Experiments on the Effects of Power Actuated Fasteners on the Strength of Open Web Steel Joists

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ABSTRACT

A series of 10 full scale tests were conducted on roof sub-assemblages consisting of open web steel joists and roof deck. In these specimens the roof deck was fastened to the joists by using either ⅝-in. diameter puddle welds or by using power-actuated fasteners (PAFs). PAFs are small high strength nails pneumatically or powder driven through the roof deck into the joist top chord angles. In these tests, the joists and roof deck were loaded to failure under downward acting vertical loads. The purpose of these tests was to determine if the presence of the PAFs produced any detrimental effects on the gravity load capacity of a joist roof system. The test results showed essentially identical performance for specimens using puddle welds as for specimens using PAFs. The PAFs produced no detrimental effects on the load capacity of the joists in these tests.

INTRODUCTION

The use of open web steel joists covered by steel deck is a common form of roof construction. The roof deck is normally fastened to the top chord angles of the joists by arc-spot welds, commonly referred to as puddle welds. The number, size and location of fasteners are usually determined by diaphragm design or uplift requirements.

Power actuated fasteners (PAFs) are an alternative to puddle welds for fastening roof deck to steel joists. PAFs are small high strength nails that are driven through the roof deck into the top chord angles of the joists. PAFs can be driven by either powder actuated tools or by pneumatically driven tools. PAFs may offer several advantages over puddle welds, including greater speed of installation and more consistent quality (Glaser and Engelhardt, 1994).

Dimensions, material properties and structural performance characteristics of PAFs are not standardized among manufacturers, as is the case, for example, with high strength bolts. Rather, each manufacturer produces their own proprietary line of fasteners and installation tools. Load capacity values and other design related information for particular fasteners can be found in manufacturers' literature and in reports published by the ICBO Evaluation Service and other code approval bodies. Safety related issues, particularly with respect to powder actuated systems, are covered by several standards, regulations and industry guidelines (ANSI, 1995; OSHA, 1981; PATMI, 1991). Test methods are covered by ASTM Standard E 1190 (ASTM, 1987).

The use of PAFs for fastening roof deck has long been common practice in Europe, where puddle welding is virtually unknown. A recent survey indicated that within the US, acceptance of PAFs has been rather slow, and the use of puddle welds still predominates for fastening roof deck (Glaser and Engelhardt, 1994). This survey further indicated that structural engineers have been hesitant to specify PAFs because of an overall lack of familiarity and information on these fasteners. In the case of roof joists, concerns have also been raised that the PAFs may damage the very thin top chord angles sometimes found in roof joists and thereby impair the load capacity of the joists. A potential concern is that driving PAFs into thin chord members may produce localized distortions of these members. These distortions, in turn, may potentially adversely affect the buckling capacity of the top chord members.

This paper describes a test program on roof sub-assemblages constructed of open web steel joists and roof deck. The vertical load capacity of specimens constructed with puddle welds was compared to nominally identical specimens constructed with PAFs. The purpose of these tests was to determine if the presence of the PAFs produced any detrimental effects on the gravity load capacity of the roof sub-assemblages. More complete details of this test program are reported in Kates, Engelhardt, and Beck (1999).

DESCRIPTION OF TEST SETUP AND SPECIMENS

An experimental setup was developed to test a roof sub-assemblage consisting of two joists with steel roof decking. Typical test specimens are shown in Figure 1. Each specimen consisted of two simply supported joists approximately 26 ft in length, spaced 4 ft apart and covered by steel roof decking. As shown in Figure 1, the specimens

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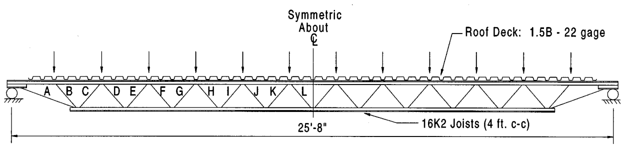
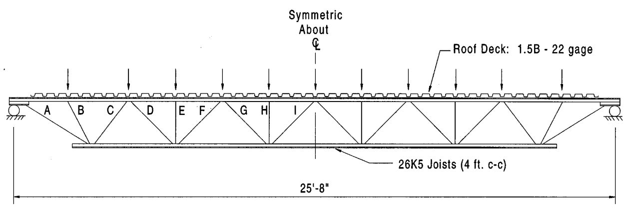
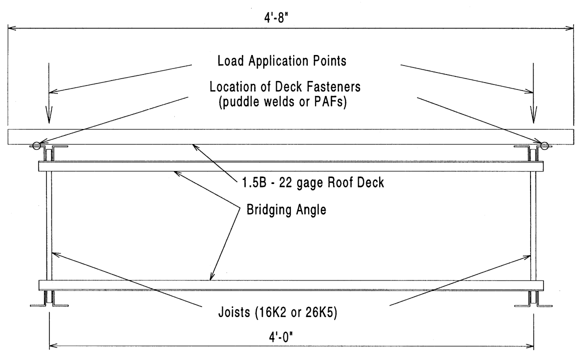


