Complete information is a key to success

Value Engineering for Steel Construction

By David T. Ricker, P.E.

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/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, in LRFD, the designer should indicate whether or not the reactions are factored.

- 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. The fabricator also needs to know if allowable stresses can be increased (ASD). If the designer has a preference for bracing work point location it should be shown.

- The fabricator and erector 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/erector needs 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/erectors is when designers don't share the infor-

Figure 1: Mill Extras

(as reported by Nucor Yamato Steel)

The following sections are available at the same cost for A36, A572 Gr. 50, A992, and CSA 40.21 Gr. 350W:

W24x68-103, W24x55-62, W21x62-93, W21x44-57, W18x50-71, W18x35-46, W16x67-77, W16x36-57, W16x26-31, W14x61-82, W14x43-53, W14x30-38, W14x22-26, W12x53-58, W12x40-50, W12x16-22, W10x49-77, W10x33-45, W10x22-30, W8x31-67, W8x24-28, W8x18-21, W6x15-25.

For other shapes, A572 Gr. 50, A992 and CSA 40.21 Gr. 350W require surcharges as follow:

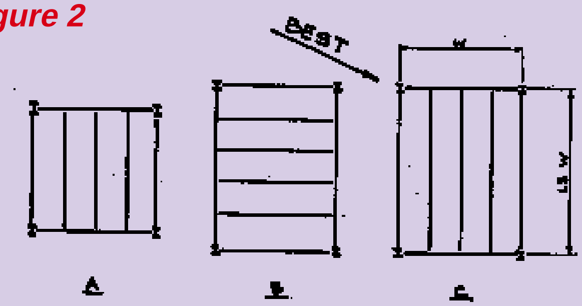
sections to 150 lbs./ft. inclusive $1.25/CWT
all sections over 150 lbs./ft. to 300 lbs./ft. inclusive $2.00/CWT
all section over 300 lbs./ft. $2.25/CWT
(note: CWT = 100 lbs)

Minimum item quantity requirements
A588 or equivalent
Mill 1 ....Inquire
Mill 2 ....20 tons
ABS-AH36 ....20 tons

Length Requirements
stock lengths 30'-80' inclusive ....none
under 30'-25' $1.00/CWT
non-standard lengths >30' - <80' $0.25/CWT
Over 80' $0.25/CWT

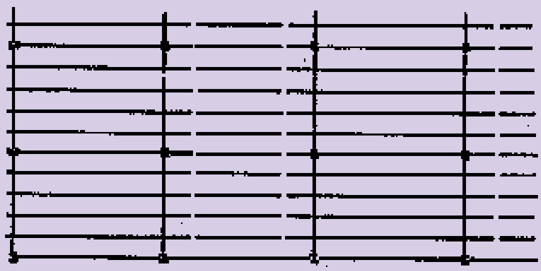
Standard lengths (30', 35', 40', 45', 50', 55', 60', 65' 70' & 80') are available in bundle quantities only.

Figure 2



Select the optimum bay size

Figure 3



Consider a cantilevered roof system.

mation 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, special conditions, etc. But when it comes to the design drawing, the engineer all-to-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.**
  - Good weight-to-strength ratio
  - Efficiency of pre-assembly.
  - Speed of delivery and erection.
  - Strength in three directions.
  - Ease of modification/renovation.

- **A designer should keep current on the cost and availability of the various steel products he/she prescribes.** A steel fabricator can supply basic steel prices and mill extras (*see figure 1*). 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% - 75%. Full composite design is often inefficient and uneconomical. The cost of one shear stud in place equals the cost of approximately 10 lbs. of steel. Unless this ratio can be attained, the addition of more studs will prove uneconomical.

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

- **Select optimum bay sizes.** An exhaustive study by John Ruddy, P.E., of Structural Affiliates International in Nashville (AISC Engineering Journal, Vol 20, #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 underdo 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 (LRFD *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 (that is, of uniform thickness with no drips, runs or sags).

