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Behaviour of Open Web Steel Joist in Composite Deck Floor System

Aiman M. Ibrahim Muhammad
University of Windsor

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Behaviour of Open Web Steel Joist in Composite Deck Floor System

By

Aiman Muhammad

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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by

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September 18, 2015
Declaration of Originality

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Abstract

The objective is experimentally investigate the ability of other simple and fast to apply shear connectors like puddle-welds and Hilti-screw to develop composite action between the slab and girders. Two full-scale tests each consisted of 2400 mm wide, 6700 mm long and 65 mm thick concrete deck cast on top of corrugated steel sheets. The deck slab is supported over two OWSJ each of 250 mm depth and spaced transversally at 1200 mm with 600 mm overhang on each side. The composite floor system is simply supported in the longitudinal direction over 6400 mm span and was loaded monotonically till failure under two line loads. Test results are presented in terms of load-strain and load-deflection relationships at different locations over the concrete deck and across the depth. Test results showed that significant composite action is developed at service load and can be considered in design when puddle weld is used.
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University of Windsor
Aiman Muhammad

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<td>Canadian Institute of Steel Construction</td>
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<td>OWSJ</td>
<td>Open Web Steel Joist</td>
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<td>HSS</td>
<td>Hollow Structure Section</td>
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<td>NBCC</td>
<td>National Building Code of Canada</td>
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<td>CD-W</td>
<td>Puddle-Welding</td>
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<td>N.A.</td>
<td>Neutral Axis</td>
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<td>LVDT</td>
<td>Linear Variable Differential Transform</td>
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CHAPTER 1: INTRODUCTION

1.1 General

Composite construction is when more than one material is used in one structural element to resist loads. Famous example is Reinforced concrete, which has internal composite action between the concrete and the reinforcing bars. Another example in bridges, steel girders is used to support the concrete deck, which can be called external composite action. Composite construction is important because it allows the better use of each material strengthens where needed, i.e. concrete in compression zone and steel in tension zone. Therefore, composite steel-concrete structures are used widely in modern bridge and building construction. Composite construction simply aims to make both materials perform better together, or to strengthen the weaknesses of each material. The challenge is to ensure that forces are transmitted effectively and safely between the two materials and there is full strain compatibility (composite action) at the interfaces. Composite construction as we know it today was first used in both a building and a bridge in the U.S.A. over a century ago. The first forms of composite structures incorporated the use of steel and concrete for flexural members, and the issue of longitudinal slip between these elements was identified by Moore (1987). Nowadays, the construction community is looking for the best way to achieve composite action between steel and concrete in terms of reducing the installation time, higher safety level for workers, and lowest cost.

Composite steel construction is considered also as one of the most economical systems for constructing building floors. Composite floor systems typically involve
structural steel beams, Open-Web Steel Joist (OWSJ), girders, or trusses with shear connectors supporting a concrete slab, forming an effective T-beam flexural member resisting primarily gravity loads (Liew, 2003). Canadian Institute of steel Construction (2011), defines Open Web Steel Joists (OWSJ) as steel trusses of relatively low mass with parallel or slightly pitched chords and triangulated web systems proportioned to span between walls, structural supporting members, or both, and to provide direct support for floor or roof decks. Specifically, joists can be designed to provide lateral support to compression elements of beams or columns, to participate in lateral-load-resisting systems, or as continuous joists, cantilevered joists, or joists having special support conditions. The advantages of OWSJ include enlarged effective depths with minimal increases in material as oppose to W-shape beams of similar depths, making them very efficient. The effective depth of an OWSJ is the distance between the centroid of the top and bottom chords. Due to the slim cross section of the top cord of OWSJ, it is highly desirable not only to ensure composite action, but also to ensure that the top cord is subjected to tension, and the N.A. lies in the concrete flange. Furthermore, Open-Web steel joists shall be designed for loads acting in the plane of the joist applied to the top chord assume to be prevented from lateral buckling by the deck.

The main advantages of combining the use of steel and concrete materials for building construction are:

- Composite systems are lighter in weight (about 20 to 40% lighter than concrete construction). Because of their lightweight, site erection and installation are easier, and thus labour costs can be minimized. Foundation costs can also be reduced.
- The construction time is reduced, since casting of additional floors may proceed without having to wait for the previously cast floors to gain strength.

- The steel decking system provides positive moment reinforcement for the composite floor, requires only small amounts of reinforcement to control cracking, and provides fire resistance.

- The construction of composite floors does not require highly skilled labor. The steel decking acts as permanent formwork. Composite beams and slabs can accommodate raceways for electrification, communication, and air distribution systems. The slab serves as a ceiling surface to provide easy attachment of a suspended ceiling.

- The composite floor system produces a rigid horizontal diaphragm, providing stability to the overall building system, while distributing wind and seismic shears to the lateral load-resisting systems.

