

# FABRICATION

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ITS RELATION TO DESIGN, SHOP  
PRACTICES, DELIVERY, AND COSTS

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## I. INTRODUCTION

General requirements for various operations used in fabricating steel for bridges are carefully delineated in the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges,<sup>1</sup> Standard Specifications for Welding of Structural Steel Highway Bridges<sup>2</sup> and the American Welding Society (AWS) Structural Welding Code, AWS D1.1.<sup>3</sup>

The requirements, set forth in these standards, begin with the consideration of flatness and straightness of steel prior to its dimensional layout or other work. This is followed by setting limitations for punching and shearing, and establishing rules for fitting, assembling, and reaming connections, as well as for bolting, welding and riveting.

The purpose of the discussion that follows is twofold: to emphasize and enlarge upon those fabrication factors that may help the engineer-designer obtain sound structures with improved efficiency and economy. Included, are fabrication and design guides — items to consider while preparing designs and specifications for highway bridges—plus a discussion of the control both of the work and the major operations or fabrication phases.

In some cases, fabrication and design guides are presented as “rules of thumb,” while in other instances, the guidelines are stated in terms of specific points that are to be considered when making design decisions based on alternative fabrication procedures, material savings

and fracture control. The “rules of thumb” are offered as broad generalities rather than hard and fast rules—strict, incontrovertible rules are a rarity in structural fabrication. Well-established rules of one fabricator may be inappropriate when another shop — either larger or smaller—is involved.

There is, in all probability, only one universally accepted rule: *fabrication costs money*. Manpower, machines, material, plant facilities, and tools all have a dollar value that collectively contribute to the ultimate cost of the fabricated product. Thus, it follows that economy can be realized only when fabrication is minimized; attempting to obtain greater economy simply by using the least amount of material is often negated by a concomitant need for increased fabrication costs.

To develop a truly economical design, it is necessary to compare the costs of various systems, and to select fabricated details that are least complex and yet will be able to perform the required function. The guidelines set forth in this publication should enable the designer to evaluate various alternative systems. Finally, there is yet another cost-saving factor to bear in mind: *design duplication*. Usually, this means that the greater the number of elements and details on a given job that are replicated, the lower will be the fabrication costs. Moreover, this practice may also lower engineering design costs.

## II. DESIGN GUIDES

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### SPECIFICATIONS

Bridge design and fabrication are covered in many specifications. Ordinarily, specifications for any given structure are stipulated by the agency responsible for its construction. In this publication, only the more commonly employed codes or specifications are referred to.

The basic fabrication requirements for welded bridge structures are given in the AWS Structural Welding Code, AWS D.1.<sup>3</sup> Additionally, all highway bridges must comply with AASHTO Standard Specifications for Highway Bridges<sup>1</sup> and AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges.<sup>2</sup> Various States have developed modifications of the foregoing. Another AASHTO publication "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members"<sup>4</sup> is mandatory on Federally funded single load path bridges. Obviously, it is very important to know which specifications will govern, and, in the case of fracture-critical members, which portions of the structure will be affected since material and fabricating costs are directly involved.

Where projects involve rail loadings, the American Railway Engineering Association (AREA) specifications<sup>5</sup> are the normal industry standard. As to material specifications, these are issued by the American Society for Testing and Materials (ASTM).<sup>6</sup> The properties and acceptance requirements for all the available structural steels are included in the ASTM specifications. The AASHTO bridge specifications<sup>1</sup> list five ASTM specifications with yield strengths of 36 ksi, 50 ksi and 100 ksi. By using these three strength levels, designers can meet most of today's material requirements. This practice should be encouraged.

An obvious preliminary guideline that can help obtain sound designs, is to use — wherever possible — standard specifications for both bridge materials and fabrication. Special provisions either in the specifications or as noted on the drawings should only be employed where modifications to the standard specifications are really necessary. Special provisions will also indicate specific supplements to the standard speci-

fications—such as specifying whether the material is to have atmospheric corrosion resistance, or a given level of toughness, or to specify the location and amount of non-destructive testing, or the amount of shop assembly.