(a) Test Specimen with 16K2 joists



(b) Test Specimen with 26K5 joists



(c) Section through test specimen

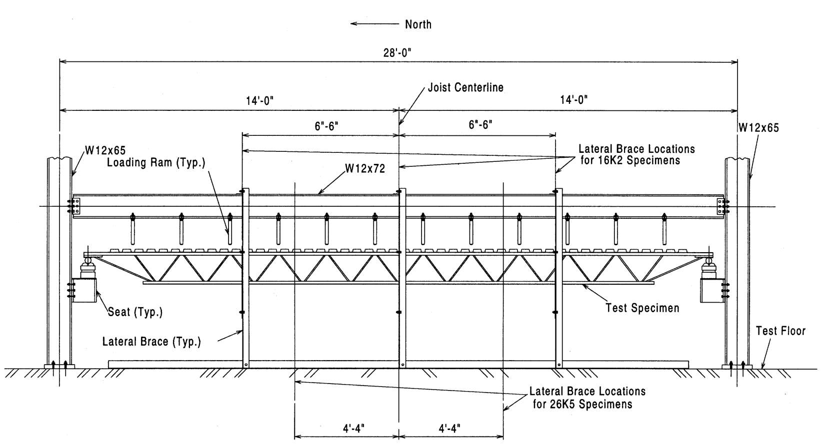
Fig. 1. Test specimens.

were loaded with equal downward point loads applied at the panel points of each joist.

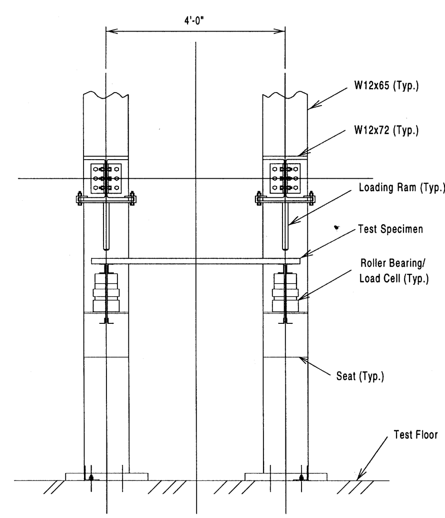
The experimental setup is shown in Figure 2. The test frame consisted of four W12×65 columns that were bolted to the laboratory floor. Each joist end was supported on a roller, which in turn rested on a stiffened seat that was bolted to the columns. Two parallel W12×72 beams spanned between the columns. These beams provided a reaction for the hydraulic loading rams and supported the upper end of the lateral bracing system. Two single angles were bolted to the laboratory floor under each reaction beam to serve as the support for the lower end of the lateral bracing system. Photographs of the test setup are shown in Figure 3.

A roller support was provided at each of the joist ends. The upper portion of the support consisted of the roller assembly. The roller assembly provided a vertical reaction but offered minimal horizontal or rotational restraint, in keeping with the simply supported design assumption. The lower portion of the support consisted of a load cell to measure joist end reactions.

Table 1 provides a listing of the ten specimens tested in this program. Two different joist sizes were tested: 16K2 and 26K5. These two sizes were chosen to represent a relatively light joist (16K2) and a somewhat heavier joist (26K5). The manufacturer's allowable total uniform load for these joists with a 26 ft span are 216 lbs./ft for the 16K2 and 542 lbs./ft for the 26K5. The joists were constructed with double angle members for both the top and bottom chords. The diagonal members consisted of either single angles or round bars. The type and size of member for each element of the joists are listed in Tables 2 and 3. The member designations listed in these tables correspond to the des-



(a) Overall view of the test setup



(b) Cross-section of the test setup

Fig. 2. Test setup.

