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Innovative Composite Cold Formed Steel Floor System

D.M. Fox¹, R.M. Schuster², and M. Strickland³

Abstract

Presented in this paper is a new, unique and innovative composite cold formed steel floor system developed by iSPAN Technologies, called the “iSPAN Composite Floor System”. The joist sections are fabricated by fastening two cold-rolled flange chord elements with cold-driven rivets to a flat web element. This makes it possible to create a section where the flange chord elements can be of a different steel thickness with respect to the web element, resulting in a most efficient structural cross section and numerous design alternatives. The joist sections have lip-reinforced web openings spaced at 4 ft o.c. along the joist length to accommodate the usual service items. The joists are typically spaced 4 ft o.c. with a 7/8 in. corrugated steel deck spanning between the joists to support the concrete during casting. Featured in this paper are the results from push-out tests that have been carried out to establish the interlocking capacity of the concrete with the top chord of the joist section. The results of a full-scale laboratory structural test are also presented to substantiate the calculated strength and stiffness characteristics. Finally, the results of a field test during construction are presented.

Introduction

Composite joists have been used since the mid 1960’s and early composite joists were developed based on open web steel joist architecture, using either elevated

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bar web members or welded shear studs to provide the required interlocking
capacity between the concrete and joists [1-2]. To date, welded shear studs,
such as Nelson Studs, are commonly used as one of the popular methods of
interlocking the concrete slab with joists. However, concerns over the studs
acting as tripping hazards have necessitated field installation of the studs [3],
which can be labour intensive and difficult to control the quality of installation.

Various alternatives to the welded stud shear connectors have been developed,
such as the Hambro ‘S’ shaped top chord, the Vescom embossed chord, and the
Tafrus perforated top chord [4]. These alternatives are all based on open web
steel joist concepts, and are therefore labour intensive to fabricate. Furthermore,
top chord bearing joists can be difficult to install on light steel framed walls,
requiring heavy distribution members to accommodate the large end reactions.

Attempts have been made to use C-sections to provide the steel component of
composite joist slabs, which typically involves the installation of shear
connectors to the top flanges of the joists. In some other cases, the top flanges of
C-sections have been embedded into the concrete slab, which can be difficult to
install the associated formwork.

iSPAN Technologies has recently introduced a fully cold-formed stay in place
composite floor, called the “iSPAN Composite Floor System”. The system was
designed specifically for the light steel framing industry, resulting in simple
fabrication and installation without the requirement of specially trained labour.
Included in the system is the composite joist, where the top chord provides the
required interlocking capacity with the concrete. In addition, the top chord also
provides the required support for the steel deck during construction. A
schematic diagram of the composite floor system and a section of the joist are
illustrated in Figure 1. Presented in this paper are the results of the interlocking
capacity tests of the top chord (push-out), a full scale composite flexural test,
and an in-situ field deflection monitoring test during concrete placement.

**Top Chord Interlocking Capacity – Push-out Tests**

Push-out tests were conducted to establish the interlocking capacity of the
embedded top chord with the concrete slab. Symmetrical specimens were
fabricated; each specimen was comprised of two composite top chords
connected to a web by rivets spaced at 8 in. o.c. The specimens were supported
such that the chords were allowed to slip between the concrete elements when
the load was applied. A photograph of a typical test setup is shown in Figure
2(a). A bearing plate was positioned over the exposed portion of the steel chords
and the load was applied at the center of the bearing plate. Failure occurred by
slippage of the concrete along one or both chords; interlocking capacity was provided by a combination of chemical bond and rivet head interlocking. A typical bearing failure in the concrete at the location of a rivet head is shown in Figure 2(b). Two different specimen lengths were tested as summarized in Table 1, which also includes the test results.

