PDHonline Course S267 (3 PDH)

Engineered Wood Connections For Snow Loads

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Course Content

1. Introduction

Engineered wood products have become common in light frame construction but use of prefabricated products requires special attention to connection details. This course focuses on two common framing members---prefabricated wood I-joists and structural insulated panels---and presents methods to design connections for these members.

Typical framing details for the prefabricated wood I-joists are readily available from manufacturers but the manufacturers' details may, at best, be valid only within the limitations of the prescriptive requirements of the IRC--- a maximum 70-psf ground snow load. In snow country it is not unusual to see ground snow loads in excess of 100-psf. Connection details for designs that exceed the limits for prescriptive requirements must be supported by calculations consistent with basic engineering principles.

Structural Insulated Panels (SIPs) differ in that all connections require substantiation via submitted engineering calculations. Although the manufacturers' literature may include standard details, a review of the Evaluation Services Reports (ESRs) for SIPs indicates that engineered design is required for connections.

This course reviews roof snow load calculation, solving for the reactions of statically determinate sloped beams and presents connection details for prefabricated wood I-joists and structural insulated panels. Although the details have been developed for higher snow load applications, the calculation methods and details are applicable to all gravity loading conditions.

2. Prescriptive Versus Performance Based Requirements

This seems like a good place to introduce a review of prescriptive requirements versus performance based requirements. Prescriptive requirements are just what the word says; they are elements or pieces of a system that are specified (prescribed) by various building codes. The International Residential Code (IRC) is an example of a code with primarily prescriptive requirements. The intent of the IRC is that one could design a safe and serviceable building by conforming to all the prescriptive structural requirements of the code. Since it is difficult to produce prescriptive requirements to account for all loading conditions, some loading conditions are outside the scope of the code's prescriptive requirements.

From the latest version of the IRC:

> When a building of otherwise conventional construction contains structural elements exceeding the limits of Section R301or otherwise not conforming to this code, these elements shall be designed in accordance with accepted engineering practice. (R301.1.3 Engineered design)
>
> Buildings in regions with ground snow loads greater than 70 pounds per square foot shall be designed in accordance with accepted engineering practice. (R301.2.3)

Performance based requirements must meet a specific level of performance. Accepted engineering practice is performance based. Structural members and connections are sized to support imposed loads within a specific margin of safety. The codes produced by industry associations, such as the National Design Specification for Wood Construction (NDS) from the American Forest and Paper Association and the American Wood Council, are performance based codes. Provisions of the industry codes are either directly incorporated into the International Building Code (IBC) or are incorporated by reference.

3. Responsibility of the Design Engineer

Building codes have introduced the concept of "Design Professional in Responsible Charge". When there is an architect on a project, they are generally considered the Design Professional in Responsible Charge and it is their duty to review submittals prepared by other members of the design team. However, engineers have a responsibility to the public including other design professionals.

When conflicts arise we should seek to educate owners and other members of the design team, including building officials and plan reviewers. If individuals charged with reviewing and approving a design do not have the technical expertise necessary to understand the engineering involved, that does not give one license to violate any provisions of the applicable codes.

4. Physical Problem and Analysis

To develop roof framing details for snow loads we will use, as an example, a stick framed, gable roof for a 24-ft wide structure with 2-ft overhangs at the eaves. The roof pitch is 6:12. The ground snow load is 80 psf, 14% higher than the prescriptive limit of the IRC. Ground snow loads can be found in both the IRC and IBC and should be available from the local building department.

4.1 Determination of Roof Snow Load

Roof snow loads must be calculated from the specified ground snow load. The American Society of Civil Engineers (ASCE) publishes a standard (ASCE7) titled Minimum Design Loads for Buildings and Other Structures that governs the calculation of roof snow load along with other building loads such as wind, seismic, flood, et cetera.

Per ASCE7, flat roof snow load (Pf) is calculated as 0.7 times the ground snow load, modified by factors for exposure, thermal performance of the roof, and importance.

$$Pf = 0.7CeCtIPg, \qquad \text{(Eq. 1)}$$

Where Ce is the exposure factor dependent on surrounding terrain features, Ct is the thermal performance factor for the roof and I is the importance factor. The adjustment factors are tabulated or read from graphs in ASCE7. For our case, Ce = 1.0 (terrain category B or C), Ct = 1.0 (warm roof) and the importance factor is 1. So Pf = 56 psf.

Pitched roof snow load (Ps) is calculated as Pf times the slope adjustment factor (Cs). For a 6:12 pitch with a roof surface that does not meet the criteria for an unobstructed slippery surface Cs is 1.0, so Ps = 56 psf.

ASCE7 requires the designer to consider unbalanced snow load on roof structures where the roof slope is less than 70˚ or greater than the larger of 2.38˚ or 70/W + 0.5 where W= the distance from the edge of the eave overhang to the ridge. In our case, 70/W + 0.5 = 5.5˚ so unbalanced snow load must be considered. For a gable roof with prismatic members spanning 20 feet or less from the ridge to the eave, unbalanced snow load is calculated as I * Pg. So our rafters and connections will be designed for 80 psf snow load in addition to roof dead load. I selected 20 psf as the roof dead load.

In this example I added a 100 plf line load for ice. Current ASCE7 snow load requirements include ice dams and icicles along eaves only for two specific roof conditions: warm roofs with an insulation R value of less than 30; and cold roofs with R less than 20. Neither condition exists in this example so the requirement to check for 2Pf on the overhanging portion of roof does not exist. In my engineering judgment, however, the addition of some line load for ice is reasonable so I included the line load. A summary of the physical problem is shown in Figure 1.

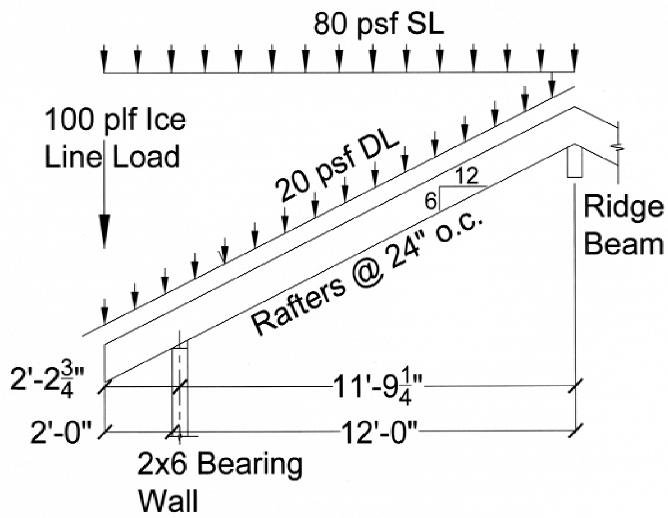

Figure 1. Physical Problem

Note that the snow load is applied to the horizontal projected area of the roof whereas the dead load is applied along the length of the member.

4.2 Analysis

Charles-Augustine de Coulomb, an 18th century engineer whose work with electrostatic attraction was fundamental to the development of the theory electromagnetism, also developed many concepts in the field of structural mechanics. He is reported to have said that any problem can be solved if one can draw an accurate Free Body Diagram of the problem. Figure 2 shows the Free Body Diagram for this problem.

I have elected to use a traditional pin-roller model (a hinge on one end and a roller on the other). In two dimensions, there are three unknown reactions: a vertical and horizontal reaction at the pin (hinge) and a vertical reaction at the roller. These are designated Rx1, Ry1 & Ry2. In two dimensions there are also three available equations of static equilibrium:

$$\sum Fx = 0, \qquad \text{(Eq. 2)}$$
$$\sum Fy = 0, \qquad \text{(Eq. 3)}$$
$$\sum M = 0, \qquad \text{(Eq. 4)}$$

Since the number of unknowns equals the number of independent equations, the system is determinate and can be solved by simple statics. From Eq. 2 we find that Rx1 is zero since there are no horizontally applied loads. Applying Eq. 4 and summing moments about the pin support, with counterclockwise being positive:

$$(2.23)(200) + (40)\left(\frac{2.23}{Cos\ 26.6°}\right)\left(\frac{2.23}{2}\right) + (160)\left(\frac{2.23^2}{2}\right) - (40)\left(\frac{11.8^2}{2 * Cos\ 26.6°}\right) - (160)\left(\frac{11.8^2}{2}\right) + (11.8)\left(R_{y2}\right) = 0$$

So Ry2 = 1130#. Using Eq. 3 we can then solve for Ry1:

$$R_{y1} = 200 + \left(\frac{11.8 + 2.23}{Cos\ 26.6°}\right)(40) + (11.8 + 2.23)(60) - 1130 = 1940$$

Figure 2 shows the solution for the member supports.

