Engineering Value into Your Project

BY DAVID T. RICKER, P.E., UPDATED BY CHARLIE CARTER, P.E., S.E.

Design economy is a topic that never grows old. Here's an update of a classic MSC article outlining ways to keep your steel projects on time and on budget.

Dave Ricker has been retired and enjoying his explorations in Payson, Ariz. for some time now. His sage advice and years of experience live on, however, in AISC *Engineering Journal* papers, which we often reference when answering questions that come into the AISC Steel Solutions Center.

Recently, while searching for an old article in the archives of MSC, I happened across an article Dave wrote on how to engineer value into a project. It read almost as if he had written it yesterday, since so much of the information remains perfectly applicable today.

Following is a slightly updated version of Dave's April 2000 article. It is interesting how much of the spirit of what Dave recommended years ago is emboldened in a document the Council of American Structural Engineers (CASE) wrote more recently: CASE 962D *A Guideline Addressing Coordination and Completeness of Structural Construction Documents*.

Perhaps the best way to implement Dave's time-honored recommendations is to build a relationship of teamwork among project participants. Everyone brings something to the table that can help the others. Start by talking about Dave's below recommendations and see what other ideas can be harvested.

—Charlie Carter

SUCCESSFUL PROJECTS REQUIRE A TEAM EFFORT, WITH THE OWNER, DESIGNER, FABRICATOR, AND ERECTOR WORKING TOGETHER TO CREATE THE FINISHED STRUCTURE. Each of the key team members have specific roles, and with these roles come responsibilities to the other team members. For example, one of the fabricator's and erector's key roles is to correctly interpret and comply with the designer's instructions. In order to accomplish this goal, however, he or she requires loads, dimensions, and member sizes to be summarized as outlined below:

- ✔ Beam end reactions for gravity, axial, and torsion loads, as well as moments, should be shown. Likewise, the designer should indicate if live load reductions have or can be taken. And, the designer should indicate whether or not the reactions given are LRFD loads or ASD loads.
- ✔ Column loads—not only axial, but also shear loads at splices and at the base, plus any moments at beam ends, brackets, and splices—can be shown on the column schedule.
- ✔ The designer should indicate diagonal axial loads and whether they are in tension, compression, or both. If the designer has preferences for bracing work-point locations or bracing connection design methods, they should be shown.
- ✔ The fabricator and erector both need to know all special floor and roof loads and point loads for special equipment or service requirements, such as beams supporting construction equipment storage areas or jump cranes, during erection. Such items should be discussed at the pre-construction conference.
- ✔ The fabricator and erector need to know which beams, if any, are subject to vibration loads such as from machine rooms and elevator beams.
- ✔ Reactions for special load conditions—such as cantilevered members, two- and three-span beams, beams with both uniform and concentrated loads, and beams with non-uniform snow-drift loads—should be shown.
- ✔ Specific column stiffener and doubler plate requirements should be shown—including sizes and locations. However, designers should consider the oftentimes more economical option of increasing the column size to eliminate the need for stiffening.
- ✔ If painting or galvanizing is required, the fabricator and erector need to know the specific requirements, such as surface preparation, which members are to be painted, the type of paint, etc. This information should be expressed using standard SSPC notation.
- ✔ Special attention should be given to details where steelwork structurally interacts with the work of other trades, such as web openings, support for fascia panels, support for metal deck, etc.

One of the most perplexing situations for fabricators and erectors is when designers don't share the information developed during the design process. During the design process, the structural engineer develops all of the information required to fabricate and connect the structural steel members, including loads, reactions, stiffening requirements, special conditions, etc. But when it comes to the design drawing, the engineer all too often merely shows the member sizes.

Skimping on the design drawing always comes back to haunt the designer in the form of questions, higher bids, change orders, arbitrating disputes, a slower review/approval process, and a dragging construction schedule. If it is a question of time, then the designer is fooling himself or herself. The time the fabricator spends deriving all of the needed information is passed back to the owner in the form of higher fees. And the engineer's approval reviewer has to spend additional time analyzing the questions and change orders.