Figure 1 - iSPAN Composite Floor System

Figure 2 - Photographs of Typical Push-out Tests
Table 1 - Summary of Push-out Test Results

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Embedment Length (in.)</th>
<th>No. of Rivets</th>
<th>Failure Load (kip)</th>
<th>Failure Mode(^1)</th>
<th>Interface Shear, (q_u) (lb/ft)</th>
<th>Average (q_u) (lb/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2R - 8(^{\circ})c - 1</td>
<td>12</td>
<td>2</td>
<td>-</td>
<td>Premature</td>
<td>-</td>
<td>16,806</td>
</tr>
<tr>
<td>2R - 8(^{\circ})c - 2</td>
<td>12</td>
<td>2</td>
<td>17.2</td>
<td>Slip 1</td>
<td>17,249</td>
<td></td>
</tr>
<tr>
<td>2R - 8(^{\circ})c - 3</td>
<td>12</td>
<td>2</td>
<td>17.4</td>
<td>Slip 1</td>
<td>17,436</td>
<td></td>
</tr>
<tr>
<td>3R - 8(^{\circ})c - 1</td>
<td>20</td>
<td>3</td>
<td>26.8</td>
<td>Slip 2</td>
<td>16,064</td>
<td></td>
</tr>
<tr>
<td>3R - 8(^{\circ})c - 2</td>
<td>20</td>
<td>3</td>
<td>27.7</td>
<td>Slip 2</td>
<td>16,629</td>
<td></td>
</tr>
<tr>
<td>3R - 8(^{\circ})c - 3</td>
<td>20</td>
<td>3</td>
<td>27.8</td>
<td>Slip 1</td>
<td>16,652</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1) Failure modes describe as follows:
   (a) Slip 1: Specimen failed by slippage along one chord
   (b) Slip 2: Specimen failed by slippage along both chords

Flexural Test

A full scale composite floor system was tested, where the span length was 21.5 ft and two joists were spaced at 3 ft o.c. The floor joists were 12 in. in depth, and the thickness and the yield strength of the steel were 0.057 in. and 57.5 ksi, respectively. 7/8 in. deep corrugated steel decking was installed by supporting it on the wings of the top chord, and a 6x6 6/6 welded wire mesh was draped over the joists and steel deck. Concrete was placed such that a 1 in. cover was maintained over the top chord, resulting in a slab whose overall depth, \(t_s\), measured from the bottom of the deck flute, was 2-3/4 in. The slab was cantilevered 18 in. on each side of the joist in order to provide two symmetrical composite sections. An overview of the test setup and specimen is presented in a schematic diagram in Figure 3, with a photograph of the actual test setup shown in Figure 4.

A four line load test setup was used in order to approximate a uniformly distributed load. The specimen was loaded until failure, as can be observed from the load displacement plot shown in Figure 5. Failure occurred by yielding of the bottom chord as is exhibited by the ductile load displacement curve. The test was stopped at a maximum deflection of 3.30 in. at which the recorded ultimate load was 21,290 lbs.
Figure 3 - Schematic Full Scale Composite Flexural Test Setup

Figure 4 - Photograph of Full Scale Flexural Test Setup
Figure 5 - Load Displacement Curve of Full Scale Composite Flexural Test

Analytical Analysis

An analytical analysis was performed to determine the required interlocking capacity, which was accomplished by using an elastic shear flow approach and an ultimate strength approach. Both of these methods have shown to yield good correlation with test results. Finally, a comparison of calculated flexural strength and stiffness to the tested values was performed.

Elastic Shear Flow Approach

The well known elastic shear flow expression is given in Equation (1):

\[ q_{\text{max}} = \frac{V_{\text{max}} Q}{I_{\text{sc}}} \]  

(1)

The first moment of area, \( Q \), can be calculated from the following expression:

\[ Q = \frac{b_e}{n} \left( y_{gc} - \frac{t_e}{2} \right) \]  

(2)
For the purpose of calculating elastic shear flow, the effective concrete flange width, $b_e$, can be taken as the maximum possible width equal to the joist spacing. The effective slab depth, $t_s$, was taken as the overall slab depth, $t_{s0}$, less the steel deck depth, $t_d$. It was assumed that the concrete below the deck does not contribute to the strength of the composite section.

