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ANALYSIS OF ULTIMATE STRENGTH OF COLD FORM STEEL JOIST

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ABSTRACT

The floor joists of cold-formed steel (CFS) structures often require large web openings. Reinforcement of such openings may mitigate the detrimental effects arising from such web openings. This project is based on an experimental investigation conducted to establish reinforcement schemes for CFS joists with a large web opening located in high shear zones. A large web opening cutout in the middle of the web of aliped channel section. Circular and square large web openings of about 65% web depth were considered.

The shear tests involved short span specimens, simply supported at the ends, and subjected to a mid-span point load. This project presents the experimental set-up, and the test results associated with 27 individual tests consisting of nine sets of three identical tests. These test sets consist of cold-formed steel joists sections with (a) no web opening, (b) webs having circular or square openings, (c) webs having reinforced openings. Three different reinforcement schemes were investigated, however, the test results indicated that the proposed "Vierendeel truss type reinforcement scheme" can restore the original shear strength of such sections having circular or square openings. Overall, this study establishes a cost effective reinforcement scheme for cold-formed steel joist having a large web opening in high shear zone.

I. INTRODUCTION

An open web steel joist is a simply supported truss, with parallel or slightly pitched chord and a triangulated web system. Joists are commonly used in roof and floor construction as secondary load carrying member spanning between primary framing members. The top chord considered to provide continuous support for the floor or roof decking.

Joists are commonly designated as short, intermediate or long span although this designation is somewhat ambiguous. Short span joists are generally produced with continuous bent bar webs which are welded to the chords using either resistance or arc welding. Intermediate span joists generally have web member in subdivided warren configurations which are welded to the top chord so as to provide support in the plane of the joists at intervals of two feet. For long span joists, the process of manufacture is similar to intermediate joists except that the web configuration generally corresponds to a Pratt Truss and the panel spacing may vary. This study is restricted to joists generally designated as intermediate.

Brief History

A number of differentloading conditionsand beam geometries are investigated. In addition, a proposedanalytical expression for determining the bearing capacity will be discussed in relation tothe results obtained using the finite element (FE) analyses. The results obtained in the FE-analyses are compared also with available experimental results. The aim is to understandthe mode of failure of the I-joists for different loading and support conditions and fordifferent beam geometries.

The first experimental research conducted in the U.S. utilizing open-web steel joists as part of a composite joist system was carried out in the [Lembeck, ; Wang and Kaley,].Composite action in the earlier testing program was achieved by inverting and lowering the top chord angles so that the webs extended above the top chord into the concrete slab. Additional shear connection was created by the use of 1/2 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 the later experimental program, composite action was achieved by providing a longitudinal shear key along the one-piece top chord of the joists. Supplemental shear connection was provided in some of the tests by adding continuous metal chairs into the top chord that were shaped like a bulb. In both research projects, the results indicated that it was possible to achieve composite action in open-web steel joist construction.

ISSN 2277 – 5528
Impact Factor- 3.145

Objective

The floor joists of cold-formed steel (CFS) structures often require large web openings. Reinforcement of such openings may mitigate the detrimental effects arising from such web openings. This project is based on an experimental investigation conducted to establish reinforcement schemes for CFS joists with a large web opening located in high shear zones. The investigation considered a large web opening cutout in the middle of the web of a liped channel section. Circular and square large web openings of about 65% web depth were considered.

The shear tests involved short span specimens, simply supported at the ends, and subjected to a mid-span point load. This project presents the experimental set-up, and the test results associated with 27 individual tests consisting of nine sets of three identical tests. These test sets consist of cold-formed steel joists sections with (a) no web opening, (b) webs having circular or square openings, (c) webs having reinforced openings. Three different reinforcement schemes were investigated, however, the test results indicated that the proposed "Vierendeel truss type reinforcement scheme" can restore the original shear strength of such sections having circular or square openings. Overall, this study establishes a cost effective reinforcement scheme for cold-formed steel joist having a large web opening in high shear zone.

II. PROPOSED SYSTEM FABRICATION AND ERECTION

Introduction

Fabricating open-web steel joists is a labor-intensive assembly line process, but is necessary to create a system in which each individual member is efficiently used. This is especially true for the web members (round bar or crimped angles). The reduction in a joist's web material, compared to the amount of web material present in a rolled Web section, for example, is significant, but comes with the price of individually welding each web diagonal to the chord "flange" members. Through years of experience, manufacturers have created assembly processes that have evolved, using techniques that efficiently assemble joists. As a result, less effort is needed in the field to install joists. The author's goal is to continue this fabrication philosophy and take joist efficiency into a new phase of two-way design.

