unloaded, the concrete deck and OWSJ revert back to unloaded behaviour.

COW-P1 is compared alongside COW-S in Figure 4.3-b, where concrete and bottom chord deflections are displayed. Minimal separation can be seen in COW-P1 and more deflection is realized by COW-P1 in cycle 3, but as the deck was unloaded, minimal residual deflections and separation was visible. This speaks to the resilience of the alternative shear connector.

Almost no separation between the concrete and OWSJ is seen in the case of COW-P2 in cycle 3 as seen in Figure 4.3-c. The two series overlap each other as the beam was loaded and unloaded up to 30 kN, or an applied moment of 23 kN-m.
Figure 4.3: Phase I top concrete and bottom chord load vs. deflection

(a) COW-S vs. COW-W
(b) COW-S vs. COW-P1
(c) COW-S vs. COW-P2
4.1.2. Strain Behaviour

Strain results in phase I are presented in terms of load vs. strain relationships as well as ultimate capacity which were all measured during the monotonic loading stages laid out in Figure 3.5. Several strain gauges have been selected to produce load vs. max strain characteristic plots for each of the four composite beams at the 30 kN load; or the max load in cycle 3. Each of the three shear connector alternate beams is compared alongside the shear studded control beam. The top chord strain and the extreme tension fiber of the bottom chord in the OWSJ are shown together on each plot; the top chord is represented by the broken line and the bottom chord is the solid line. The larger strain is utilized between the left and right sides of the beam; this is to display the critical strain at the corresponding load.

A load vs. strain plot is shown in Figure 4.4-a, the graph compare COW-S to COW-W. Both the top and bottom chord are labelled according to the legend, at 30 kN. The bottom chord strains for both beams are below yielding of 1750 micro-strain and deforming linearly elastic as expected. The bottom chord in COW-W displayed about 37% more tensile strain than the control beam. The top chord strain of COW-S is positive indicating that composite action is present while the top chord in COW-W is negative, indicating partial composite action. The neutral axis has fallen out of the concrete flange; however, the strain values are very slight. Lastly, the composite beams remained fairly elastic, leaving a small amount of residual strain after cycle 3.
Figure 4.4: Phase I load vs. max strain in top chord and bottom chords
The load vs. strain results for COW-P1 is compared to COW-S in Figure 4.4-b; again for the top and bottom chord strains. The bottom chord in COW-P1 behaved linearly up to the 30 kN max load and the top chord remained at near zero strain, straddling the y-axis. COW-P1 behaved very similarly to COW-S and all components remained below the yielding strain of 1750 micro-strain. The bottom chord of COW-P1 undertook 45% more tensile strain at the 30 kN load as compared to the control beam.

Load vs. strain is displayed for the top and bottom chords of COW-P2, compared alongside to COW-S in Figure 4.4-c. The bottom chords of each beam behaved very similarly through the 3rd cycle; however, COW-P2s top chord took on compressive strain indicating partial composite behaviour. The max negative/positive strain values are all far below yielding at 1750 micro-strain and this is also evident by the linear elastic behaviour of the bottom chord. In addition, the bottom chord in COW-P2 only exhibited 7% more tensile strain than the control beam, COW-S, less than the other two alternate shear connectors.

4.1.3. Cross Sectional Strain Profiles

In this section, graphs are provided showing the cross sectional strain profile for the respective phase (or composite structure), at specific loadings relevant to service and ultimate live loading conditions. The graphs are strain profiles across the depth of the composite structure at the same load, ie. Each graph contains multiple gauges, organized according to depth from the top concrete surface, to the bottom of the OWSJs bottom chord; all at the same monotonic load. Additionally, two horizontal lines are plotted alongside the strain profile to help differentiate where the neutral axis(s) lie according to the profile of the composite structure. The lines represent the change in continuity of the
concrete slab and shearing plane or deck-joist connection, see Figure 4.5. The neutral axis(s) lie where the strain profile intersects the y-axis. Sample strain gauges have been illustrated across the depth as well.

**Figure 4.5:** Cross sectional strain profile fundamentals

Figure 4.6-a-b-c shows cross sectional strain profiles of the four composite beams: one for each of the shear stud alternatives, compared to a shear stud beam. The strain profiles are taken at the 13 kN, service live load in the second cycle in each test; the order is as follows: COW-W vs. COW-S, COW-P1 vs. COW-S and COW-P2 vs. COW-S. The continuous, solid black line is the control COW-S strain profile and the lighter line corresponds to the respective alternate shear connector.