**Ultimate Strength Approach**

For most composite joist sections, such as composite trusses and open web steel joists, it is typical to consider only the bottom chord in the calculation of the flexural strength [5, 6]. These joist sections tend to have non-solid web elements which do not contribute significantly to the flexural strength of the section. However, the composite joist considered herein includes a solid web which does contribute to the flexural strength. However, the high slenderness ratio of the web does not allow the entire cross section of the web to yield. Since the web is subjected to a stress gradient (see Figure 6), the resultant tensile force can be calculated according to Equation (3):

$$T_s = T_{sc} + T_{sw}$$

![Figure 6 - Force Components for Composite Flexure Calculations](image)

Based on the assumption of full composite action, the interlocking capacity in the slab element must be greater than the tensile force in the steel. The average required interlocking shear flow between the points of minimum and maximum moments is therefore:
A traditional reinforced concrete approach was used to calculate the flexural resistance of the composite joist [7]. As per traditional reinforced concrete design:

\[ M_r = T_{sc} \left( y_{sc} - \frac{a}{2} \right) + T_{sw} \left( y_{sw} - \frac{a}{2} \right) \]

where

\[ a = \frac{T_s}{\alpha_i \phi f_c b_e} \]

As recommended by Clause 17.4.1 of CSA S16-01 [5], the effective slab width, \( b_e \), was taken as the lesser of:

1. Joist Spacing
2. Span divided by 4

Finally, the composite moment of inertia was calculated based on traditional transformed section procedures where the effective slab width was considered to be equal to the joist spacing divided by the modular ratio:

\[ I_{xc} = I_{sj} + A_j D_j^2 + \frac{b_c}{n} \frac{t_c^5}{12} + \left( \frac{b_c}{n} t_c \right) D_c^2 \]

**Test Result Comparisons**

The computed flexural capacity, \( M_r \), maximum shear flow, \( q_{\text{max}} \), ultimate shear flow, \( q_u \), and composite moment of inertia, \( I_{xc} \) (expressed in equivalent steel) were computed in order to compare the calculations with the test data presented above. The parameters for the 12 in. composite joist floor system are presented in Table 2 along with the results of the calculations, where all resistance factors were set equal to 1.0.
Table 2 - Test Result and Comparison of Flexural Test

<table>
<thead>
<tr>
<th>Parameters of 12 in. Composite Joist Tested</th>
<th>(per joist)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_1$ =</td>
<td>0.81</td>
</tr>
<tr>
<td>$A_1$ =</td>
<td>1.68 in.</td>
</tr>
<tr>
<td>$A_{sw}$ =</td>
<td>0.66 in.</td>
</tr>
<tr>
<td>$b_2$ =</td>
<td>36 in.</td>
</tr>
<tr>
<td>$D_j$ =</td>
<td>5.17 in.</td>
</tr>
<tr>
<td>$D_c$ =</td>
<td>1.00 in.</td>
</tr>
<tr>
<td>$f'_c$ =</td>
<td>4.07 ksi</td>
</tr>
<tr>
<td>$f_{sc}$ =</td>
<td>57.5 ksi</td>
</tr>
<tr>
<td>$I_{xj}$ =</td>
<td>34.6 in.$^4$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calculated Values and Comparisons with Tested Values</th>
<th>(per joist)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_{max}$ =</td>
<td>6,099 lb/ft</td>
</tr>
<tr>
<td>$q_u$ =</td>
<td>5,521 lb/ft</td>
</tr>
<tr>
<td>$M_r$ =</td>
<td>43.2 k-ft</td>
</tr>
<tr>
<td>$I_{xj}$ =</td>
<td>90.9 in.$^4$</td>
</tr>
</tbody>
</table>

As shown in Table 2, the required interlocking capacity calculated either by the elastic or the ultimate approach, $q_{max}$ and $q_u$, respectively, are both less than the provided interlocking capacity, $Q_r$, determined from push-out tests as described above. This confirms that the assumption of full composite action was valid.

