FREQUENTLY ASKED

Framing Questions

Four engineers shed light on common framing mysteries, myths & misconceptions

When to Double Rafters

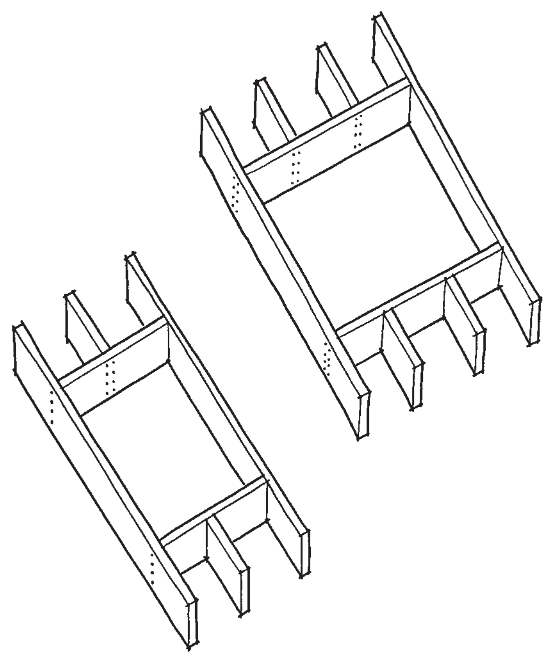
Q. *When you remove one rafter to install a skylight, do you have to double the two rafters at the sides of the opening? What about the headers?*

A. When you're framing a skylight that requires you to remove no more than two rafters, a common rule of thumb says to double the rafters on either side of the opening and use double headers. This is often required by building inspectors, but in fact, it's a very conservative guideline that often results in unnecessary framing. Only in rare cases are the doubled headers required. And in many cases, particularly when 2x10 or 2x12 rafters are chosen for insulation thickness rather than for strength, doubled rafters may not be required.

In my experience, the inspectors also often insist on joist hangers at the headers, but these are also rarely needed. Usually, an adequate number of 16d nails, as many as 8 nails for a 2x12 connection, depending on the loads) can handle the reaction forces.

Watch out, though: As the opening size increases to the point where you're removing three or more rafters, even doubling the perimeter framing may not be sufficient.

—R.R.



If deep rafters have been selected to accommodate insulation requirements rather than structural requirements, it may be unnecessary to double trimmer rafters. It would rarely be necessary to double the opening headers.

Thanks to the structural engineers who contributed to this article:
***Christopher DeBlois**, P.E., Palmer Engineering, Tucker, Ga.*
***Scott McVicker**, S.E., McVicker & Associates, Palo Alto, Calif.*
***Robert Randall**, P.E., Randall Engineering, Mohegan Lake, N.Y.*
***Frank Woeste**, P.E., Professor of Wood Construction, Virginia Tech, Blacksburg, Va.*

Plywood in Built-Up Headers

Q. *Do the layers of plywood in typical built-up headers add significant strength to the header?*

A. The most important thing the plywood adds is thickness. Of course, the plywood does add some strength, but for several reasons engineers almost never count on this strength in their designs.

Only the layers of plywood with the grain oriented horizontally (parallel with the direction of the header) are really adding any strength. A quick look at the thicknesses involved shows that the additional strength is small. If half the layers in $1/2$-inch plywood are horizontal, that's $1/4$ inch of extra material. Compared with 3 inches of 2x10, that's an increase of only 8%. What's more, you only get the full effect of this extra thickness if there are no splices in the plywood near the middle of the span, or better yet, no splices at all. For headers at openings wider than 8 feet, that's not often the case. But it's these longer headers that will most likely need some extra strength.

Combine these drawbacks with size limitations and the plywood almost never makes a critical difference in safety. What I mean by size limitations is that when I design a header, the numbers may tell me I need two 2x9s. Since two 2x9s are about 30% stronger than two 2x8s, the 2x8s plus 8% from $1/2$ inch of plywood wouldn't be strong enough. And I wouldn't ask the framer to rip some 2x9s, I'd simply call for 2x10s. What's more, he'll probably use double 2x10s for all his headers, big and small. Because headers only come in certain depths, there's usually extra strength in the 2x10s to begin with. And that extra strength in the 2x10s means that the small extra strength from the plywood is rarely important. But the thickness is helpful.

—C.D.