It can be noticed that COW-S exhibited composite action with the neutral axis lying within the fluted portion of the concrete deck at the service live load. In Figure 4.6-a, the strain discontinuity of COW-W is evidence that there is partial composite action at the service load level and both the concrete deck. The max strain in the bottom chord of
COW-W was less than 600 micro-strain, less than half of the yielding strain, nor did the concrete reach crushing strain of 3500 micro-strain.

COW-P1 in Figure 4.6-b and the strain profile is more similar to COW-S. The profile was steeper sloped and the OWSJ exhibited small amount of strain at the service live load (13 kN).

Referring to Figure 4.6-c, featuring COW-P2 vs. COW-S, the shear stud alternative had already lost full composite action at 13 kN and was exuding partial composite characteristics. However, the bottom chord strain for COW-P2 was below 1000 micro-strain, far below yielding and the top chord was experiencing small compression forces. The concrete also remained far below crushing strain at about -250 micro-strain.
Figure 4.6: Phase I cross sectional strain profile comparisons
4.1.4. Failure Modes

For a full and ideal composite section, the failure mode has to be initiated by yielding of the bottom cord of the OWSJ while the top cord is under tension forces. This means that the neutral axis is located within the concrete slab and no premature failure due to buckling of the top cord would occur. Also, when the top cord exhibits a small or insignificant compression force and the neutral axis is located outside the concrete deck, within the depth of the OWSJ, the failure should be initiated by yielding of the bottom cord in tension.

Figure 4.7: Phase I failure mode

Figure 4.6 shows comparisons of the strain distribution, strain profile, across the depth of the test specimens near failure. It can be noticed that, at failure, test specimens COW-W, COW-P1 and COW-P2 were losing composite action and only resembled
partial composite action. The failure was initiated by in-plane and out-of-plane, local buckling of the top cord, followed by delamination between the metal steel deck sheets which lead to a shear failure of the concrete slab right under the load as shown in Figure 4.7. For the test specimen COW-S, the strain profile shows a perfect composite action at failure and both the bottom and top cords have a maximum strain that is less than the yielding strain.

4.2. Phase II: Full-scale Composite Floor Decks (CD-xx)

4.2.1. Deflection Behaviour

4.2.1.1. Puddle weld composite deck (CD-W)

The live load deflection results are presented in the form of load vs. deflection in each case or cycle.

Load vs. deflection plots are provided below for two of the LVDT’s used in phase II; the concrete deck and bottom chord deflections at \(1/3\) span shown in Figure 4.8. Also, the allowable live load deflection for the simply supported composite deck is 21 mm or \(L/300\) (Canadian Standards Association, 2009).

The cycle 1 load vs. deflection behaviour is displayed in Figure 4.8-a; this is the deflection at midspan in each case. Both the top concrete deck and bottom chord deflection are presented, and both series almost overlap each other up to the 30 kN load. The small gap between each series is attributed to small slippage within the composite deck; this is typical in the settling load cycle. CD-W deflected about 5 mm under the monotonic live load in cycle 1.
Figure 4.8-b displays a load vs. deflection graph similar to Figure 4.8-a, but for the 2nd load cycle. Both the concrete deck and bottom chord deflect bi-linearly up to the 80 kN, monotonic, live load. The 80 kN load surpasses the service and ultimate live loadings; corresponding to loads of 40 kN and 58 kN (National Research Council, 2010). The total deflection for the concrete deck and bottom chord was about 13 mm and 14 mm respectively, still below the allowable live load deflection of 21 mm (Canadian Standards Association, 2009). After the loading was removed, CD-W relaxed back to a residual deflection of about 1.25 mm.

CD-W deflected bi-linearly through cycle 3 up to a load of about 127 kN; more than double the ultimate live load rating for residential dwellings. The concrete deck and bottom OWSJ remained intact through some plastic deformation, right up to global failure at around 45 mm of deflection at the max load. This corresponds to an increase of 233% in deflection while only a 57.5% increase in max load from cycle 2 to cycle 3 up to the failure load. However, during cycle 3, CD-W deflected 28.5 mm (59% of Δ at P_{failure}) from loads 100 kN to 127 kN at ultimate. This indicating most of the deflection took place after sufficient loads were sustained, then large deflections occurred which would warn occupants of impending, global failure. Lastly, CD-S2 did not reach the allowable live load deflection (21 mm) until at loads greater than 2.5 times the service load.
Figure 4.8: Phase II CD-W load vs. deflection concrete and bottom chord
4.2.1.2. 6.35 mm Hilti-screw composite deck (CD-S2)

The first set of graphs depicts the load vs. deflection readings of structure before any cyclic loading had commenced. Monotonic load cycles 2, 4 and 6 (up to 40 kN, 50 kN and 70 kN respectively) are all shown in Figure 4.9, for a load vs. bottom chord deflection plot.