### PLATE GIRDER WEBS AND WEB STIFFENERS

From the standpoint of material costs, it is usually desirable to make girder webs as thin as design considerations will permit. But it should be noted that this may not always produce the best *overall* economy. Fabricating considerations may control minimum web thickness. For example, extremely light webs may buckle during fabrication when the preheated flange contracts as it cools after welding, or a thin web may deform during welding to an extent that could result in an objectionable change in camber. Also, the question of whether stiffeners will permit use of thinner web plates requires careful economic analyses.

#### Transverse Stiffeners

When selecting the appropriate thickness for a plate that is to serve as a plate girder web, it is important to bear in mind the fact that where stiffeners will be used, transverse stiffeners placed on one side of the web will, almost always, be more economical than if used as pairs on both sides of the web. Transverse stiffeners should not be specified to bear on both the top and bottom flanges unless this is a design requirement. To achieve this condition is quite difficult since all other tolerances (tilt of flange, etc.) are thereby voided.

In this case, the often-asked detailing question, "How much extra material is justified in order to avoid another shop operation?" becomes "How much thicker does it pay to make a plate girder web in order to reduce the number of transverse web stiffeners or eliminate them?" The following guidelines should help determine whether increasing the web thickness, for the purpose of eliminating transverse web stiffeners, is justified.

To begin with, web thickness must be determined for both cases, i.e., with and without web stiffeners.

Referring to the formulations below for individual stiffeners, it is economical to use the thicker web if the thickness increase does not exceed the amounts shown.

*For 36 ksi stiffeners and web*

$$\text{Increase in } t_w \leq N (36 + W_{st})/41L$$

*For 36 ksi stiffeners and 50 ksi web*

$$\text{Increase in } t_w \leq .85 \times \text{Increase for 36 ksi web}$$

*For 50 ksi stiffeners and web*

$$\text{Increase in } t_w \leq N (28 + W_{st})/41L$$

where

$t_w$  = web thickness in inches.

$N$  = number of stiffeners to be removed.

$W_{st}$  = weight in lb/linear ft of *one* stiffener.

$L$  = length of web in feet to be increased.

$\leq$  = equal to or less than.

Note that 100 ksi web material is not included here. It is rarely, if ever, used because  $D/t$  restrictions make it uneconomical.

For example, consider the alternative designs given as Examples 3 and 4 on pp. 2-90 through 2-93 of the American Institute of Steel Construction (AISC) Manual (1980).<sup>7</sup> In each of these cases, the girder is 85 ft long and is designed in steel having  $F_y = 36$  ksi.

*Material Weights for a Design Without Stiffeners:*



Web:  $7/16'' \times 50''$

$$@ 74.4 \text{ lb/ft}$$

2 Flange Plates:

$1 1/8'' \times 18''$

$$@ 137.7 \text{ lb/ft}$$

$$212.1 \text{ lb/ft} \times 85 \text{ ft} = 18,029 \text{ lb}$$

*Material Weights for a Design With Stiffeners:*



Web:  $5/16'' \times 50''$

$$@ 53.1 \text{ lb/ft}$$

2 Flange Plates:

$7/8'' \times 25''$

$$@ 148.8 \text{ lb/ft}$$

$$201.9 \text{ lb/ft} \times 85 \text{ ft} = 17,162 \text{ lb}$$

6 Stiffeners: each  $1/2'' \times 5 1/2'' \times 48'' @ 37.4 \text{ lb} =$

$$\frac{224 \text{ lb}}{17,386 \text{ lb}}$$

$$17,386 \text{ lb}$$

The design with stiffeners will save 643 pounds of steel. However, the overall economy should be checked; doing so, we find that:

where

$N$  = number of stiffeners to be removed = 6

$W_{st}$  = weight per ft of one stiffener = 9.35 lb/ft

$L$  = length of web = 85 ft.