- Concrete provides corrosion and thermal protection to steel at elevated temperatures. Composite slabs of a 2-h fire rating can be easily achieved for most building requirements.

### 1.2 Composite Action

Composite action occurs when two or more components act as a single structural element, such as a concrete bridge and a steel girder. This composite action results in an increase in strength and stiffness of the bridge girders compared to non-composite action beam. When steel joist and concrete deck floor is subjected to bending, the deck and joist tend to slip due to longitudinal shear at the interface, unless they are rigidly connected.
with shear connectors are used for this purpose, which creates strain compatibility behavior between the joist and deck, provided composite action. Composite action is achieved by connecting the steel girder to the concrete slab to permit transfer of horizontal shear force at the steel-concrete interface (Kwon, 2008).

1.3 Problem statement
To date, CSA S16-09 mandates the use of shear studs in order to obtain adequate composite action in composite flooring. Shear studs require extra material costs and also require long time to install. They’re also a trip hazard for the workers as well as welders have to be hooked to the ground, which is a big concern especially in large roofs. Currently, for roof applications, designers used to count only on the OWSJ to resist the load and ignore any composite action to avoid using shear studs. The practice of building composite flooring without shear studs provides a competitive advantage. Without having to weld shear studs, money can be saved from material costs, faster and easier construction. However, it results in using deeper cross section and reduced the effective useable height. The objective of this research project is to determine if in fact, alternative simple connectors can be used to achieve composite action without the use of shear studs.

1.4 Objectives and Scope

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 mostly due to the lack of researched allotted to this topic. This research investigates alternative methods to connect the deck slab to the OWSJ (puddle welding or Hilti-screw pins), which greatly reduce the
cost and speed the construction. The objective of this experimental program is to determine if in fact, composite action can be achieved without the use of shear stud reinforcement.

1.5 Organization of Thesis

The general approach of this thesis is to explore the design of shear connectors such as Puddle weld and Hilti-screw, with more focus on the behavior mechanism between Open web steel joist and corrugated steel with concrete deck. The thesis is organized as follows: Chapter 2 introduces a background of related research work. In Chapter 3, the Proposed research program which includes the design concept, material properties, and test setup. The evaluation methodology and attained results are presented in Chapter 4 including the comparison with these shear connectors alternatives. Finally, Chapter 5 concludes all the contributions made in this thesis, and outlines future research directions.
2.1 Composite Construction

Composite Construction is when more than one material is rigidly connected to each other to perform as one body. The purpose of this type of construction practice is to bring out the strengths of each material as well as to strengthen the weakness of each individual material. Due to the modular-like nature of composite flooring with Open-Web Steel Joists (OWSJ), several floors can be quickly constructed and easily repeated so it is an ideal design for buildings with the same floor after floor applications. In a composite floor system, the concrete deck is poured onto steel decking sheets which act as formwork for the concrete and supported over the span between OWSJ. This construction method saves a tremendous amount of time and materials needed under typical flooring construction, leading to great savings to the bottom line.

In comparison to non-composite flooring, composite flooring is stiffer, causing less deflection as shown in Figure (2.1). Increased span lengths are possible with composite flooring. Composite flooring also has a larger moment capacity, allowing for smaller section sizes. The metal decking in composite flooring acts as formwork for concrete decreases construction times. When used properly, composite construction will result in reduced building costs.
Figure (2.1): Non-composite vs. Composite beam

(source: http://www.thecivilbuilders.com/2013/02/applications-of-composite-beams.html)

Figure (2.2): Non-composite vs. Composite- Neutral Axis

(source: http://www.fgg.uni-lj.si/~/pmoze/ESDEP/master/wg10/10200.htm)
Composite action has a great effect on the stress and strain of a beam. Without composite action, there is no strain compatibility at the interface between the two materials, two neutral axis, causing the steel and concrete to operate under both compression and tension, which leads to a smaller yielding/buckling load. With sufficient composite action, there will be only one neutral axis (better if within the concrete slab), allowing the concrete to take all the compression forces, while the steel beam takes almost all the tension forces. Also, under the same load, a composite action beam will have a much lower deflection than a non-composite beam as shown in Figure (2.2).

2.2 Open-Web Steel Joist (OWSJ)

Open Web Steel Joist (OWSJ) is an attractive structural engineering option for increased strength and stiffness while offering sufficient opening for air ducts and other services as shown in Figure (2.3). Composite joists are lighter and more economical than non-composite joists. Composite joists with a deck-slab system will have higher stiffness as well as better ductility than non-composite systems.

Based on Samuelson, D. (2002), the benefits by using composite steel joists include the following:

- Ability to route the mechanical heating, ventilating, plumbing, and electrical lines through the joist open webs. Customized web openings and configurations can be provided for large ducts.
- Ease of relocating and/or moving future HVAC during the life of the building.
- Better plenum space utilization.
- Floor-to-floor height can potentially be reduced by not having to run the
mechanical lines under the joists. Also the more efficient and stiffer composite design makes it possible to support a given load with a shallower joist.