Fabrication

There are two different types of joists used in the proposed system. One direction of joists has dominant joists (16 in. depth), while the other direction consists of non-dominant joists (14 in. depth). Both joist designs mimic standard K-series joist dimensions and member sizes as closely as possible. Top and bottom chord members are 2L2x2 angles, and the web members are 3/4 in. diameter round bars (with the exception of 7/8 in. diameter round bar used at the joist ends). Joist seat details (2 1/2 in. depth) in the proposed system are the same as those describing a traditional configuration. Figure 3.2.1.shows a non-dominant joist. The depth of 14 in. is 2 in. less than the shallowest joist depth available in the standard selection tables (SJI 2005), given a span of 30 ft.

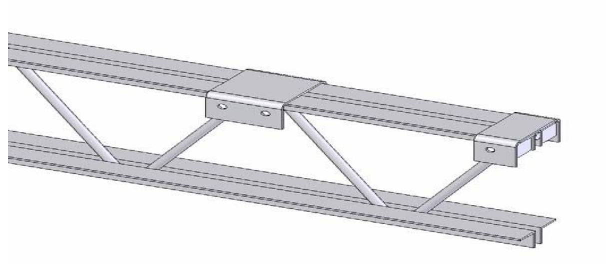

Fig1:Non-Dominant Joist

This reduction in depth is necessary to facilitate the coexistence of the joist top chord with the top chord present in the transverse (dominant joist) direction. A total of four "special" panel points (10'-3" and 11'-10" inward from the joist ends) are needed along the top chord to form this connection.

Inverting the bottom chord is vital to address clearance issues brought about by intersecting the joists. This inversion reduces the chord's section modulus, but is necessary to ensure that the flanges of the non-dominant joist bottom chord do not come into contact with the web members in the dominant direction. A distance between panel points of 19 in. was selected because it reflects a typical panel dimension used when the joist depth is 14 in. Increasing this panel dimension to 24 in. would cause an appreciable lack of moment of inertia of the cross section. The joist manufacturer has the option of cambering the non-dominant joists because both the top and bottom chord members are continuous. The joist system in this study, however, did not take advantage of this opportunity.

Dominant joist design (Figure 3.2.2) in the proposed system deviates much further from traditional design than non-dominant joist design. The core of the member is adapted from a 16K9 joist (a 24 in. panel length was maintained and the chord sizes were very similar). The most pronounced adaptation is the discontinuous top chord member.

A cut in the top chords is made every 6 ft. to accommodate the non-dominant joists.

Additional web members are added within the vicinity of each cut; this includes four small angles (2L1.5x1.5x0.113) and two vertical round bars (3/4" dia.). There are two primary functions of the web

ISSN 2277 – 5528
Impact Factor- 3.145

angles welded to the outside of each chord. The webangles are stiff enough to ensure that the non-dominant joist does not deflect an amount great enough to cause contact between the bottom chords.

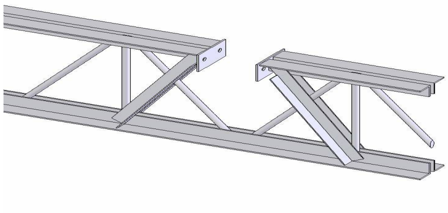

Fig2:Dominant Joist

The web angles also "calm" the moment distribution in the top chords by forming triangles. It should be noted that the web angles are coped at their upper ends to allow a fastening tool to enter unobstructed.

Figure shows the different connecting elements needed to form the orthogonal intersections of the two rows of joists. The piece shown in Figure 3.2.3 is welded to the flanges of the non-dominant joist's top chord The "C" channel formed by cutting a HSS cross section in half may have to be substituted with another structural piece (possibly 3 plates welded together) if clearance becomes an issue as the chord member size increases with load demand. The two chamfered plates in Figure3.2.3 serve as stiffening elements, restraining the vertical portions of the HSS shape from acting as small cantilever beams. Finally, the plate in Figure 3.2.3.is welded to the top chord of the dominant joist. Bolting is accomplished in the field, and welding is done in the shop.