- **Show all necessary loads on the design drawing** to avoid costly overdesigning of connections or—worse yet—underdesigning. 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 ositive.

• **Make sure the general contractor or construction manager indicates who is responsible for any "grey 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 sub-contractor 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 is well-documented in the current bolt 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-tight" bolts and bearing values.

• **Allow the use of tension control (twist-off) high-strength bolts.** These bolts are as reliable as other methods of measuring bolt tension 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".

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

Figure 4

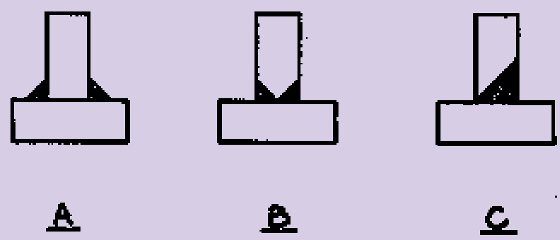


Figure 5

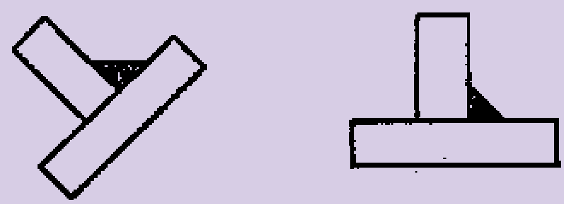
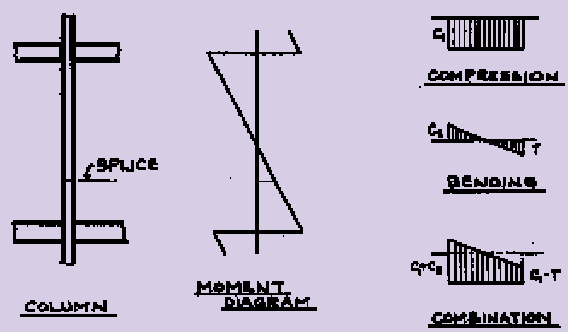


Figure 6



• **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 lbs. of A992 steel. The fabricator should study the situation carefully before he decides 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' 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 the point of maximum bending and seldom do the bending stresses result in a condition that

Figure 7

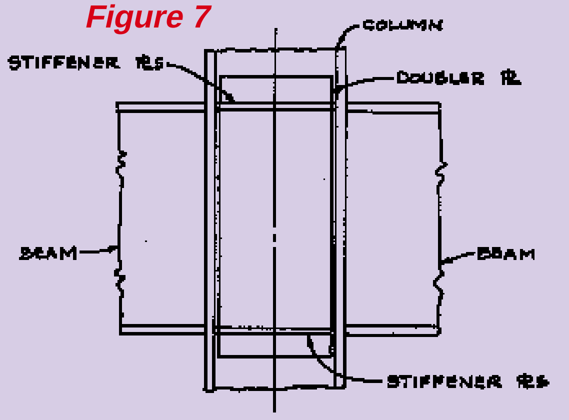
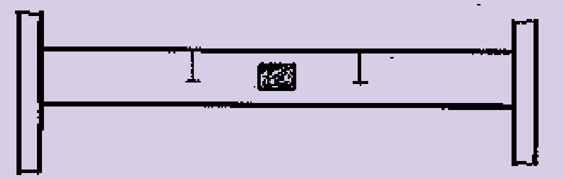


Figure 8



Locate web penetrations at areas of low shear and at mid-depth of beam

wold require a full-strength splice. The column has axial compression stresses. The excess capacity is allotted to bending stress 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 never into full allowable tension of the magnitude that would require a full-strength splice. Thus, except in high seismic construction there is little justification for ever 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 lbs. of A992 steel if the stiffeners are fillet-welded. If they are groove welded, the cost skyrockets to the equivalent of 1000 lbs. of A992 steel. The cost of one installed doubler plate is about the same as 350 lbs. of A992 steel (*see figure 7*). Considering that for an average two-floor column there could be as many as four pair of stiffeners and two or more doubler plates, at least 1900 lbs. 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. To order AISC publications and software, visit www.aisc.org or call 800/644-2400.)