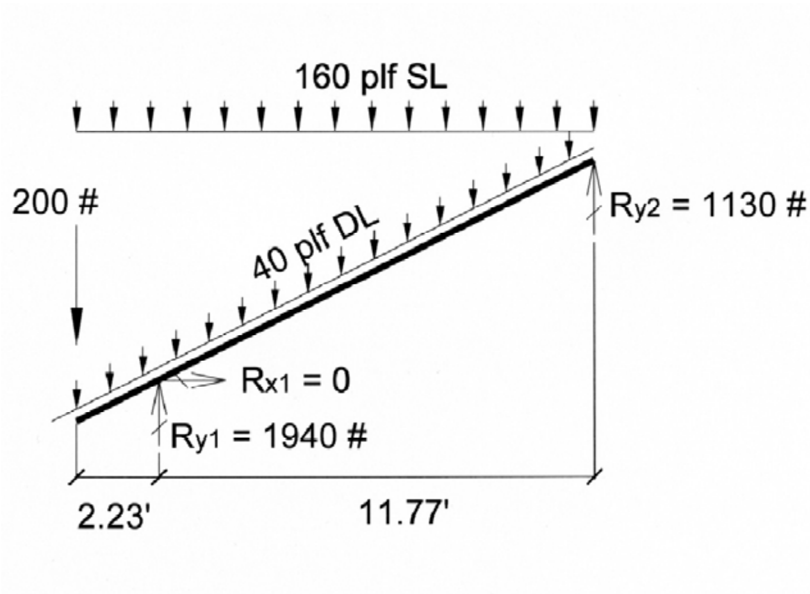

Figure 2, Free Body Diagram

Once we have the reactions, we can solve for the internal member forces. We can do this by solving a Free Body Diagram of the member cut along its axis or by recognizing that the internal member force vectors at a support must equate to the reaction at that support. Essentially this reduces a Free Body Diagram to a Vector Force Diagram. Figure 3 shows the resulting internal shear and axial force diagrams for the rafter.

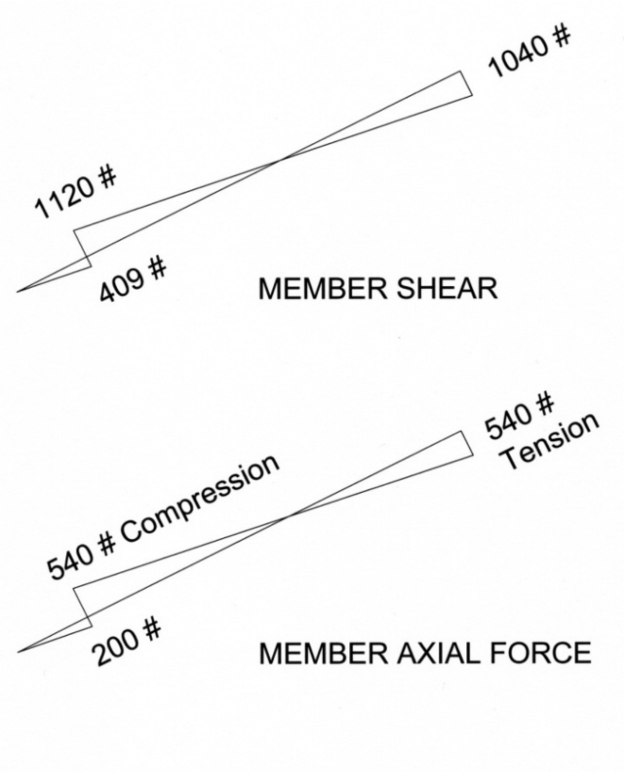

Figure 3. Internal Member Force Diagrams

If the rafter is supported on a beveled plate at the exterior wall, the components of the reaction on that surface are required. The components are determined by the geometry and we can find those by solving the right triangle. The pitch angle of a 6:12 slope is 26.6 degrees. The reaction normal to the bearing surface is 1940# * Cos 26.6°. The up-slope reaction (applied down-slope force) is 1940# * Sin 26.6°. The vector components of the reaction may also be determined from a graphical analysis by carefully drawing the components to scale at the proper angles. Figure 4 shows the resolved components of the lower reaction on a beveled plate.

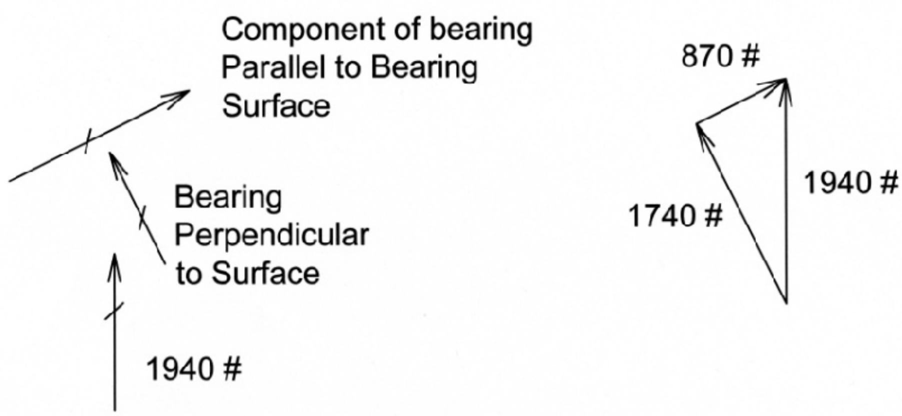

Figure 4. Lower Reaction Resolved into Components

Before moving on, let's examine the internal member forces more closely. Figure 5 shows a Free Body Diagram of the lower end of a solid sawn rafter with a seat cut at the exterior wall plate. The rafter is simply supported (pin-roller supports) with no eave overhang. The internal moment (M) is zero (free end). The internal shear and axial forces are computed as:

$$V = R(\cos \emptyset) \qquad \text{(Eq. 5)}$$
$$F = \mathrm{R}(\sin \emptyset) \qquad \text{(Eq. 6)}$$

Where Ø is the pitch angle of the roof.

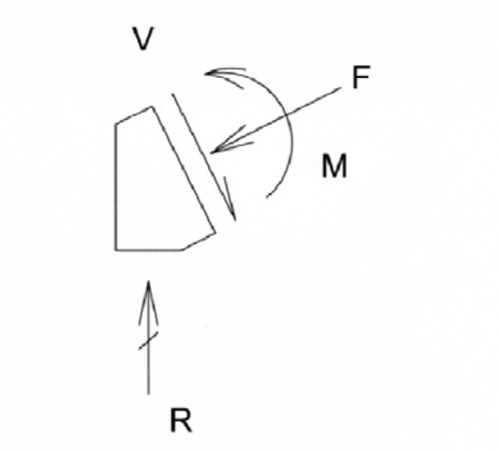

Figure 5. Free Body of Rafter Free End

I've included this to show that the internal member forces are a function of the member geometry and load and are not affected by the bearing condition. We could resolve the end reaction on any sloped surface we choose without changing the over-all reaction or the internal member forces. Note that the seat cut rafter is still supported by a beam or bearing wall at the ridge. A conventional rafter/ceiling tie system would have different support conditions and different reactions.

For practicing engineers who solve similar problems every day this review is basic but; I have been surprised, in reviewing submitted plans and calculations, to find a number of engineers who struggled with the concept of the incurred down-slope force of a rafter supported on a beveled surface. In traditional framing, using solid sawn lumber, rafters were seat cut on level wall plates and this issue never arose. Installing a rafter-to-rafter strap at the ridge, although an essential piece of some connection details, cannot change the internal member forces or the reaction at the lower bearing. A review of manufacturers' suggested roof details for engineered products seems to indicate that the issue may be more ubiquitous than we would like.

The design engineer must provide detailing that accounts for the forces at each end of the rafter. The axial tension at the ridge (see Figure 3) gives the force for design of the rafter-to-rafter strap across the ridge. Note that, in comparing Figures 3 and 4, the bearing normal to the surface is the absolute sum of the member shears across the support and the reaction component parallel to that surface is the absolute sum of the member axial forces across the support. The specified connection must account for the reaction component parallel to the bearing surface.