Strength of Toe-Nails vs. End-Nails

Q. *What is the strength of toe-nails compared with end-nails when nailing studs or joists?*

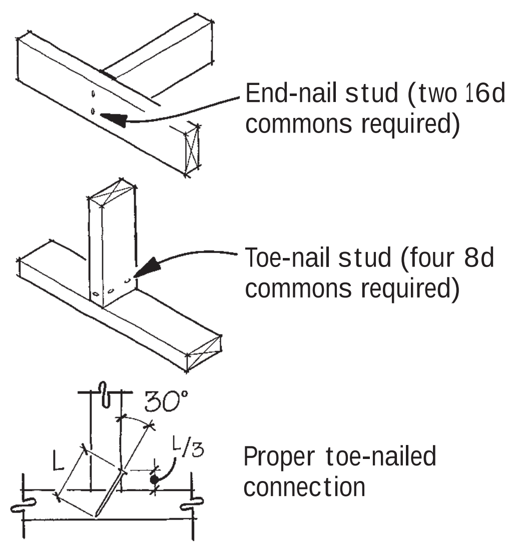
A. The answer to this question is found in the *National Design Specification for Wood Construction (NDS),* published by the American Forest and Paper Association (800/890-7732). The *NDS* gives the design capacities for nails loaded laterally and in withdrawal. The nail-strength tables are further categorized by wood species and the size of the members being joined. The nail capacities given in the tables assume that the two members are being joined side by side, the way you nail overlapping joists to one another above a girder or bearing wall (single shear connections). Toe-nailed and end-nailed applications require strength reductions.

For toe-nails loaded laterally (for example, the nail loading that results from wind pressure on a stud wall), the reduction factor is 0.83. For toe-nails loaded in withdrawal (for example, uplift on a stud wall due to wind suction on the roof), the reduction factor is 0.67.

For laterally loaded end-nails, the reduction factor is 0.67 (called the "end grain factor"). The *NDS* (widely used as the basis for code requirements) doesn't allow nails driven into end grain to be loaded in withdrawal.

As you can see, a correctly installed toe-nail (see illustration) is stronger than an end nail of the same size. Of course most carpenters use smaller nails when toe-nailing to avoid splitting, so this also has to be taken into account. For example, when attaching a 2x4 stud to the sole plate, the BOCA code prescribes two 16d nails driven through the plate into the stud. If toe-nailing the stud in place, the code prescribes four 8d commons. In this case, the four toe-nails would be stronger in lateral loading than the two end-nails. Get a copy of the 1997 *NDS* for specifics.

—F.W



When nailing a stud to a plate, a toe-nailed connection is typically stronger against lateral forces than an end-nailed connection.

Deflection of Plywood vs. OSB

Q. *Does OSB sag more than plywood when installed horizontally over 24-inch-center rafters?*

A. It depends on the materials used to make the OSB, which can be manufactured from a variety of species. These include aspen, southern pine, sweet-gum, yellow poplar, and birch. The Modulus of Elasticity (MOE) of the wood used will determine the relative flexibility of an OSB panel. The list at right shows the MOE values for some of the woods used to make OSB and plywood (from *NDS Supplement)*.

On the same roof with rafters spaced at 24-inch centers, a plywood panel made from high-grade Douglas Fir-Larch veneers is going to deflect less than an OSB panel made from Aspen. Run the same test using an OSB panel made from similar materials and you will most likely find no difference in their deflections.

Wood Species	MOE
Aspen	800,000 to 1,100,000
Yellow Poplar	1,100,000 to 1,500,000
Beech-Birch-Hickory	1,200,000 to 1,700,000
Douglas Fir-Larch	1,300,000 to 1,900,000
Southern Pine	1,200,000 to 1,900,000

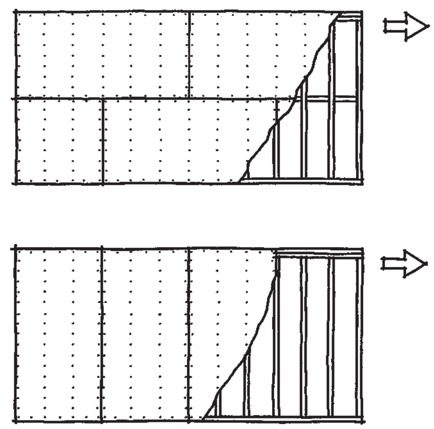
Probably the reason that OSB has the reputation for flexing more than plywood is that much of the OSB sold is manufactured from lower grade fibers. This is why OSB typically costs a lot less than good plywood.