• **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. To order AISC publications and software, visit www.aisc.org or call 800/644-2400 to order publications and 312/670-5444 to order software.

• **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 his stock 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 it 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".

• **Avoid the "nebulous" specification** calling for stiffeners, roof frames, reinforcing of beam web penetrations, etc., "as required". The fabricator and erector are rarely furnished 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 question-

Figure 9

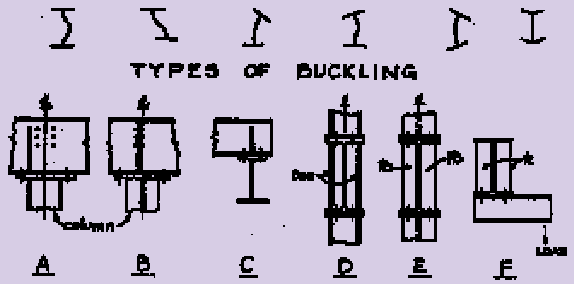
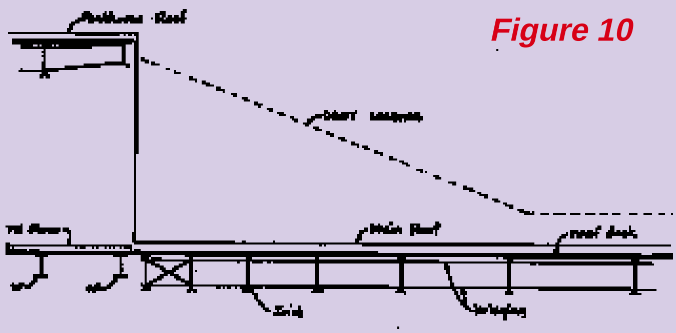


Figure 10



able 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. This miniscule "giveaway" 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.

Designing for Steel Joist Economy

The engineer-of-record is not expected to design steel joists or other proprietary products such as window walls, large atrium skylights or modular storage towers. However, they must clearly state the project's requirements regarding loads and performance to enable the joist manufacturer to produce a compliant product.

The key to economical steel joist design is "standardization." Open web steel joists were originally conceived as single-span members carrying uniform loads and as such they are at their best. Joists are a standard product with relatively few variables.

Departure from the standard will generally increase the cost of the joist. The following is a list of "do-nots":

• Do not specify non-standard joist depths.

• Do not call for non-parallel chords for "K" series joists.

• Do not specify severe top chord slope for long-span joists and joist girders.

• Do not call for clip angles, brackets or any such superficial attachements to be fastened to the joists by the manufacturer, as this would disrupt his normal handling and shipping system.

• Do not specify non-standard camber (see Steel Joist Institute Specifications for more information—phone 843/626-1995; fax 843/626-5565; web: steeljoist.org.)

• Do not specify a special joist paint, surface preparation or method of paint application.

• Do not specify a special paint color other than the manufacturer's standard (this is usually grey or reddish brown).

• Do not prescribe a special paint thickness.

• Do not specify special chord or web sizes.

• Do not call for a special web profile.

• Do not specify special steel material. The SJI Specification permits a broad range of acceptable material.

• Do not stipulate hot-rolled or cold-rolled material to the exclusion of the other. The SJI Specification permits both types.

• Do not prohibit or limit the number of joist chord splices, restrict the splice locations or require the splice to be made using a particular welding procedure. The SJI prescribes splicing procedures that have passed the test of time with flying colors.

• Do not call for holes in highly stressed portions of joist chords.

• Do not call for joist top chord extensions to be too long or too heavily loaded. Follow manufacturer's recommendations.

• Do not call for concentrated loads on joists that are beyond their capacity to resist. The SJI has established maximum limits.

• Do not specify bottom-bearing joists if underslung ends will suffice.