![Figure 4.9: Phase II CD-S2 load vs. bottom chord deflection prior to cyclical loading](image)

These loads closely correspond to service (36 kN) and ultimate (54 kN) live load ratings for residential dwellings as well as the ultimate load for industrial buildings (72 kN) (National Research Council, 2010). There is about 3.5 mm of deflection before the start of cycle 2 which is attributed to dead load deflection and settling. The allowable live load deflection for CD-S2, similarly to CD-W, is 21 mm (Canadian Standards Association, 2009) and this limit was not reached while loading up to the ultimate loading condition for residential dwellings, indicating CD-S2 had passed all deflection
checks prior to fatigue testing. The max deflection seen at the max load in cycle 6 was just less than 20 mm, an increase of 52.5% from the service load in cycle 2.

After the cyclical cycle loading was calibrated and tested in cycle 6, the cyclical loading began and followed the scheme in Table 3.3. Figure 4.10 below displays the load vs. bottom chord deflection during cyclical loading. The fatigue response, displays the load vs. deflection for the concrete deck and bottom chord during the fatigue cycling. The starting and ending responses are taken at during 1000 and 49 000 cycles into the 50 000 cycles, all from a max load of 40 kN and a minimum of 8 kN.

![Figure 4.10: Phase II CD-S2 deflection fatigue response](image)

The bottom chord and concrete deck did not accumulate much deflection over the fatigue testing, and there was not much separation between the two series either. This indicates that the composite deck remained intact over the course of many cycles. The deflection amplitude is about 8.5 mm.
After the 50 000 cycles, another monotonic load cycle (cycle 19) was conducted at the ultimate live load (70 kN) to assess any degradation due to fatigue loading. Figure 4.11 compares pre-fatigue to post-fatigue load vs. deflection at the ultimate live load level.

**Figure 4.11:** Phase II ultimate live loading before and after fatigue loading

Thereafter, the specimen was loaded monotonically up to failure in four consecutive load cycles, each at increasing load levels. (cycles 20-23, Table 3.3). The bottom chord deflections are shown below in Figure 4.11 for cycle 19, 20 and 23. These three cycles relate to the service and ultimate live load ratings for residential dwellings, and the last cycle is the ultimate capacity of CD-S2.
**Figure 4.12:** Phase II CD-S2 load vs. bottom chord deflection—cycles 20-23

The bottom chord deflection is plotted against load in Figure 4.12 for monotonic load cycles after the cyclical cycling. There was little residual deflection from all of the load cycles and the bottom chord was still exhibiting linear to bi-linear elastic behaviour up until global failure around 130 kN. For a span of 6.4 m, the allowable live load deflection is 21 mm (Canadian Standards Association, 2009) which is not surpassed until 60 kN. While reloading up to the ultimate residential live load, post fatigue testing, the deflection limit was surpassed at about 65 kN. After the first 18 load cycles (includes 50,000 cyclical cycles), CD-S2 only exhibited about 10 mm of residual deflection, loading into cycle 19. From monotonic cycles 19 to cycle 22, very little residual deflection was observed and the deck remained fairly elastic. In cycle 23, CD-S2 was loaded till failure and larger deflections were measured before global failure occurred.
4.2.2. Strain Behaviour

4.2.2.1. Puddle weld composite deck (CD-W)

Several distinct strain gauges have been selected to generate load vs. max strain plots for each of the three load cycles. Steel gauges are chosen to display the max strains in the top and bottom chords and concrete gauges for the concrete deck.

Test specimen CD-W showed a very promising performance in phase II. Full-composite action was observed during the full range of loads applied in the three successive cycles. Composite action was eventually lost and the structure failed in a distinct fashion. The load vs. max strain is shown below for the top concrete deck fiber, the top chord of the OWSJ and the bottom (chord) tensile fiber of the OWSJ.