Therefore the maximum economical increase in web thickness:

$$\text{Using the expression } \frac{N (36 + W_{st})}{41 L}$$

$$\frac{6 (36 + 9.35)}{41 \times 85} = .078$$

The actual increase in web thickness is  $1/8$  inch (0.125). It follows, therefore, that the design with stiffeners will be more economical.

In some cases, girders of this depth — designed for AASHTO loading — would require more stiffeners. Under these circumstances, the formula may show that a greater web thickness would prove more economical. An increase in web thickness may also be justified by other considerations. As for example, when stiffeners are used, design stresses are established by AASHTO Category C stress range values for fatigue, and may control the flange area required; eliminating the stiffeners should make it possible to use the higher stresses of Category B. If, of course, other details — such as studs on a composite girder — place the design in Category C, or if fatigue stress does not govern, this additional saving cannot be realized.

### Longitudinal Stiffeners

The following guides can be used for increasing the web thickness as a means of eliminating *longitudinal* stiffeners. To eliminate *one* longitudinal stiffener, web thickness can be increased economically by the maximum amounts shown for the given conditions:

*For 36 ksi stiffener and web*

$$\text{Increase in } t_w \leq [(36 + W_{st})/41D] L_s/L_w$$

*For 36 ksi stiffener and 50 ksi web*

$$\text{Increase in } t_w \leq .85 \times \text{increase for 36 ksi web}$$

*For 50 ksi stiffener and web*

$$\text{Increase in } t_w \leq [(28 + W_{st})/41D] L_s/L_w$$

where

$t_w$  = web thickness in inches

$D$  = depth of web in *feet*

$W_{st}$  = weight in lb/linear ft of *one* long. stiffener

$\leq$  = equal to or less than

$L_s$  = length of longitudinal stiffener in feet

$L_w$  = length of web whose thickness will be changed, in feet

## FLANGE AND WEB SPLICES

### Flange Splices

Occasionally, fabrication cost savings may be obtained by eliminating a flange splice. There are frequent cases where a heavy flange plate is needed in those portions of a span that have the highest bending moments; elsewhere, however, a lighter flange plate can be used. Thus, the appropriate question becomes:

*If the heavier flange plate is extended to eliminate the splice, how much additional flange plate material is economically justified?*

The following guides can be used to make approximate determinations of the pounds of material that must be saved in order to justify the labor cost of a splice in a plate girder flange.

*36 ksi yield material*

$300 + 25 \times \text{cross-sectional area (inches}^2\text{) of the lighter flange plate}$

*50 ksi yield material*

(Same pounds as for 36 ksi material)  $\times 0.85$

*100 ksi yield material*

(Same pounds as for 36 ksi material)  $\times 0.65$

### Web Splices

The elimination of girder web splices may help achieve fabrication economies. It is, of course, necessary to retain some web splices since available mill lengths and lengths restricted by shipping limitations are frequently shorter than the required girder length. Designers should also consider using horizontal web splices in haunched girders. This may help create a less costly girder than would be the case in one made by cutting the web out of a single piece. The use of horizontal web splices eliminates width extras, and allows more efficient nesting of material.

The girder web thickness needed for regions of low shear stress (located at some distance from an interior or exterior support) is always less than the required maximum thickness. Here, the question arises: *Should two splices be placed symmetrically about a pier with an accompanying change in web thickness, or should the splice be eliminated while maintaining a constant web thickness?*

Following are approximate guides, giving the pounds of web material that can be added economically in lieu of a single welded web splice:

*36 ksi yield material*

$750 + 10 \times \text{cross-sectional area (inches}^2\text{) of the lighter web at the splice}$

*50 ksi yield material*

(Same number of pounds as for the 36 ksi material)  $\times 0.85$

## WEIGHT VS. COST

From the aforementioned guides — for stiffeners and splices—it should be evident that the simple expedient of decreasing material weight does not necessarily lower the ultimate cost of a fabricated girder. Although the cost of material may be lessened, fabrication costs will remain unchanged unless there is also a reduction in the number of detail pieces and shop operations. Similarly, if thinner high-strength material is used, the slower speed of punches and drills, increased requirements for welding control, the necessity for more stringent inspection, etc., will tend to keep fabrication cost-per-piece about the same as that for the thicker lower-strength material.

