$ave More Money

BY CHARLES J. CARTER, P.E., S.E., AND THOMAS J. SCHLAFLY

Smart design and detailing can add up to big savings in the total cost of fabricated structural steel.

WHEN A STEEL FABRICATOR prepares a cost estimate for a typical project, the following steps are common:

- ✓ Perform a detailed material and labor takeoff.
- ✓ Weigh and price all materials, including waste materials, for which payment is based upon weight, such as structural shapes, plates, and bolting products.
- ✓ Add the cost of supplemental materials for which payment is not based upon weight, such as welding and painting products.
- ✓ Estimate the labor hours required to fabricate the project and calculate the cost, including overhead.
- ✓ Add the cost of all outside services required, such as prefabrication materials preparation, galvanizing, shipping, and erection.
- ✓ Add the cost of shop drawings.
- ✓ Add the cost of buyout items such as steel deck and steel joists.
- ✓ Evaluate the risk and need for contingency, bonding, and insurance requirements and add the appropriate amount.
- ✓ Factor in schedule requirements and add the appropriate amount.
- ✓ Determine the profit required and add the appropriate amount.

All of the components of the total cost identified in the foregoing estimating process can be classified into one of four categories:

Material costs: This category includes the structural shapes, plates, steel joists, steel deck, bolting products, welding products, painting products, and any other products that must be purchased and incorporated into the work. It also includes the waste materials, such as short lengths of beams (called "drops") that result when beams are cut to the specified length. By an order of magnitude, the most influential component of these products on the total material cost of a building structure is the weight of the structural shapes. Also of impact is how much material can be purchased in mill-order quantities directly from a mill and how much must be purchased in smaller quantities through a steel service center.

As illustrated in the chart (next page), the typical material cost has rebounded in recent years from its low of 20% of the total cost in 1998. Nonetheless, the current percentage remains one-third lower than 25 years ago.

Fabrication labor costs: This category includes the detailing and fabrication labor required to prepare and assemble the shop assemblies of structural shapes, plates, bolts, welds and other materials and products for shipment and subsequent erection in the field. It also includes the labor associated with shop painting. The total fabrication labor cost is simply the cost of the detailing and shop time required to prepare and assemble these components, including overhead and profit.

The typical fabrication labor cost has increased slightly in recent years from 30% of the total cost in 1983 to 33% in 2008. This represents a 10% increase in fabrication labor costs over the last 25 years.

Erection labor costs: This category includes the erection labor required to unload, lift, place and connect the components of the structural steel frame. The total erection labor cost is simply the cost of the field time required to assemble the structure, including overhead and profit.

The typical erection labor cost has increased in recent years from 19% of the total cost in 1983 to 27% in 2008. This represents a 42% increase in erection labor costs over the last 25 years.

Other costs: This catch-all category includes all cost items not specifically included in the three foregoing categories: outside services other than erection, the additional costs associated with risk, the need for contingency, and the schedule requirements of the project.

The typical cost in this category has increased slightly in recent years from 11% of the total cost in 1983 to 13% in 2008. This represents an 18% increase in other costs over the last 25 years.

Obviously, very few projects, designers, fabricators, and erectors are exactly alike. Given this, the exact distribution of the total cost among

This article was updated from a previous version written by Carter, along with Thomas M. Murray, P.E., Ph.D., and William A. Thornton, P.E., Ph.D. in April 2000.