- Weight savings resulting from the efficient joist design reduces overall building costs.
- Simplified erection and connections provide for fastest construction.
- Large column free areas give the building tenant maximum flexibility when selecting a floor layout plan.
- Ability to provide customized composite joist designs for any given loading and serviceability requirements.

Figure (2.3) Composite truss system with HVAC ducts
Composite Trusses or joists must provide sufficient factored resistance against occupancy loading, deck placement, and concrete placement. The factored resistance against collapse of OWSJ is dependent on the factored resistance of each of the following individual components:

- Steel top chord member
- Concrete deck-slab as a top flange
- Steel bottom chord member
- Web framing members
- Shear studs

2.2 Experimental Studies on Composite Construction

A considerable amount of research has been performed on composite steel joists over the past 50 years by experimentally testing the effective parameters of shear connections. The previous studies were experimentally conducted in the U.S. utilizing open-web steel joists in composite joists system in 1960’s (Samuelson, 2008) by connecting open web steel joists with deck slab using shear studs only. The first testing of composite joists was found in a 1965 Master of Science Thesis by H.G. Lembeck Jr. (Lembeck, 1965); followed by Wang and Kaley (1967). In these earlier test specimens, composite action was achieved by lowering the top chord angels so that the webs extended above the top chord into the concrete slab. Corrugated steel forms resting on the horizontal legs of the top chord angles supported the concrete slab. The extra shear
connection was created by the use of ½ in. (12.7 mm) diameter filler rods welded to the top chord between the panel points. The tests were compared to conventional joists with the same theoretical design load and the results showed that the composite steel joists were stiffer, having about a 20 percent reduction in deflection at the design load. The composite joists also attained an ultimate moment approximately 14 percent higher than the conventional joists that were tested. In both research projects, the results indicated that it was possible to achieve composite action in open-web steel joist construction.

Galambos and Tide (1970) performed tests on five composite steel joists that used 3/8 in. (9.5 mm) diameter x 2 in. (51 mm) long shear studs welded to the joist top chords. A 3 in. (76 mm) concrete slab was cast over each of the joist specimens. The main purpose of the research was to investigate the degree of composite action that could be obtained by studying the stud shear connector behavior in a composite system comprised of open-web steel joists, a cast-in place concrete slab, and mechanical shear connectors holding the two together. The researchers varied the type and size of the joist top and bottom chords as well as the number and location of stud shear connectors. The web members were over-designed in all the test specimens to ensure there would be no web failures in the experiments.

Cran, (1972) and Atkinson and Cran, (1972) tested composite steel joists supporting 1/2-in. (38 mm) deep steel deck. Results from their study suggested that for joists spaced more than 5 ft (1,524 mm) apart and with joist spans greater than 36 ft (10.97m), that composite joists were more economical than non-composite joists.
Azmi (1972) conducted six tests on composite joists with 50 ft. (15.24 m) spans. In addition to the testing, a design model was developed that showed good correlation with the experimental data. The model was based on three levels of shear connection: Under-connected, balanced, and over-connected which related the stud shear strength to the tensile yield force in the bottom chord of the joist. Fahmy (1974) developed a finite difference method to analyze the behavior of composite steel joists in both the elastic and inelastic regime that considered two different methods for shear connection, puddle welds and shear studs.

Robinson and Fahmy (1978) presented the experimental results and analysis of a number of composite open-web joists with metal deck. The idea was to demonstrate that composite with ribbed metal decking, would have sufficient ductility, and attain their computed ultimate flexural capacities. The experimental programs tested four types of OWSJ provided by three different manufacturing. They noticed that they have greater stiffness, strength, and ductility than non-composite open-web joists. The load was applied to the composite open-web joists at two points by means of a spreader beam. Test spans of the joists were either 50 ft. (15.25 m) or 51 ft. (15.6 m). Each joist supported a 4 in. (101.6 mm) thick concrete slab incorporating a 19 in. (38.1 mm) deep ribbed metal floor of 14-gauge material. Stud connectors and arc spot welds were placed between the load points at approximately the same spacing as in the shear spans. Test results showed that composite open-web joists with ribbed metal floors have greater stiffness, strength and ductility than non-composite open-web joists. All but one of the composite open-web joists tested attained at least or more than the calculated ultimate flexural capacity. The mode of failure was very much influenced by the degree of connection. Provided
sufficient of them are used (a balanced or over-connected composite open-web joist) arc spot welds provide an effective shear connection. In such a case vertical connection relies on the bond between the ribbed metal deck and the concrete slab. It is likely that composite embossed deck would be beneficial.

