PDHonline Course S116 (1 PDH)

Special Vertical & Lateral Load Considerations for Open Web Steel Joists and Joist Girders

Instructor: D. Matthew Stuart, P.E., S.E., F.ASCE, F.SEI, SECB, MgtEng

2013

PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone & Fax: 703-988-0088
www.PDHonline.org
www.PDHcenter.com

VERTICAL LOADS:

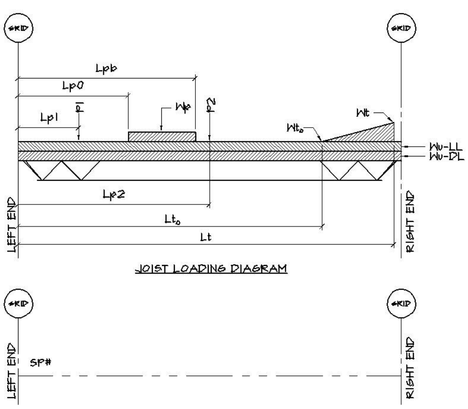
Most joist manufacturers recommend that the most economical option (from a material quantity stand point) for the support of concentrated or non-uniform loads is to designate the use of special joists through the use of specific load diagrams. In addition to providing a load diagram to the manufacturer, it is also recommended that the design engineer verify that the end seat of the special joist is compatible with the end seat depth of the adjacent joist series. In general, the maximum shear capacity of a standard 2.5 inch deep joist end seat is limited to approximately 9.2 kips. End reactions greater than this will require that the special joist be supplied with a deeper seat. Alternatives to submitting load diagrams to the manufacturer include:

1. Using a heavier standard joist that is capable of supporting an equivalent uniform load that provides a shear and moment envelope capacity that is greater than the actual imposed loads.

2. Specifying a KCS series joist that provides a shear and moment envelope that is greater than that imposed by the actual loads.

3. Substitute a wide flange beam capable of resisting the imposed loads.

Special gravity load diagrams and plans showing net roof uplift loads should always be provided because only the joist manufacturer is capable of designing for special loadings and checking members for the stress reversals associated with uplift loads. The following is an example of a multi-purpose loading diagram for joists.



LEGEND
Wu-LL -- FULL UNIFORM LIVE LOAD
Wu-DL -- FULL UNIFORM DEAD LOAD
Wp1,Wp2 -- PARTIAL UNIFORM LOAD
LP1a -- DISTANCE FROM LEFT END TO START OF Wp
LP1b -- DISTANCE FROM LEFT END TO END OF Wp
Wt -- MAXIMUM TRAPIZOIDAL LOAD
Wto -- ZERO TRAPIZOIDAL LOAD
Lt -- DISTANCE FROM LEFT END TO Wt
Lto -- DISTANCE FROM LEFT END TO Wto
P1,P2 -- CONCENTRATED LOADS
LP1,LP2 -- DISTANCE FROM LEFT END TO P1, P2, ETC.
L -- TOTAL SPAN (C/C OF SUPPORT)

NOTES:
1. UNIFORM LOADS ARE IN UNITS OF (PLF)
2. CONCENTRATED LOADS ARE IN UNITS OF KIPS
3. VERTICAL LOADS SIGN CONVENTION; POSITIVE (DOWN), NEGATIVE (UP)
4. AXIAL LOADS; COMPRESSION--POSITIVE; TENSION--NEGATIVE
5. AXIAL LOADS APPLIED TO TOP OF CHORD, CUMLATIVE ALONG ASSOCIATED COLUMN LINE B/T EDGE OF DECK & TENSION LEG OF X-BRACING.

REMARKS:
1. ALTERNATE STANDARD JOIST SIZE GIVEN DOES NOT INCLUDE UNIFORM AXIAL LOAD. JOIST MANUF. SHALL INCLUDE AZIAL FORCES IN DESIGN OF JOIST.