Figure 11

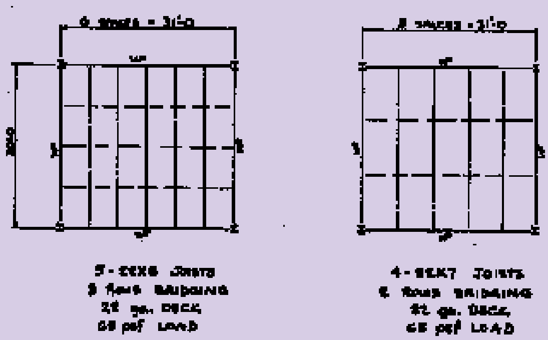


Figure 12: Approximate* Min. Joist Lengths

H & K Series Open Web Joists Depth (in.)	Min. Length* (ft.)	LH Series Longspans Depth (in.)	Min. Length* (ft.)
8	4-0	18	6-4
10	4-6	20	7-0
12	5-7	24	8-4
14	5-11	28	9-8
16	6-10	32	11-0
18	7-6	36	12-4
20	7-10	40	13-8
22	8-2	44	15-0
24	8-6	48	16-4
26	8-10	52	17-8
28	9-2	56	19-0
30	9-6	60	20-4

varies by manufacturer

Figure 13

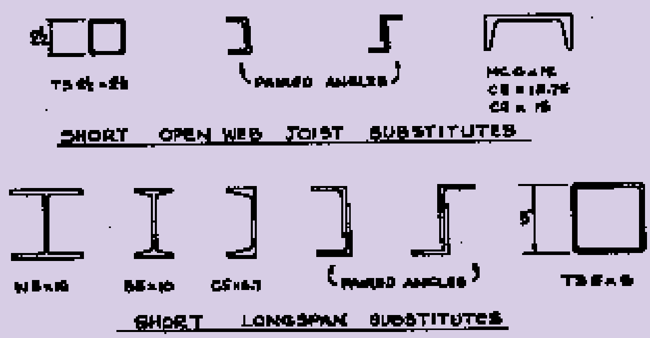
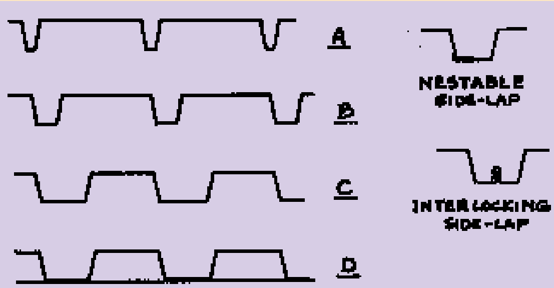


Figure 14



- Do not call for cross-bridging where the SJI Specifications permit the use of horizontal bridging.
- Do not specify that the welds used by the joist manufacturer be made using a special weld process or electrode.
- Do not indicate "ceiling extensions" if the ceiling is "hung" below the joists.
- Do not require that modest concentrated loads be delivered at a panel point. A joist top chord is designed to support a 400 lb. concentrated load anywhere between panel points in addition to the normal uniform load.

There are other ways to realize cost savings in joist construction. Sometimes it is less expensive to use a heavier joist if it reduces the bridging requirements *(see figure 11)*. Bridging installation is very labor intensive. For example, a 12K1 joist 17' long requires two lines of bridging whereas a 12K3 joist of the same length requires only one line of bridging. This is a 50% reduction in bridging and installation and the cost of the heavier joist is only about $3 more.

When a span is so short as to make it impractical or impossible to use an open web joist, a joist substitute can be used such as angles, channels, small wide-flanges, HSS or combinations thereof. *Figure 12* shows approximate limitations for minimum lengths of short joists. These may vary among manufacturers. *Figure 13* shows some of the shapes that can be utilized in place of short joists.

Today, some manufacturers are willing to customize their product, often to an amazing degree. However, for ordinary joist construction there are practical and economical limitations on the amount of customization that can be performed on a joist. If the designer sees the steel joist becoming too complex, chances are he or she should consider using a wide-flange alternative.

Designing for Metal Deck Economy

Formed metal deck floors and roofs play a significant role in steel building construction. There are enough variables in deck design to make it important for designers to put the right combination together to maximize value.