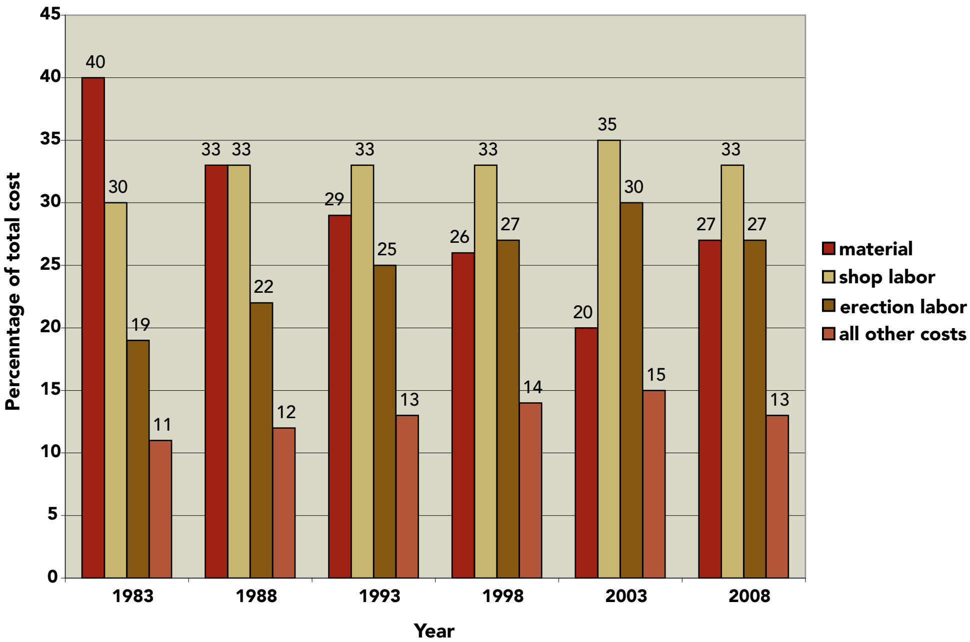
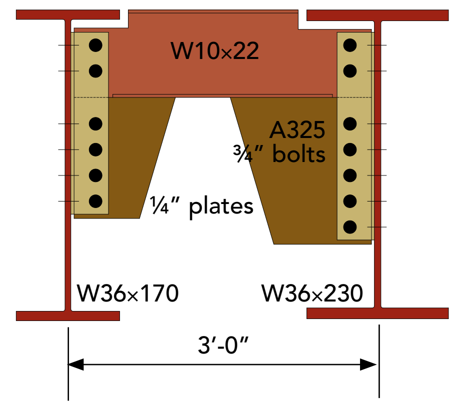


Figure 1. Material, shop labor, erection labor, and other costs—1983 through 2008.



these four categories can and will vary based upon the specific characteristics of a given project, including the design and construction team. In some specialized cases, any one of the four cost centers may dominate the total cost. Nonetheless, it can be stated that the current distribution of cost, rounded to the nearest 5% increment, among these four centers for a typical structural steel building is approximately as follows:

Material costs..................................... 25%
Fabrication and erection labor costs .. 60%
Other costs... 15%

Cost Conclusion: Thus, in today's market, labor in the form of fabrication and erection operations typically accounts for approximately 60% of the total constructed cost. In contrast, material costs only account for approximately one quarter of the total constructed cost. Clearly then, least weight does not mean least cost. Instead, project economy is maximized when the design is configured to simplify the labor associated with fabrication and erection.

Ways to Save Time and Money

Given these factors, the following are basic suggestions that you can use in your office practice today to work smarter—and to improve the economy of steel building construction.

Communicate! With the division of responsibilities for design, fabrication, and erection that is normal in current U.S. practice, open communication between the engineer, fabricator, erector, and other parties in the project is the key to achieving economy. In this way, the expertise of each party in the process can be employed at a time when it is still possible to implement economical ideas. The sharing of ideas and expertise is the key to a successful project. Indeed, the Construction Industry Institute (CII) has noted that the earlier construction design decisions are made, the more money those decisions can save.

Take advantage of a pre-bid conference. When in doubt about a framing detail or construction practice, consult a knowledgeable fabricator and/or erector. Most will gladly make themselves available at any stage of the game for a pre-bid conference, such as to help with preliminary planning or discuss acceptable and economical fabrication and erection practices. A pre-bid conference can also be used to communicate the requirements and intent of the project to avoid misunderstandings that can be costly. Many times, fabricators and erectors can provide valid cost-saving suggestions that, if entertained, can reduce cost without sacrificing quality.

Issue complete contract documents when possible. Design drawings and specifications are the means by which the owner, architect, and/or engineer communicates the requirements for structural steel framing to the fabricator and erector. Care in preparing these and other contract documents is important, not only to assure responsive bids or estimates, but also to minimize the potential for misrepresentation, errors and omissions in both bidding and the final product. The most clear, complete, and accurate design drawings and specifications will generally net the most accurate and competitive bids. Certainly, they are the starting point for economical, timely construction in steel. For guidance on what constitutes complete contract documents, consult the AISC *Code of Standard Practice*, particularly Section 3, (**www.aisc.org/code**) and CASE 962D, *A Guideline Addressing Coordination and Completeness of Structural Construction Documents*. When the nature of the project is such that it is not possible to issue complete contract documents at the time of bidding, clearly provide the scope and nature of the work as far as what the framing will be and what kinds of connections are required.