Table 1. Test Specimens			
Specimen Design	Joist Size	Deck Fastener Type	Deck Fastener Pattern
16K2-PW-1	16K2	puddle welds	36/7
16K2-PW2	16K2	puddle welds	36/3
16K-DX-1	16K2	PAFs	36/7
16K2-DX-2	16K2	PAFs	36/3
16K2-ND	16K2	no deck	no deck
26K5-PW-1	26K5	puddle welds	36/7
26K5-PW-2	26K5	puddle welds	36/3
26K5-DX-1	26K5	PAFs	36/7
26K5-DX-2	26K5	PAFs	36/3
26K5-ND	26K5	no deck	no deck

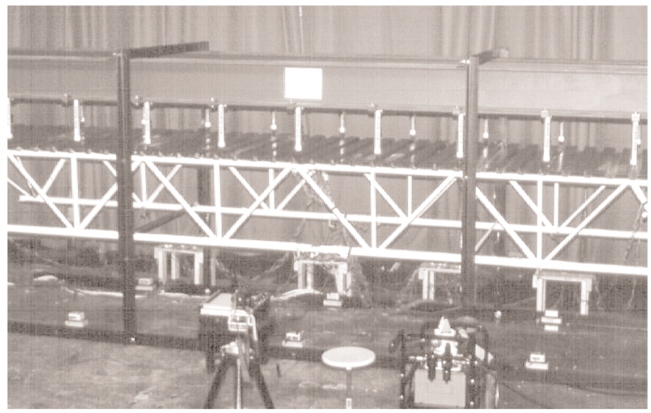
ignations shown in Figures 1(a) and 1(b). As indicated by these tables, the chord angles are quite thin.

The joist loading system consisted of a series of hydraulic rams secured to the W12×72 reaction beams using brackets, as shown in Figure 2. The rams were centered over each of the upper chord panel points and over the longitudinal centerline of each joist. The 16K2 specimens required 12 hydraulic rams for each joist. The 26K5 specimens required 11 hydraulic rams for each joist. The arrangement of hydraulic rams was intended to approximate a uniformly distributed load on each joist.

For all joists of a particular designation, the same heats of steel were used for the different members making up the joist. For example, all of the top chord angles of the 16K2 joists were taken from the same heat, etc. This was done to minimize variations in specimen performance due to material variations. Tensile coupon tests were conducted on samples of selected members from the joists. The results are listed in Tables 2 and 3.

The roof deck used in this test program was designated as a Type 1.5B galvanized roof deck with a 22 gage (0.0295 in.) thickness. The depth of the deck was 1.5 in., with ribs spaced at 6 in. The length of the deck was 56 in., as shown in Figure 1(c). The deck was supplied in standard 36 in. wide panels. Side lap connections between deck panels were made using #10 self-drilling screws. Eight deck panels were used for each specimen.

Each specimen was provided with horizontal bridging angles. The bridging locations and member sizes were chosen according to recommendations of the joist manufacturer and the Steel Joist Institute. The specimens with 16K2 joists were provided with three lines of horizontal bridging, spaced at about 6.5 ft along the length of the specimen. The specimens with 26K5 joists were provided with two lines of horizontal bridging, spaced at about 8.6 ft along the length of the specimen. For both joist sizes, the horizontal bridging consisted of single angles (L 1× l× 7/64) welded to the top chords and bottom chords of both joists, as indicated in



(a) Overall view

(b) Section View

Fig. 3. Photos of test specimen.

Figure 1(c). At the bridging locations, external lateral supports were also provided. Consequently, out-of-plane displacement of the specimens was completely prevented at the bridging locations. The locations of lateral supports and horizontal bridging are shown in Figure 2(a).

The primary variable in this experimental program was the deck to joist fastening system. Two different types of fasteners were used: puddle welds and PAFs. The type of fastener used for each specimen is listed in Table 1. Specimens with puddle welds have a "PW" in the specimen designation, whereas those with PAFs have a "DX" in the specimen designation.

All puddle welds were nominally ⅝ in. in diameter. The puddle welds were made by shielded metal arc welding process (stick welding) using an E7010-A1 3⁄32-in. diameter rod. A welder experienced in welding steel deck recommended this electrode. No weld washers were used.