Some of these variables include:

- Type (roof deck, centering, deformed floor cell, cell deck).
- Profile (narrow, intermediate, wide rib).
- Depth.
- Gauge.
- Finish.
- Side laps.
- End laps.
- Span and deflection.
- Fastening systems.
- Acoustic or non-acoustic.

Roof Deck

Roof deck is generally available in 1-1/2” and 3” depths. Deeper deck is available from a few manufacturers. Deck is available in thicknesses from 16 ga through 22 ga and with painted, G60 galvanized or G90 galvanized finishes. Roof deck is available in acoustic and non-acoustic styles, with or without cells. Three rib profiles are available—narrow, intermediate and wide. Side laps are either nestable or interlocking (*see figure 14*). Deck ends are usually overlapped for a minimum of 2”. Die set (swaged) ends were offered at one time as an aid to deck nesting but modern deck profiles are such that this is no longer necessary. In fact, there are distinct disadvantages regarding the erectability of deck panels that have die set ends. The span is important because it has a direct effect on several other deck properties, namely the profile, gauge, depth, stress and deflection limits.

Narrow-rib deck is so inefficient that it is seldom used today. Intermediate-rib deck is competitively priced with wide-rib deck; 24 ga thickness intermediate-rib deck is offered by some manufacturers but should only be used in special cases. For a given span and gauge, wide-rib deck is stronger than intermediate-rib deck, which, in turn, is stronger than narrow-rib deck. As a rule, all things being equal thick deck costs more than thin deck, and 3” deck costs more than 1-1/2” deck. Also, G90 galvanized deck generally costs more than G60, which costs more than painted roof deck. Some deck manufacturers offer other finishes, such as electro-galvanized, phosphorized and galvanized/painted, which may be cost effective. Manufacturers should be consulted regarding pricing and availability. Because of the volatility of the deck industry, it is not recommended that a specific manufacturer be specified; rather, it is recommended that the manufacturer be a member in good standing with the Steel Deck Institute (ph: 847-462-1930; fax: 847-462-1940; web: sdi.org).

The following example of value engineering illustrates how several deck variables can be selected in order to provide a suitable product at a good price:

- For the metal deck of a large warehouse, the designer has made several efficiency and cost studies and determined that a wide-rib, 20 ga, 1-1/2”-deep deck on a 7’ three-span condition meets his requirements for load carrying ability and deflection.
- Wide-rib was selected because intermediate-rib would have required an 18 ga thickness. Narrow-rib was not considered because the allowable span would have been so short that it would have required many more supporting members.
- 3”-deep deck was considered and would have allowed the span to increase to 10’ and the deck lightened to 22 ga; however, the supporting members increased in size and the spacing did not adapt well to the chosen bay of 35’.
- 20 ga deck is among the most common used, with most manufacturers carrying a stock of 20 ga and 22 ga coils, which results in good availability and quick delivery.
- Painted deck was selected because the interior of the building is dry, well-ventilated and not subject to corrosive atmosphere, condensation, humidity or any other condition that would have required a more expensive coating.
- The designer needed some diaphragm strength from his deck system so he selected a deck with nestable side laps rather than interlocking side laps. Interlocking side laps are normally fastened with a button-punching devices but this method is unreliable in transmitting diaphragm shear forces. Nestable side laps can be welded, screwed or pop riveted. In 20 and 22 ga deck, there is a danger of blowthroughs when welding side laps, so a self-drilling screw system was selected. Plain rather than die set ends were stipulated since there was no reason to require die set ends. As the probability existed that the deck might have to