Don't forget to include the basics. Show a North arrow on each plan. Show a column schedule. Include "General Notes" that cover the requirements for painting, connections, fasteners, etc. in a manner that is consistent, complementary, and supplementary to the specification.

Late details can cost a lot. Even simple detail items like roof- or floor-opening frames can cost a small fortune if delayed, particularly when the delay forces installation after the steel deck is in place. Check the real costs the next time an opening frame gets moved, and then ask what the original detail costs to fabricate and install. You'll be amazed at the ratio of these numbers.

Show all the structural steel on the

structural design drawings. As indicated in the AISC *Code of Standard Practice*, structural steel items should be shown and sized on the structural design drawings. The architectural, electrical, and mechanical drawings can be used as a supplement to the structural design drawings, such as by direct reference to illustrate the detailed configuration of the steel framing, but the quantities and sizes should be clearly indicated on the structural design drawings.

Make sure the general contractor or construction manager clearly defines responsibilities for non-structural and miscellaneous steel items. Structural and non-structural steel items are identified in AISC *Code of Standard Practice* Section 2. Many items, such as loose lintels, masonry anchors, elevator framing, and precast panel supports, could be provided by more than one subcontractor. Avoid the inclusion of such items in two bids by clearly defining who is to provide them.

Avoid "catch-all" specification language. Language like "fabricate and erect all steel shown or implied that is necessary to complete the steel framework" probably sounds good to a lawyer, but it really does not add much to quality or economy, because it is nebulous and ambiguous. What is implied? Such language probably results only in arguments, contingency dollars, or change orders—and legal fees.

Avoid language that is subject to interpretation. Vague notations, such as "provide lintels as required," "in a workman-like manner," "standard," and "to the satisfaction of the engineer" are subject to widely varying interpretations. Instead, when required, specify measurable performance criteria that must be met.

Use standard practices and tolerances. ASTM A6/A6M defines standard mill practice. The AISC *Code of Standard Practice* defines fabrication and erection tolerances. The RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (**www.boltcouncil.org**) covers bolting acceptance criteria. AWS D1.1 establishes weld acceptance criteria. These and other documents provide standard tolerances that are acceptable for the majority of cases. Generally, they present the most efficient practices. Practices common to the industry work in a context and with the infrastructure routinely available in building construction.

In some cases, more restrictive tolerances may be contemplated for compatibility with the systems and materials that are supported by the structural steel frame. Or, tolerances may need to be defined for highly specialized systems or when steel and concrete systems are mated. All non-standard practices should be cost justified.

Changes in practices and tolerances require planning and resources that are not common and cause disproportionate increases in time and cost. Changes in tolerances, if made, need to reflect common construction practices and the available workpoints.

Clearly state any inspection requirements in the contract documents. The scope and type of inspection of structural steel should be indicated in the project specification. Make sure that the requirements for inspection are appropriate for the application. For example, the inspection of groove welds that will always be in compression during their service life is probably not required. Also, make sure shop inspection is scheduled so that it does not disrupt the normal fabrication process.

Avoid the use of brand names when specifying common products. When many manufacturers make a product, or there are acceptably equivalent products, avoid specifying the product by brand name. When it is necessary to indicate a brand name for the purposes of description, be sure it is a current, readily available product. Whenever possible, allow the substitution of an "equal." One excellent example: paint.

Try to avoid them entirely, but when you can't, clearly identify changes and revisions. Changes and revisions that are issued after the date of the contract generally have some cost associated with them. For example, material may have already been ordered, shop drawings may have already been drawn, and shipping pieces may have already been fabricated. Thus, it is best to avoid a default reliance on the change and revision process as a means to expedite schedules. However, when changes or revisions are necessary or desirable, they should be clearly identified so that all parties can recognize them and account for them.

Provide meaningful and responsive answers to requests for information. When the fabricator asks for a design clarification through an RFI, the most prompt and complete response, within the limitations of the available information, will be beneficial to all parties. If the RFI involves information on a shop drawing approval submission, it is best to provide the most specific answer possible. Try to avoid responses such as "architect to supply," "general contractor to supply," or "verify in field."

Specify materials in the appropriate—and usual—grade. See Part 2 of the 13th edition AISC *Steel Construction Manual* (available at **www.aisc.org/bookstore**) for a guide to the appropriate and usual grades for all the various structural steel materials.