Leon and Curry (1987) and Curry (1988) reported on the testing of two full-scale, 36 ft. (10.97 m) long span composite steel joists to failure. Each test specimen was constructed with 2 in. (51 mm) composite steel deck, 3/4 in. (19 mm) diameter headed shear studs, and normal weight concrete with a nominal strength of 4 ksi (27.6 MPa). Alsamsam (1988) tested another two full-scale specimens to failure. The major result of the four tests was that the composite beam model could be used to predict the ultimate moment capacity of composite steel joists. Patras and Azizinimini (19910 tested two full-scale composite joists. The composite steel joists were 36 feet (10.97 m) long with a nominal depth of 12 inches (305 mm). Top and bottom chords of both specimens consisted of two equal leg angles welded back to back. Web members consisted of equal leg angles placed on the outside of the chords. Galvanized deck supported the 4 inch (102 mm) total concrete slab. Shear connectors, 3/4 inch (19 mm) diameter x 3.5 inches (89 mm) long after welding, were welded through the metal deck to the steel joist top chord angles. Light weight concrete was utilized for both specimens. Test specimen CH-1 was designed for a nominal strength of 3 ksi (20.7 MPa) while CH-2 was designed for a nominal strength of 12 ksi (82.8 MPa). Crushing of the concrete adjacent to the shanks of the “Weak” position shear studs in CH-1 was observed while there was no noticeable concrete crushing in CH-2 in the vicinity of the shear studs. Test results also showed that the higher strength concrete in CH-2 exhibited a higher stiffness as expected. Ultimate
load-carrying capacities were accurately predicted for both test specimens.

Kennedy and Brattland, (1992) studied the effect of concrete shrinkage on the behavior of composite steel joists. The authors tested two full-scale 38 ft. (11.58 m) specimens to failure, one at 65 days and the other at 85 days. It was found that the majority of the shrinkage occurred in the first 30 days. The failure loads that the specimens attained closely matched predictions based on an ultimate strength method with only the bottom chord in tension. Easterling et al. (1993) discussed the composite joist and slab systems. They tested four composite beams each of a single W16*31 section with a composite slab and composite deck with a total of 6 in. thickness. The span of each specimen was 30 ft. Welded wires were placed directly on the top of the deck and a total of 12 headed shear studs, ¾ in.*5 in. Wang et al. (2011) conducted twelve push-out test specimens of stud shear connectors with large diameter and high strength. The researchers noticed that the use of studs with large diameter and high strength can simplify the composite structure, save construction time and make the steel and the concrete work together better. In addition, the shear resistance and shear rigidity were higher than the normal studs used in composite structures and can be better used in bridge structures. The specimens were designed according to the Eurocode 4, and 12 specimens were conducted by considering different diameters and different strengths of studs, and were divided into 4 groups. The length of each stud was 200 mm, and the diameters were 22mm, 25mm, and 30mm. The tested average compressive strength was 70.3 MPa after 28 days. Two specimens in each group were tested under monotonic load, and the other one was tested under cycle load. The shear resistance and shear rigidity of studs with large diameter and high strength are all-higher than the normal studs used in composite
structures and can be better used in bridge structures.

Hedaoo et al. (2012) presented the structural behavior of composite concrete slabs. The slab is created by composite interaction between the concrete and steel deck with embossments to improve their shear bond characteristics. However, it fails under longitudinal shear bond. Eighteen specimens are split into six sets of three specimens each in which all sets are tested for different shear span lengths under static and cyclic loadings on simply supported slabs. Lakshmikandhan et al. (2013) investigated the longitudinal shear transfer mechanism at the interface between steel and concrete. Three types of mechanical connectors schemes were investigated experimentally which were exhibited full shear interaction with negligible slip. These experimental were improved strength and stiffness of the deck and can effectively reduce the cost of formworks. The experts were noticed that the composite slab without shear connectors slips and fails at the earlier load level. The insertion of shear connector modifies the brittle behavior of the composite slab into ductile.
CHAPTER 3: DETAILS OF THE EXPERIMENTAL PROGRAM

3.1 Introduction

The proposed research is an experimental program on full scale composite deck prototype specimens. One test prototype used puddle welding between steel deck and steel joist while the other test used Hilit-screw pin. The objective of this chapter is to describe the proposed program, introducing the design concept, material properties, test setup and instrumentation for the proposed specimens.

3.2 Experimental Investigation

The main objective for the proposed experimental program is to investigate the behavior of open-web steel joist in composite deck floor system with different shear connectors.