4.3 Alternate Analysis

Prescriptive connections for roof framing systems that ignore member spans and snow load magnitude have existed in codes for some time. Several articles have been written on prescriptive connections and why framing consistent with those requirements seem to work despite what traditional engineering computations would show for the forces involved. Similarly, framing that violates the principles of mechanics presented here may survive appreciable snow load without obvious damage.

Just as the map is not the territory, the mathematical model isn't the structure. In most cases, a mathematical model is intended to simplify a physical system to enable quick and easy computation of forces in order to produce a safe design (consistent with standard engineering principles) which will work within the time constraints of a design office. As a design engineer, one is always free to perform a more detailed analysis that may do a better job of modeling actual behavior of the physical system. The analysis presented, although simplistic, produces verifiable designs without the need to solve an indeterminate structure or produce a three dimensional model of all elements of the system.

5. Rafter Selection

Prefabricated I-joists are an engineered wood product with structural panel webs connected to wood flanges. These days the webs are usually OSB (Oriented Strand Board), typically 3/8" thick, glued into a slot milled in LVL (Laminated Veneer Lumber) flanges. Flanges may be as thin as 1 1/8" for some joists.

The joist selected for the rafters should be based on worst case conditions. The rafter bending moment and upper reaction should be calculated for no live load on the overhang as building owners may remove snow from only the overhanging portion of the roof. In our example the design bending moment is 3490 ft-lb. Worst case shear is 1150 lb and the design bearing condition is 1940 lb. The typical mode of bearing failure for I-joists is where the web splits the joist flange. Bearing values for manufactured joists vary with each manufacturer but are usually not the same as the allowable shear capacity of the joist. Although much of the manufacturers' literature is filled with load and span tables for prescriptive designs, the capacity of the joists will be given in a table of properties.

6. A Word on Nails

Fasteners for wood construction have allowable values based on deflection limits not on ultimate failure. You may have used a single nail to attach a piece of scrap to the support for your basement storage shelf and used it to support your weight as you reached for Christmas lights. In fact that one nail may have supported you and your overweight brother-in-law who's currently living in the basement. Nails may have a considerable reserve capacity beyond the deflection limit at which they are considered to have failed. As engineers, we should always use published fastener values for connection design or calculate the capacity of the fastener using published dowel bearing strength and accepted engineering principles.

Nail values are sometimes given for common nails. Most carpentry work is accomplished with nail guns that use thinner nails. Gun nails may be equal to, or thinner than, box nails. When specifying connections relying on nails, one should be aware of the nails used in the field and adjust connection values accordingly. ANSI standard nail sizes and allowable loads are available in the NDS and other sources.

When specifying connections with framing hardware, such as that produced by Simpson Strong Tie or United Steel Products, one should require that the connectors be installed with the manufacturers' specified fasteners or adjust connector values for other fasteners. Some engineers specify hand nailing of all light gage framing connectors.

7. Wood I-Joist Connection Details

The following details are similar to those found in manufacturer's catalogs. In many cases CAD details are available free from the manufacturer. The ones in this report were produced by the author.

7.1 Ridge Connection

As mentioned above, the internal axial tension at the upper end of the rafter gives the force required to be resisted by a strap across the ridge. The manufacturer's suggested details may specify a strap such as an LSTA18 or LSTA24. For my details I have used products from the latest Simpson Strong Tie catalog.

Nails should be excluded from the ends of the joists to prevent splitting of the flange, so the allowable load of the strap must be adjusted for reduced nailing. (TrusJoist specifies that nails be excluded from the end 2 1/8" of the joist.) Using an LSTA18 strap, this would reduce the nails in one half of the strap from 7 to 5 so the allowable strap capacity should be adjusted accordingly. For Simpson LSTA straps full nailing would include an additional nail near the end of the strap (as seen in the plan view of the strap). Neglecting this nail may eliminate some concern regarding splitting of the joist flange but will further reduce the strap capacity.

Where the straps are applied directly to the joist flange, nailing of roof sheathing is compromised by the presence of the straps. A better detail is to apply the straps over the roof sheathing. This also facilitates inspection of roof framing and sheathing nailing at the same time. Given nominal 5/8" (40/20 rated) sheathing, 1 ½" nails would penetrate 0.91" into the joist flange (1.5" – 19/32" = 0.91"). This is more than the minimum 6 diameter penetration required by the NDS to achieve the full lateral value of 10d common nails (6 x 0.148" = 0.89"). In addition, since the sheathing is continuously attached to the rafters, nailing the strap to the rafter through the sheathing may improve the over-all performance. However, Simpson Strong Tie's catalog states that 10d x 1 ½" nails may be substituted for 10d nails at 100% of the table load except when straps are installed over sheathing. So it is the intent of Simpson that the full length of a 1 ½" shank penetrates into the joist and longer nails should be used where the strap is installed over sheathing.

Backing out the nail values from the tabulated load for a Simpson LSTA18 strap indicates that the nails are at 86#/ nail at 100% load duration. This is less than the 90#/nail given in the current NDS for 10d (common) nails with a 20 gauge steel side plate.

How Simpson's requirement for full 1 ½" penetration relates to use of the strap with joist flanges to 1 1/8" thick is beyond the scope of this course. Designers must exercise engineering judgment in producing details for construction documents.

Figure 6 shows a typical framing detail at a dropped ridge. The ridge strap has an installed capacity of 630# for each rafter (LSTA18 @ 100% duration = 770#, 770x1.15x5/7=630#). This is greater than the required tension of 540#. Note that I-joists are never seat cut at the upper end.

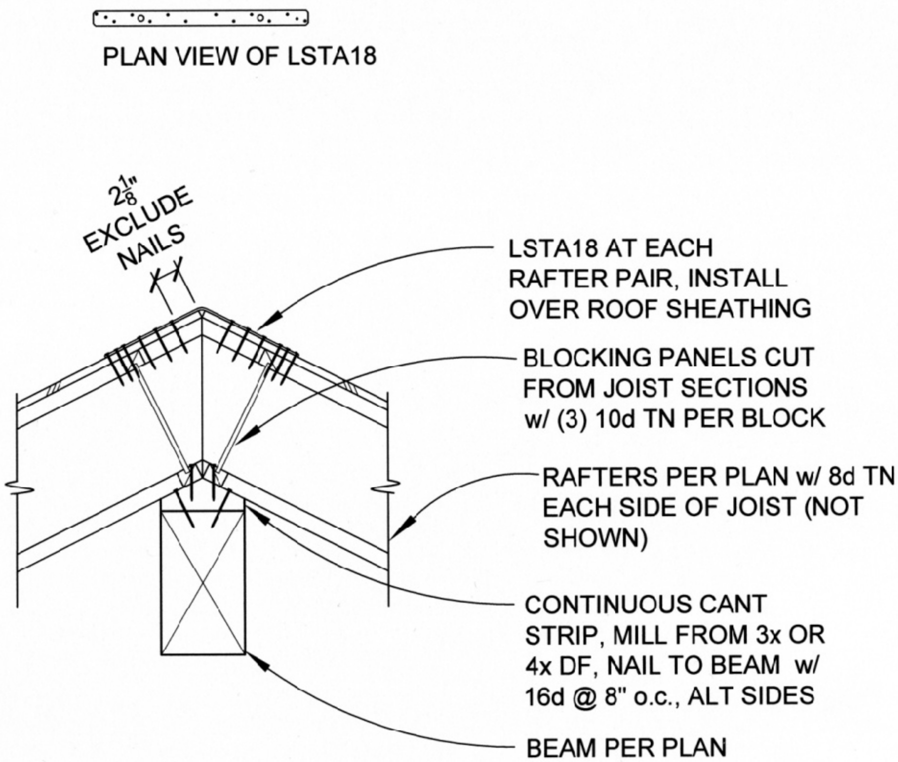

Figure 6. Typical Connection at Drop Ridge Beam

Some carpenters prefer to mill the top surface of beams to create the sloped bearing. This may be allowed for solid-sawn, LVL or PSL (Parallel Strand Lumber) provided the designer has increased the required depth of the beam to accommodate the removed material. Milling the top surface of glued-laminated beams should be discouraged.