—S.M.

Horizontal vs. Vertical Sheathing

Q. *I've always heard that installing plywood sheathing horizontally (perpendicular to the direction of the studs), with joints staggered, is stronger than installing it vertically. True? Is this true of roof sheathing too?*

A. This is true for wall sheathing in many instances, but not for roof sheathing. To understand why, we need to look at how the grain of the plies is oriented relative to the direction of the applied force. Each layer of wood in plywood is oriented either parallel or perpendicular to the long direction of the sheet. Most of the shear force is resisted by those plies whose grain runs parallel to the direction of the applied force. So for 3-ply plywood, for instance, which has two face plies running parallel with the long dimension of the sheet, and a single central ply running perpendicular, most of the wood fibers are oriented parallel to the length of the sheet, so that is the plywood's stronger direction.



When all the panel edges are supported by solid framing, 1/2-inch plywood sheathing is stronger against racking forces when installed horizontally (top sketch). If there's no blocking at the 4-foot mark, the sheathing is stronger installed vertically, as long as all edges are supported (bottom sketch).

This fact is reflected in the *Uniform Building Code's* nailing schedule for structural panel shear walls (1997 *UBC*, Table 23-II-I-1), which permits the allowable shear for $^{3}/_{8}$-inch and $^{7}/_{16}$-inch panels, if oriented horizontally across the wall studs, to be increased to that of corresponding $^{15}/_{32}$-inch panels. As plywood gets thicker, this rule is less important because the overall percentage of fibers running parallel with the long dimension decreases as the number of plies increases.

Note that the UBC table applies only to *fully blocked* shear walls; in other words, all the plywood edges have to be supported by a minimum of 2-by framing. Regardless of plywood orientation, a plywood panel fully supported at all edges is always stronger than a panel with some edges unsupported (see "The Strength of Plywood Sheathing," *Practical Engineering*, 11/96).

So far we've talked only about wall sheathing, which mainly resists lateral loads from high wind or earthquakes. Roof sheathing is another matter, since roofs experience forces applied both parallel and perpendicular to the long direction of the plywood. We could, in theory, credit a plywood panel installed perpendicular to the rafters with the higher shear force in that one direction, but we would be forced to accept the basic code value in the opposite direction. In such a case, the designer generally assigns the lower shear value to the plywood in both directions. If a greater shear value is needed, the designer may specify increased nailing or thicker plywood.

—S.M.

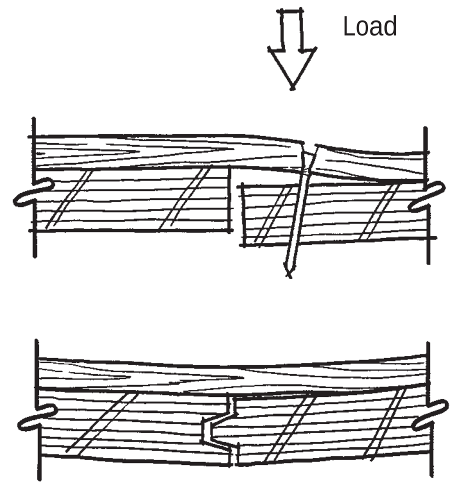
Purpose of T&G Plywood

Q. *Does tongue-and-groove plywood add extra strength or stiffness to a floor system, or does it just help prevent floor squeaks?*

A. The tongue-and-groove joint doesn't add strength, but it does help to distribute loads to adjacent panels, improving the perceived stiffness of the floor. T&G plywood was developed as a labor-saving alternative to installing solid wood blocking at unsupported panel edges.

Without the tongue and groove, a load on one panel edge causes that panel to deflect relative to the adjacent sheet. A wood floor that spans across the joint would experience a wedging action, causing a floor squeak. Tongue-and-groove plywood is actually more effective than solid blocking at preventing squeaks, because over time the blocking will shrink, leaving unsupported edges.

—S.M.



Unsupported joints in square-edged plywood floor sheathing cause squeaks when someone steps on them. T&G plywood sheathing prevents squeaks and makes the floor feel stiffer, though it doesn't actually increase the design strength of the floor system.