3.2.1 Test specimens

The experimental program consists of assembling two composite decks without any shear studs, one deck utilized puddle welding as a shear transfer mechanism and the second deck utilized small screws for the same purpose. Each composite floor system consists of three main elements; concrete slab, corrugated steel sheets and two OWSJ. Two OWSJ, each is 250 mm in depth, were spaced 1200 mm apart in the transverse direction and were simply supported over 6700 mm span in the longitudinal direction. Corrugated steel sheets, each of 900 mm wide and 2400 mm long, were welded (test specimen CD-W) or screwed (test specimen CD-S) to the top chords at single spot on each flute of the corrugated steel sheets. The concrete slab has a clear depth of 65 mm and overall width of 2400 mm and was supported over 1200 mm span with two overhangs.
each of 600 mm wide. A welded wire fabric was placed at mid-height of the concrete slab to mitigate cracking on the surface of the deck. The tested parameters in this experimental investigation are the ability of puddle-weld and Hilti-screw pins to develop composite action between the deck slab and supporting girders. Figure (3.1) shows schematic drawing of test specimens.
3.2.2 Details of constructions

Initially the two joists were rested parallel to one another at 1200 mm apart and on two large steel reinforced I-Sections that were leveled and stabilized. The joists were then secured to the I-Section using c-clamps. The decks were laid on top of the two trusses with the flutes perpendicular to them, and were centered so that an overhang of 600 mm on both ends was achieved. The decks were fastened temporarily at the ends using c-clamps until they were welded or pin-screwed to the joist. Figure (3.1) shows the initial layout of corrugated steel panels on Open Web Steel Joist.
For the first test prototype, a professional welder was hired to apply 19 mm in diameter Puddle-welds to only one angle of the top cord of each joist spaced at 300 mm in the longitudinal direction using the arc weld machine to fasten the meal decking to the trusses as shown in figure (3.2). Also deck sheets were secured to one another by crimping the sections where the decks overlap. However, on the second test prototype, Hilti-screws pins, 4 mm in diameter, were used instead of the puddle welds at the same locations as shown in figure (3.3).

(a) Layout of Corrugated steel panels
(b) Puddle weld

Figure (3.2): Initial Layout of Corrugated steel panels and Puddle-Welds
After the decks were fastened to the joists, wood formwork was built underneath the deck to support the fresh concrete placement. Two 8 by 4 foot sheets of plywood, laminated on the one side, were used as the side edge of the forms to give the concrete a smooth and even finish. Fourteen 2 by 4 by 10 feet lumber were served as the support for the laminate plywood sides and lateral supports as well as to support the overhang weight of fresh concrete. The 2 by 4’s were placed on top of, and perpendicular to, the joist under every other flute of the deck as shown in figure (3.4).

The 48 by 96 inches laminate boards were cut, using a table saw, into eight equal pieces of 6 by 96 inches. Two sheets of 48 by 96 inches plywood boards were each cut into 4 equal pieces of 12 by 96 inches to be used as a ledge lying horizontally over top of the 2 by 4 inch lumber to help secure the laminate boards and provide a ledge for the
wedges to be secured on. The Laminate sheets were notched to fit on top of the 2 by 4 inch lumber so that the height of the form extends 65mm above the top of the flute.

![Image of formwork construction]

**Figure (3.4): Formwork construction**

The laminate boards at the short side of the structure were rested on two 2 by 4 inch pieces that were secured by C-clamps to the I-section supporting the trusses. Three 2 by 4 by 10 feet lumber were each cut into ten equal pieces 1 foot in length, which in turn were cut into two pieces from corner to corner forming two triangular wedges that were used to secure the laminate plywood boards in place. The 6 by 96 inch laminate plywood sheets were placed vertically over top of the plywood, three on each long side and one on each short side. They were placed tight to the edge of the metal decking to ensure that concrete will not flow out once poured. Figure (3.5) shows the details of formwork. Figures (3.6) and (3.7) show the placement of the formwork and composite deck under the loading frame. Once all parts of the form were cut to size, they were oiled to prevent the concrete from sticking to them and to allow ease of removal once the concrete dries. A welded wire fabric was used to control the shrinkage cracking of the
concrete slab and was located at mid-height of the 65 mm concrete slab 1 ½ inch ‘plastic chairs’ as shown in figure (3.8).

Figure (3.5): Formwork details

Figure (3.6): Placement under the actuator
Figure (3.7): Forklift Placement

Figure (3.8): Welded wire fabric
A ready mix concrete with 28 days compressive strength of 30 MPa was used to cast the deck slab. Figure (3.9) shows casting the concrete on the top of corrugated steel and weld wire fabric were already placed.

![Figure (3.9): Casting Concrete](image)

A portion of concrete was set aside for some material testing (the slump test, compressive strength). A slump test was performed and a slump of 70 mm was recorded. Fifteen 4 by 8 inches and five 6 by 12 inches concrete cylinders were prepared for the compressive and tensile splitting test. Figure (3.10) shows the details of casting cylinders and slump test.
Figure (3.10): Casting cylinders - Slump test

Figure (3.11): Finishing of the Concrete Surface

The plastic and burlap sheeting in figure (3.12), was used for the moist curing of the concrete for 7 days. The concrete cylinders were removed out of the moulds and placed on top of the deck in order to have similar curing conditions. Figure (3.13) shows the whole system after curing and ready to install the instrumentation.
3.2.3 Materials properties

All test specimens were constructed using normal weight, ready-mixed concrete with a targeted 28-day concrete compressive strength of 30 MPa. All test slab prototypes were cast and kept in the laboratory, for 7 days, wrapped with plastic sheets in humid environment for curing. The actual concrete compressive and tensile strengths were
determined based on the average value of compressive and tensile splitting tests carried out on standard cylinder specimens of 100x200 and 150x300 mm, respectively, on the day of testing of the slabs. The standard cylinder specimens were cured under the conditions as their reference slabs. The obtained average concrete compressive and tensile strengths were about 32 MPa. The OWSJ used is CSA grade 350 steel, with 375 MPa yield strength and 200 GPa elastic modulus.