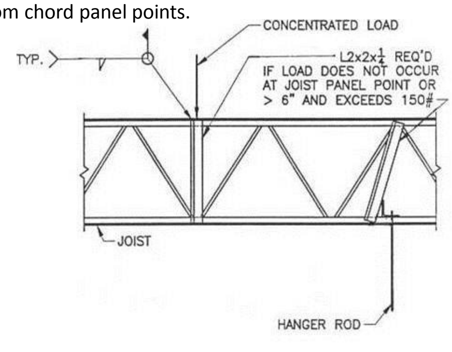
When specifying standard joist sizes as an alternate to special joists, the following precautions should be taken:

1. The actual uniform load calculated from the load diagram must fall completely within the equivalent shear diagram of the standard joist selected. The allowable shear diagram can be constructed from information derived from the joist load tables and Steel Joist Institute (SJI) specifications.

2. The maximum allowable end shear is equal to the allowable uniform load times half the span for a given joist. The allowable shear at the center of the joist is a percentage of this end shear value. The percentage is given in the SJI specifications for all joist series available. K series joists are designed for centerline shear of 25% of the maximum end shear.

3. The point of zero shear for the special load diagram should also be determined. If this point is not relatively close (±1-foot) to the center of the joist span, there may be diagonal members that are subject to stress reversal. If stress reversals are present with the use of a standard joist, a KCS series joist must be specified as an alternate to the special joist.

Collateral hanger loads that do not exceed 150# can be accounted for as a part of the uniform loading. When hanger loads exceed 150 pounds, they should be accounted for with special concentrated load diagrams. An exception to this rule that commonly occurs includes the support of sprinkler lines, cable trays and ducts. For these items, it is common for hanger loads to exceed 150#. Joist manufacturers indicate that for these type of hanger loads, the use of uniform loads to account for the actual loadings has proved to be reasonable and economical. However, it is recommended that the design engineer specify on the drawings the hanger spacings that are required in order to help minimize the magnitude of the reactions. In addition, for these types of hangers, web reinforcement should be installed for hangers that occur at locations greater than 6" from the bottom chord panel points.



TYPICAL CONCENTRATED LOAD ON JOIST DETAIL

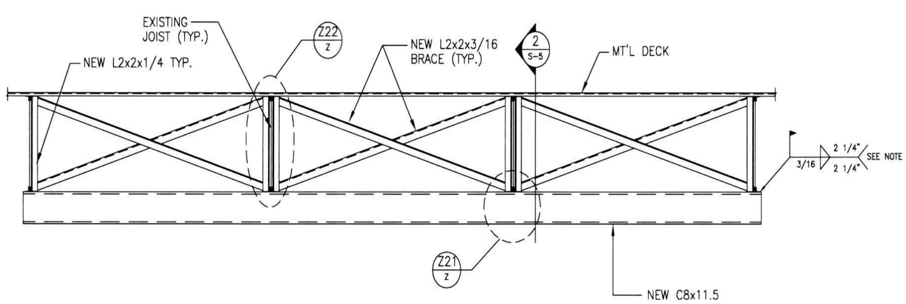
Typically the method for determining the proper distribution of loads on a series of adjacent joists is based on a rigid support transfer element resulting from static equilibrium, however, there are situations where the elastic nature of the supports and joists results in a different distribution of the support reactions. The relative stiffness (β) of a series of joists based on a given distribution beam equals:

$$\beta = \sqrt[4]{\frac{\left(\frac{K}{S}\right)}{(4EI)}}$$

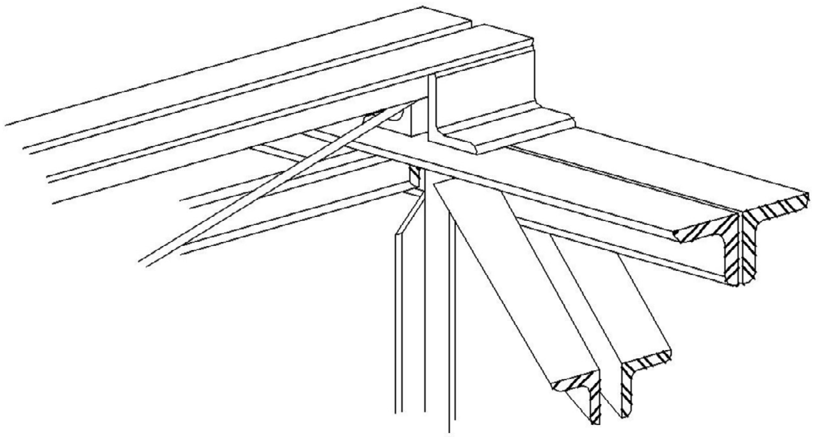
Where: K = The stiffness of the joist, kips/inch.
S = The spacing of the joists.
E = The modulus of elasticity for the beam.
I = The moment of inertia of the beam.

If S is less than Π/4β the beam on elastic support calculations are applicable. If the spacing limit is not exceeded and the length of the beam is less than 1/β, the beam may be considered rigid with respect to the supporting joists and the reactions to the joists may be determined by static equilibrium

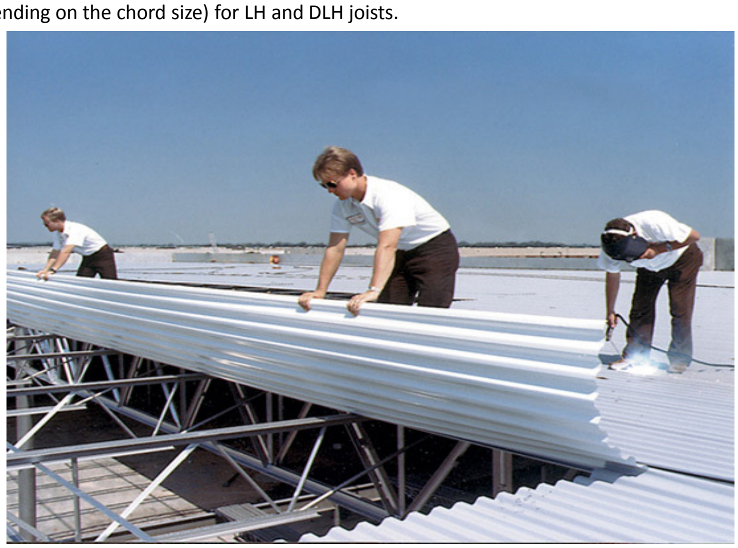
In lieu of using spreader beam below or above the joist it is also possible to field fabricate a truss within and between the joists to distribute the load more effectively between several members.



Joist manufacturers recommend that for eccentrically loaded joist girders or beams, the joist spacing should be staggered on each side of the girder by at least 6" to allow for the bearing seats of the joists to extend past the center line of the girder. This offsetting helps to compensate for the twisting action induced on the girder due to differences in joist reactions from opposing sides of the girder. Extreme situations may actually require the addition of a bottom chord extension brace from the supported joist to limit the twisting action.

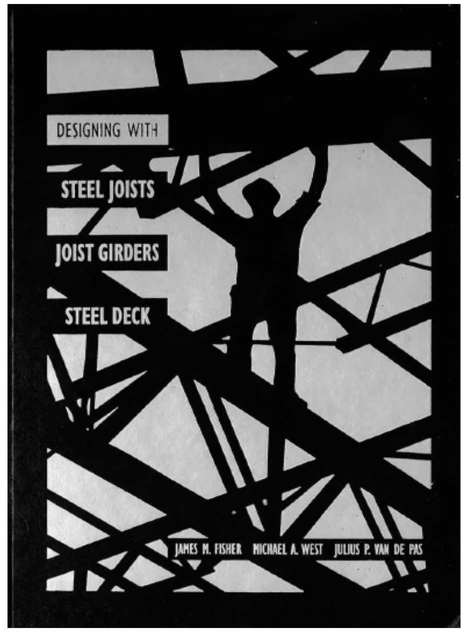


The minimum deck fastening requirements as required by SJI specifications for adequate joist top chord stability is 100 PLF (300# at 3'-0" o.c.) for K-series joist and from 120 PLF to 250 PLF (depending on the chord size) for LH and DLH joists.