The question of when to use high-strength steels usually hinges on a number of factors. These include such considerations as:

- 1) the cost-strength ratio,
- 2) whether lower maintenance costs may be expected if unpainted weathering steels are used,
- 3) whether savings can be achieved by employing hybrid members with flanges of higher-strength steels than are used in the web,
- 4) whether the tonnage of each high-strength steel is adequate to justify its use.

The table below is a guide to making approximate estimates when considering the substitution of A572 Grade 50, A588, or A514 material for A36 material in welded plate girders. For the same quality level of surface preparation and shop painting, the weight reduction below the weight for A36 should be greater than the listed percentages to justify substitution of the higher strength material.

### Percentage Weight Reduction

#### A36 to High-Strength Steels

A572, Grade 50	14%
A588	-20%*
A514	-60%

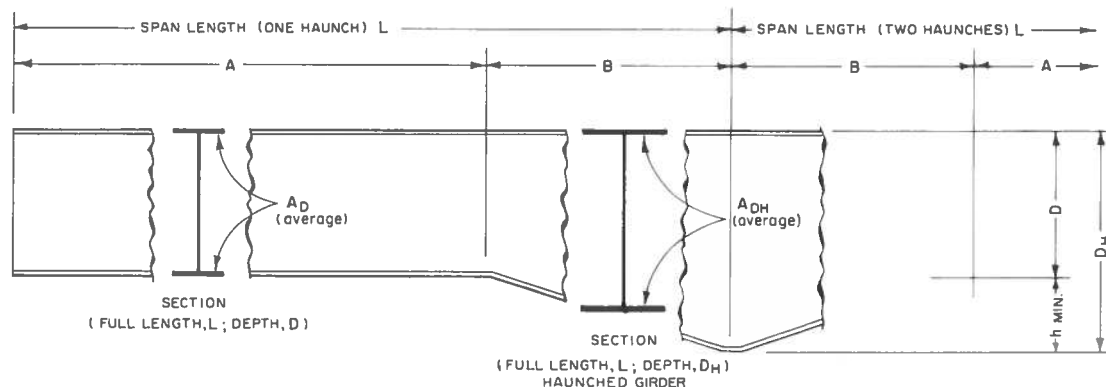
## HAUNCHED GIRDERS

For multiple-span continuous girders, site conditions or esthetics may require that girders be haunched at the piers instead of having parallel flanges throughout. A properly designed haunch will, of course, reduce the amount of material used. But, from the standpoint of final fabricated cost, the question remains: *When is a haunch justified?*

Clearly, a shallow haunch saves relatively little material and thus does not justify its added fabrication cost. The guides given below suggest the minimum depth for a haunch. Haunches that are deeper than  $h_{\min}$ , but less than  $0.5 D$ , will probably be economical.

\* If A588 steel is left unpainted, initial cleaning and painting are eliminated, and if the cost of re-painting every five years is thereby eliminated, this would make the use of unpainted A588 and painted A36 girders of the same dimensions cost about the same.





**Figure 1. Haunched girder:**

A & B are in feet;  $h_{min}$  in inches =  $D_H - D$  (both in inches)

$A_D$  (in sq inch) = average flange area for L with parallel flanges and web depth equal to D

$A_{DH}$  (in sq inch) = average flange area for L with a haunch of  $D_H$  depth

t (in inches) = thickness of web of girder without a haunch.

### Guides

for 36 and 50 ksi yield strength

Span with two haunches

$$h_{min} \times t = 16 B/A + (A_D - A_{DH})$$

Span with one haunch

$$h_{min} \times t = 8 B/A + (A_D - A_{DH})$$

for 100 ksi yield strength

Span with two haunches

$$h_{min} \times t = 8 B/A + (A_D - A_{DH})$$

Span with one haunch

$$h_{min} \times t = 4 B/A + (A_D - A_{DH})$$

### FIELD SPLICES

Costs can be reduced by the expediency of choosing proper locations for field splices. On evaluation, it is immediately apparent that the cost of the splice material itself will be lowest if the splice is positioned at a point that requires a minimum cross-section. Similarly, costs will be lowered if the field splice does double duty at a location where the flange thickness and/or web thickness will be changed, or where a shop splice would ordinarily be made to suit available mill lengths.