3.3 Test Set-up, Instrumentations and procedure

The composite floor system is simply supported in the longitudinal direction over 6700 mm span. It was loaded monotonically till failure under two line loads at the third points as shown in Figure (3.14). Very stiff loading system was used to ensure uniform load is applied over the entire width. The load was applied under load-controlled rate of 10 kN/min. three loading/unloading cycles were applied on each specimen. Figure (3.15) shows the full view of the test setup.

![Figure (3.14): Test setup](image)
The testing procedure followed the Performance Testing Procedure according to the CISC (1980) as follows:

- Apply the load monotonically up to 25% of the ultimate load, then unloaded to settle the joist.
- Apply the load monotonically up to 60% of the ultimate load and record the deflection.
- Unloading the system and record the residual deflection.
- Apply the load monotonically up to 100% of the test load and record the deflection.
- Unloading the system and record the residual deflection.
- Apply the load monotonically up to failure.

The ultimate load capacity was predicted to be 130 kN. Therefore, the peak load levels in the three load cycles were set to 32, 80, and 130 kN respectively. Test load was determined from factored loads
Several electric foil strain gauges, 10 mm long, were installed on the OWSJ elements at critical locations, including top and front faces of the top cord as well as bottom and front faces of the bottom cord, to monitor the strain distribution across the depth of the composite deck during the test. Also, several electric foil strain gauges, 70 mm long, were installed on the top and front face of the concrete deck at different locations to monitor the strain distribution across the width and depth of the concrete slab. Three Linear Variable Displacement Transducers (LVDT’s) were mounted at mid-span on the bottom cord, top cord and on top of the concrete slab over the OWSJ to record the deflection values during the test. Hydraulic Loading actuator, 500 KN capacities with +/-1250 mm stroke, was used to apply the load. It was mounted vertically in stiff steel frame that was tightly bolted to a rigid floor. Figure (3.16) shows a photo for the
instrumentations. A data acquisition system, monitored by a computer is programmed to record the readings of all strain gauges, LVDTs, and the load cell.

(a) Strain gauges on top concrete surface
Figure (3.16): Instrumentations
The following table explains the strain gauges identifications and locations:

<table>
<thead>
<tr>
<th>Gauges</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-TCC1</td>
<td>Placed on the top surface of concrete slab at the mid line</td>
</tr>
<tr>
<td>C2-TMC2</td>
<td>Placed on the top surface of concrete slab between C1 and C3.</td>
</tr>
<tr>
<td>C3-TEC3</td>
<td>Placed near the edge of the top surface of the concrete slab</td>
</tr>
<tr>
<td>C4-ESC1</td>
<td>Placed near the top edge on the side of the concrete slab</td>
</tr>
<tr>
<td>C5-ESC2</td>
<td>Placed near the bottom edge on the side of the concrete slab</td>
</tr>
<tr>
<td>S1-UDL</td>
<td>On the front truss, placed on the underside of the diagonal on the far left side</td>
</tr>
<tr>
<td>S2-UBL</td>
<td>On the front truss, placed on the underside of the first bottom chord on the left side</td>
</tr>
<tr>
<td>S3-UTC</td>
<td>On the front truss, placed on the underside of the top chord under the centre line</td>
</tr>
<tr>
<td>S4-FTC</td>
<td>On the front truss, placed on the face of the top chord under the centre line</td>
</tr>
<tr>
<td>S5-UBL</td>
<td>On the front truss, placed on the underside of the bottom chord to the left of the centre line</td>
</tr>
<tr>
<td></td>
<td>On the front truss, placed on the face of the bottom chord to the right of the centre line</td>
</tr>
<tr>
<td>----</td>
<td>--------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>S6-FBR</td>
<td></td>
</tr>
<tr>
<td>S7-UBR</td>
<td>On the front truss, placed on the underside of the bottom chord to the right of the centre line</td>
</tr>
<tr>
<td>S8-UBL</td>
<td>On the back truss, placed on the underside of the bottom chord to the left of the centre line</td>
</tr>
<tr>
<td>S9-UTC</td>
<td>On the back truss, placed on the underside of the top chord under the centre line.</td>
</tr>
</tbody>
</table>
CHAPTER 4: ANALYSIS OF THE EXPERIMENTAL RESULTS