Consider the use of hollow structural sections (HSS). Square and rectangular HSS are available in ASTM A500 grades B and C with 46 ksi and 50 ksi yield strengths, respectively. Round HSS are available in ASTM A500 grades B and C with 42 ksi and 46 ksi yield strengths, respectively. Although their material cost is generally higher, HSS generally have less surface area to paint or fireproof (if required), excellent weak-axis flexural and compressive strength, and excellent torsional resistance when compared with wide-flange cross-sections.

Be careful when specifying beam camber. Don't specify a camber of less than ¾ in.; small camber ordinates are impractical, and a little added steel weight may be more economical anyway. Also, do not overspecify camber. Deflection calculations are approximate and the actual end restraint provided by simple shear connections tends to lessen the camber requirement. Consider specifying from two-thirds to three-quarters of the calculated camber requirement for beams spanning from 20 ft to 40 ft, respectively, to account for connection and system restraint. In any case, watch out when rounding up the calculated camber ordinate, particularly with composite designs. Shear studs are unforgiving in that they can protrude through the top of the slab when too little camber is relieved under the actual load. Alternatively, allow sufficient slab thickness to account for reduced actual deflection.

Another thing to keep in mind: The minimum length of a beam that is to be cambered is about 25 ft. Why? Because the fabrication jig that is used to camber beams is usually configured with pivot restraints that hold the beam from 18 ft to 20 ft apart. To make sure there is adequate beam extending beyond this point to resist the concentrated force from the cambering operation, a 25-ft beam is generally required.

Favor the use of partially composite action in beam design. Although shear stud installation costs vary widely by region, one installed shear stud, on average, equates to 10 lb of steel. Fully composite designs are not usually the most economical, because the average weight savings per stud is less than 10 lb. Sometimes, the average weight savings per stud for 50% to 75% composite beams can exceed the point of equivalency. In some cases, non-composite construction can be most economical. A caveat: Make sure that the beam in a composite design is adequate to carry the weight of the wet concrete.

When composite construction is specified, the size, spacing, quantity and pattern of placement of shear stud connectors should be specified. It should also be compatible with the type and orientation of the steel deck used.

When evaluating the relative economy of composite construction, keep in mind that most shear stud connector installers charge a minimum daily fee. So, unless there are enough shear stud connectors on a job to warrant at least a day's work, it may be more economical to specify a heavier non-composite beam.

Shear stud connectors should be field installed, not shop installed. Otherwise, they are a tripping hazard for the erector's personnel on the walking surface of steel beams.

Consider cantilevered construction for roofs and one-story structures. Cantilevered construction was invented primarily to reduce the weight of steel required to frame a roof. Although today we are less concerned with weight savings than labor savings, cantilevered construction may still be a good option. Why? Because the associated connections of the members are generally simple to fabricate and fast and safe to erect. So cantilevered construction is still very much a potential way to save money.

Use rolled-beam framing in areas that will support mechanical equipment. It always happens. The structural design is performed based upon a preliminary estimate of the loads from the mechanical systems and units. Later, the mechanical equipment is changed and the loads go up—way up—sometimes after construction has begun. Rolled-beam framing offers much greater flexibility than other alternatives to accommodate these changing design loads.

Optimize bay sizes. It is still a good idea to design initially for strength and deflection. Subsequently, geometry and compatibility can be evaluated at connections, with shape selections modified as necessary. John Ruddy's assessment in a 3rd Quarter 1983 AISC *Engineering Journal* paper (**www.aisc.org/epubs**) suggested that using a bay length of 1.25 to 1.5 times the width, a bay area of about 1,000 sq. ft, and filler beams spanning the long direction combine to maintain economical framing. But…

Avoid shallow beam depths that require reinforcement or added detail material at end connections. Detail material such as reinforcement plates at copes and haunching to accommodate deeper, special connections is typically far more expensive than simply selecting a deeper member that can be connected more cleanly. If the beam is changed from a W16×50 to a W18×50, the simplified connection is attained virtually for free. And...

Don't change member size frequently just because a smaller or lighter shape can be used. Detailing, inventory control, fabrication, and erection are all simplified with repetition and uniformity. Keep in mind that economy is generally synonymous with the fewest number of different pieces. This same idea applies when selecting the chords and web members in fabricated trusses.