Nailing Patterns for Built-Up Beams

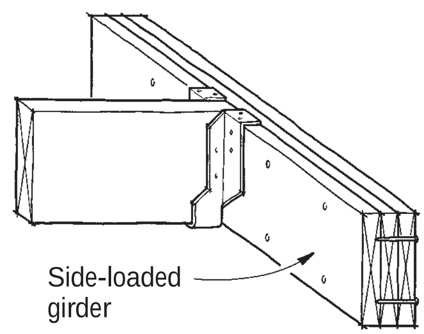
Q. *What's the best nailing pattern for built-up beams?*

A. The critical issue with built-up beams is that all the layers must deflect together and by the same distance in order to be properly sharing the load. For beams where the load comes down evenly on top of the beam, such as drop beams or beams directly under bearing walls, the nailing pattern is not all that critical. All you need are enough nails to hold the layers together and keep them from twisting. For beams loaded from the side, however, and especially for beams loaded from one side only, the nailing pattern is critical.

When beams are loaded from the side, there must be enough nails to transfer the load through the loaded member and into the attached members. For example, if a beam consists of three 2x10s loaded from one side only, the loaded member should only carry $^{1}/_{3}$ of the weight. To transfer the rest of the load into the attached members there must be enough nails from the loaded 2x10 into the center 2x10 to transfer $^{2}/_{3}$ of the load, and enough nails from the far side 2x10 into the center 2x10 to transfer the final $^{1}/_{3}$ of the load into that outer member. These numbers assume that all three 2x10s rest fully on the supports; the situation gets more complicated when the members are not all the same size or material. The bottom line, though, is that if all the pieces deflect together and equally, the beam should perform as designed.

At a minimum, I recommend pairs of 16d nails every 12 inches along the beam, with the top row of nails $1^{1}/_{2}$ inches or so from the top of the beam, and the bottom row $1^{1}/_{2}$ inches or so up from the bottom. Use the same nailing pattern on both sides for triple beams, and check with an engineer whenever you think the loads involved might be unusually heavy.

—C.D.



Sideloaded beams must be carefully nailed together, to ensure that all the beam members share the load.

Safety Factor in Wood Construction

Q. *Doesn't the safety factor in wood construction mean that most wood structures are way overbuilt?*

A. The idea that "wood structures are way overbuilt" may be the greatest myth in the wood construction field. It is possible that at one time in history wood structures were overbuilt, but it is certainly not true today. The safety factor for bending strength for visually graded dimension lumber is 1.3; by contrast, the safety factor for structural steel, which has much less variability from piece to piece, is as much as 2.

So how are safety factors applied? To arrive at the design values used in wood design, thousands of pieces of lumber of representative sizes, grades, and species have been tested. These tests are run for about ten minutes to determine the stress that will cause a piece of lumber to fail. The test data for every piece of lumber of a given grade, size, and species is recorded. In a test of bending strength, for example, the values from a batch of lumber might range from 3,000 to 15,000 psi. By convention, the value of the 5th percentile is calculated (in other words, 95% of the pieces tested fall above this number, 5% fall below). Choosing a value at the 5th percentile is a way of accounting for the wide variability in the strength of pieces of visually graded lumber (due to knots, slope of grain, etc.).

This number — let's say it's 4,000 psi — is then divided by 1.62 to convert it to a ten-year duration value, which is the load duration that is used in the design of wood floor systems. (Remember, the test lasts only ten minutes; lumber can resist more stress for short periods of time.) Finally, the ten-year value is divided by a safety factor of 1.3. So a 5th percentile value of 4,000 psi would become 1,899 psi. This is the number that is published in the allowable design stress tables.

It's a grave mistake to make design decisions based on an assumption that the wood safety factor is excessive.

— F.W.