Figure 7 shows a typical connection with a flush framed ridge beam. Hangers must be sloped to accept the rafter. Manufactured I-joists are never slotted or seat cut to accommodate flat seat connectors. Field adjustable hangers are available from both Simpson and USP. Table values for those hangers are given for sloped applications and are good to a 45 degree pitch. When using hangers to support manufactured I-joists one must always check the joist capacity for the bearing length provided by the hanger.

Web reinforcement (called web fillers or web stiffeners) are required each side of the joist for the field sloped hangers. Manufacturers' data includes web fillers required for the different joists and information on how the fillers must be attached to the joists. A gap should be left between the top of the filler and the underside of the joist flange. This is to prevent any possibility of the filler prying up on the flange.

The hangers used in the figure are Simpson LSSUI hangers. Note that a tension strap has been provided even though the specified nails to resist the down-slope force (7-10d x 1 ½) may be adequate to provide the required force (130#/10d nail with 18 Gauge side plates and 115% load duration factor). Typical joist manufacturers' details show the strap and it is good practice to provide the separate tension tie. The design engineer may calculate the allowable force provided by the joist nails and elect to eliminate the strap but should pay attention to any spacing issues with hanger nails installed through the flange of the joist.

In this example the ridge is a 6 ¾" glue-lam beam and the strap allowable capacity is calculated as the ratio of number of available nails to number of nails for full nailing. For the LSTA24, 770# x 1.15 x 5/9 = 490# < 540#. Therefore an LSTA30 was selected. The Simpson tabulated load for the 30" strap is the same as the tabulated load for the 24" strap but use of the longer strap allows more nails to be used.

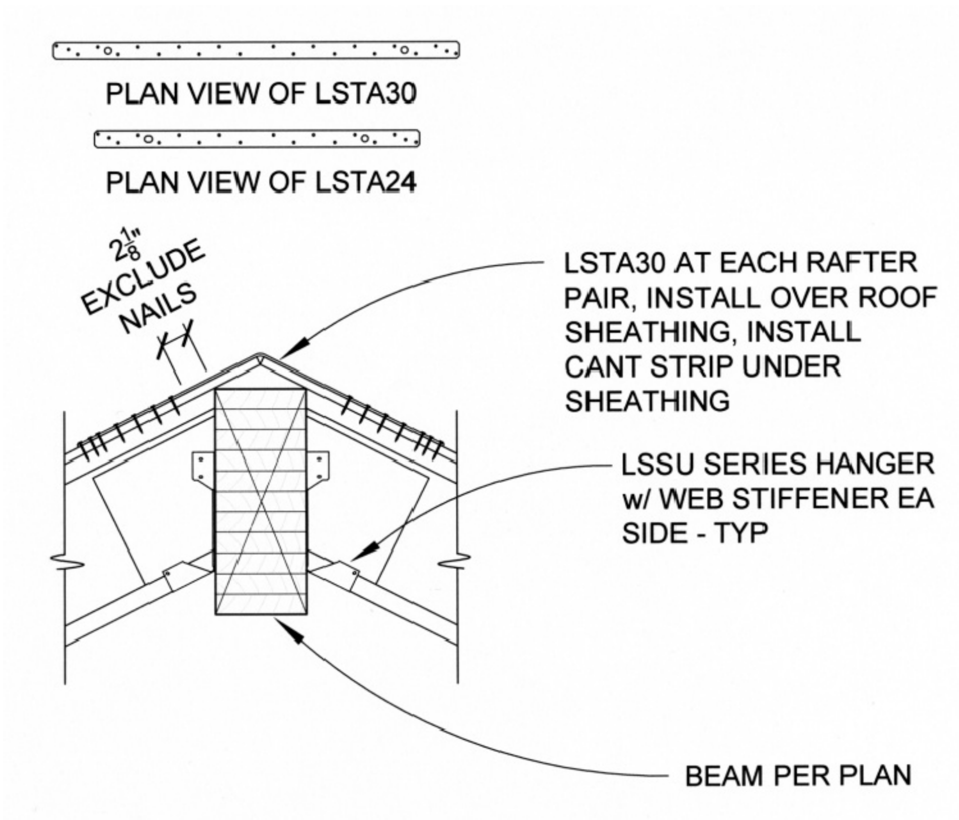

Figure 7. Typical Connection to Flush Ridge

Occasionally framing may be pushed up to provide a clean ceiling. In Figure 8 a glue-lam ridge has been installed into the cold roof sleeper space. For this detail, our analysis is no longer correct. Per ASCE7, a cold roof with an insulation R value of 25 or more between the ventilated space and the heated space would have a thermal factor of 1.1. (With a ground snow load of 80 psf I think we can assume the required roof insulation will be greater than R25.) So the calculated uniform roof snow load would be 62 psf. Note that the thermal factor doesn't enter into the calculation for unbalanced snow load so our rafter selection and connection forces aren't changed.

The tension strap is shown at the cold roof sleepers. This strap is an important part of the connection and should not be omitted. One could provide additional toe nails from the cold roof sleepers to the rafters to transfer the tension force but, since the cold roof sleepers must be continuously attached to the rafters to prevent the entire cold roof system from sliding off the roof (it has happened), extra nails are not required. The cold roof sleepers are usually attached to the rafters with toe nails each side of the sleepers but some designers prefer to use framing anchors pre-installed to the rafter (over the first layer of sheathing) and nailed to the sleepers. The down-slope force used to design the cold roof sleeper to rafter connection is calculated as the Sine of the roof pitch times the sum of the cold roof weight plus snow load times the rafter spacing.

$$Fv = (SL + DLcr)(\sin\ p)s \qquad \text{(Eq. 7)}$$

In our example, using 10 psf for the weight of the cold roof system, $Fv =$ (80+10)*(Sin 26.6)*(2.0) = 81 lb/linear foot of rafter. A 10d box nail used with a 1" DF side plate has a tabulated capacity of 93#. With a 115% load factor and a 5/6 toe nail reduction factor, the allowable load is 89#. So the required cold roof sleeper to rafter connection is 10d TN @ 12" o.c. As this is a critical connection, the detail specifies 10" o.c. spacing.

This configuration may result in a reduced load for the hanger if all the specified nails can't be installed. This may be particularly true in steeper roof pitches. Minimum edge distance is 6 diameters or approximately 5/8" but I might discount any nail driven within 1" of the bottom of the beam. In the example of Figure 7 all hanger nails are required. If the bottom nail each side did not have sufficient wood available for full lateral value, the installed capacity of the hanger could be calculated as the ratio of usable nails divided by number of nails for full nailing. The tabulated value for snow load is 1275# so the capacity would be 8/10 of 1275# or 1020#. This is less than our worst case reaction of 1040# so the installation would not be acceptable. Other hangers with fixed, sloped bearing seats are available but usually as special order items incurring additional cost and lead times.

Note that there are two layers of sheathing in this detail. The first layer separates the insulated rafter space from the cold roof sleeper space to produce a true cold roof. This inner layer of sheathing is frequently designed to act as the horizontal diaphragm of the lateral force resisting system where the upper sheathing supports the superimposed live load and must be span rated for the applied load. Where the under layer is the horizontal diaphragm, the sleeper space may be constructed free of blocking to improve the performance of the cold roof. However, where the lateral system resists seismic loads in snow country, a portion of snow load may be on the roof during a seismic event and the cold roof system could fail even though damage to the remainder of the building structure is prevented by proper design of lateral force resisting elements.