Select members with favorable geometries. Watch out for connections at changes in floor elevations; a deeper girder may simplify the connection detail. Also, watch out for W10, W8, and W6 columns, which can have narrow flanges and web depth; connecting to either axis is constrained and difficult. It is often most helpful to make rough sketches of members to approximate scale in their relative positions to discover geometric incompatibilities.

Use repetitive plate thicknesses throughout the various detail materials in a project. Just like with member sizing, the use of similar plate thickness throughout the job is generally more economical than changing thicknesses just because you can. For example, use one or two plate thicknesses for all the column base plates. This same idea applies for other detail materials such as transverse stiffeners and web doubler plates.

Design floor framing to minimize the perceptibility of vibrations. Floor vibration can be an unintended result in service when floors are designed only for strength and deflection limit-states and an absolute-minimum-weight system is chosen. Today's lighter construction, when combined with the lack of damping due to partitionless open office plans and light actual floor loadings (in the era of the *nearly* paperless office), has exacerbated the potential for floor vibration problems. Fortunately, design criteria to prevent perceptible floor vibrations from occurring are available; see AISC's *Design Guide No. 11* (**www.aisc.org/epubs**). There is also a helpful guide article by Christopher Hewitt and Thomas Murray in the April 2004 issue of MSC (**www.modernsteel.com/backissues**).

When designing for snow-drift loading, decrease beam spacing as the framing approaches the bottom of a parapet wall. Reduced beam spacing allows the same deck size to be used and the same beam size to be repeated into a parapet against which snow may drift. This is generally more economical than maintaining the same spacing and changing the deck and beam sizes.

Minimize the need for stiffening. When required at locations of concentrated flange forces, transverse stiffeners and web doubler plates are labor-intensive detail materials. For the sake of economy, using 50 ksi steel and/or a member with a thicker flange or web can often eliminate them. In the latter case, consider trading some less expensive member weight for reduced labor requirements. Always remember to reduce the panel-zone web shear force by the magnitude of the story shear. This can often mean the difference between having to use a web doubler plate and not. For further information, see AISC *Design Guide No. 13* (**www.aisc.org/epubs**).

Economize web penetrations to minimize or eliminate stiffening. Web penetrations in beams are often a cost-effective means of minimizing the depth of a floor system that contains mechanical or electrical ductwork. However, if they are numerous and require stiffening, it is probably more economical to eliminate them and pass all ductwork below the beams, if possible. Thus, stiffening at web penetrations should be called for only if required. The use of a heavier beam, a relocated opening, a change in the size of the opening, and the use of current design procedures can often eliminate the need for reinforcement of beam web penetrations. If web penetrations are to be used and stiffening is required, the most efficient and economical detail is the use of longitudinal stiffeners above and below the opening. For more information, see AISC *Design Guide No. 2* (**www.aisc.org/epubs**).

Eliminate column splices, if feasible. On average, the labor involved in making a column splice equates to about 500 lb of steel. Consider the elimination of a column splice if the resulting longer column shaft remains shippable and erectable. If a column is spliced, locating the splice at 4 ft to 5 ft above the floor will permit the attachment of safety cables directly to the column shaft, where needed. It will also allow the assembly of the column splice without the need for scaffolding or other accessibility equipment. If the column splice design requires welding in order to attain continuity, consider the use of PJP groove welds rather than CJP groove welds for economy.

Configure column base details that are erectable without the need for guying. Use a four-rod pattern, base-plate thickness, and an attachment between column and base that can withstand gravity and wind loads during erection. At the same time, make sure the footing detail is also adequate against overturning due to loads during erection. For further information, see AISC *Design Guide No. 10* (**www.aisc.org/epubs**). This reference contains minimum column base details for various column heights, and recommended wind exposures. And...

Make your column base details repetitive too. The possibility of foundation errors will be reduced when repetitive anchor-rod and base-plate details are used. Keep your anchor-rod spacings uniform throughout the job. Use headed rods or rods that have been threaded with a nut at the bottom if there is any calculated uplift. Otherwise, hooked rods can also be used if desired. Be sure to identify both the length of the shaft and the hook if so.