The calculated composite flexural strength of 43.2 k-ft compares well to the tested flexural strength of 44.1 psf; the additional moment due to dead loads (self-weight and loading apparatus) were included in the computation of the tested flexural strength. As shown in Figure 5, the predicted flexural capacity was within 2% of the tested capacity. Finally, the calculated moment of inertia for the tested joist is 90.9 in.$^4$. From Figure 5 it can be seen that the calculated composite stiffness of the floor matched well with the test. It can be noted that an effective moment of inertia approach, similar to that recommended in codes to account for interfacial slip [5, 8] or for web deformation and joint eccentricity [4, 6], was not required to properly reflect the test data.

**In-situ Monitoring of Concrete Placement**

In-situ monitoring of a floor system during concrete placement was conducted to confirm the accuracy of the non-composite design method, specifically with
respect to predicting the deflection during concrete placement. The selected project was near Toronto, Ontario, and was constructed of 15 in. composite floor joists spaced at 48 in. o.c. with a clear span of 24.3 ft. The specified slab depth was 3-3/4 in. measured from the bottom of the deck flutes.

Joist strength (flexural and shear) and moment of inertia for deflection calculation were calculated according to the AISI S100 (CSA S136) [9] with modified buckling coefficients as recommended by Fox et al [10]; the moment of inertia for deflection calculation, $I_{xd}$, of the specified joist is 77.7 in.$^4$. A displacement transducer was installed at midspan of a joist as shown in Figure 7. During concrete placement, the deflection was monitored and recorded; a plot of midspan deflection over the course of the pour is shown in Figure 8.

The floor system was designed for the non-composite phase as per the recommendations given in CSSBI 12M-06 [11]:

1. strength must resist the effects of system dead loads combined with either a 21 psf uniform load or a 137 lb/ft transverse line load at midspan, and
2. deflections based on system dead loads are to be limited to the smaller of $L/180$ or $\frac{3}{4}$ in. Calculated deflection is increased by a ponding factor, $Y_p$, of 1.10 to account for possible concrete ponding or to account for a slab thickness greater than that specified.

![Figure 7 - Photographs of In-Situ Deflection Monitoring Equipment](image-url)
The dead load of the system, considering steel system self weight, metal deck, welded wire mesh, and wet concrete was estimated at 47.0 psf. Considering a ponding factor of 1.1, the expected permanent deflection due to dead loads is 0.71 in. If the ponding factor is set equal to 1.0, then the expected permanent deflection would be 0.65 in.

Figure 8 – Mid-span Deflection During Concrete Placement

During concrete placement, three distinct regions of deflection were experienced, as can be observed in Figure 8:

1. concrete placement away from the joist; observed deflection is a result of movement of the superstructure,
2. placement of concrete over the monitored joist’s tributary area; a sustained midspan deflection of 0.52 in. is observed, and
3. placement of concrete away from the joist being monitored; deflection is a result of movement of the superstructure.

In order to confirm that the permanent deflection of the joist was 0.52 in. and also to establish the amount of concrete ponding, measurements were taken after the concrete had hardened, with the results summarized in Table 3.

The recorded data shown in Figure 8 and the measurements taken under the joist after concrete hardening confirm that the permanent joist deflection due to self weight during concrete placement was 0.52 in. Considering a ponding factor of
1.10, the recorded deflection was 27% less than the predicted deflection. If the ponding factor is set to 1.0, the recorded deflection would be 20% less than the predicted deflection.