Figure 15

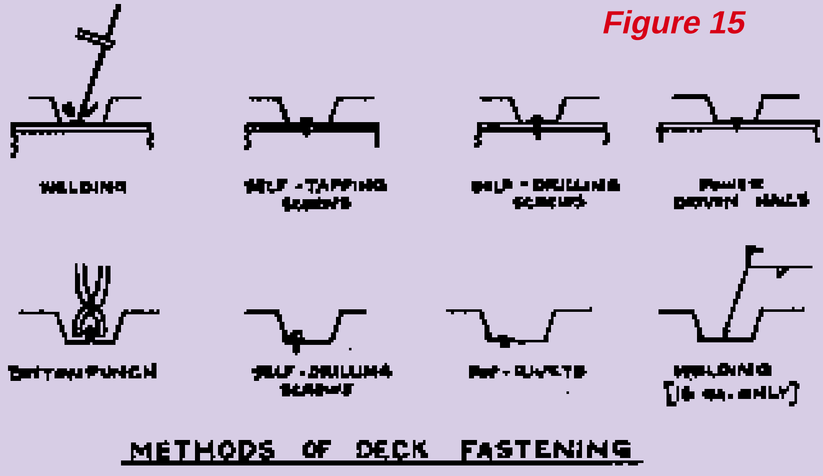
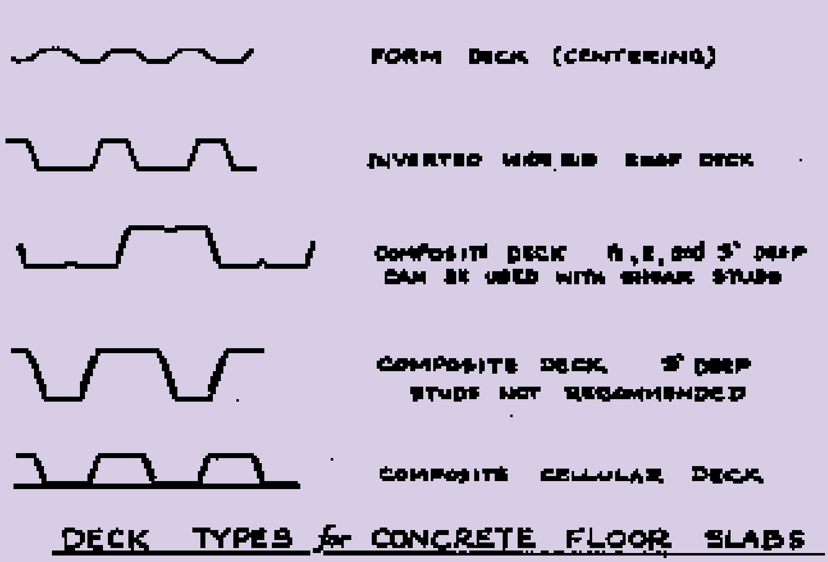


Figure 16



be laid in a patch-work manner and die set ends would have created a severe hardship for the erectors since die set deck is normally laid out end-to-end all in one direction.

• Because the deck was supported by members with relatively thick flanges, it was decided that welding would be the most efficient manner of fastening *(see figure 15)*. If the supporting material had been thinner, screw type or power-driven fasteners would have been considered. As it was, the designer wisely permitted all three methods in the specification. Since the deck was thicker than 0.028", weld washer were not needed or specified.

Floor Deck

Similar economical advantages can be realized in the selection of floor deck.

Floor deck for the support of poured-in-place concrete is available in several styles (*see figure 16*):

- Form deck (centering), which usually comes in a modified corrugated profile varying in nominal depths from ½" to 1-1/4";
- Composite (deformed) floor deck ;
- Cellular deck;
- Deck for use with shear studs;
- Deck that cannot be used with shear studs;
- Vented or unvented floor deck; and
- Floor deck with provisions for ceiling hangers.

Floor deck is available in 1-1/2", 2" and 3" depths and some manufacturers have special deeper decks. Inverted roof deck also can be used as a form for placing wet concrete. Floor deck is available in thicknesses from 16 ga to 28 ga depending on the type selected. It is available in uncoated, galvanized, painted and certain combinations thereof, again depending on the manufacturer and the type of deck required. Floor deck comes with either nestable or interlocking side laps.

As with roof deck, cost-conscious designers will only specify those features deemed necessary for proper maintenance and performance and will reject unnecessary features that serve no particular feature but add to the cost of the project.

