If you've ever asked yourself "why" about something related to structural steel design or construction, *Modern Steel Construction's* monthly *Steel Interchange* column is for you!

This month's questions were submitted to AISC's Steel Solutions Center.

Cambered Beam Connections

On a cambered beam that has welded double angles on both ends, how should the angles be oriented in the fabrication shop? Should the angles be plumb to vertical or should they be perpendicular to the curve of camber? If they are plumb then what happens when the beam comes under load? If they are fit with the beam how do you bolt up in the field?

Usually, the double angles are fit relative to the flanges at the end of the beam when the beam is not in a cambered state. Once the beam is cambered, the double angles will be slightly tilted away from the vertical at the ends of the beam. When the beam is loaded with dead load, such as that from a concrete slab, the double angles will once again be in a near vertical orientation.

In many fabrication shops, holes for bolted double angles are punched or drilled prior to cambering the beam. Welded double angles are fit on the end of the beam perpendicular to the camber curve. Therefore, the connection angles in buildings are set to be parallel after the beam deflects to the zero camber position.

If the usual assumption that double angles are simple connections is valid, either method would be acceptable from an engineering standpoint. In most simple beam connections the elements are flexible enough to be pulled tight by the bolts. While neglecting the variation due to camber works for most simple beams, this is not true for deep plate girders and trusses and deep flange plate connections. Details or erection procedures must account for camber in these connections.

It may be of interest to note that the opposite applies to bridge splices. These splices are usually on cantilevered ends and are drilled during assembly.

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Stitching Requirements

We have quite a few fairly short (~3′) double angles to detail and are being questioned regarding using two intermediate connectors. The 3rd edition *LRFD Manual* appears to require a minimum of two intermediate connectors for all lengths (the tables go down to 2′). For such a short length, it seems excessive to have two intermediate connectors. Is there a way out?

If you want to eliminate the stitches, you could design the angles as a pair of single-angle compression members. That changes the potential behavior a bit (flexural-torsional buckling can occur in each angle independently, for example). But at the length you've got, it should not be hard to calculate a size that will work.

Charles Carter, S.E. P.E.
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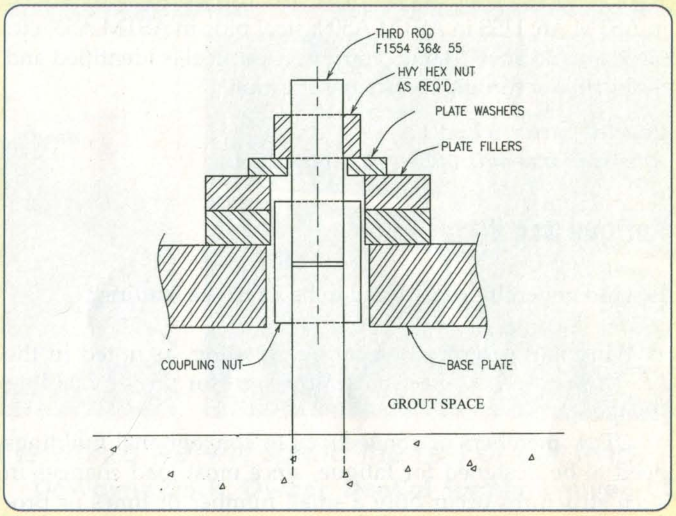
Threaded Coupler for Anchor Rods

On a job where the anchor rods came up short, the contractor suggested using a threaded coupler to extend the rods. What detail can be used with a threaded coupler?

One possible detail is shown (below). Note that it will likely be necessary to enlarge the base plate hole to make room for the coupler, which is typically the same width as a heavy hex nut. The use of plate fillers and plate washers is common in order to accommodate the coupling nut.



Keith Mueller, Ph.D.
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Hole Spacing

The *LRFD Manual of Steel Construction* (3rd edition) assumes a 3″ center-to-center hole spacing for most, if not all, of the examples utilizing bolted connections. Is this 3″ spacing a code requirement?