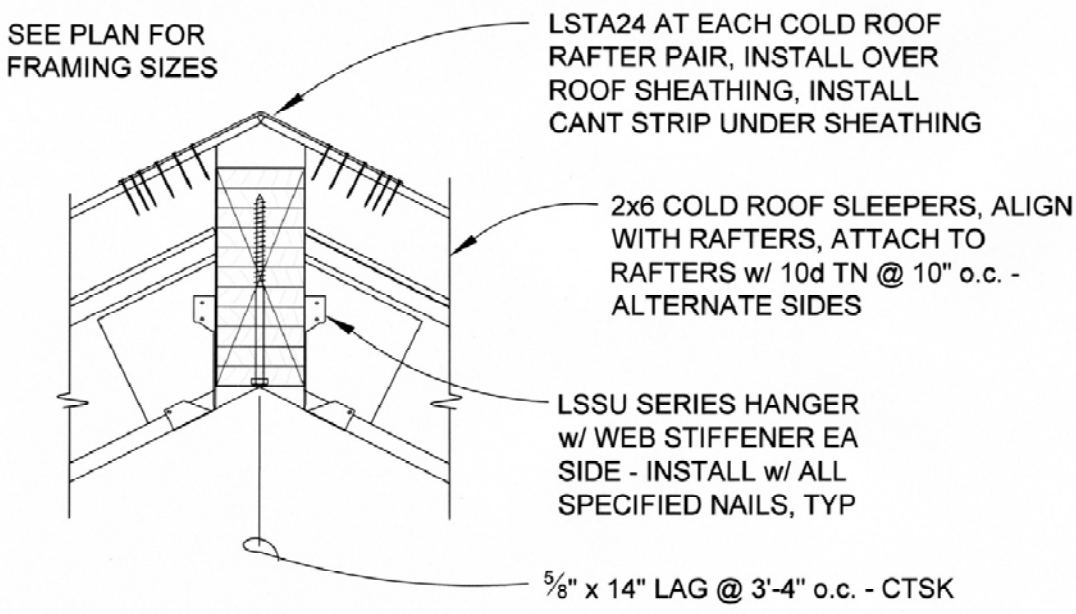

Figure 8. Connection to Raised Ridge

For this detail, the designer must also be concerned with the capacity of the ridge beam to support loads applied to the lower portion of the beam. This is not desirable detailing as loading a beam in this manner could cause splitting of the beam. Per the latest NDS, "Designs that induce tension stress perpendicular to grain shall be avoided whenever possible. When tension stress perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist all such stresses shall be considered." Therefore we must calculate the load on the ridge. The uniform snow load would control (not the unbalanced load) so the superimposed load to the ridge is 62*12 + 20*(12/Cos 26.6˚) #/ft. Self-weight of the beam is not considered in this instance. So we must mechanically reinforce the ridge beam for 1010 plf. The tabulated value for 5/8" lag bolts in withdrawal from DF is 447#/inch which, adjusted for 115% load duration, becomes 514#/inch. A 5/8 x 14" lag would have 6.59 inches of usable thread so the capacity per lag is 3,390# and one lag each 40" of beam length is required to resist the perpendicular-to-grain stress.

7.2 Connection at Exterior Wall

It is usually at the low end of the rafter that designers run into trouble. The reaction parallel to the bearing surface must be accounted for.

When using engineered lumber, the capacity of the connection should always be verified by the designer. For I-joists on low slope roofs, manufacturers may specify an 8d toe nail each side of the joist. For roof slopes in excess of 3 ½ : 12 a twist strap is required. No capacities are given for the suggested connections, however. The designer should verify the forces involved and capacity of actual nails in use for the connections.

The following two details are for illustration only. The first, Figure 9, shows an I-joist installed on a beveled plate with a twist strap. The strap is a Simpson TS22--- a 16 Gauge strap that is the longest TS strap Simpson currently has in their catalog. The allowable value of the strap in tension is 870# at 115% or precisely what is needed for our example. However, the strap allowable load is for installation with 16d common nails (0.162" x 3 ½"). The required value for our example cannot be achieved by nailing into an OSB web filler attached to a 3/8" OSB web. The designer could use a thicker web reinforcement piece, sufficiently attached to the joist web to transfer the force, and make this detail work.

The second detail, Figure 10, shows an I-joist attached to a beveled plate with four ¼" screws. The allowable load for ¼" x 3" Simpson SDS screws is 280# at a 100% duration factor installed through a nominal 2x DF or SP side member. Using a 115% load duration factor and adjusting the value for a 1 1/8" thick side member, gives 270#/screw. Four screws would develop 1070#, or more than enough to resist the force in our example. However, the spacing limits for nails installed in the flanges of some joists are 6" for common nails (0.162" diameter). Although some designers may feel comfortable specifying the fasteners shown (installed in pre-drilled holes), there may be some question as to whether the warranty on the framing product would still be valid. One may want to verify such a connection with the joist manufacturer.

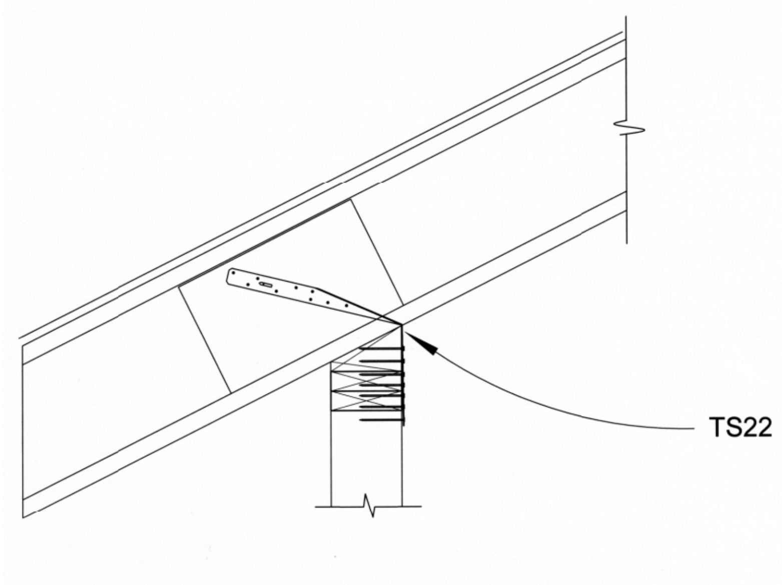

Figure 9. Sloped Bearing with TS Strap

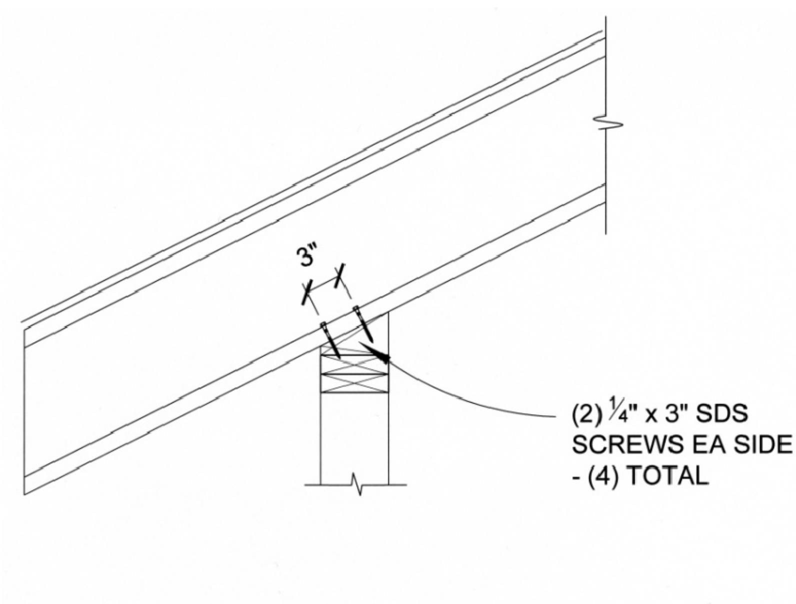

Figure 10. Sloped Bearing with SDS Screws

For our example the best support option is to use the I-joist version of a seat cut. This is referred to as a bird’s mouth cut and the basic idea is shown in Figure 11. Attachment of the joist to the bearing isn’t shown but shouldn’t be neglected.