Designing for Hollow Structural Section (HSS) Economy

Hollow structural sections have an advantage over wide-flange shapes in regard to painting and fireproofing, wind resistance and stiffness about the minor axis. Additionally, many architects prefer their aesthetic quality.

Square, rectangular and round HSS make economical columns. They are an excellent choice when stiffness about both axes is required. They can be used as hollow members or filled with concrete. There is no great strength advantage to filling small HSS with concrete; however, for larger columns there is a distinct advantage. For example, the design strength of an HSS 3x3x1/4x10' is a little more than 10% higher when filled with 3000 psi concrete. But the design strength of an HSS 8.625x0.322x12' is about 40% higher when filled with 3000 psi concrete than unfilled.

HSS have less surface area than equivalent wide-flange members, with round HSS having the least surface area. For example, listed here are the surface areas per linear foot of three common sizes:

- W8x31 = 3.89 sq. ft.
- HSS 8x8x1/4 = 2.60 sq. ft.
- HSS 8.625x0.233 = 2.26 sq. ft.

This can be a significant cost factor if members require an exotic surface coating or fireproofing.

HSS offer excellent resistance to torsional forces and can be used to advantage to support eccentric loads such as relieving angles for brick veneer, stone, or precast concrete panels (*see figure 17*).

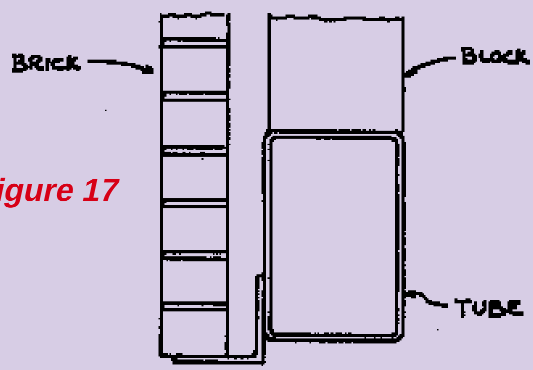


Figure 17

Figure 18

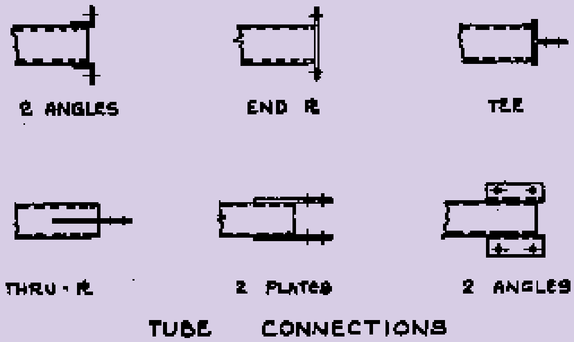
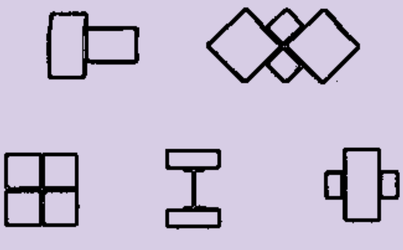


Figure 19



HSS also make efficient bracing members. It is usually easier to hide HSS diagonal bracing systems within interior walls than it is to conceal wide-flange shapes, angles or channels. Figure 18 shows various methods of connecting HSS and steel pipe. They also can be combined with other structural shapes to produce some startling aesthetic effects (*see figure 19*).

For more information on designing with HSS, AISC offers a *Hollow Structural Sections Connection Manual* (call 800/644-2400 or visit www.aisc.org to order).

Uneconomical Design Practices

A significant portion of a fabricator's overhead is spent on estimating. For every job, a fabricator may prepare 20 or more unsuccessful bids. Some recent project specifications have been written in such a way as to require the fabricator to perform significant portions of the steel design in order to prepare an accurate cost estimate.

A complete design is the best assurance that those who must use that design will accurately interpret the intent of the designer. There will be far less chance for ambiguities, misinterpretations, errors and/or omissions.

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