Obviously, from an initial cost viewpoint fewer splices are more desirable, but there remains this question: *Can the piece, between field splices, be moved to the site, then unloaded and erected with reasonable safety?* To answer this, recognition must be given to the fact that it is quite difficult to establish firm rules, since site conditions and distances between plant and site vary greatly from job to job. Nonetheless, the guides which follow under the headings: *handling*, *transportation*, and *erection* should be of some value.

### HANDLING

Usually, a length of approximately 120 ft, and a weight of approximately 90 tons are the two maximums for

pieces that can be handled efficiently either in the shop or at the site.

### RAIL TRANSPORTATION

The standard railroad flat car is 53'-6" long by 10'-8" wide, and 3'-6" above rail. Its average capacity is about 70 tons. Longer loads may be shipped in standard gondola cars; these are 65'-0" long by 8'-6" to 9'-0" wide.

Pieces 10-ft high by 8-ft wide by 60-ft long, weighing about 20 tons do not normally require railroad clearance. Carriers should be consulted regarding pieces exceeding these dimensions.

For restricted rail movements, widths up to approximately 13 ft can be handled, depending on the route, the configuration of the load, and the mid-ordinate and end-of-overhang on curved alignments. Long pieces may be shipped—supported on bolsters—on two flat cars, at opposite ends of the load, connected by idler cars. Such bolsters run as much as 1'-6" high above the car floor, reducing the net height available for the load by that amount.

In recent years, truck/train "piggyback" cars have been used for long loads. These cars vary in length up to 85 ft, and can handle loads over 100-ft long if the overhang beyond the end of the car is accommodated by idler cars. High blocking is needed to provide clearance above the idler car, but the blocking need not accommodate movement as in the case of a bolster.

Once again, it must be emphasized that each project's requirements for rail movement should be investigated on an individual basis.

### TRUCK TRANSPORTATION

Pieces 8-ft high by 8-ft wide by 60-ft long with axle or gross weight limited by the respective public agencies, can normally be shipped without special authorization. It is, however, common practice to ship members as long as 120 ft with permits.



## BARGE TRANSPORTATION

Often, the most economical means of transport are barges; this is true when 1) long distances are involved, 2) the site location and condition permit erection direct from the barge, 3) tow clearances are sufficient, and 4) when fabricating plant location permits easy barge loading.

Approximate barge dimensions are:

For Hopper (Cargo) Type:

26 ft x 160 ft x 11 ft = 1300 ton capacity

For Deck Type (River Use):

40 ft x 128 ft x 9'-3" = 1100 ton capacity

## GENERAL OBSERVATIONS ON TRANSPORTATION

Shipping larger, heavier pieces will usually require one or more special provisions. These include: additional blocking, specific types of railroad cars or multi-axle trailers, restricted hours for highway use, escort vehicles, and so forth. While such steps entail additional costs along with added shop handling costs, these may be offset by significant erection savings as a result of the need for fewer pieces and splices.

## ERECTION

Field splices should be located close enough to each other so that individual pieces will be stable without requiring special stiffening trusses, falsework, etc. A widely used rule may be stated as follows:

*The unsupported length of the shipping piece divided by the minimum width of flange material in compression should be less than about 85.*

A final important suggestion for minimizing the cost of splices: show bolted splices with acceptable welded alternatives on the design drawings. This makes it possible for each fabricator to use his most economical splice in his bid quotation.