The 3″ spacing satisfies spacing requirements for the most common bolt diameters used in structural steel buildings ($3/4$ in., $7/8$ in., and 1 in.). However, this spacing is not a code requirement. Experimental research conducted by Kulak et al (ref. Kulak, Geoffrey L., Fisher, John. W., and John H. A. Struik, *Guide to Design Criteria for Bolted and Riveted Joints, Second Edition*, AISC, 2001) on bolt spacing indicates that spacings greater than three times the diameter of the bolt, *d*, (center of fastener to edge of adjacent bolt hole) ensure the maximum design strength in bearing. When adjusted to a basis from center of hole to center of hole, this is the basis of AISC's minimum spacing requirement of $2\frac{2}{3}d$, found in Section J3.3 of the 1999 *LRFD Specification*. The Commentary to this *Specification* does recommend a spacing of 3*d* to better provide for entering and tightening clearances.

Keith Mueller, Ph.D.
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Salvaged Steel

It seems like I increasingly have clients that want to use "recycled" steel on their projects. The problem is that what they mean by "recycled" is really salvaged material without any history or certs. Is there a standard for use in re-certifying used steel to permit reuse? I assume you would need to run tensile and Charpy capacities to see what you really have. Are there any other tests or issues that one should consider to permit reuse or is this just a bad idea?

Reuse is generally fine and the same tests a mill would run on mechanicals, chemicals, etc. can be run by any testing lab with knowledge of the appropriate tests. For example, if you want to identify W-shapes for re-use, the testing is described in ASTM A6; HSS in ASTM A500; steel pipe in ASTM A53; etc. Once you do such testing, your recycled steel is identified and designable according to that specification.

Charles Carter, S.E., P.E.
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Fatigue and Wind

Is wind generally considered to be a fatigue loading?

Wind is not typically a fatigue loading, as noted in the *LRFD Specification,* Section K3 (Design for Cyclic Loading [Fatigue]):

"Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions."

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Stub girder construction

How common is "stub-girder" construction?

Stub-girder construction is not as common today as it once was, primarily because the stub-girder system trades reduced steel weight for increased shop labor—an uneconomical trade in today's market. This type of construction, illustrated in the figure (modified Figure 1 from the Colaco paper cited below), was developed in the early 1970s. It creates a deeper member as an assembly of shallower pieces, but with so many more pieces to assemble in the shop, the labor costs add up quickly. Lower material costs for steel—and the advent of the beam line—made deeper W-shapes more practical. In addition, deeper W-shapes can be provided with web penetrations to accommodate ductwork and other mechanical systems.

For more information, the following articles discuss the stub-girder system.

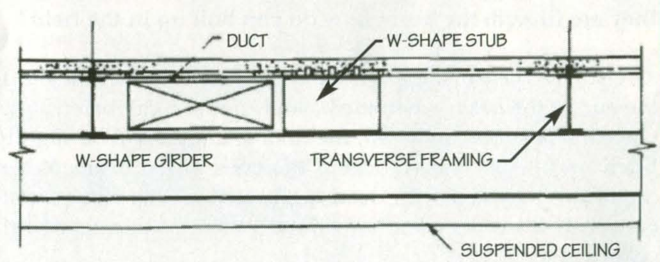
"A Stub-Girder System for High-Rise Buildings," Joseph P. Colaco, *Engineering Journal*, July, 1972.

"Computer-Aided Design of Stub-Girder System," Leon Ru-Liang Wang and John A. Gotschall, *Engineering Journal*, second quarter, 1980.

"Some Aspects of Stub-Girder Design," Reidar Bjorhovde and T.J. Zimmerman, *Engineering Journal*, third quarter, 1980.

"Stub-girder Design," Reidar Bjorhovde, *National Engineering Conference Proceedings*, AISC, 1987.

The *Engineering Journal* references noted above are available free online for *e*Pubs subscribers at **www.aisc.org/epubs**, or they may be purchased online at **www.aisc.org/bookstore**. For a copy of the conference proceedings paper, call AISC's Steel Solutions Center at 888.ASK.AISC.



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Steel Interchange is a forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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If you have a question or problem that your fellow readers might help you to solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact *Steel Interchange* via AISC's Steel Solutions Center:

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