For specimens with PAFs, the fastener chosen for these tests is illustrated in Figure 4. The fastener is manufactured by Hilti Corp. and is designated as "X-EDNK 22THQ12M." The slightly tapered fastener is 0.146 in. in diameter and approximately 1 in. in length. In addition to the knurled shaft, the fastener has a flat steel washer that assists in aligning the fastener in the installation tool. The washer also provides for a large bearing area on the steel deck, and is intended to prevent the deck from pulling over the top of the fastener. As shown in Figure 4, there is an additional conical washer between the head of the fastener and the flat washer. This conical washer is compressed during fastener installation. This compressed washer is intended to serve as a spring, to assist in keeping the decking in contact with the joist angle and to adjust for variations in energy requirements to properly install the fastener. According to the manufacturer's literature, this fastener is specifically intended for use in fastening steel deck to steel joists, with joist angle thickness in the range of about ⅛ in. to ¼ in. As indicated in Table 2, the top chord angles of the 16K2 joists were just under ⅛ in. in thickness. The fasteners were installed using a pneumatically driven tool. During the installation process, the joists were supported only at their ends, to simulate the flexibility of the roof system during fastener installation. Figure 5 shows views of installed PAFs. Installation of the PAFs into the thin top chord angles of the 16K2 joists caused slight permanent distortions of the chord angle, as seen in Figure 5(b). As described earlier, the effect of these distortions on the buckling strength of the top chord members was of particular concern.

For specimens with either puddle welds or PAFs, two different fastener patterns were employed as indicated in Table 1. For specimens with a 36/7 pattern, a fastener was provided in each deck rib, resulting in seven fasteners per 36-in. deck panel. For specimens with a 36/3 pattern, a fastener was provided in every third deck rib, resulting in

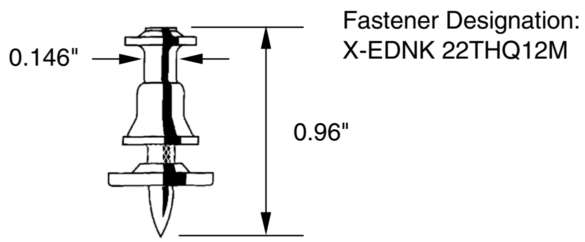
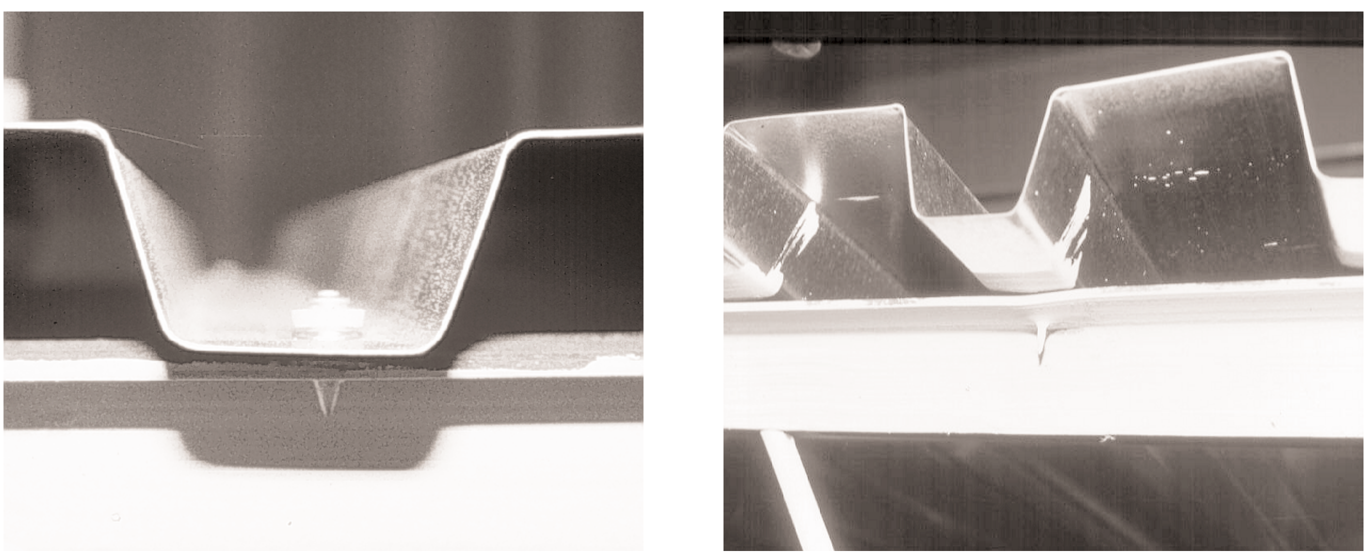


Fig. 4. PAF used in test specimens.



(a) PAF in 26K5 joist *(b) PAF in 16K2 joist*

Fig. 5. PAF installed in joist.