Source: Loadmaster Systems

The capacity of joists and joist girders to function as chord members and collector elements for diaphragm loadings is discussed extensively in "Designing With Steel Joists, Joist Girders, Steel Deck" by Fisher, West & Van De Pas. The first edition of this manual was published by Nucor Corporation, however, it is no longer in publication. Your local joist manufacturer or steel fabricator may be able to provide you with a used copy, or a newer edition. Pertinent information worth noting from this same referenced text is provided on the following slides.



The maximum chord force permitted by K-series joists is 6.8 kips.

THE MAXIMUM ECCENTRIC AXIAL LOAD CAPACITY FOR A K JOIST CHORD CAN BE DETERMINED BY FINDING THE ALLOWABLE AXIAL LOAD AND BEARING MOMENT COMBINATION FOR THE TOP CHORD ANGLES IN THE JOIST.

JOIST CHORDS

THE SEAT DETAIL IS SHOWN IN THE FIGURE ABOVE. THE MAXIMUM CHORD SIZE FOR A K-SERIES JOIST IS TWO ∠2x2x1/4. THE SEAT DEPTH IS 2.5 INCHES, THUS THE MOMENT IN THE CHORD EQUALS P(2.5-.592) = 1.91P

ASSUMING THE DECK LATERALLY SUPPORTS THE TOP CHORD, AND THAT THE MAXIMUM UNBRACED LENGTH OF THE DOP CHORD ABOUT ITS x-x AXIS IS 48 INCHES, THE CHORD CAPACITY CAN THEN BE DETERMINED FROM THE AISC BEAM-COLUMN INTERACTION EQUATIONS:

$$\frac{r_a}{F_a} + \frac{C_m r_a}{\left(1 + \frac{f_s}{F'_c}\right) F_s} \leq 1.0$$ AISC Eq. (H1-1)

$$\frac{r_a}{0.6F_Y} + \frac{f_S}{F_a} \leq 1.0$$ AISC Eq. (H1-2)

WHERE,

r_a = .609
S_a = .494
KL/r_a = 48/.609 = 78.8
F_a = 19.2 KSI (F_y = 50 KSI)
f_a = P/A = P/1.88
f_b = M/S_a =2.06P/S = 2.06P/0.494 = 4.17P
F_b = $0.6F_y$ = 0.6(50) = 30 KSI
F'_e = 24.05 KSI
C_m = 0.2 (REVERSE CURVATURE ASSUMED

SOLVING EQUATION (H1-1) YIELDS P ≈ 15 KIPS
SOLVING EQUATION (H1-2) YIELDS P ≈ 6.8 KIPS

THUS, A MAXIMUM ECCENTRIC JOIST CHORD FORCE OF 6.8 KIPS CAN BE USED.

SINCE THE ENGINEER DOES NOT KNOW THE EXACT CONFIGURATION OF THE JOIS AND SINCE CHORD FORCES DUE TO THE HORIZONTAL COMPONENT IN THE END DIAGONAL ALSO EXIST, THE MAXIMUM CHORD FORCE CAN BE USED ONLY AS AN ESTIMATE. THE ENGINEER OF RECORD MUST INDICATE THE JOIST CHORD FORCE REQUIREMENTS ON THE CONTRACT DOCUMENTS.

The maximum chord force permitted by joist girders weighing more than 30 PLF is 20 kips.