## WELDED BOX MEMBERS

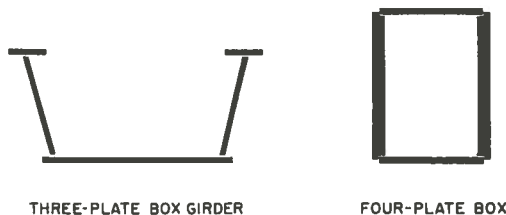


Figure 2.

Welded box-shaped members are employed in many applications for highway and railroad bridge construction. This kind of member, typified as a four-plate box, is found in truss bridges, arch rib and tie members, as well as in a three-plate box in girder-type highway bridges (Fig. 2).

Welding and inspection requirements specified for box member fabrication will have a significant effect on

cost—particularly in the case of four-plate members. In box members, where stresses permit, greater economy can be achieved by connecting their component parts with fillet welds rather than full penetration welds.

When it is not deemed necessary to make provisions for maintenance inspection of box member interiors, cost savings may be realized by placing a sealing diaphragm near the end of the member. This is a frequently used device in truss-type bridges where the box interior is too small to permit entry by a workman. In such cases, savings will accrue since there is no need to paint the box interior between sealing diaphragms.

## CURVED MEMBERS

In current design practice, bridge geometry is fit to meet highway alignment requirements rather than bending the highway to match a simpler bridge geometry. This practice, plus appearance considerations, has increased the use of horizontally curved members. But, before specifying horizontal curvature, a designer should be aware of the resulting increase in fabrication costs. (These costs are over and above the added cost of increased material that is needed to take the torsional stresses arising from curvature.)

The designer should also take into account the fact that for long radii, girders with low transverse moments of inertia (about their Y-Y axis) may often be easily forced into position and pinned in the field to the desired curvature using come-alongs or similar equipment. In this case, no shop curvature would be required.

Where an in-shop procedure is necessary, members can be curved by 1) cold-bending, 2) pre-cutting the curved flanges and bending the web to fit the flanges, or 3) heat-curving the members after fabrication (see p.I/7.14 et seq.). All three methods leave residual stresses in the cross-section. Generally, cold bending is only used for rolled shapes with a moderate required curvature.

Within the limits on curvature specified by AASHTO, it is usually more economical to heat-curve than to pre-cut curved members. Quenched and tempered steels, however, should never be heat-curved, since changes in their metallurgical structure will occur. As noted earlier, the designer should place as few restrictions on choice of method as is possible, since fabricators' facilities vary so widely.

To make a rough dollar estimate for girders with radii over 1000 ft, it may be assumed that heat-curving a straight girder will add about 10% to that of the straight girder fabrication cost. As the radius shortens, this percentage increases, although not linearly. For the pre-cut flange method, the added cost for curving could be as much as 50% of that for the straight girder—depending on the number of pieces to be curved using special jigs, and so forth. In this connection, for spans with a long radius of curvature, the designer should check whether the same radius might be specified for each line of girders since the change in the mid-ordinate

will be rather small. By repeating the same radii, a better opportunity is afforded for duplicating the girders.

## WELDED JOINTS

In the design of a welded joint, the primary concern is for the amount of weld required to transmit the connection stress. *Over-welding wastes money.* Fillet welds are almost always the most economical choice; groove welds can increase the welding cost from 150 to 200%. Where groove welds are necessary, the type required should be stated only as complete joint penetration (CP) or partial joint penetration (PP). (If PP is used, the required effective throat must also be specified.) Adhering to this procedure permits the fabricator to use the most economical groove weld for his particular situation, equipment, etc.

The two most important cost factors in a welded joint are 1) the amount of joint edge preparation required and 2) the amount of weld metal deposited. Fillet welds and square-groove butt welds require the least amount of preparation and usually the least amount of weld material and should be used when thicknesses are relatively small. Note that, with proper welding procedures, one-quarter inch of penetration can be achieved with each fillet weld. Frequently, partial penetration

instead of full penetration can be used where one side of a joint is inaccessible.

Several welding processes with a number of variations are in current use. Each of these, when used in accordance with American Welding Society (AWS) Part 4 Techniques,<sup>3</sup> or similar specifications, will provide a welded joint capable of meeting the design requirements — and this includes field as well as shop welding. Here again, it is best to leave the selection of a welding process to the fabricator, subject to the approval of the design engineer.