In Figure 4.13-a, the concrete deck displayed very elastic behaviour through cycle 2 up to 80 kN and into the 3rd cycle. The 2nd and 3rd load cycle ranges exceed the service load value for residential and industrial settings which are about 36 kN and 72 kN respectively for this size of composite floor and load setup used. (2.4kPa and 4.8 kPa surface live loads – National Research Council, 2010). Non-elastic behaviour is noticed at loads above 100 kN which is due to the global loss of composite action and impending failure. The deck remained far from the concrete crushing strain of 3500 micro-strain during testing indicating that the concrete deck is of sufficient dimensions for these load ratings. Figure 4.13-b is displaying the load vs. strain in the top and bottom chords of test specimen CD-W, in each of the three load cycles.
a) Extreme compression fiber – top concrete deck strain

b) OWSJ strain – top and bottom chord

**Figure 4.13**: Phase II CD-W load vs. strain results – concrete and OWSJ
The bottom chord strains are illustrated on the right with a solid line whereas the top chord strains are represented by a dotted line on the left. It can be seen that the top chord is exhibiting positive strain/ tensile strain, elastically, well into the 3rd load cycle up over 100 kN. This indicates the neutral axis remains in the concrete deck above the OWSJ.

The strain values are far from yielding strains of 1750 micro-strain and there is no residual strain before loading into the 3rd cycle in the top chord. This indicates that the deformations and failure was due to in/out-of-plane buckling as opposed to sectional failure of the chords. The bottom chord is showing completely positive and elastic strain as expected with little to no residual strain between cycles. The strain becomes bi-liner in the 3rd cycle just prior to the global failure of the deck and where yielding would occur, but the bottom chord is not responsible for the global failure.

4.2.2.2. 6.35 mm Hilti-screw composite deck (CD-S2)

Tests specimen CD-S2 underwent extensive cyclical loading in addition to monotonic cycles as described in Chapter 3; results are organized in a chronological fashion and only the monotonic load cycle results are shown.

The first set of graphs depicts the load vs. strain readings of CD-S2 before any cyclical cycles have commenced. Plots for top chord, bottom chord and concrete deck, load vs. strain are provided in Figure 4.14. Cycles 2, 4 and 6 are displayed corresponding to 40 kN, 50 kN and 73 kN. These are closely representative of the service (40 kN) and ultimate (70 kN) live load ratings for residential according to NBCC 2010 (Canadian Research Council, 2010).
The load vs. strain plot in Figure 4.14-a displays strain results for the top concrete surface of CD-S2, for load cycles 2, 4 and 6. The strains were all far below the concrete crushing strain of 3500 micro-strain and the series are fairly linear and elastic. There was very little residual strain from the time of shoring removal all the way through cycle 6.

**Figure 4.14:** Phase II CD-S2 load vs. strain prior to cyclic loading
In each case the top chord was exuding negative or compression strain, meaning the neutral axis has already fallen out of the concrete slab, CD-S2 was showing partially composite characteristics. The max strain values are far below the yielding strains and the strain behaviour is linear or bi-linear in the top chord. The gaps between residual strains originate from the cycles in between the cycles show, ie. Cycles 1, 3 and 5. The bottom chord was linear elastically deforming in each cycle; explaining the minor residual strain over the successive load cycles. The max strain values were also far below the yielding strain of 1750 micro-strain.

Figure 4.15: Phase II CD-S2 fatigue response – top and bottom chords

After cycle 6, the cyclical loading began and followed the scheme in Table 3.3. Figure 4.15 displays the top and bottom chord strains again at two key load levels: at the beginning and the end of the fatigue/cyclical loading.
A fatigue response is presented here for CD-S2, this time for load vs. strain in the top and bottom chords of the OWSJ. The bottom chord strain is on the right, tension, and the top chord is on the left, or in compression strain. Neither of the two chords developed much residual strain during the fatigue cycling and also the chords remained far below yielding at 1750 micro-strain.

After the 50 000 cycles, the structure was loaded monotonically up till failure in successive cycles (cycle 19-23, Table 3.3). The load was increased for each monotonic cycle up to about 130 kN in cycle 23 when global failure occurred. The top concrete deck strain along with the top and bottom chord strains are shown below in Figure 4.16 for the ultimate (70 kN) live load ratings for residential dwellings before and after the fatigue loading for CD-S2.

Figure 4.16-a showcases the concrete compression strain at the ultimate live load level (70 kN), before and after the fatigue testing (cycle 6 and 20). The concrete strain is very small compared to the concrete crushing strain of 3500 micro-strain; further supporting that the concrete deck proportions are more than sufficient. Small residual strain and the repetitious behaviour of the concrete deck were seen, showing that the fatigue testing had not permanently/plastically damaged the deck.
The top and bottom chord strain results are shown in Figure 4.16-b. Less than 600 residual micro-strain (compression) accumulated in the top chord over the fatigue testing and the chord behaved bi-linearly, similarly to cycle 6. The top chord is

Figure 4.16: Phase II CD-S2 load vs. strain at ultimate live loading
displaying negative strain indicating that partial composite action is present; however, the
dock maintained strength up to ultimate loadings after fatigue testing.