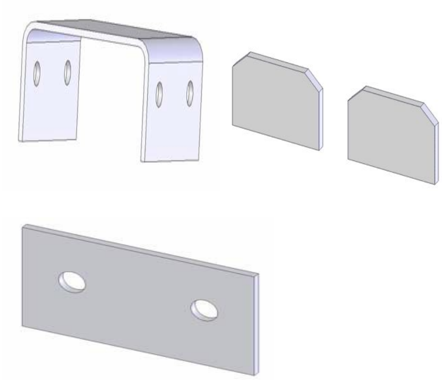

Fig4:Connection Elements

The general panelized erection sequence begins with the delivery of the joists to the job site. The dominant joists have discontinuous top chord members; therefore, temporary restraint is provided at 6 ft. intervals. Otherwise, excessive lateral and torsional deformations may take place during construction. The temporary restraints will likely be sacrificial dowels placed through the bolt holes of the connection plates. When the joists are picked up and moved, workers may elect to handle them "up-side-down" so that the continuous bottom chord (now on top) is the member in compression.

A flat spot needs to be established on the job site (on the ground or perhaps on a floor bay already formed in the building). The dominant joists are then arranged in a parallel manner, held in place with some sort of jig (e.g. 2 x 4 framing) that inhibits rollover. Traditional lateral bridging could be attached at 6' intervals to the joists' top chords during this phase of erection.

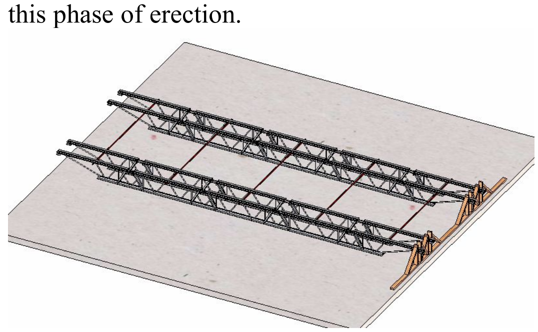

Fig3:Phase 1 of General Erection Sequence

Phase 2 of the general erection sequence entails removing the temporary top chord restraints and setting the non-dominant joists into place (Figure 4.3.1). Four 5/8 in diameter bolts are fastened at each top chord intersection. The author feels that bolting is faster and more economical than welding. It should be noted that a small vertical void (on the order of 1/8 in.) exists between the two bottom chords at the joist intersections. This demands that the load transfer from one joist direction to the other takes place only through the top chords.

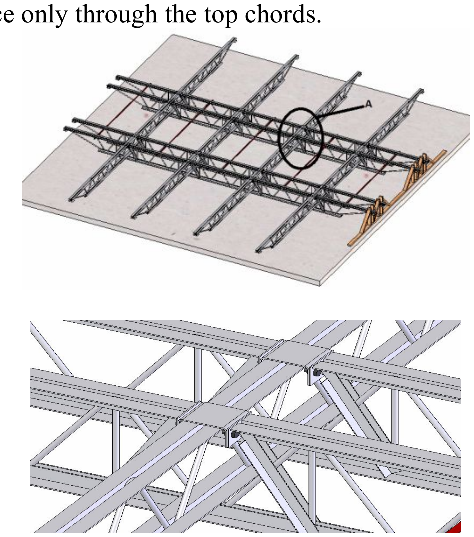

Fig5:Phase 2 of General Erection Sequence

In phase 3 of the general erection sequence, the interlocked joists (together weighing

ISSN 2277 – 5528
Impact Factor- 3.145

approximately 2800 lbs) are hoisted into the air with a crane and set onto the awaiting steel girders. The absence of structural members in the corners of the panels allows crane operators and iron workers to easily maneuver the system. The joist seats are either bolted or welded to the girders in a manner no different from the manner in which traditional open-web joists are connected.

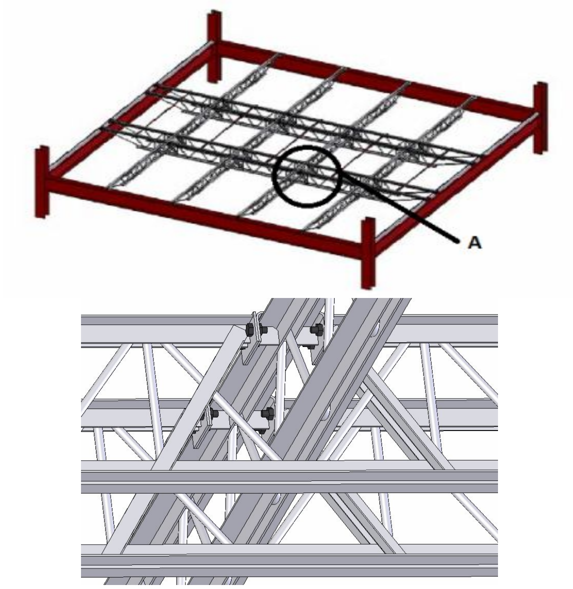

Fig6:Phase 3 of General Erection Sequence

Steel decking is welded into place in phase 4 of the general erection sequence. A contractor may decide to have some of the decking attached to the joists prior to setting

The system onto the girders. As shown in Figure 3.3.4 Phase 4, the steel decking runs perpendicular to the non-dominant joists and is welded to the top chords of these joists at increments consistent with traditional joist construction.