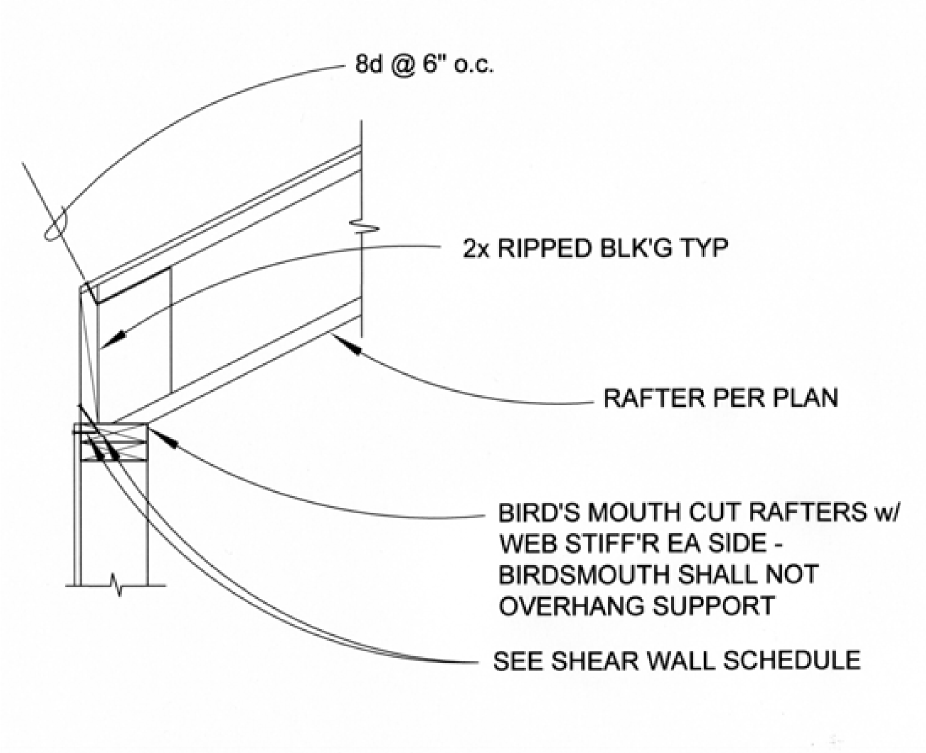

Figure 11. I-Joist with Bird's Mouth Cut

The use of a bird’s mouth cut eliminates the problem of the down-slope force but creates a problem with support of the overhang. Figure 12 was modeled on one of several typical details shown in TrusJoist’s Specifier’s Guide. The standard details should be verified for project specific loading. It is important to keep the 2x extension clear from the underside of the joist flange to prevent prying action of the 2x against the underside of the flange. Using a 2x filler on top of the extension facilitates this given the depth of the flange.

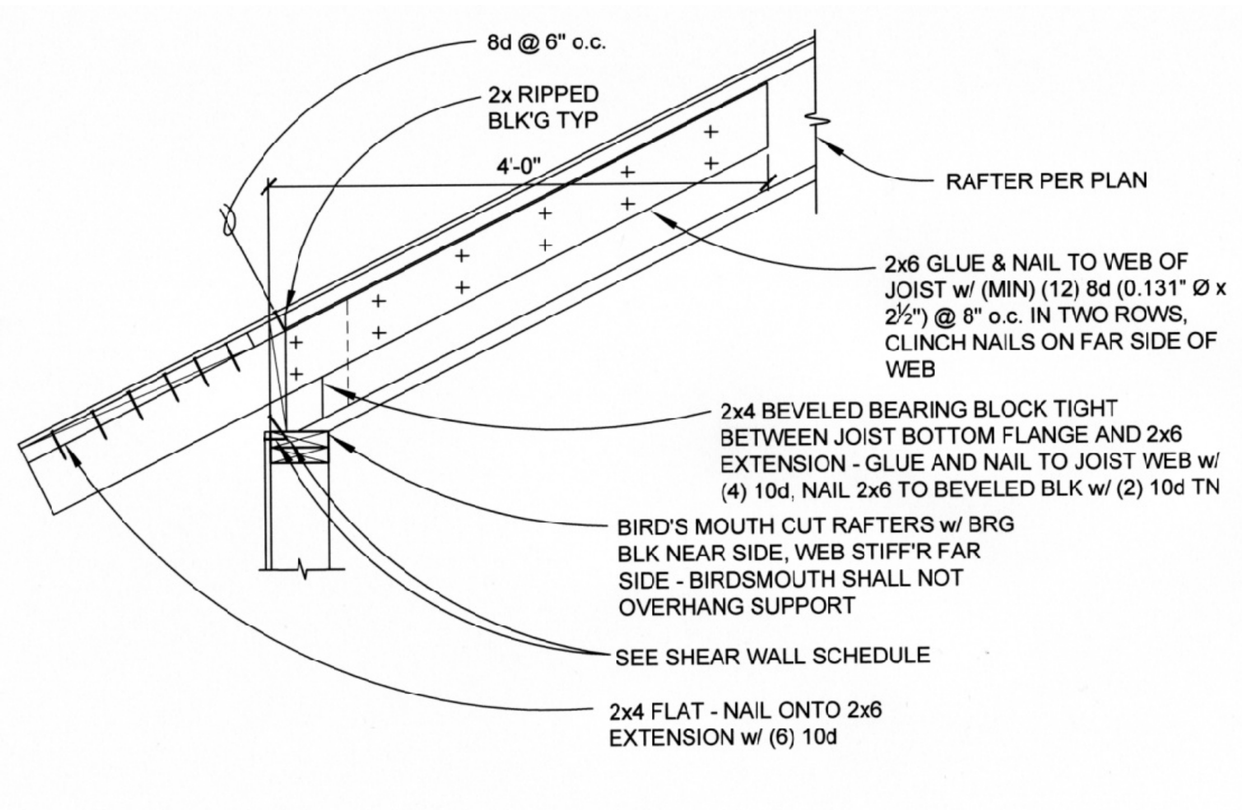

Figure 12. Bird's Mouth Cut with Sawn Lumber Overhang

Alternately, a 2x can be nailed to the joist flange with a bearing block under the 2x. This detail is also shown in manufacturers' catalogs with a sloped bearing block. Be careful of nail spacing when nailing into the edges (into the laminations) of the LVL flanges. One could modify such a detail to use a 2x extension, seat cut on a bearing plate or block.

7.2 Connection at Interior Supports

The connection at an interior support may be designed in a similar manner. Resisting the component forces of an interior support can be challenging. Using web reinforcement thick enough to provide adequate nail values for a twist strap is one solution. Another is to break the member over the support. This can be accomplished with hangers each side of a support or with a bird's mouth cut for the upper rafter and a hanger for the lower side as shown in Figure 13. The designer must check that no nails are located within the edge limits of either the beam or the continuous ledger. Otherwise the hanger capacity must be reduced.

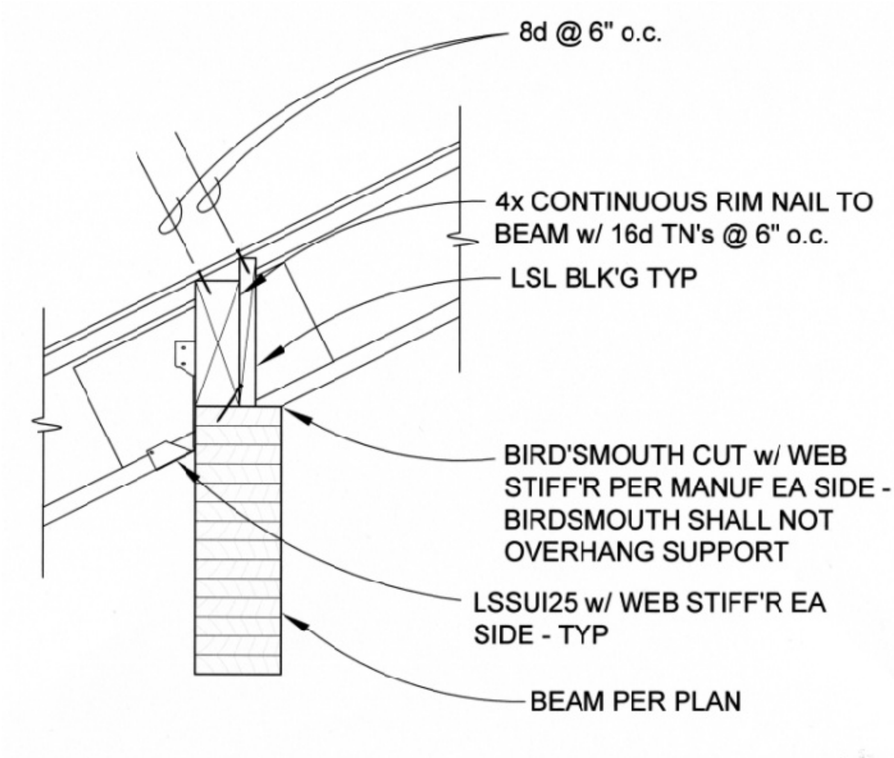

Figure 13. Bearing at Interior Support

8 SIP Connection Details

Structural Insulated Panels (SIPs), sometimes called stressed skin panels, have been developed over the last 20-years and are seeing increasing use, especially in residential construction. Early manufacturers of foam panels sold structural and non-structural panels. The structural panels had structural panel sheathing on both sides whereas the non-structural panels had sheathing on the exterior side only and were sold as combined insulation/roof sheathing system. SIPs are typically manufactured with 7/16" OSB skins bonded to a foam core of expanded polystyrene (EPS) or extruded polystyrene (XPES). Splines are used to connect the edges of panels together in the field. Splines may be dimensional sawn lumber, prefabricated wood I-joists, insulated I-sections, engineered lumber or surface splines of OSB strips. The Structural Insulated Panel Association (SIPA), a relatively new industrial association, has published an Engineered Design Guide that covers the design of the panels. That guide does not have any connection details.