4.1 Introduction

For a full-composite section, the failure need to be initiated by yielding of the bottom cord of the OWSJ while the top cord is under tension forces or insignificant compression force to avoid out of plane buckling. In another words, the neutral axis should be located above the OWSJ top cord, i.e., within the concrete deck slab. From a finite element analysis using STAAD Pro, it was concluded that, under these conditions, the ultimate load capacity of the composite deck/OWSJ is about 130 kN. Therefore, the peak load values of the three load cycles were chosen as 32 kN, 80 kN and 130 kN. The preliminary test results are presented in terms of Load - deflections, Load- strains relationships as well as ultimate capacity and failure load which were all measured during the monotonic loading steps that were carried out on each test specimen. Also, sectional strain distributions across the depth of the composite OWSJ are also provided to examine the level of composite action.

4.2 Ultimate capacity and failure mode

Test specimen CD-W where welding was used to develop the composite action showed a very promising 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 128 kN. The failure mode was mainly due local out-of plane buckling of the top cord followed by shear-bond failure between the concrete slab and the corrugated sheets by and shear failure of the concrete deck and local yielding of the top
cord under the load as shown in Figure 4.1. However, specimen CW-S failed to show a competitive behavior. The failure occurred at load level of 80 kN with excessive deformation and delamination between the concrete and the deck slab. The failure was initiated by slippage/bending between the hilit-screws and the deck followed by buckling of the top cord.

(a) Test specimen Puddle-welds (CD-W) after failure

(b) Shear-bond failure (delamination) between the concrete slab and corrugated sheets
4.3 Load-Deflection Behaviour

Figures 4.2 to 4.5 show comparisons between the load vs mid-span deflection relationships measured at the concrete top, top cord and bottom cord of each specimen during the three load cycles. The consistency of the deflection behaviour and values measured at the three levels in specimen CD-W reflects no slippage or delamination between the components until high load levels, around 80 kN. This confirms the significant composite action developed by the puddle welds. Specimen CD-S showed incompatibility of deflection measured at the three levels due to the slippage occurred at the interface between the deck and the top cord due to bending and sometime rupture of the hilti screws. Figures 4.6 shows a comparison of the load-deflection behaviours
between CD-W and CW-S during the three load cycles. It can be noticed Test specimen CD-W showed a perfect linear load-deflection behaviour throughout all the loading history up to 80 kN load without any residual (plastic) deformations after load cycles 1 and 2. The measured deflection at 40 kN "live load deflection" in figure (4.3) is about 6 and 12 mm for CD-W and CD-S respectively where the allowable is around 16 mm. On the other hand, specimen CD-S showed a totally non-linear behaviour during all the loading steps with significant residual deflections after all load cycles. This deflection behaviour is in good agreement with the strain distribution showed before which confirm the non-composite action behaviour of specimen CD-S.

(a) Specimen CD-W
Figure (4.2): Comparisons of the load-deflection behaviour across the depth of test specimens during cycle 1

(b) Specimen CD-S

(a) Specimen CD-W
Figure (4.3): Comparisons of the load-deflection behaviour across the depth of test specimens during cycle 2

(b) Specimen CD-S

(a) Specimen CD-W
Figure (4.4): Comparisons of the load-deflection behaviour across the depth of test specimens during cycle 3

(b) Specimen CD-S

(a) Specimen CD-W
Figure (4.5): Comparisons of the load-deflection behaviour of test specimens measured at bottom cord

(b) Specimen CD-S

(a) 1st load cycle – Bottom Cord
Figure (4.6): Comparisons between the load and mid-span deflection relationships of test specimens
4.4. Load - Strain Behaviour

Figure (4-7) to (4-11) shows comparisons of the load vs strain behavior of CD-W and CD-S for the diagonal, top and bottom cords. It can be noticed that specimen CD-W also showed a perfect linear behavior throughout all the load cycles up to about 80 kN load level with almost no residual strains. The maximum recorded tensile strain was 1600 micorstrain which is less than the 1750 microstrain yielding strain. CD-W top showed a tensile strain duo to 80 kN thereafter it changed gradually to a compressive strain of about 1150 microstrain at failure. This indicated that the neutral axis located in the top concrete deck until a load level of about 80 kN. On the other hand, the top cord strains were always a compression strains in CD-S and reached a high value of 1400 microstrain at failure, only at 80 kN. Since load cycle #1, CD-S had the neutral axis below the top cord.