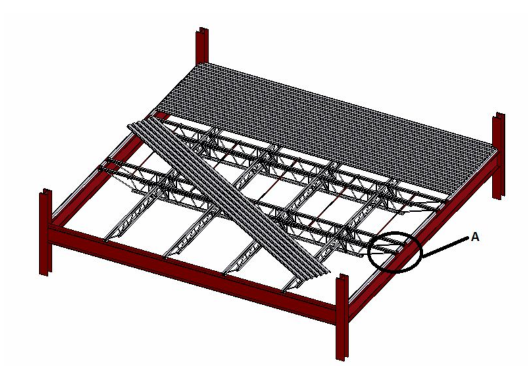

Fig7:Phase 4 of General Erection Sequence

The decking will "bubble-up" a small amount in the vertical direction at the joist intersections due to the presence of the connection elements. This misalignment, equivalent to the thickness of the HSS piece (3/16 in.), is assumed to be negligible in the design.

Direct contact is assumed to be non-existent between the dominant joists and the decking.In other words, the dominant joists only receive loading via the top chord connections to the non-dominant joists. Note that the steel decking in Figure 7. is shown to be terminated at the girders. The figure is illustrated in this manner for clarity. In actuality, the deck is continuous over the girders because the panel is located in an interior bay.

The proposed system uses the girders (which run parallel to the non-dominant joists) to directly carry some of the decking. In other words, each of these girders will have point loads from dominant joist reactions as well as uniform line loading from a 6 ft. tributary width of deck (assuming that a symmetrical adjacent bay is present). To facilitate the bearing of the deck, a small steel shape with a depth of 2 1/2 in. (to match the depth of the joist seat) needs to be welded to the top of the girder. A cold-formed steel channel is shown in Figure 7, but a variety of options are available depending on the contractor's preference. The type of detail used depends on whether or not the girders are designed for composite construction. If composite construction is desired, using a structural tee (Rongoe 1984) may be preferred to provide a more direct load path (through the stem of the tee) from the shear stud to the girder flange.

Similar to a traditional system, the girders occupying the orthogonal column line do not directly carry the steel decking. Due to the load distribution of the system, these girders (running parallel to the dominant joists) will be smaller than the girders in the other direction. If a member is needed to fill the void between the girder flange and the deck (such would be the case if a bearing wall was placed directly over the girder), a concrete pour stop detail could be used (detailed no differently than a traditional system).

The final phase of the general construction sequence is shown in Figure 8. A Mat of welded wire fabric is set into place and a 4 in. concrete slab is poured over the decking. Normal-weight concrete was assumed in the design of the proposed system.

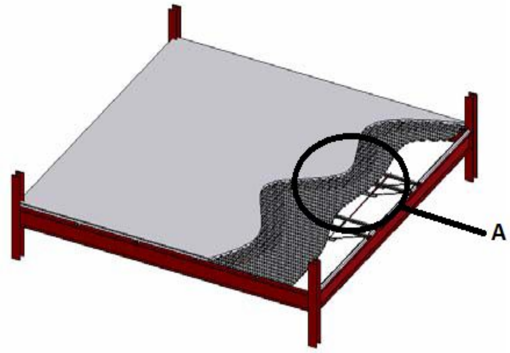

Fig8:Phase 5 of General Erection Sequence

III. TYPES (SERIES) OF STEEL JOISTS

Since the Steel Joist Institute adopted the first standard specification and first load table in 1928 and 1929, respectively, the building designer has been able to specify standard joist designations rather than design each structural component of each steel truss. The current Steel Joist Institute's Standard Specification Load Tables and Weight Tables for Steel Joists and Steel Girders contains three standard specifications for three distinct series.