Figure 4.17: Phase II CD-S2 load vs. strain at ultimate loading - OWSJ

Linear elastic behaviour was seen in the bottom chord right on through to the
ultimate load of 130 kN in cycle 23 (Figure 4.17). Minimal residual strains were realized
from cycle to cycle and the yielding 1750 micro-strain was not surpassed until just prior
to the ultimate load of the composite deck. This means there is the possibility that plenty
of plastic deformation can occur after the deck has failed, leaving time to escape in the
real event of a disaster.

4.2.3. Cross Sectional Strain Profiles

4.2.3.1. Puddle weld composite deck (CD-W)

The phase II, CD-W cross sectional strain profiles are presented in Figure 4.18, the
two monotonic loads are 30 kN and 70 kN. These loads closely correspond to service
(40 kN) and ultimate (70 kN) loadings for residential dwellings (National Research Council, 2010).

![Diagram showing strain profiles at 30 kN and 70 kN](image)

**Figure 4.18:** Phase II CD-W C.S.S.P. at service/ultimate live loads

Test specimen CD-W showed a very promising and competitive performance. Full-composite action was observed during the full range of loading applied during the three load cycles up to failure. The analysis of the strain measurements showed a linear
strain distribution across the depth where the neutral axis was located within the concrete
deck past the ultimate live load level.

4.2.3.2. 6.35 mm Hilti-screw composite deck (CD-S2)

The cross sectional strain profile presented in Figure 4.19 contains two series
corresponding to the service live load before (cycle 2) and after (cycle 12) the fatigue
loading. The monotonic load in each cycle is 40 kN.

![Figure 4.19: Phase II CD-S2 cross sectional strain profile at service load levels](image)

The strain profiles were not continuous but partial composite action remained
present throughout the fatigue testing. The max strain values in the joist are realized in
the top chord but are below half of yielding strains at 1750 micro-strain; negating
material failure. The bottom chord had shown great resilience over the 50 000 cycles with
little to no buildup of stress/strain. The concrete deck was only experienced minor
compressive strains, far below crushing strains. A small amount of compressive strain built up in the top chord which could indicate possible minor, local deformation.

4.2.4. Failure Modes

4.2.4.1. Puddle weld composite deck (CD-W)

Test specimen CD-W where puddle welding was used to develop the composite action showed a very promising and competitive performance. A full-composite action was observed during the full range of load applied during the three load cycles up to failure. The specimen failed at a total load of about 130 kN. The failure mode was mainly due to shear-bond failure between the concrete slab and the corrugated sheets by followed by shear failure of the concrete deck and local yielding of the top cord under the load, shown in Figure 4.20. The full-composite action limit of about 100 kN is equivalent to a service live load of 5.78 KPa (factored live load of 8.68 KPa) which is more than twice the NBCC load demand (National Research Council, 2010).
a) Test specimen CD-W after failure

b) Shear-bond failure (delamination) between the concrete slab and corrugated sheets

c) Out-of plane buckling of top cord

**Figure 4.20:** Phase II CD-W failure mode

### 4.2.4.2. 6.35 mm Hilti-screw composite deck (CD-S2)

CD-S2 also performed very well in phase II which utilized the 6.35 mm Hilti-screw for a shear connector. A slim, composite structure, the deck was subjected to large deflection and curvature as it approached failure. The curvature, in addition to longitudinal shear, caused a delamination of the steel decking from the concrete slab. This was a shear-bond failure which initiated a loss of composite action and the
subsequent diagonal shear-compression failure of CD-S2. As the concrete deck separated from the OWSJs, composite action was lost and this lead to a local, out-of-plane buckling of the top chord due to an abundance of compressive stresses. The OWSJ was now subjected to extreme shear causing a buckle of the web members (1st diagonal) and ultimately the global failure of the composite structure. The failure is displayed in Figure 4.21.

![Delamination of steel decking](image1.png) ![Shear connector at global failure](image2.png)

a) Delamination of steel decking   b) Shear connector at global failure

![Yielding in top chord](image3.png) ![Floor deck at failure](image4.png)

c) Yielding in top chord   d) Floor deck at failure

**Figure 4.21:** Phase II CD-S2 failure mode

Large deflection and curvature set in near ultimate failure and the structure refused to support any addition loading and therefore at capacity. However, the structure remained intact and most of the deformation receded once CD-S2 was unloaded. This
indicates that the floor system failed before any one element in the composite deck which is the preferred failure behaviour. The steel decking yielded around the Hilti-screw and all the shear connectors remained intact.