Shielded metal-arc (manual or “stick”), the oldest of today’s processes, is only used either for short, out-of-position types of welds, or when assembling other equipment is uneconomical. For bridge members, submerged-arc welding, either fully automatic or semi-automatic (i.e., with manually controlled travel), has largely replaced manual welding in the shop. Generally, however, it is limited to the flat position for groove welds and to the horizontal or flat positions for fillet welds (Fig. 3).

To permit automatic welding in any position, both flux cored-arc (FCAW) and gas metal-arc (GMAW) welding processes have been developed and are increasing in usage, particularly FCAW. The newest processes, electro-slag (ES) and electro-gas (EG), were developed for single-pass, high deposition rate welding. They are limited to the vertical position and to weld lengths which justify a setup. At the present time, a temporary moratorium is in effect on the use of ESW in tension members of Federally funded bridges. The ESW and EGW processes should not be used for splices in main tension members (including members in flexure) until further research supports the practice.

Welding techniques are continually being improved and additional processes will undoubtedly be developed. (See also Operations, Welding pp. 18, 19).

The applicable Codes give the type and scope of non-destructive examination (NDE), and the acceptance criteria to be used in all cases. For the most part, simply meeting these requirements is usually all that is necessary to determine that a structure has been properly designed to suit its purposes. Unless there are special design considerations (e.g., fracture-critical members) redundant use of inspection procedures will increase costs out of proportion to the small quality improvement that may be obtained. Where specifications permit alternative inspection procedures, the choice should be left to the fabricator and, at the same time, remain subject to the Engineer’s approval. This subject is discussed in greater detail on pp. 14, 15.

## FIELD CONNECTIONS

The question of whether to use field bolts or field welds is inevitable. If bolting is used, the ultimate cost will include drilling or punching the splice material, shop assembly for reaming, and the combined cost of bolts, nuts and installation. A marked reduction in current fabricating costs can be attributed to the growing prac-

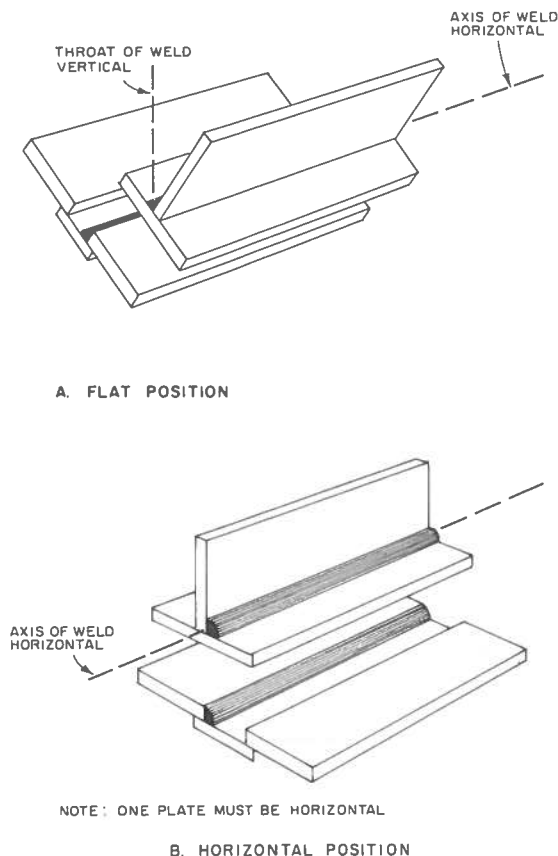


Figure 3.

tice of numerically controlled (NC) drilling with its inherent accuracy. In some instances, this has been used in lieu of ream assembly; a number of engineers are accepting NC drilling with the proviso that sufficient "proof-of-fit" checks are carried out. On the other hand, if welding is used, joint preparation and shop assembly costs will be lower, but the cost of field welding will be more than that for field bolting.