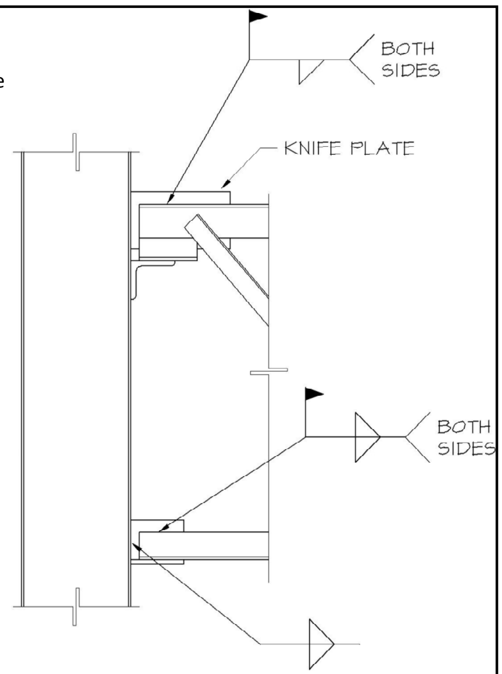
Vulcraft has done extensive testing of the moment capacity for joist girders used in the Basic Connection. Based on their test program, the maximum chord forces using standard six inch deep joist girder seats are presented in the table below.

Joist Girder Weight Per Foot	Maximum Horizontal Load Capacity For Standard 6" Deep Joist Girder Seats*
Less Than 30	10 Kips
30.1 & Greater	20 Kips

* Capacities can be increased by one-third for load cases involving wind or seismic loads.

These values are based on using ¾ inch A325 bolts and a minimum of two ¼ inch fillet welds 5 inches long along the sides of the seat. The loads can be increased by one-third for loads containing wind or seismic forces.

Chord forces greater than these values can be obtained through extensive modification to the standard chord/bearing assembly of each section, however, most joist manufacturers recommend the use of wide flange spandrel beams in lieu of joists for bracing/chord forces greater than 20 kips. Joist girders can be modified to provide chord force limits of up to 200 kips.



KNIFE PLATE CONNECTION

For situations in which the perimeter edge roof angle, acting as a diaphragm collector, is perpendicular to the span of the joists, the magnitude of the collector force should be compared to the rollover capacity of the joist seat. The rollover capacity of a typical joist is about 1650#. For a typical joist spacing of 6'-0" o.c., this capacity would equate to a maximum collector force of 275 PLF. Tests performed by Vulcraft indicate the actual upper bound of the ultimate rollover strength limit of some H-series joists was 9.0 kips. Examples of a rollover analysis are illustrated on this and the next two slides.

IT IS OFTEN NECESSARY TO EVALUATE THE RESISTANCE OF A JOIST SEAT RELATIVE TO A LATERAL FORCE APPLIED TO THE TOP OF THE JOIST SEAT. THIS SITUATION FREQUENTLY EXISTS AT THE PERIMETER OF A ROOF OR FLOOR DIAPHRAGM WHEN SHEAR COLLECTORS ARE NOT PROVIDED. BASED ON AN ELASTIC ANALYSIS, THE CALCULATED ROLLOVER RESISTANCE OF THE JOIST SEAT TO A LATERAL FORCE IS EXTREMELY LOW. AN ULTIMATE STRENGTH APPROACH PROVIDES SIGNIFICANTLY HIGHER RESISTANCE VALUES. THE DIFFERENCE BETWEEN THE TWO APPROACHES IS SHOWN IN THE EXAMPLE BELOW.

JOIST SEAT ROLLOVER RESISTANCE EXAMPLE

DETERMINE THE RESISTANCE TO ROLLOVER OF THE SEAT SHOWN BELOW.