4.2.5. Full-scale Composite Deck Comparison

4.2.5.1. SERVICE live load comparison

Both full-scale composite floor decks performed very well in phase II. This section compares the load vs. deflection and load vs. strain results from CD-W to pre/post-fatigue results from CD-S2. The monotonic cycles are taken immediately before and after the 50 000 cyclical cycles. CD-W and CD-S2 were loaded to 33 and 40 kN respectively.

Load vs. live load deflection is presented in Figure 4.22 for two decks in phase II, at the service live loading. Both decks deflect less that the live load limit of 21 mm (Canadian Standards Association, 2009) and only about 5 mm of residual deflection is built up during the fatigue testing in CD-S2.
The load vs. strain results at the service live load rating for residential settings is presented in Figure 4.23. The top and bottom chord strains are located on the left and right respectively and pre/post fatigue is included for CD-S2. Each chord exhibited linear behaviour and strains below half of that of yielding. This is also indicated by the low residual strains.
Figure 4.23: Phase II service live load strain comparison

4.2.5.1. ULTIMATE capacity comparison

Bottom chord deflection is presented for each composite deck in Figure 4.24. CD-W and CD-S2 both failed at about 130 kN each but CD-S2 underwent fatigue loadings in addition to monotonic loadings. Each deck behaved very similarly to each other with large deflections and about 40 mm of residual deflection at their ultimate capacity.
The top and bottom chord strain at ultimate loading are shown in Figure 4.25 for phase II. Linear to bi-linear behaviour is seen in both chords, for both floor decks, till failure. Negative (compression) strain is exhibited in the top chord for each deck; however more so in CD-S2, indicating full to partial composite action up to failure.
Nevertheless, both chords in each deck did not yield and (1750 micro-strain) indicating that there was still the potential for more deflection and warning near failure, in addition to strain hardening.
CHAPTER 5: DISCUSSION AND CONCLUSIONS

Composite floor systems consisting of a concrete deck slab poured on top of corrugated steel sheets, supported over Open-Web Steel Joist (OWSJ) are widely used in industrial and commercial building repeated floors, workshops and warehouses. Composite flooring offers larger moment capacity, allowing for a more economical, smaller cross section size. Composite floors in design and construction saves a tremendous amount of time and materials needed under typical flooring construction. It also greatly reduces the cost and accelerates construction.

In the recent past there has been a drive to bring the vast benefits of composite flooring to the low-high residential market. Under residential applications (as opposed to industrial conditions), the floors are designed for lighters live loads (National Research Council, 2010) and smaller spans. Shallower joist are also selected to minimize storey height. With these reduced requirements, the composite design has been reanalyzed in order to make the practice more economical and feasible.

Currently, the Canadian Institute of Steel Construction (Canadian Standards Association, 2009) mandates the use of shear studs in composite floors as the only means to develop full composite action. However, during construction, arc spot weld (commonly called puddle welds) and/or screw pins are utilized in order to temporarily fasten and secure the metal deck sheets in place on top of the supporting members, in industrial loading/flooring applications.
To date, all design codes do not account for any level of composite action that puddle welds or screw pins might provide. The objective of this research is to determine if composite action can be achieved without the use of shear studs utilizing puddle welds and Hilti-screw pins as an alternate for shear stud connectors. To date, neither the puddle weld’s nor the screw pins strength is considered in the design and no composite action should be utilized in the analysis due to the lack of researched allotted to this topic.

Currently, there has been related research conducted in the United States. Scholars have developed a smaller connector that is installed similarly to a screw, but also has more of a head that protrudes into the concrete deck. The protrusion is designed to generate a superior steel-concrete bond like a shear stud, but with the quickness and easy installation of a Hilti-screw. This connector has been patented in the US which has directed this research towards a simpler screw connector.

The large Hilti-screw (CD-S2) performed nearly as well as the puddle weld connector when comparing stiffness and ultimate capacity behaviour. Due to the great advantages the screws provides over the welded type connectors (Shear studs, Puddle welds), the screws are a much more promising and feasible shear stud alternative for composite flooring in residential applications. The screws do not require specialized welders to install and also do not require ground/weld connections on each consecutive floor during construction. This greatly speeds up construction time and reduced costs considerably.
This research focuses on the test results of the two phase experimental program to explore the effectiveness and to optimize the effect of different test parameters on the performance of the new shear connectors. According to the analysis of test results, the following conclusions were obtained:
1- Significant levels of composite action can be achieved in composite constructions without the use of traditional shear studs,

2- Puddle welds showed a very competitive performance in terms of ultimate capacity and stiffness/deflection performance. The alternate connector behaved very similarly to the current practices at a whole range of loads relating to service and ultimate ratings for residential flooring applications.