The solution is greater teamwork and a consciousness of the importance of value engineering. The team member with the greatest impact on the economic success of the project is the designer. The team members all live or die with the engineer's design.

The following is a checklist of items designers should consider while designing a steel project:

Capitalize on steel's strengths. Steel offers good weight-to-strength ratio, efficiency of pre-assembly, speed of delivery and erection, strength in three directions, and ease of modification/renovation.

Keep current on the cost and availability of the various steel products. A steel fabricator can supply basic steel prices and guidance if any of the non-usual grades of steel applicable to a given product should be considered (see Figure 1 and the related information in Part 2 of the 13th Edition AISC *Steel Construction Manual*). A designer also should be aware of where the money is spent on steel construction: approximately 30% on material, 30% on shop costs, 30% on erection, and 10% of other items such as shop drawings, painting, and shipping. Labor is more than 60%!

Consider using partial composite design of floor beams—something in the range of 50% to 75%. Full composite design is often inefficient and uneconomical. The cost of one shear stud in place equals the cost of approximately 10 lb of steel. Unless this ratio can be attained, the addition of more studs will prove uneconomical.

Table 2–3
Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58-80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▒	
			50	62								▒		
	A501		36	58								▒	▒	
	A529[c]	Gr. 50	50	65-100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70-100	▒	▒	▒	▒	▒	▒	▒			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65[d]	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. I & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A913	50	50[h]	60[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50-65[i]	65[i]	■	▒	▒	▒	▒	▒	▒			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▒									
			46[k]	67[k]	▒			▒			▒			
			50[l]	70[l]	▒	▒	▒	▒	▒	▒	▒			
	A588		50	70	▒	▒	▒	▒	▒	▒	▒			
	A847		50	70								▒	▒	

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

[a] Minimum unless a range is shown.
[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.
[c] For shapes with a flange thickness less than or equal to 1½ in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).
[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾-in. thick and under. For wall thicknesses over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.
[j] For shapes with a flange thickness greater than 2 in. only.
[k] For shapes with a flange thickness greater than 1½ in. and less than or equal to 2 in. only.
[l] For shapes with a flange thickness less than or equal to 1½ in. only.

Figure 1

Take advantage of live-load reductions if governing codes permit.

Select optimum bay sizes. An exhaustive study by John Ruddy, P.E., formerly of Structural Affiliates International in Nashville and now with AISC (AISC *Engineering Journal*, Vol. 20, No. 3, 1983), indicated that a rectangular bay with a length-to-width ratio of approximately 1.25 to 1.50 was the most efficient. The filler members should span in the long direction with the girder beams in the short direction (see Figure 2).

Tailor the surface preparation and the painting requirements to the project conditions. Do not overdo or under-do the coating requirements. An extensive examination of a multitude of aged structures with steel frames indicates that the presence or absence of a shop primer is immaterial as long as the structural steel is kept dry (see the AISC *Specification* Commentary Chapter M). These same studies indicate that shop primer alone affords very little protection if a structure develops a serious leak.

In recent years, the trend has gone toward not painting. There are many side benefits to be gained by the omission of paint: no masking around bolt holes, better adhesion for concrete and/or fire proofing, easier weldability, ease of inspection, ease of making field repairs/alterations, etc. If shop painting is necessary, bear in mind that a shop coat is by definition a temporary coat—usually serving less than six months in duration. As such, there is little justification that the coat be perfect (i.e., of uniform thickness with no drips, runs, or sags).

Show all necessary loads on the design drawing to avoid costly overdesign of connections or—worse yet—under-design. The designer who provides a complete design will find that the subsequent review and approval process of shop drawings will be much quicker and more positive.