(a) Puddle-Welding
Figure (4.7) the relationship between strain and Load – 1\textsuperscript{ST} Diagonal

(b) Hilti-Screw

(a) Puddle-Welding
Figure (4.8) the relationship between strain and Load – Bottom cord

(b) Hilti-Screw

(a) Puddle-Welding
Figure (4.9) the relationship between strain and Load – Top Cord

(b) Hilti-Screw

(a) 2nd cycle
Figure (4.10): Comparisons of the load and strain relationships of test specimens – Top cord

(a) 2\textsuperscript{nd} cycle

(c) 3\textsuperscript{rd} cycle
Concrete strain gauges c1, c2 and c3 were used to ensure that the concrete compressive strain is uniformly distributed across the top slab width, i.e. even load distribution across the effective flange width. Figure (4-12) shows that uniform concrete compressive strain/stress already developed in the concrete slab of CD-W while figure (4-13) showed that due to failure of the hilti screws and yielding of the top cord of one joist, the stress distribution in the top slab is not even. It can be noticed that the maximum compressive strain of CD-W was about 175 microstrains at 128 kN while in CD-S was about 220 microstrains at 80 kN.
Figure (4.12): Comparisons of the top concrete strain across the width of test specimen CD-W
Figure (4.13): Comparisons of the top concrete strain across the width of test specimen CD-S
Figure (4.14) the relationship between strain and Load – concrete gauge 3
4.5 Strain Profile

Figures (4-16) to (4-21) show the distribution of the measured strains across the depth of the test specimens up to the equivalent service load, 40 kN in cycle 2. The analysis of the strain measurements showed a linear strain distribution across the depth where the neutral axis was located within the concrete deck up to a load level of 40 kN in test specimen CD-W. Test specimen CD-S where screw pins were used to develop the composite action showed very poor performance compared to specimen CD-W. It can be noticed from the strain distribution that no composite action was developed. No strain compatibility was observed at the bottom of the concrete slab and the top of the OWST. Two neutral axes can be distinguish, one within the OWSJ and the second is within the top deck. Local yielding in the top cord of the OWST was observed at different locations early during the 2nd load cycle.
Figure (4.16) Strain profile across the depth for Puddle-welds – at 30 kN

Figure (4.17) Strain profile across the depth for Hilti-screws – at 30 kN
Figure (4.18) Comparison of the strain profile across the depth – at 30 kN

Figure (4.19) Strain profile across the depth for Puddle-welds – at 40 kN
Figure (4.20) Strain profile across the depth for Hilti-screws – at 40 kN

Figure (4.21) Comparison of the strain profile across the depth – at 40 kN
CHAPTER 5: CONCLUSION AND RECOMMENDATIONS

5.1 Summary

Composite floor system consists of concrete deck slab poured on top of corrugated steel sheets supported over Open-Web steel Joist (OWSJ) is widely used in industrial and commercial building repeated floors, workshops and warehouses. Composite flooring offers larger moment capacity, allowing for smaller cross section sizes. Composite action is the term used to describe the behaviour of composite structure which ensure strain compatibility across the section which has a great effect on the stress and strain of beam and floor composite systems. However, to achieve the desirable composite action, extensive shear studs have to be welded on the top of the girder before concrete casting. Such method imposes extra cost, time and construction difficulties. Some alternative methods to connect the deck slab to the supporting girders (such as puddle welding or Hilti-screw pins), are widely used during construction. 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 mostly due to the lack of researches allotted to this topic. This research investigates the ability of puddle welding or Hilti-screw pins to develop composite action, which will greatly reduce the cost and speed of the construction. The objective of this research is to promote the application of composite floor system to residential applications with the use of simple, yet effective connectors, without the use of shear stud reinforcement.
Two full-scale test prototypes each consisted of 2400 mm wide, 6700 mm long and 65 mm thick concrete deck cast on top of 3 mm thick corrugated steel sheets. The deck slab is supported over two OWSJ each of 250 mm depth and spaced transversally at 1200 mm with 600 mm overhang on each side. The composite floor system is simply supported in the longitudinal direction over 6400 mm span and was loaded monotonically till failure under two line loads at the third points. The test parameters were the type of the shear connector, puddle welds and hilti screw pins. Test results are presented in terms of comparisons of load-strain and load-deflection relationships at different locations and load levels. Based on the research findings and analysis of test results, the following conclusion can be drawn.

1- It is feasible to obtain a significant level of composite action without the use of shear studs.

2- 19 mm in diameter puddle welds installed on one side of the top cord angles at the flute of the corrugated sheets and spaced 300 mm in the longitudinal direction showed a very strong performance in terms of the deflections and strains. Its performance meets the serviceability and ultimate code requirements.

3- 4.2 mm hilti screws on one side of the top cord angles at the flute of the corrugated sheets and spaced 300 mm in the longitudinal direction showed a very poor performance and could not maintain any composite action.
5.2 Recommendations for future research

1- Additional experimental testing is still needed to investigate the behavior of different sizes (diameter) and layout of Hilti-screws.

2- The use of lightweight concrete to reduce the thickness and the self-weight of composite slabs should be investigated.

3- The use of fibre-reinforced concrete (FRC) to eliminate the wire mesh inside the concrete slab and speed the construction.
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