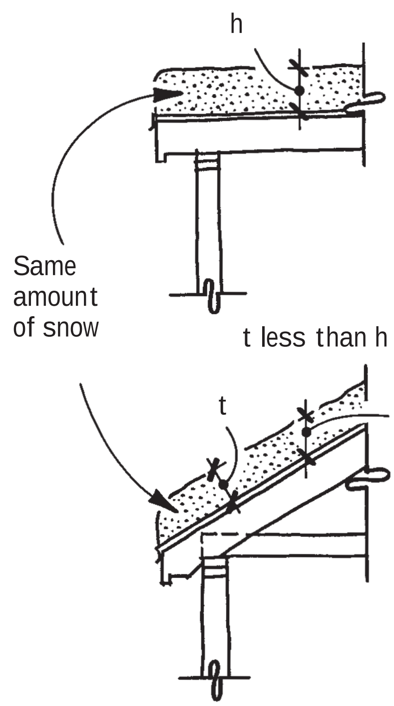
Calculating Loads on Sloped Roofs

Q. *I often see roof loads calculated based on the horizontal run of the roof. But isn't it more accurate to figure the weight of snow and roofing materials by measuring along the actual length of the rafter? Thus, as the roof gets steeper, the rafter gets longer, and the weight of roofing materials increases.*

A. Actually, with snow, you don't get more load along the slope of the rafter; you get the same load as a flat roof with the same run would get. This is because as snow falls vertically it spreads itself further along a sloping rafter and so accumulates less depth. The BOCA code recognizes this and allows you to use the horizontal projection of the roof when calculating snow loads. BOCA also allows you to reduce the snow load for roofs with slopes greater than 30 degrees, presumably because snow will slide or blow off steeply pitched roofs.

For dead loads, you are correct. Technically you should use the actual rafter length when adding up the weight of roofing materials. However, in my practice, I typically use the horizontal run of the roof for both types of load. To do this, I use conservative (too heavy) dead loads and full snow loads regardless of pitch. I ignore the slope factor altogether for snow load reduction which adds another measure of conservatism. (Slope length cannot be ignored for wind load analysis, though.)

—R.R.



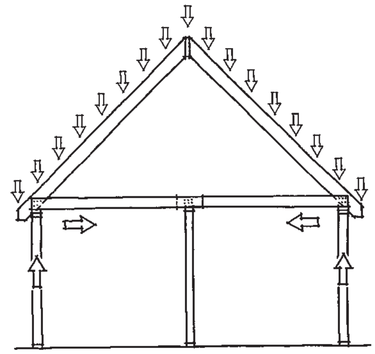
When adding up the weight of snow of a sloping roof, use the horizontal run of the roof, since the same amount of snow would accumulate on a perfectly flat roof. The code also allows you to apply a slope reduction factor, to account for snow blowing or sliding off a sloping surface.

When Are Collar Ties Needed?

Q. *Collar ties don't seem necessary in attics where the rafters come all the way down to the ceiling joists. Can you remove some of them to create headroom?*

A. The most common reason for installing collar ties is to prevent rafters from spreading apart under load. However, in a conventionally framed peaked roof, like the kind you describe, collar ties would probably serve little or no function, since the attic floor joists serve as ties to prevent the rafters from spreading. Note that the connections between the rafters and the joists must be adequate, and that the overlapping joists at midspan must also be properly nailed (see *Practical Engineering,* 5/96).

There are some exceptions, however, when collar ties might be useful even in a conventional attic roof. For example, very long rafters in a relatively steeply pitched roof (slopes above 6/12, for instance) may benefit from a stabilizing effect if *adequately connected* collar ties are installed on every rafter pair. In this case, the collars serve not as ties but as spreaders. Also, in high wind situations with lower pitched roofs, collar ties may help hold the ridge assembly together, although steel strap ties installed just below the ridge board would probably work better.



Collar ties are usually not needed in conventional gable roof attics, as long as the floor/ceiling joists are properly connected to handle the tension forces created by the outward thrust of the roof.

My call is that in the vast majority of such cases, collar ties can be removed with no detrimental effect. In most of the cases I have observed, the existing connections between the collar ties and the rafters are inadequate to provide any meaningful beneficial effect anyway.

—R.R.

Shear Strength of Gypboard

Q. *I've heard that engineers give no structural "credit" to gypboard, but I know it greatly stiffens partitions when I nail it up. How much shear strength does drywall really have, and why not credit it in the design?*

A. As you suspect, properly fastened gypboard does have the capacity to resist racking and/or lateral forces. The 1997 Uniform Building Code (Table 25-1) gives shear values for both gypsum wallboard and gypsum sheathing. In fact, the allowable lateral force on a wall with fully blocked 5/8-inch gypboard on both sides nailed at 4-inch centers (350 plf) actually exceeds that of a wall with 1/2-inch Structural I plywood fastened with 10-penny nails at 6-inch centers (340 plf). Be careful, though: If you are working in seismic Zones 3 or 4, note that even with fully blocked edges you must reduce the allowable lateral load on gypboard by 50%.