Make sure the general contractor or construction manager indicates who is responsible for any "gray areas" such as loose lintels, masonry anchors, elevator sill angles, elevator sheave beams, fastenings for precast concrete spandrel beams, etc. Unless the responsibility is specifically delegated, it is likely that the cost of these items will be included in the bids of multiple contractors, which means the owner will pay more than once for the same article.

Don't require the steel subcontractor to perform work normally done

by other trades, such as installing masonry anchors, ceiling hangers, toilet partition supports, window wall supports, etc. Information required to perform this work is often slow to develop, resulting in needless delay to the fabricator. The most efficient steel jobs are those on which the fabricator and erector are allowed to concentrate on the steel frame while unencumbered by the intricacies pertinent to other trades. This reduces coordination requirements and allows the steel framework to be turned over to the other trades in far less time than would otherwise be possible.

Consider the use of cantilevered rafters and purlins to save weight on roof design (see Figure 3).

Do not design for minimum weight alone. The savings in material cost will often be negated by the need for more members, more connections, and more costly shop work and field erection.

Excessively stringent mill, fabrication, and erection tolerances beyond state-of-the-art construction practices will reduce the number of bidders and raise the cost of the project. ASTM A6 tolerances and those established by AWS and AISC have served the industry well for many years and should be adhered to except under extraordinary circumstances where some special condition dictates a more strict treatment.

Design the proper type of high-strength bolt value. The correct application of each type (snug-tightened, pretensioned, and slip-critical) is well documented in the current AISC and RCSC specifications. Do not specify "slip-critical" bolt values for the purpose of obtaining an extra factor of safety. The trend in recent years is toward the use of snug-tightened bolts and bearing values.

Allow the use of tension control (twist-off) high-strength bolts. These bolts are as reliable as other methods of pretensioned installation and save labor costs in both shop and field.

Where possible, specify fillet welds rather than groove welds. Groove welds are more costly because of the joint preparation required and the generally greater volume of weld (see Figure 4).

Use single-pass welds where possible. This involves keeping fillet welds to a maximum of 5⁄16 in.

Favor the horizontal and flat welding positions. These welds are easier and quicker to make, and are generally of high quality (see Figure 5).

Don't specify more weld than is necessary. Over-welding creates excessive heat, which may contribute to warping and shrinkage of the members resulting in costly straightening expense.

Grant the fabricator the option of eliminating some column splices. The cost of one column splice equals the cost of approximately 500 lb of A992 steel. However, the fabricator should study the situation carefully before deciding to omit the column splice, as the resulting column may be too long for safe erection. Multi-tier columns should be designed to have splices every two or four floors. Three-floor columns are to be avoided due to erection difficulties. The higher up in a tall building, the less desirable it is to use four-floor columns due to higher wind speed and difficulties in guying.

Avoid designing column splices at mid-story height. These are often too high for the erector to reach without rigging a float or scaffold. If the splice can be located no higher than 5 ft above the tops of the steel beams, it saves the expense of the extra rigging and still will be in a region of the column where bending forces are relatively low (see Figure 6).

Except where dictated by seismic considerations, do not design column splices to "develop the full bending strength of the governing column size." Seldom is the splice located at