Table 2. Member Size and Material Properties for 16K2 Joists

Member Location	Member Size	Tensile Coupon Properties		
		F_y (ksi)	F_u (ksi)	Percent Elongation
Top Chord	2L 1 1/2×1 1/2×0.113	52.5	76.2	31.4
Bottom Chord	2L 1 1/4×1 1/4×0.109	53.2	75.6	30.3
A	9/16 Round			
B	19/32 Round			
C	19/32 Round	52.0	80.4	29.0
D	19/32 Round			
E	19/32 Round			
F	19/32 Round			
G	19/32 Round			
H	1/2 Round			
I	1/2 Round			
J	1/2 Round			
K	1/2 Round			
L	1/2 Round			

Table 3. Member Size and Material Properties for 26K5 Joists

Member Location	Member Size	Tensile Coupon Properties		
		F_y (ksi)	F_u (ksi)	Percent Elongation
Top Chord	2L 1 3/4×1 3/4×0.155	56.3	83.4	28.8
Bottom Chord	2L 1 1/2×1 1/2×0.123	56.0	74.4	30.1
A	7/8 Round			
B	L 1×1×0.109			
C	L 1 3/4×1 3/4×0.155	56.3	84.8	30.2
D	L1×1×0.109			
E	L1×1×0.109			
F	L1 1/2×1 1/2×0.113			
G	L1×1×0.109			
H	L1×1×0.109			
I	L1 1/4×1 1/4×0.109			

three fasteners per 36-in. deck panel. These two patterns were chosen to represent reasonable bounds on the least and most number of fasteners used in typical design practice.

For specimens with PAF deck fasteners, several PAFs were also installed in the bottom chord angles on the centerline of the joists. These were intended to examine the effect of PAFs on the tensile capacity of thin steel members. PAFs might be used in the bottom tension chord of a joist for supporting suspended ceilings, ductwork, piping, etc. Further, the top chord of the joist may be subject to tension if there is significant uplift on the roof. The PAFs installed in the bottom chord also had a diameter of 0.146 in.

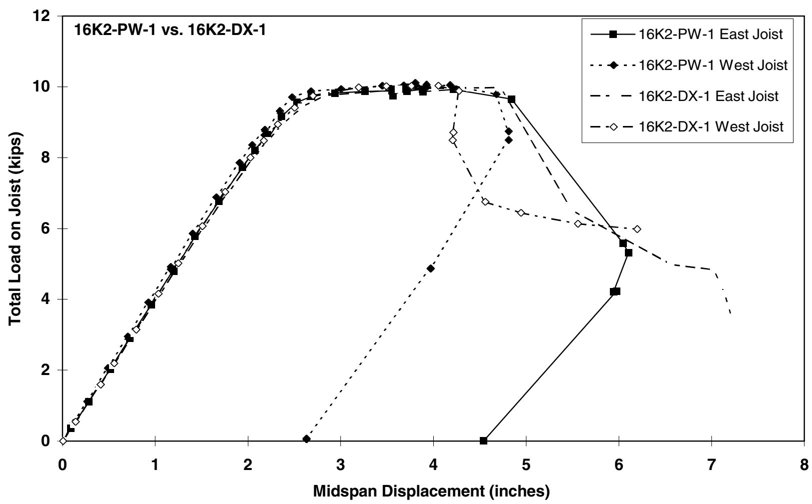
Two of the test specimens (16K2-ND and 26K5-ND) were provided with no decking and no deck fasteners. These specimens were included for comparison purposes. Although these two specimens had no roof decking, they were provided with the same horizontal bridging as all other specimens.

TEST RESULTS

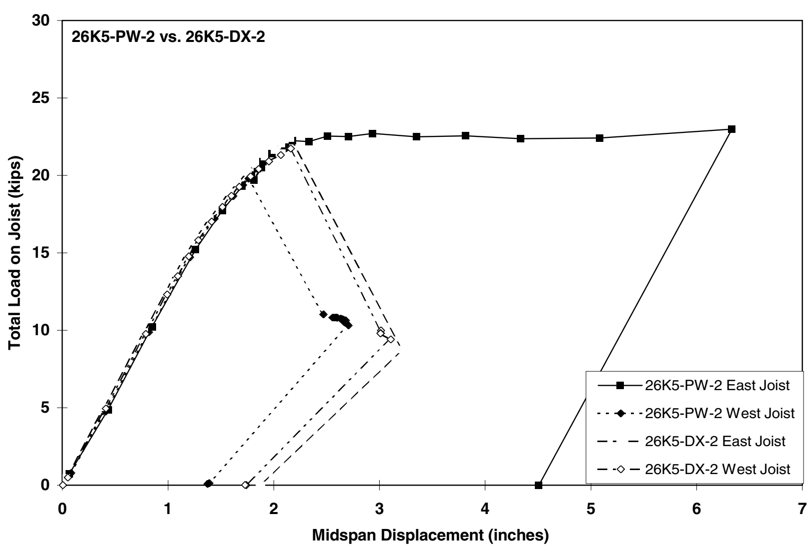
Each specimen was subjected to slowly applied downward vertical loads until failure of the joist. Load cells were provided under the joist end roller supports as described earlier. The total load on each joist was therefore simply computed as the sum of the two end reactions. Vertical displacements were monitored at the center and quarter points of each joist. In addition, critical members of each joist were instrumented with strain gages.