Table 3 - Measurements of Monitored Joist after Concrete Hardening

<table>
<thead>
<tr>
<th>Location</th>
<th>Distance from datum string to joist/concrete (in.)</th>
<th>Maximum Deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End 1</td>
<td>End 2</td>
</tr>
<tr>
<td>Under Joist</td>
<td>25/32</td>
<td>⅛</td>
</tr>
<tr>
<td>Above Joist</td>
<td>1-3/8</td>
<td>1-9/16</td>
</tr>
</tbody>
</table>

The amount of ponding at mid-span can be determined by subtracting the deflection of the top of the slab, $\delta_a$, from the deflection of the bottom chord of the joist, $\delta_u$, (values are listed in Table 3, and locations are shown in Figure 9). It can be concluded that the maximum amount of ponding that occurred at midspan was 0.18 in. The ponding observed represents approximately a 5% increase in slab thickness at midspan with respect to the specified slab depth. The ponding factor of 1.1, which in effect assumes a 10% added weight, is a conservative estimate of the degree of ponding observed.
Conclusions

A new composite floor system, named ‘iSPAN Composite Floor System’, specifically designed for light steel framing was introduced. The composite joist section is comprised of a unique top chord that enables simple installation and provides the required interlocking capacity for composite action. Results from push-out tests, a full scale flexural test, and in-situ deflection monitoring during concrete placement are summarized. Based on the test data, the following conclusions can be made:

1. the interlocking capacity of the top chord is more than sufficient to enable full composite action between the concrete slab and the joist,
2. the flexural capacity of the composite joists section can be predicted conservatively based on current Standards/Specifications, the flexural test indicates that the web can be considered in the flexural calculations in order to better reflect the composite behaviour,
3. the stiffness of the composite section can be accurately predicted using standard transformed section properties, and

The conclusions drawn regarding composite flexural stiffness and strength are based on one test. A test program is currently underway to carry out additional flexural tests in order to fully substantiate the conclusions presented herein.
References


Notations

a  depth of effective compressive stress block (in.)
A_{sc}  area of steel in bottom chord (in.²)
A_{sw}  area of steel in web (in.²)
A_j  total area of steel in joist (in.²)
b_e  effective width of concrete flange (in.)
d_c  concrete cover over top chord of joist (in.)
δ_s  measured deflection of concrete along joist at midspan (in.)
δ_u  measured deflection of bottom chord of joist at midspan (in.)
D_c  distance from composite joist to concrete flange center of gravity (in.)
D_j  distance from composite joist to steel joist center of gravity (in.)
E_c  modulus of elasticity of concrete (ksi)
E_s  modulus of elasticity of steel (ksi)
F_y  yield strength of steel (ksi)
f'c  compressive strength of concrete (ksi)
γ  density of concrete (lb/ft³)
I_{xc}  composite moment of inertia in equivalent steel (in.⁴)
I_{xd}  Moment of inertia for deflection calculation (in.⁴)
I_{xj}  Moment of inertia of steel joist (in.⁴)
L_q  distance between points of maximum and minimum moment (ft)
M_r  calculated composite flexural strength (k-ft)
M_t  tested composite flexural strength (k-ft)
n  modular ratio = E_s/E_c
φ_c  resistance factor for concrete
φ_s  resistance factor for steel
Q  first moment of area of concrete flange in composite joist (in.³)
Q_{it}  interlocking capacity of top chord to concrete slab (lb/ft)
q_{max}  maximum shear flow (lb/ft)
q_u  ultimate interlock capacity required for full composite action
t_c  effective slab depth (in.)
t_d  steel deck depth (in.)
t_s  total slab depth to bottom of steel deck flute (in.)
T_s  total tension force (lbs)
T_{sc}  tension force developed in chord (lbs)
T_{sw}  total tension force developed in web (lbs)
y_{cgc}  center of gravity of composite joist from top of slab (in.)
y_{cgs}  center of gravity of steel from top of slab (in.)
$Y_p$ factor to account for concrete ponding
$V_{\text{max}}$ maximum shear force (lbs)