As to crediting the design for the strength of the gypboard, this decision is based on the materials selected for the particular structure. If you build a house with rigid-foam insulation panels on the exterior (under finishes) and gypboard on the interior, then the gypboard *is* the lateral force-resisting material. However, if the interior gypboard is combined with plywood sheathing on the exterior (or with diagonally braced structural steel studs), then the strength of the gypboard is discounted. In the latter case, the plywood is considered the primary lateral-force-resisting material because of its greater strength and stiffness. In both instances, the designer must make certain that the primary lateral-force-resisting material is sufficiently fastened to the framing to resist the *total* lateral load despite the presence of other secondary materials.

In reality, it is the combination of all the primary and secondary materials that will resist the applied lateral loads. However, should the loading persist, the repetitive cycles of load/release will cause fatigue of the weaker materials (like gypboard) until essentially only the primary lateral material remains functional. If we were to credit the strength of the gypboard towards the total lateral load (and reduce the plywood nailing accordingly), our structure would lack critical capacity after the time when the gypboard had yielded. This is the reason gypboard receives no credit for its strength.

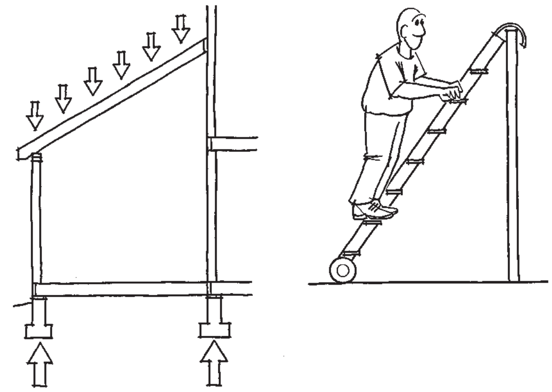
—S.M.

Rafter Thrust in a Shed Roof

Q. *Does a cathedral shed roof addition need collar ties to restrain the outward thrust of the rafters?*

A. A shed roof with a proper shear connection at the ridge *has no lateral thrust*. Think of a ladder leaning against a building. Imagine the ladder has wheels at the bottom. With no restraint at the top, the ladder will roll away from the building. Attach the ladder to the building at the top, however, and it stays put. A shed roof is similar; as long as it's properly attached at the top, the bottom can't move. "Collar ties" are an exercise in futility. Use them as ceiling joists if needed; otherwise, leave them out.

—R.R.



Collar ties are not necessary in a cathedral shed roof, since there's no outward thrust to restrain. The situation is analagous to a ladder leaning against a wall, and attached at the top. Even if it's on wheels, the ladder can't move away from the wall.

Strength of PT Lumber

Q. *Does CCA pressure treatment adversely affect the strength or durability of framing lumber?*

A. According to the *National Design Specification for Wood Construction* (1997), pressure preservative treatment with CCA (chromated copper arsenate) does not affect the strength of lumber except in the case of impact loads (loads that last about a second). Fortunately, impact loads are not typical in residential construction.

The bigger concern with PT lumber is that in use it is typically exposed to the weather. Thus, its design strength is subject to a "wet service" reduction factor, and thus most fasteners have weaker values. Therefore, most lumber properties are lower and most connections are weaker. One way to avoid a moisture penalty for connections in PT lumber is to use threaded hardened-steel nails, which have been shown in testing to have full rated strength even in wet lumber. When working with pressure-treated lumber, choose fasteners that resist corrosion from CCA, such as hot-dipped galvanized. *— F.W.*

Using Rafter Purlins

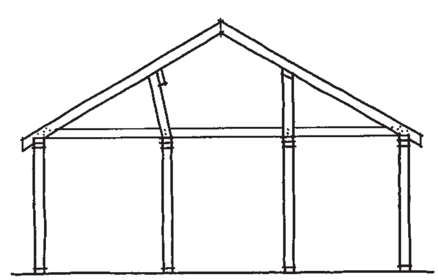
Q. *Does using purlins and struts at midspan allow you to cut the roof span in half compared with what's given in a rafter table?*

A. Yes. Actually, in theory, a rafter with purlin support at midspan could be a little *longer* than twice the maximum allowable single span length. This is due to the effect of *moment continuity* across the support. This means that the roof load on one side of the purlin has a slight lifting effect on the other side of the purlin.