Table 4 summarizes the peak load capacity and failure mode for each joist of each specimen. For most specimens, one of the joists typically failed at a load slightly different than the other. For such cases, after failure of the first joist occurred, loading of that joist was terminated. Loading was then continued on the other joist until failure. Readings of the joist end vertical reactions indicated that there was no sharing of vertical load between the joists. That is, after failure of the first joist occurred in a specimen, there was no redistribution of vertical load to the remaining intact joist. The out-of-plane stiffness of the roof deck over the 4-ft span between the joists was apparently insufficient to transfer vertical load between the joists. Consequently, the ten specimens permitted essentially 20 independent joist tests.

Typical load-deflection plots for several joists are shown in Figure 6. The total load on the joist versus the measured midspan deflection is shown for several 16K2 joists in Figure 6(a) and for several 26K5 joists in Figure 6(b). Based on these load-deflection plots, a ductility ratio was computed for each joist and is listed in Table 4. The ductility ratio was computed as the deflection at joist failure (when unloading occurred) divided by the deflection at yield. Yield was defined at the knee of the load-deflection curve for joists that exhibited a plateau in the load deflection curve, such as the 16K2 joists plotted in Figure 6(a). Several joists exhibited no significant yielding prior to fail-



(a) 16K2-PW-1 and 16K2-DX-1 specimens



(b) 26K5-PW-2 and 26K5-DX-2 specimens

Fig. 6. Typical load-deflection response for joists.

Table 4. Summary of Test Results				
Specimen	Joists	Maximum Load (kips)	Ductility Ratio	Failure Mode
16K2-PW-1	West	10.11	1.7	yielding of bottom chord, followed by local bucking of top chord
	East	9.99	1.7	yielding of bottom chord, followed by in-plane buckling of top chord
16K2-PW-2	West	10.06	1.6	yielding of bottom chord, followed by local buckling of top chord
	East	9.97	2.1	yielding of bottom chord, followed by in-plane buckling of top chord
16K2-DX-1	West	10.04	1.7	yielding of bottom chord, followed by local buckling of top chord
	East	9.98	1.9	yielding of bottom chord, followed by in-plane buckling of top chord
16K2-DX-2	West	9.74	1.8	yielding of bottom chord, followed by in-plane buckling of top chord
	East	10.08	1.2	yielding of bottom chord, followed by in-plane buckling of top chord
16K2-ND	West	9.65	1.0	out-of-plane buckling of top chord
	East	9.85	1.0	out-of-plane buckling of top chord
26K5-PW-1	West	23.86	3.5	yielding of bottom chord
	East	22.45	1.0	buckling of diagonal member
26K5-PW-2	West	19.79	1.0	buckling of diagonal member
	East	22.98	3.4	yielding of bottom chord
26K5-DX-1	West	21.66	1.7	failure of weld on tension diagonal
	East	21.59	1.6	buckling of diagonal member
26K5-DX-2	West	21.73	1.0	buckling of diagonal member
	East	22.41	1.0	buckling of diagonal member
26K5-ND	West	20.00	1.0	buckling of diagonal member
	East	21.82	1.0	buckling of diagonal member

ure, such as three of the 26K5 joists plotted in Figure 6(b). For these cases, the deflection at failure was taken equal to the deflection at yield, resulting in a ductility ratio of 1.0. Thus, joists with a ductility ratio of 1.0 in Table 4 exhibited essentially no ductility prior to failure.