3- Hilti-screws are a more economical shear connector for composite flooring when considering residential loading scenarios. They proved as an effective shear connector, remaining intact after global failure and large deflection/curvature. The showed promising behavior at residential design loads but inferior to the puddle weld connector.

4- Additional analytical and experimental research is needed before the new shear connectors can be put forth into current practice

5- Larger screw-type connectors are recommended in future experimentations. The size of screw can be varied greatly without increasing costs noticeably. Also the number of screw connections can be varied. More screws per flute as well as screws in both angles of the joist’s Top Chord have been proposed to generate greater shear resistance and composite action.
APPENDIX A – PHASE I MATERIAL PROPERTIES

Phase I ASTM C39/C39M Compression Test

Table A.1: 30 day concrete compressive strength

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Phase I ASTM E8/E8M Tensile Test on steel coupons for the OWSJ chords

Figure A.1: ASTM E8/E8M – tension test on OWSJ angle

\[ F_y = 398 \text{ MPa} \quad E = 195 \text{ GPa} \]
APPENDIX B – PHASE I CONNECTION TESTING

Puddle weld Connection Testing (19 mm diameter)

\[ P_{\text{max}} = 11.3 \text{ kN} \quad \text{(steel deck yielded around puddle weld, puddle weld intact)} \]

6.35 mm Hilti-screw Connection Testing (6.35 mm diameter)

\[ P_{\text{max}} = 4.7 \text{ kN} \quad \text{(steel deck yielded around Hilti-screw, Hilti-screw intact)} \]

Figure B.1: Double shear connection test specimens
a) Load vs. deflection – puddle weld connection

b) Load vs. deflection – 6.35 mm Hilti-screw

**Figure B.2:** Load vs. deflection – double shear specimens
APPENDIX C – PHASE II MATERIAL PROPERTIES

CD-W ASTM C39/C39M Compression Test

Table C.1: 5 day concrete compressive strength

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stress (MPa)</th>
<th>Load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27.72</td>
<td>224724</td>
</tr>
<tr>
<td>2</td>
<td>34.73</td>
<td>281527</td>
</tr>
<tr>
<td>3</td>
<td>34.94</td>
<td>283262</td>
</tr>
<tr>
<td><strong>AVERAGE</strong></td>
<td><strong>32.3</strong></td>
<td><strong>263171</strong></td>
</tr>
</tbody>
</table>

Table C.2: 30 day concrete compressive strength

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stress (MPa)</th>
<th>Load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>43.35</td>
<td>351454</td>
</tr>
<tr>
<td>2</td>
<td>45.31</td>
<td>367378</td>
</tr>
<tr>
<td>3</td>
<td>45.12</td>
<td>365777</td>
</tr>
<tr>
<td>4</td>
<td>44.05</td>
<td>357103</td>
</tr>
<tr>
<td>5</td>
<td>45.63</td>
<td>369914</td>
</tr>
<tr>
<td>6</td>
<td>45.23</td>
<td>366711</td>
</tr>
<tr>
<td><strong>AVERAGE</strong></td>
<td><strong>44.8</strong></td>
<td><strong>363056</strong></td>
</tr>
</tbody>
</table>

CD-S2 ASTM C39/C39M Compression Test

Table C.3: 30 day concrete compressive strength

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stress (MPa)</th>
<th>Load (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24.3</td>
<td>196960</td>
</tr>
<tr>
<td>2</td>
<td>24.83</td>
<td>201270</td>
</tr>
<tr>
<td>3</td>
<td>25.66</td>
<td>208030</td>
</tr>
<tr>
<td>4</td>
<td>26.43</td>
<td>214260</td>
</tr>
<tr>
<td>5</td>
<td>23.83</td>
<td>193180</td>
</tr>
<tr>
<td><strong>AVERAGE</strong></td>
<td><strong>25.0</strong></td>
<td><strong>202740</strong></td>
</tr>
</tbody>
</table>

High and low cylinders already removed prior to tabulation
MasterFiber® MAC 100 Plus
Macrosynthetic Fiber

Description
MasterFiber MAC 100 Plus product, a macrosynthetic fiber, is manufactured from a proprietary blend of polypropylene resins, in compliance with ASTM C 1116/C 1116M “Standard Specification for Fiber-Reinforced Concrete.”