K-Series

Open Web Steel Joists or K-series are defined as simply supported uniformly loaded trusses that can support a floor or roof deck (see Figure 1). The top chord of the joist is assembly braced against lateral buckling by the deck. The K-series is distinguished by the depth range of 8" to 30" with a maximum span of up to 60' and standard seat depth of 21/2". Maximum uniform load for K-series joists is 550 plf. The standard load table found in the Steel Joist Institute Specification uses standard designations which define the joist depth, a series designation, the total load capacity, live load capacity

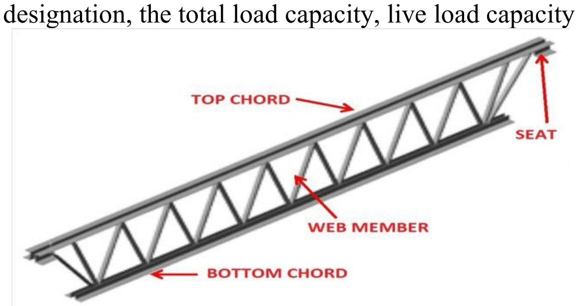

Fig9:K-Series

based on *L*/360 allowable deflection, erection stability bridging requirements and approximate joist weight. It also includes the K-series economy table so the lightest joist for a given load can easily be selected. The ends of K-series joists must extend at least 2 ½" over steel supports

LH AND DLH JOISTS

The second series, the Long Span and Deep Long Span steel joist or the LH and DLH series, is defined as simply supported uniformly loaded trusses (see Figure 3). LH series may support a floor or a roof deck. DLH series may support a roof deck. Both series are designed assuming the top chord is braced against lateral buckling by the deck. Its depth of 18" to 48" distinguishes the LH series joist. It has a maximum span of 96' and a maximum uniform loading up to 1000 plf. Its depth of 52" to 72" and a maximum span of 144' distinguish the DLH series. It has a maximum loading of 700 plf. The

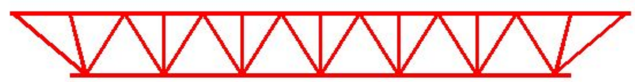

Fig10:LH and DLS Joists

The standard seat depth is 5", although a 71/2" seat depth is preferred for the larger joist designations. The ends of LH and DLH series joists must extend a distance of no less than 4" over a steel support (see Figure 4). The standard load tables found in the Steel Joist Institute's Specification use standard designations that define the joist depth, a series designation, the total load capacity, live load capacity based on *L*/360 allowable deflection.

JOIST GIRDERS.

The third series is Joist Girders designed as simply supported, primary load carrying members. Loads will be applied through steel joists and typically will be equal in magnitude and evenly spaced along the joist girder top chord. The ends of joist girders must extend a distance of no less than 6" over a steel support. Joist girder tables found within the Steel Joist Institute's Specifications include member depth, number of joist spacings, loading at each joist location and an approximate weight of the joist girder. The Steel Joist Institute's Weight Table for Joist Girders includes approximate weights for joist girders with depths from 20" up to 72" and spans up to 60'. Standard seat depth is 71/2". The ends of Joist Girder series joists must extend a distance of not less than 6" over a steel support.

When Joist Girders support equal, uniformly spaced concentrated loads, the joist girder designation provides an adequate specification of the member. For example, the joist girder designation 60G10N12K indicates the joist girder is 60" deep.

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The G indicates that it is the joist girder series, the 10N indicates the number of joist spaces, and the 12K indicates the magnitude of the concentrated load in kips. The building designer should include the self weight of the joist girder in the panel point load. The joist manufacturer will design the joist girder using the most economical web configuration, typically where the diagonals are located under the concentrated loads

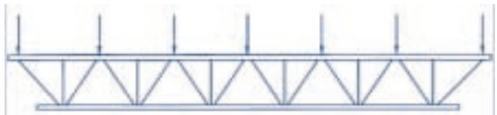

Fig11:Joist Girders.

IV. CONCLUSION

It is desirable to keep the floor height to minimum, but large web openings are often cut on the floor joists of cold-formed steel building structures. Providing appropriate reinforcements for such openings may improve the overall behavior of such cold-formed steel members and may mitigate the detrimental effects of such large web openings. This project presented the experimental results for nine sets of shear capacity tests on 203 mm(8in.)deep 1.092 mm thick galvanized lipped channel cold-formed steel sections, with unreinforced, and reinforced large web opening (65%oftheweb flat height) located in high shear zones. The test program also included tests on sections with no web opening, which provided the base line results. The test program considered circular and square openings. The effectiveness of the reinforcement scheme depends on the reinforcement type and its length, screw spacing and screw pattern. It is found that the Virendeel truss type shear reinforcement scheme, referred to herein as Scheme C, restored the original shear strength cold-formed steel sections having circular and non-circular large web opening. Overall, based on an experimental investigation, this paper establishes prescriptive, cost- effective reinforcement schemes for cold-formed steel sections having large web openings in high shear zones. These schemes are primarily intended for CFS sections widely used in one and two family dwellings. Additional experiments, numerical tests, and parametric studies are needed before design guidelines can be established for general case of shear reinforcements for cold-formed steel sections having large web openings in high shear zones.

V. ACKNOWLEDGEMENT

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