1. CALCULATE THE RESISTANCE BASED ON FIRST YIELDING.
2. CALCULATE THE RESISTANCE BASED ON ULTIMATE STRENGTH.

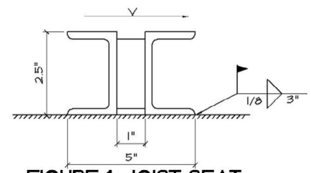


FIGURE 1: JOIST SEAT

A. THE SEAT IS MADE FROM ∠2x2x1/8 WITH A YIELD STRENGTH OF 50 KSI.

B. THE SEAT HAS SUFFICIENT INTERNAL STRENGTH (BASED ON VULCRAFT'S FABRICATION PROCEDURES) TO FORCE THE FAILURE TO BE OF THE SEAT ANGLE.

C. THE RESISTING FORCES ARE ASSUMED AS SHOWN:

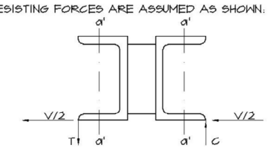


FIGURE 2: RESISTING FORCES ON JOIST SEAT

SOLUTION:

1. RESISTANCE BASED ON FIRST YIELD:
THE YIELD MOMENT CAN BE DETERMINED FROM THE SECTION MODULUS OF THE SEAT ANGLE.

THE SEAT ANGLE SECTION MODULUS, $S=bt^2/6$, WHERE b AT a'-a' IS DETERMINED AS SHOWN IN THE FIGURE BELOW.

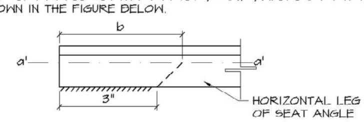


FIGURE 3: EFFECTIVE ANGLE WIDTH

b = WELD LENGTH + (2-k)
b = 3+(2-7/16) = 4.56 IN.
S = $(4.56)(.125)^2/6$ = .0119 IN^3
M_Y = SF_B
F_B = 0.75 F_Y = (0.75)(50) = 37.5 KSI
M_Y = (.0119)(37.5) = .446 IN-KIPS

THE MAXIMUM FORCE T OR C EQUALS M /(2-7/16) = 0.286 KIPS

T = C(2.5V)/5 = 0.5V

V = 570 POUNDS

IT CAN BE SEEN THAT THE STRENGTH IS CONTROLLED BY YIELDING IN THE SEAT ANGLE RATHER THAN THE STRENGTH OF THE FIELD WELD.

2. RESISTANCE BASED ON ULTIMATE STRENGTH:
AT FAILURE THE SEAT IS ASSUMED TO BE DEFORMED AS SHOWN IN FIGURE 4 BELOW.

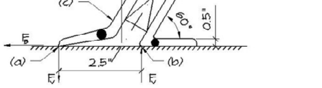


FIGURE 4: SEAT FAILURE MODE

IT IS ASSUMED THAT THE PRIMARY RESISTANCE TO OVERTURNING IS PROVIDED BY THE WELD AT POINT (a) AND BY A COUPLE FORMED BY THE FORCES F_V.

PLASTIC HINGES ARE ASSUMED TO HAVE FORMED IN THE SEAT ANGLES. THE SEAT LEG WHICH LIFTS FROM THE BASE IS ASSUMED TO HINGE APPROXIMATELY 1/2 INCH ABOVE THE BASE. THIS ASSUMPTION IS BASED UPON THE LOCATION OF THE RIGID END DIAGONAL WHICH IS WELDED BETWEEN THE ANGLES.

THE ULTIMATE WELD RESISTANCE CAN BE DETERMINED AS FOLLOWS:

WELD STRENGTH:

TAKING MOMENTS ABOUT POINT (b),

$2.5\ F_V = 2.5V$, THUS $F_V = V$

FORCES IN THE HORIZONTAL DIRECTION,

$F_H = V$

THE TOTAL FORCE AT (a) = $\sqrt{(F_V^2 + F_H^2)} = 1.414V$

THE RESISTING FORCE IN THE WELD AT POINT (a) EQUALS R = 1.414V.