As mentioned in the introduction, there are no prescriptive connections for SIP roof panels to supporting walls that this author is aware of. Connections of SIP roof panels must be designed for project specific loading conditions using accepted engineering principles. The National Association of Home Builders (NAHB) has published a prescriptive guide for connecting SIPs to concrete walls but the tabulated connections found in the guide are for resisting wind and seismic forces (uplift and laterally applied loads where the SIP screws resist the forces in tension). There may be other design guides but the issue of resisting the gravity induced reaction on a sloped bearing surface has yet to be addressed by the industry in any meaningful way, despite a plethora of details showing such an attachment.

The following was gleaned from a representative sampling of ICC Evaluation Services Reports (ESRs) on SIPs:

ESR-1138: Precision Panel Structures, Inc.

> 2.0 Uses: …The panels are alternatives to walls, floors and roofs designed in accordance with IBC Section 2306.
>
> When panels are installed under the IRC, an engineered design is required in accordance with IRC Section R301.1.3.
>
> 5.0 Conditions of Use
> 5.6: Panel connections, other than longitudinal joints at panel splines, are beyond the scope of this report and must be addressed in the design calculations and structural details.

ESR-1295: Insulspan, Inc.

2.0 Uses: … When panels are installed under the IRC, an engineered design is required in accordance with IRC Section R301.1.3.

5.0 Conditions of Use
5.5: Connection and attachment of panels are outside the scope of this report and must be addressed in the design calculation and details.

ESR-1882: Premier SIPs by Insulfoam

2.0 Uses: … The panels are alternatives to walls, floors, and roofs prescribed in IRC R502, R602, and R802. An engineered design is required in accordance with IRC Section R301.1.

5.0 Conditions of Use
5.5: Connection and attachment of panels are outside the scope of this report and must be addressed in the design calculation and details.

ESR-2139

5.0 Conditions of Use
5.3: Construction documents, including engineering calculations and drawings providing floor plans, window opening details, door opening details, and connection details, must be submitted to the code official when application is made for a permit, to verify compliance with this report and the applicable code.

ESR-1403: Temo, Inc.

4.1.2 IRC: When panels are installed under the IRC, engineered design is required in accordance with IRC Section R301.1.3.

5.4: Each structure built using Temo, Inc., structural thermal panels shall be designed by a registered design professional. Construction documents, including engineering calculations and drawings providing floor plans, window details, and connector details, shall be submitted to the code official when application is made for permit, to verify compliance with this report and the applicable code.

Recently, the structural insulated panel industry made much of the news that SIPs had been approved in the IRC and SIPA now has a link to a document on prescriptive use including connections. However, the IRC incorporated SIPs as walls only and the prescriptive connections are for wall-to-wall joints, joints within a wall, wall to foundation joints and headers.

As a design engineer you are on your own as far as specifying any roof connections to resist gravity loads. This course will cover some basic concepts and, hopefully, give some meaningful direction on how to proceed.

8.1 Use Supporting Timber Frame Members

At least two SIP manufacturers are marketing combined systems with timber frames as the gravity supporting system and SIPs as insulation and structural sheathing. This is similar to the early use of non-structural panels except the supports can be further apart. Although the systems being marketed are proprietary, design engineers are free to use similar schemes with conventional timber frame components. Connection details would be similar to timber framing details one might use with conventional sheathing and insulation.

8.2 Ridge Support Detail

Figure 14 shows a typical detail at the ridge that may be found in manufacturers' data. The long screw is a SIP screw that may be proprietary or generic. Typical diameter of the screws is 0.19" which corresponds to a No. 10 screw. Per the latest NDS, the tabulated value for a #10 screw in DF with a ½" side plate is 90#/screw. Assuming one could use dowel bearing on both pieces of exterior sheathing, ignoring the difference between ½" and the actual OSB thickness and adjusting for 115% duration, the screw allowable is 207#. So the screws might work if spaced at 4" o.c. However, tabulated data on screw capacity from manufacturers' is scarce. A discussion on using the dowel bearing strength of both faces of OSB would be outside the scope of this paper but, given the yield modes used to derive the yield limit equations for dowel type fasteners in the NDS, using twice the single sheathing capacity for these screws questionable. It might be prudent for design engineers to review Appendix I of the NDS prior to certifying such a connection.

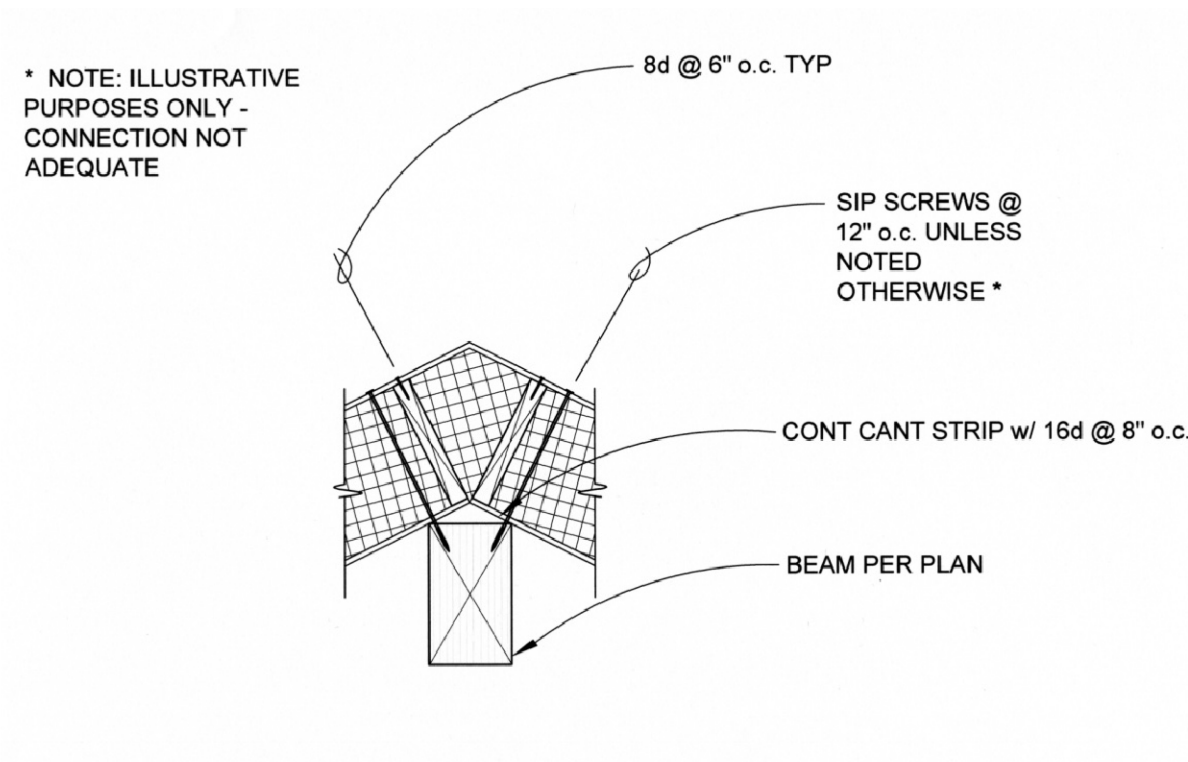

Figure 14. SIP Manufacturer's Detail At Ridge

Figure 15 shows how one might use the panel splines for the installation of a tension tie across a ridge. Using the same loads as our I-Joist example with 4-ft wide panels, the tensile force to be resisted would be double that for the I-joist example or 1080#. This would preclude use of a surface spline but should work well with solid wood splines. In Figure 14 a Simpson CS18 strap was selected. CS straps are coil straps field cut to length. Per the NDS, given an 18 gauge side plate with 115% load duration, 10d common nails are good for 130# each, so a minimum of (9) nails are required for the connection. The SIP screws are used to resist wind uplift.