When using purlins, you must be careful that the struts are properly supported. Always carefully trace the load path down to the ground, verifying the adequacy of each element.

To be most effective, struts should be installed as close to vertical as possible so as not to create lateral forces that have to be dealt with. This will depend, of course, on the location of the bearing wall below. And keep in mind that when the struts get longer than 6 feet, they may require lateral bracing.

—R.R.



Purlins and struts reduce rafter spans just as bearing walls do. The struts, or a kneewall, should be installed as close to vertical as possible, and must be properly supported below.

Weight of Steel vs. Lumber Beams

Q. *For the same loads, which is heavier, structural steel beams or lumber beams?*

A. When depth is not a restriction, it is almost always possible to design a steel I-beam that is lighter than the lightest structurally acceptable wood beam design, including glulams, LVL, and Parallam beams. And no matter how hard you try, solid timbers, built-up 2x beams, and flitch beams are almost always heavier than the lightest steel I-beam option — usually a lot heavier. Yes, it's true that steel as a material is heavier than wood given two chunks of the same size. That's because the density of steel is 12 times or so higher than the density of Southern Yellow Pine, for example. One cubic foot of steel weighs about 490 pounds, while the same size chunk of kiln-dried SYP wouldn't top 40 pounds. But because the steel can be formed into very efficient shapes, like I beams, the overall weight of a steel beam is often lower than the lightest wood option.

In some cases, steel may be the only type of beam that will solve a problem. A good example is that common remodeling problem of removing a loadbearing wall without having the new support beam project below the ceiling. For long spans in a 2x10 floor, you can't get enough stiffness from 9 1/4-inch LVLs or 9-inch glulams, but 8-inch steel I-beams come in a variety of widths and weights to handle almost any situation like this. In such a case, the framer may complain that the steel beam is very heavy, but it's not heavier than the alternatives when there are none. There are also times when steel is ideal not because it can hold up a lot of weight, but because it can be welded into rigid frames. The modern two-story window wall leaves little room for plywood shear panels, but in high wind and seismic areas you can't ignore the potential for racking that accompanies these lateral loads. A stiff moment frame of steel tubes or I-beams can often solve this problem when wood just won't do the job.

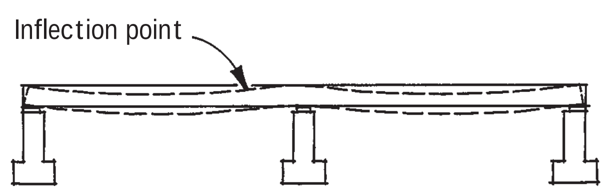
—C.D.

Splices in Built-Up Beams

Q. *Is it necessary to place splices in built-up lumber beams directly above the support posts?*

A. The easy answer to this question is Yes, but it's not entirely true. What is true is that you can't run into trouble locating all splices directly over support posts.

In reality, the most efficient location for splices is at points of inflection. The sketch at right shows the expected deflection of a uniformaly loaded beam without any splices spanning from wall to wall across a center post. Note how the beam sags near the centers of the spans, while the deflection curve turns upward over the post. The points where the curvature of the beam transitions from concave down over the post to concave up between the posts are the inflection points. At those points, stresses in the wood due to bending are lowest — in fact, they are zero. Unfortunately, shear stresses won't be zero at these points, so if you spliced all the members of a built-up lumber beam at inflection points, you would still need some type of steel or wood shear plates nailed or bolted across the splice to transfer the loads from one section of beam to the next. That's a trick that's common in commercial steel construction, but that becomes a pain for wood framing.



Bending stresses disappear at a beam's inflection points, making this a good place for splices in a built-up beam, as long as metal shear plates are used to handle shear stresses. But because it's difficult to figure out exactly where the inflection points are, it's always a safe bet to place splices directly over posts.

A second problem is that wood beams aren't flexible enough to see the shape of the curvature and reveal the inflection points; their locations must be calculated. Since the location of each inflection point depends on the relative length of adjacent spans, the number of spans, and the variations in load along the beam, there is no easy rule of thumb for locating the inflection points and hence the best location for splices.

So my suggestion is to take the safe route and set all your splices in multiple span built-up beams directly over the posts.

—C.D