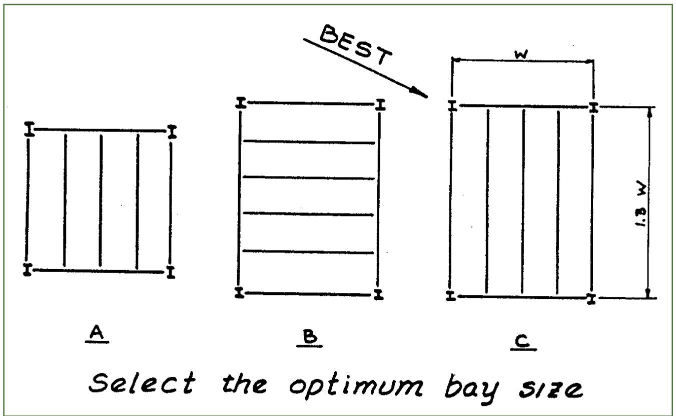

Figure 2

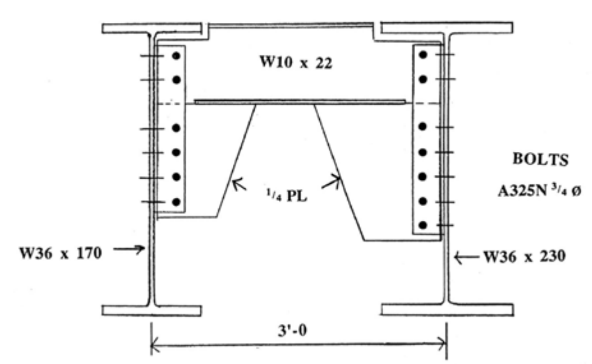

Figure 3

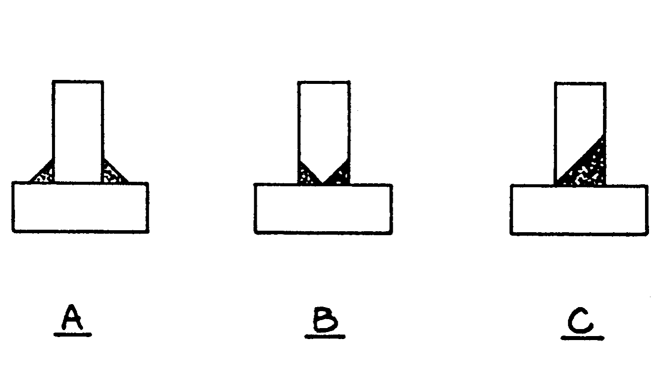

Figure 4

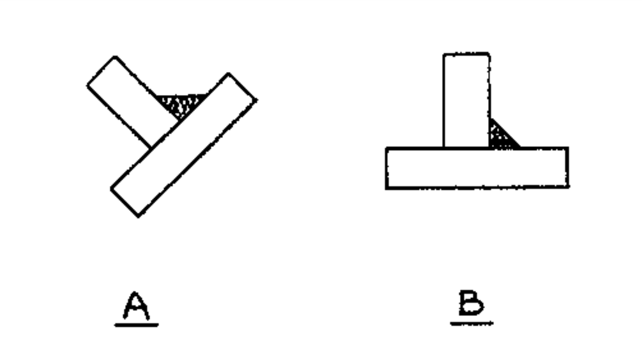

Figure 5

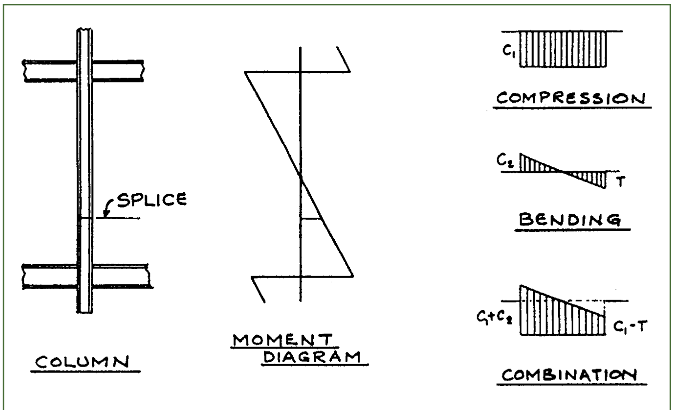

Figure 6

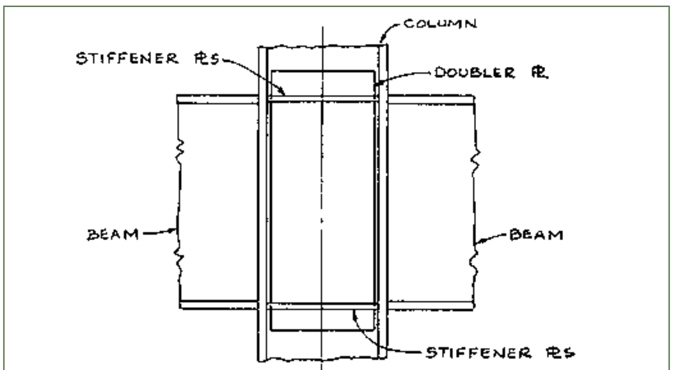

Figure 7

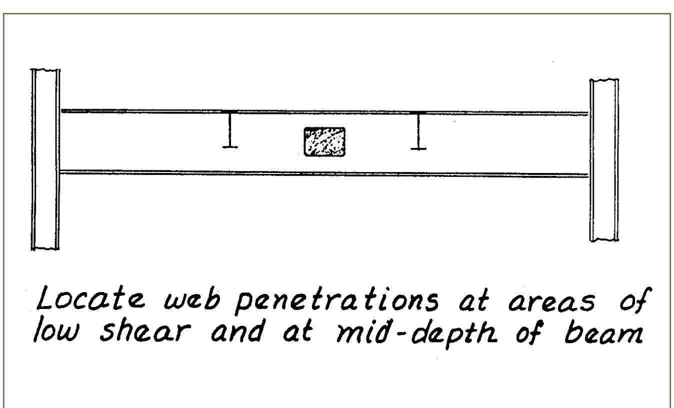

Figure 8

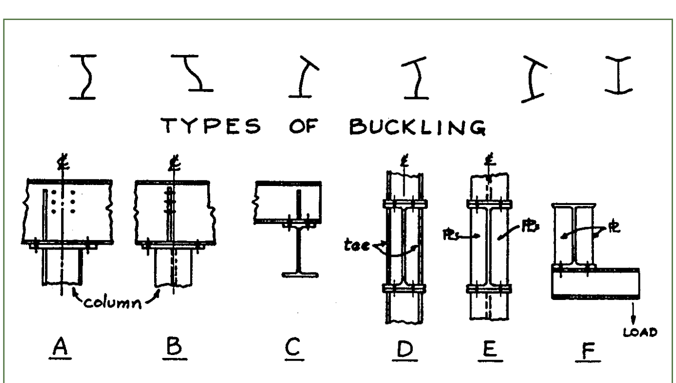

Figure 9

the point of maximum bending and seldom do the bending stresses result in a condition that would require a full-strength splice. The column has axial compression stresses. The excess capacity is allotted to bending stresses that occur as compression in one flange and tension in the other. The compression forces are added to each other at one flange while at the other flange the tension force is subtracted from the compression force. Seldom does this other side of the column ever go into tension and almost never into full allowable tension of the magnitude that would require a full-strength splice. Thus, except in high-seismic construction, there often is little justification for requiring a full-strength column splice (see Figure 6).

Consider using a heavier column shaft to eliminate the need for web doubler plates and/or column stiffeners opposite the flanges of moment-connected beams. One pair of stiffeners installed costs approximately the same as 300 lb of A992 steel if the stiffeners are fillet welded. If they are groove welded, the cost skyrockets to the equivalent of 1,000 lb of A992 steel. The cost of one installed doubler plate is about the same as 350 lb of A992 steel (see Figure 7). Considering that for an average two-floor column there could be as many as four pairs of stiffeners and two or more doubler plates, at least 1,900 lb of A992 steel could be sacrificed in order to save the time and expense of making the lighter shaft compliant. For more information, see AISC *Design Guide 13: Wide-Flange Column Stiffening at Moment Connections*.

Avoid designing heavy or awkward members in remote, hard-to-reach portions of the structure. This may eliminate the need for larger, more expensive hoisting equipment.

Reinforce beam-web penetrations only where necessary. It may be less costly to use a beam with a thicker web, to move the opening to a less critical location, or to change the proportions of the opening to something less demanding (see Figure 8). To help in designing web openings, AISC published *Design Guide 2: Design of Steel and Composite Beams with Web Openings*. AISC also offers a software program, Webopen, to help in designing web openings.