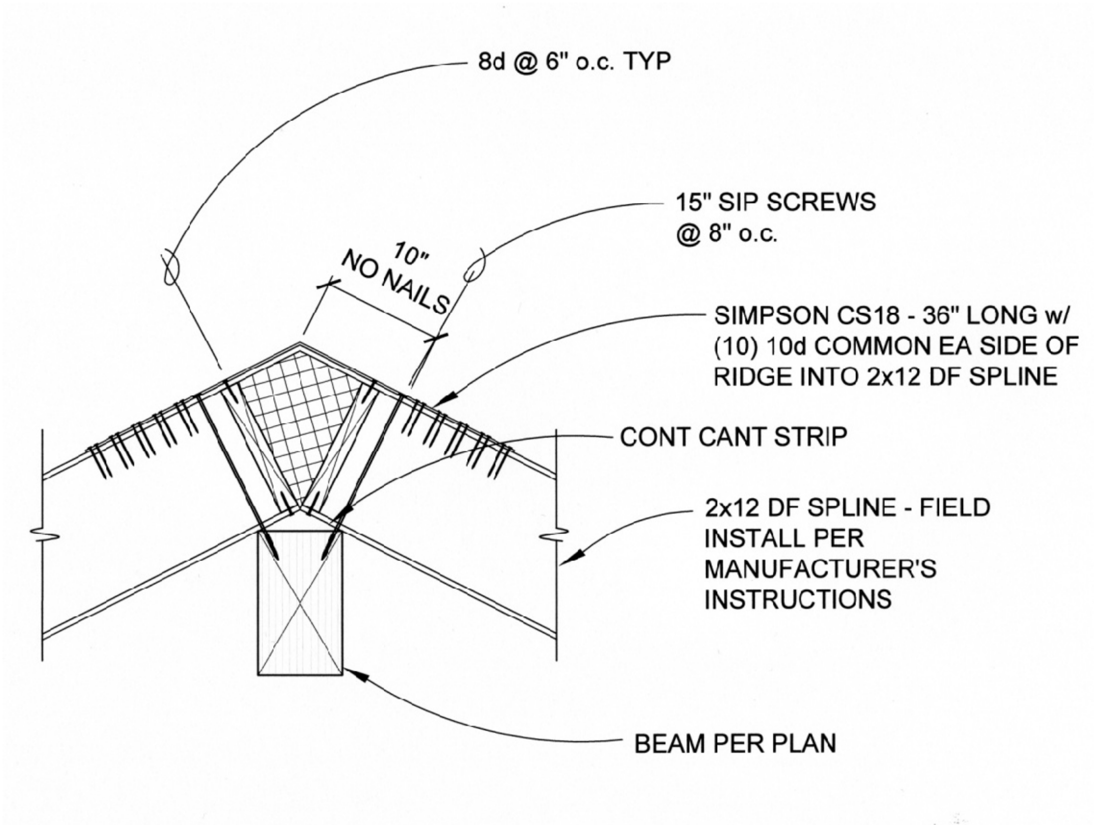

Figure 15. Improved Ridge Detail

8.3 Exterior Wall Detail

Figure 16 shows another typical industry detail. Figure 17 shows the author's recommended detail with a SIP version of a seat cut. Although I found a similar detail in a literature search some years ago, I was unable to find any industry recommended detail with this type of configuration while preparing this report.

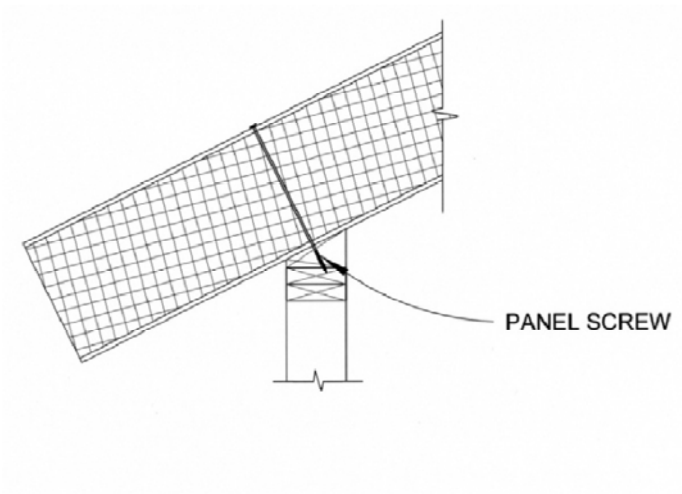

Figure 16. Industry Detail at Eave

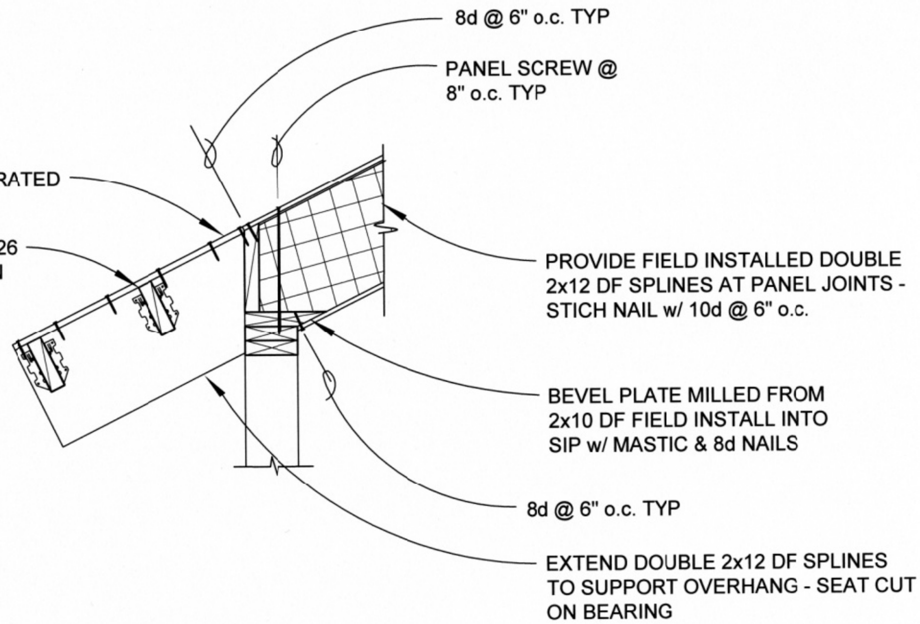

Figure 17. Improved Eave Detail

The detail of Figure 17 is presented only as an illustration to prompt thought and discussion, not as a finished detail.

8.4 A Word about Field Applied Glue

I have observed a design professional claim that the adhesive specified in manufacturers' instructions for SIP installations, constituted the structural fastener. There are many approved adhesives in the construction industry but, as of this writing, structural adhesives for wood-to-wood connections have only been approved for use in factory controlled conditions. One problem with wood-to-wood glues is that the moisture content of the wood can affect the performance of the glue. Other conditions that may be difficult to control in the field may also affect glue strength.

Traditionally, going back to the Uniform Building Code (UBC) days, glued nailed floor systems were allowed to be considered in stiffness calculations but not in strength calculations. The design engineer responsible for the performance of a structure may want to exercise extra care in any detail or specification relying on field applied adhesives.

9 Summary

In summary, prefabricated wood I-joists have revolutionized the construction industry. They have been used for floor systems for 30 years. Their use in roof framing lagged behind their use in floor framing due, in part, to unfamiliarity of required detailing by designers and carpenters. The use of I-joist framing requires some special attention to connection details.

The use of basic engineering calculations along with published data on fasteners can produce safe, verifiable connections for prefabricated wood I-joists in almost any loading condition.

Structural Insulated Panels are seeing more use. The positive insulation values offset the added expense of the panels, especially for residential construction. In many instances, however, the connection details shown for the panels are not supported by engineering calculations and may not be suitable for the imposed loads. The SIP industry is young and the construction industry is still in a learning curve when it comes to the use of SIPs. Since verifiable, industry supplied details are lacking, the design engineer must produce calculations on proposed connections for submittal to code officials whenever an application for permit is made that involves SIPs. Engineers will have to rely on engineering judgment in preparing calculations and designing connections with SIPs.

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