DISCUSSION OF RESULTS

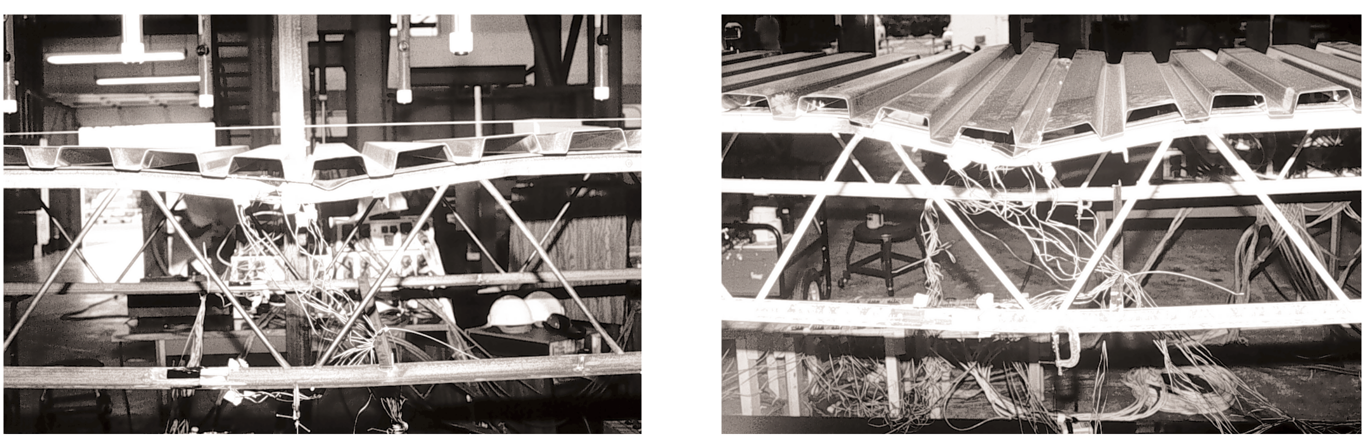
The specimens of greatest interest in this program were those with 16K2 joists. These had the thinnest top chord angles, and were expected to be most vulnerable to potential damage from installation of the PAFs. However, as indicated in Table 4, all of the specimens with 16K2 joists showed nearly the same peak load capacity, regardless of deck fastener type or pattern. The variation in load capacity over the ten joists was less than 5 percent. Further, the average load capacity of the four joists with puddle welds was within 1 percent of the average capacity of the four joists with PAFs. In addition, the 16K2 joists with either puddle welds or PAFs showed essentially the same failure modes. In each case, yielding of the bottom chord occurred first, resulting in a plateau in the load-deflection response of the joist as illustrated by the plots in Figure 6(a). This bottom chord yielding was the source of overall ductility observed in these joists. Yielding of the bottom chord was then ultimately followed by buckling of the central top chord member, which then caused unloading of the joist.

As indicated in Table 4, the top chords for most of the 16K2 joists failed by flexural buckling in the plane of the joist. Figure 7 shows buckled top chords in Specimens 16K2-PW-1 (puddle welds at 36/7 pattern) and Specimen 16K2-DX-1 (PAFs at 36/7 pattern). These photos illustrate the similarity in the joist failure modes. Observe also that for both specimens, the deck remained fastened to the top chord despite the large deformations and distortions of the top chord after buckling. This was observed for specimens both with puddle welds and with PAFs. For two of the joists with puddle welds and for one joist with PAFs, the top chord angles experienced a localized buckling mode of the angle legs as opposed to a flexural type buckling. The failure loads for these joists, however, were no different from the others.

Strain gages were mounted on the central top and bottom chord members of all joists. Ten gages were mounted at a single cross-section to permit accurate estimation of forces in the double angle chord members. The method used to compute member axial force from strain gage data is detailed in Kates et al. (1999). The axial force in the top chord members at the point of buckling determined from this analysis is listed in Table 5. This data shows that all 16K2 top chords with puddle welds or PAFs buckled at essentially the same load. The average buckling load for top chord members was 26.7 kips both for joists with puddle welds as well as for joists with PAFs. This corresponds to an average axial stress of about 40 ksi. Thus, the presence of the PAFs appears to have had no effect on the buckling load for the top chord members. Further, essentially the same buckling load was recorded regardless of the buckling mode. This suggests that the failure loads corresponding to various buckling modes for these double angle members were all quite close.

The 16K2 joists with no decking (Specimen 16K2-ND) failed by out-of-plane buckling of the top chord, at a load just slightly less than the specimens with decking. The out-of-plane buckle extended over several panels between horizontal bridging locations. Since none of the joists with decking exhibited out-of-plane buckling, it appears that the in-plane stiffness and strength of the decking and fasteners (both PAFs and puddle welds) was sufficient to serve as a lateral stability brace for the top chord.