Allow the prudent use of oversized holes and slots to facilitate fit-up and erection. They may eliminate or reduce the need for costly reaming of holes or modification of connection parts in the field.

For ordinary structures, do not specify that connection material be of one type to the exclusion of other types. Allow the fabricator to use stock materials to good advantage. However, the fabricator should recognize that certain structural situations require specific types of steel. The designer should identify these special conditions.

Avoid calling for the indiscriminate use of stiffeners. Allow partial-depth stiffeners where applicable. Stiffeners are required to prevent local deformation or to transfer load from one part of a member to another (see Figure 9). If the main members are capable of taking care of themselves, then the cost of stiffeners can be saved.

Avoid odd sections that may not be readily available or which are seldom rolled, since this could result in costly delays. Consult with a fabricator concerning the availability of specific shapes.

In areas subject to snow drift loading, arrange the purlins parallel to the drift and space the purlins closer together as the drift load increases so the same gage roof deck can be used throughout (see Figure 10).

Space floor beams so as to avoid the necessity for shoring during the concrete pour. The cost of shoring is relatively expensive and can easily be offset by varying the span, gage or depth of the floor deck.

Avoid the "catch-all" specification that reads something like this: "Fabricate and erect all steel shown or implied necessary to complete the steel framework." The bids will undoubtedly be padded to cover whatever might be "implied." Or worse: arguments and extras!

Avoid the "nebulous" specification calling for stiffeners, roof frames, reinforcing of beam web penetrations, etc., "as required." The fabricator and erector are rarely furnished with enough information at the bid stage to determine what is or is not required and therefore will include in the bid an allowance for investigating and furnishing the questionable items whether they're needed or not.

Avoid overly restrictive specifications. The more restrictions listed in the steel specifications, the greater the chance that no one will be able to meet them all. This will eliminate some competition and result in higher bids.

Design for duplication of beam sizes where possible, since this results in economies of scale. For example, in a mezzanine the edge beams often carry less load and could be made smaller but for the sake of duplication make them the same.

Likewise, design for duplication of connections. For example, if most of the filler beams on a job can be connected using a four-bolt shear plate but a few require only a three-bolt shear plate, make them all four-bolt connections. This miniscule "give-away" is more than made up by the efficiencies of duplication—both for the shop and field. Connection material rarely exceeds 5% of the job total weight. Trying to save a tiny percentage of 5% is not cost-effective if it leads to special handling, marking, sorting, and other special treatment of the members in question. MSC

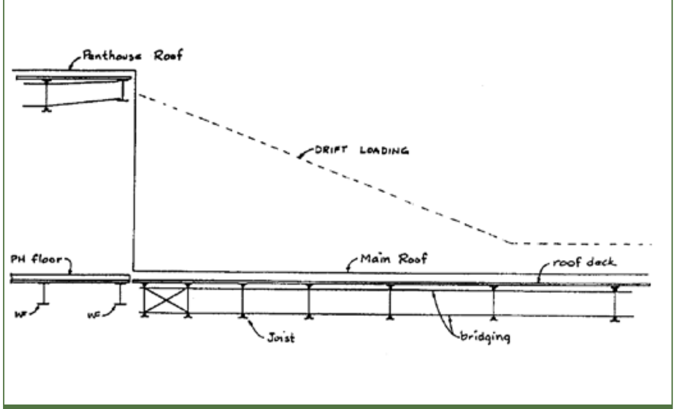

Figure 10

David T. Ricker, P.E., began his career in the American Bridge Division of U.S. Steel and later moved to Berlin Steel (Berlin, Conn.), eventually becoming the company's chief engineer and then its vice president of engineering. Now retired, he lives in Payson, Ariz.

Charlie Carter is AISC's chief structural engineer.