MasterFiber MAC 100 Plus product also conforms to the requirements of CSA B68-10 “Design, material, and manufacturing requirements for prefabricated septic tanks and sewage holding tanks.”

Applications
Recommended for use in:
- Industrial and warehouse floors
- Commercial slab construction
- Concrete pavement
- White topping and overlays
- Thin wall precast
- Shotcrete
- Septic Tanks
- Composite metal decks

Features
MasterFiber MAC 100 Plus product is specifically engineered for use as shrinkage and temperature (secondary) reinforcement and provides excellent plastic shrinkage control and reduced settlement cracking. MasterFiber MAC 100 Plus product has the following features:
- Uniform distribution throughout the concrete matrix
- Excellent finishability
- Excellent post-first crack performance

Benefits
- Eliminates the need for welded-wire reinforcement (WWR) and conventional steel bars as secondary reinforcement, depending on the application
- Effective tight crack control
- Provides excellent plastic shrinkage crack reduction and settlement control
- Improves green strengths – permits earlier stripping of forms with less rejection
- Reduces construction time and overall labor and material costs
- Increases flexural toughness, impact and shatter resistance

Performance Characteristics

<table>
<thead>
<tr>
<th>Physical Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>0.91</td>
</tr>
<tr>
<td>Melting Point</td>
<td>320 °F (160 °C)</td>
</tr>
<tr>
<td>Ignition Point</td>
<td>1,094 °F (600 °C)</td>
</tr>
<tr>
<td>Absorption</td>
<td>Nil</td>
</tr>
<tr>
<td>Alkali Resistance</td>
<td>Excellent</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>75,000 psi (515 MPa)</td>
</tr>
<tr>
<td>Length</td>
<td>1.5 in. (38 mm)</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>59</td>
</tr>
<tr>
<td>Chemical Resistance</td>
<td>Excellent</td>
</tr>
</tbody>
</table>

APPENDIX D – GLASS FIBERS FOR G.F.R.C.
MasterFiber MAC 100 Plus

Guidelines for Use
Dosage: The recommended dosage for MasterFiber MAC 100 Plus product is 3 lb/yd³ (1.8 kg/m³). It can be used at dosage rates from 3 lb/yd³ (1.8 kg/m³) up to 13 lb/yd³ (7.8 kg/m³) or the equivalent dosage of 0.2% to 0.9% by volume, unless otherwise specified.

For composite metal deck applications, minimum dosage is 4 lb/yd³ (2.4 kg/m³) as recommended by the Steel Deck Institute.

Mixing: MasterFiber MAC 100 Plus product is packaged in pre-weighed, ready-to-use, degradable bags which are designed to be introduced at any time before or during mixing of concrete produced in accordance with procedures specified in ASTM C 94/C 94M.

Engineering Specifications
MasterFiber MAC 100 Plus product, when used at an appropriate dosage, is an option for the replacement of WWP, as a safe and easy-to-use reinforcing system that is rust proof, alkali resistant, and compliant with Industry codes when used in concrete mixed in accordance with ASTM C 94/C 94M.

MasterFiber MAC 100 Plus product should be specified for use in concrete slabs:
- For enhanced residual strength, toughness and durability
- To reduce plastic shrinkage cracking
- To increase abrasion resistance
- To improve impact resistance
- To reduce permeability

BASF does not recommend the fiber for use in slabs as a replacement for any structural steel reinforcement.

MasterFiber MAC 100 Plus product at a dosage of 3.0 kg/m³ (5.0 lb/yd³) also conforms to the requirements of CSA B66-10 “Design, material, and manufacturing requirements for prefabricated septic tanks and sewage holding tanks.”

Storage and Handling
Storage Temperature: MasterFiber MAC 100 Plus product should be stored at temperatures below 140°F (60°C). Avoid storing near strong oxidizers and avoid sources of ignition. Use caution when stacking to avoid unstable conditions. Store in a sprinkled warehouse.

Packaging
MasterFiber MAC 100 Plus product is packaged in pre-weighed degradable 3 lb (1.36 kg), 4 lb (1.81 kg) and 5 lb (2.27 kg) bags that can be added directly to the mixing system.

Related Documents
Safety Data Sheets: MasterFiber MAC 100 Plus product

Additional Information
For additional information on MasterFiber MAC 100 Plus product, contact your local sales representative.

BASF Corporation
Admixtures Systems
www.master-builders-solutions.basf.us
REFERENCES


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