Allow the use of the right column-base leveling method for the job. Three methods are commonly used to level column bases: leveling plates, leveling nuts and washers, and shim stacks and wedges. Regional practices and preferences vary. However, the following comments can be stated in general: Leveling plates lend themselves well to small- to medium-sized column bases, say, up to 24 in. Shim stacks and wedges, if used properly, can be used on a wide variety of base sizes. Proper use means maintaining a small aspect ratio on the shim stack, possibly tack welding the various plies of the shim stacks to prevent relative movement and secure placement of the devices to prevent inadvertent displacement during erection operations and when load is applied. Leveling nuts and washers lend themselves well to medium-sized base plates, say, 24 in. to 36 in., but are only practical when the four-rod pattern of anchor rods is spaced to develop satisfactory moment resistance. Large column base plates, say, over 36 in., can become so heavy that they must be shipped independently of the columns and preset, in which case grout holes and special leveling devices are usually required.

Don't over-specify the details of secondary members. For example, spandrel kickers and diagonal braces can often be provided as square or bevel-cut elements that get welded into the braced member and structural element that provides the bracing resistance with a very simple line of fillet weld. In contrast, it is very costly to require that such secondary details be miter-cut to fit the profile of a member or element to which it is connected and welded all-around.

Keep relieving angles in a practical size range. The thickness of relieving angles is normally $5/16$ in. or $3/8$ in. If a greater thickness is required for strength, the basic design assumptions should be reviewed and perhaps modified. If vertical and/or horizontal adjustment of masonry relieving angles is required, the amount of adjustment desired should be specified and the fabricator should be allowed to select the method to achieve this adjustment, such as by slotting or shimming. Final adjustments to locate relieving angles should be made by the mason, preferably after dead load deflection of the spandrel member occurs.

Consider if heavy hot-rolled shapes are really necessary in lighter and miscellaneous applications. Ordinary roof openings can usually be framed with angles rather than W-shapes or channels. As another example, heavy rolled angles for the concrete floor slab stop (screed angles) are unnecessary if a lighter gage-metal angle will suffice (something in the 10-gage to 18-gage range, depending upon slab thickness and overhang). These lighter angles can often be supplied with the steel deck and installed with puddle welding, simplifying the fabrication of the structural steel. Small roof openings on the order of 12 sq. in. or less probably need not be framed at all unless there is a heavy suspended load, such as a leader pipe.

Consider the fabricator's and erector's suggestions regarding connections. To a large extent, the economy of a structural steel frame depends upon the difficulty involved in the fabrication and erection, which is a direct function of the connections. The fabricator and erector are normally in the best position to identify and evaluate all the criteria that must be considered when selecting and detailing the optimum connection, including such non-structural considerations as equipment limitations, personnel capabilities, season of erection, weight, length limitations, and width limitations. The fabricator will also know when variations in bolt diameters and holes sizes, broken gages, and a combination of bolting and welding on the same shipping piece will incur excessive and costly material handling requirements in the shop.

Design connections for actual forces. Or at least do not overspecify the design criteria. In U.S. practice, the Engineer of Record sometimes specifies standard reactions for use by the connection designer. These standard reactions can sometimes be quite conservative; look at the extreme example illustrated in the above figure. However, design for the actual forces generally allows more widespread use of typical connections, which improves economy. Axial forces, shears, moments, and other forces should be shown as applicable so that proper connections can be made and costly overdesign, as well as dangerous underdesign, can be avoided. This applies to shear connections, moment connections, bracing connections, column splices—all connections! The actual reactions are quite important for the proper design of end connections for beams in composite construction.

Use one-sided shear connections when possible. One-sided connections, such as single-plates and single-angles, have well-defined performance, are economical to fabricate, and are safe to erect in virtually all configurations. When combined with reasonable end-reaction requirements, one-sided connections can be used quite extensively to simplify construction. Sometimes, however, end reactions are large enough to preclude their use because of the strength limitations of such connections.

Avoid through-plates on HSS columns; use single-plate shear connections whenever possible. A single-plate connection can be welded directly to the column face in all cases where punching shear does not control and the HSS is not a slender-element cross-section.

Design columns to eliminate web doubler plates (especially) and transverse stiffeners (when possible) at moment connections. The elimination of labor-intensive items such as web doubler plates and stiffeners is a boon to economy. One fillet-welded doubler plate can generally be equated to about 300 lb of steel; one pair of fillet welded stiffeners can generally be equated to about 200 lb of steel. Additionally, their elimination simplifies weak-axis framing. For further information, see AISC *Design Guide No. 13* (**www.aisc.org/epubs**). MSC