The specimens with 26K5 joists showed somewhat greater variability in peak load capacity and in the controlling failure mode. This variation in performance appears to be related to variability in the joists themselves and unrelated to the type or pattern of deck fasteners. As indicated by the data in Table 4, the average capacity of the joists with puddle welds was essentially identical to the average capacity of joists with PAFs. None of the 26K5 joists failed by buckling of the top chord. Rather, the failures occurred primarily by buckling of diagonal members of the joist. For these specimens, cracking of the weld attaching the diagonal to the bottom chord was observed prior to buckling of the diagonals. One specimen experienced failure of a weld between a tension diagonal and the bottom chord. Ultimately, however, none of the failures were related to the type of fastener used for the decking. Since no failures of top chord members occurred in any of the 26K5 joists, these joists were not as revealing as the 16K2 on the potential



(a) Specimen 16K2-PW-1 (east joist) *(b) Specimen 16K2-DX-1 (east joist)*

Fig. 7. Buckling of top chord members in 16K2 Joists.

Table 5. Buckling Loads for Chords Members of 16K2 Joists		
Specimen	Joist	Force in Top Chord at Buckling (kips)
16K2-PW-1	West	26.2
	East	Not Available[a]
16K2-PW-2	West	26.6
	East	27.2
16K2-DX-1	West	27.1
	East	26.5
16K2-DX-2	West	27.3
	East	26.0
16K2-ND	West	25.3
	East	27.0

[a]Data not available for east joist of 16K2-PW-1 due to damaged strain gages

effects of PAFs installed in the top chords. Nonetheless, the PAFs installed in the top chords of the 26K5 produced no unanticipated or detrimental effects on the joists.

Overall, the results of this testing program indicate that the use of PAFs to fasten steel roof deck to open web steel joists had no detrimental effect on the vertical load capacity of the joists. The performance of specimens with PAFs was essentially identical to specimens with puddle welds. As noted earlier, concerns have been raised in the past that PAFs driven into very thin top chord joist members may adversely affect the joists. The top chord of a joist will normally be in compression, and the capacity of a top chord member will therefore be controlled by its buckling strength. The deck fasteners may influence buckling strength in several ways. First, if installation of the fasteners produces significant permanent distortions of the top chord member, the buckling capacity of the member may be reduced due to initial crookedness effects. As described earlier, this was the major issue of concern in these tests. Further, if the fasteners do not have sufficient shear strength or stiffness, the deck may no longer serve as an effective lateral brace, and out-of-plane buckling may potentially occur at a lower load. Despite the very thin top chord members in the test specimens, neither of these potentially detrimental effects was observed. Moreover, it is anticipated that the PAFs used in these tests should have no adverse effects on the strength of joists with a top chord angle thickness at least as large as the 16K2 joists tested in this program.

As described earlier, for specimens with PAFs, fasteners were also driven into the central bottom chord members. One fastener was driven into each of the two angles of the bottom chord, both for the 16K2 and 26K5 joists. Under the applied joist loads, these bottom chord members were subject to large axial tension forces. The behavior of tension members with PAFs is of interest because PAFs are sometimes used to attach items such as ceiling hangers, ductwork, electrical conduit, etc. to the bottom chord of a joist. PAFs may potentially affect tension capacity due to loss of cross-sectional area in the chord member or due to stress concentrations introduced by the fastener. In these tests, despite the very thin bottom chord members and despite the development of yield level stresses in the bottom chords, the presence of PAFs had no detrimental effect on these members. An extensive series of smaller scale tension tests on thin steel members recently completed by the writers (Kates et al., 1999) confirm this same observation. These tests indicated that a tension member with a PAF has a higher net section fracture strength than a tension member with drilled holes having the same diameter as the PAF. These results will be presented elsewhere.

CONCLUSIONS

A series of full-scale tests were conducted on roof subassemblages constructed of open web steel joists and metal roof decking. Specimens were constructed using either puddle welds or PAFs to fasten the roof decking to the joists. PAFs were also installed in the bottom chord of several specimens. All specimens were then tested to failure under vertical loads. The results showed no difference in joist load capacity whether puddle welds or PAFs were used to fasten the decking. The use of PAFs produced no detrimental effects on either the compression capacity of the thin top chord members or on the tension capacity of the thin bottom chord members of the joists.

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