THE ULTIMATE STRENGTH OF THE WELD, BASED ON THE VON MISES YIELD CRITERIA EQUALS:

$R = (70/\sqrt{3})(.125)(.707)(3) = 10.71$ KIPS

$V = R/1.414 = 7.58$ KIPS

SEAT ANGLE STRENGTH:
THE MAXIMUM STRENGTH OF THE SEAT ANGLES EQUALS THE SHEAR YIELD STRENGH OF THE SEAT ANGLE TIMES THE SHEAR AREA AT POINT (c). AGAIN, USING VON MISES YIELD CRITERIA:

$V = (50/\sqrt{3})(.125)(4.56) = 16.45$ KIPS

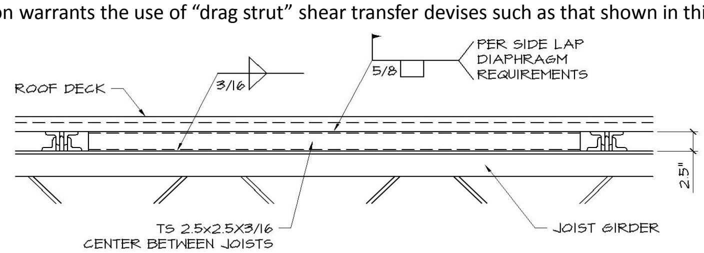
THUS, THE ULTIMATE STRENGTH IS 7.58 KIPS.

USING A FACTOR OF SAFETY OF 2.0, THE ALLOWABLE SHEAR FORCE EQUALS 3.79 KIPS.

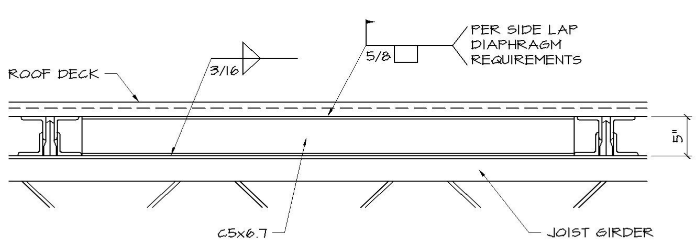
AS CAN BE SEEN FROM THE DEFORMED SHAPE, THE SEAT ASSEMBLY WOULD DISPLACE LATERALLY A SIGNIFICANT AMOUNT AT THE ULTIMATE LOAD. BASED UPON THE GEOMETRY ASSUMED IN FIGURE 4, THE LATERAL DEFLECTION WOULD BE APPROXIMATELY 1.15 INCHES. IT IS SUGGESTED THAT THE DEFLECTION AT SERVICE LOADS BE LIMITED TO 0.25 INCHES. IT SHOULD BE NOTED THAT EVEN AT THIS LIMIT, THERE WOULD BE SOME YIELDING OF THE CONNECTION.

THUS BY PROPORTIONING THE ALLOWABLE SHEAR EQUALS (.25)(7.58)/1.15 = 1.65 KIPS.

In either case, the designer should determine if the magnitude of the collector force in this situation warrants the use of “drag strut” shear transfer devises such as that shown in this slide.



SHEAR COLLECTOR WITH K JOIST



SHEAR COLLECTOR WITH LH JOIST

Laminated wood and structural wood fiber decks are not normally used as diaphragms as there has not been adequate research performed by the industry. Standing seam roofs are also not capable of functioning as an adequate diaphragm. In all of the above cases, in-plane horizontal diagonal bracing must be provided as a part of the roof system.

For flexible metal deck diaphragms it should be noted that deck shear capacities tabulated in the Steel Deck Institute (SDI) Diaphragm Design Manual already take into account the one-third increase for wind or seismic loads. It is recommended that diaphragm deflections be checked when used to brace the top of masonry or concrete walls. Deflection equations are provided in the SDI Diaphragm Manual. It is recommended that the empirical formula of this same manual be used as guideline for the limitation on diaphragm deflection when supporting masonry or concrete walls.