Shop costs can usually be estimated fairly accurately, but estimating field costs is somewhat more difficult. Field welding costs must take into account many variables such as:

- 1) Availability of qualified welders near the job site,
- 2) Weather conditions — particularly temperature and wind,
- 3) Additional falsework and erection material that may be needed for support during welding,
- 4) A tight erection schedule,
- 5) Availability of qualified firms to do specified non-destructive tests on the welds,
- 6) Variations in local work rules, such as the one requiring a welder's helper.

On the contract drawings, alternate details — for welded or bolted-girder field splices — will allow the contractor the option of using the technique best suited to his capabilities and, taking into account the above-listed factors, the most economical.

It should be noted that field welding main truss joints is rarely an economical procedure. In cases where average conditions can be anticipated, use the following guideline: Simple connections that require little or no permanent splice material will usually be about fifteen to twenty percent more economical when bolted. (Connections for lateral bracing, diaphragms and cross-frames are examples of this type.)

## SURFACE PREPARATION

Surface preparation is accomplished by many procedures. These include:

### a) *Hand Tool Cleaning*

Removal of loose rust, loose mill scale, and loose paint—to whatever extent specified—by means of hand chipping, sanding, and wire brushing.

### b) *Power Tool Cleaning*

Removal—to a specified degree—of loose rust, loose mill scale, and loose paint by means of power tool chipping, scraping, sanding, and wire brushing.

### c) *Brush-off Blast Cleaning*

Blast cleaning of all—except tightly adhering—residues of mill scale, rust, and coatings, exposing numerous evenly distributed flecks of underlying metal.

### d) *Commercial Blast Cleaning*

Blast cleaning until at least two-thirds of each element of surface area is free of all visible residues. (This technique is suggested for use where rather severe exposure conditions are anticipated.)

### e) *Near-White Blast Cleaning*

Blast cleaning almost to white metal cleanliness, i.e., until at least 95% of each element of surface area is free from all visible residues. (This technique is suggested for use in high humidity, chemical atmosphere, marine or other corrosive environments.)

### f) *White Metal Blast Cleaning*

Blast cleaning to white metal cleanliness by removal of all visible rust, mill scale, paint and foreign matter. (This is, at once, the most expensive, and rarely needed technique for paint systems on highway bridges.)

In terms of effectiveness and economy, following is a guide to the various cleaning methods: "Brush-off" blast cleaning gives greater uniformity than power cleaning while its cost approximates that of power cleaning. "Commercial" blast cleaning, usually sufficient for many paint systems, is from 90 to 100% more costly than brush-off cleaning. "Near-white" blast cleaning, specified for certain paint systems, costs about 15% more than "commercial" cleaning. The various degrees of blast cleaning are defined by the Steel Structure Painting Council (SSPC).<sup>8</sup>

## UNPAINTED STEELS

With regard to "weathering steels" such as USS Cor-Ten B (ASTM A588, Grade A), unpainted surfaces are not recommended in applications that involve:

1) exposure to highly corrosive fumes, 2) severe marine conditions with repeated wetting by salt spray or fog, 3) burial in the ground, or 4) being submerged without adequately protecting the steel, and 5) details that are subjected to run-off containing road salts, and the steel is not adequately protected by a suitable coating. Under ordinary atmospheric conditions, however, exposed weathering steel can be left unpainted, since a protective oxide coating forms on its surface governed primarily by the environment. In all cases, the designer should carefully heed the steel producer's advice on the question of painting weathering steel for a given application.

In the period immediately following erection, it is particularly important to obtain uniformity of oxide formation because the ultimate overall appearance will depend largely on the uniformity of surface preparation. Of primary importance is a clean surface, i.e., one that is free of shop markings, grease, oil, welding slag, flux or non-adhering spatter.

Blast cleaning of weathering steels should be specified only on surfaces where it is deemed necessary for the initial appearance to be uniform. As an example, beams and girders of deck bridges would be blast cleaned only on the outside surfaces of the fascia members and on the soffits of the interior members. Often, *commercial* cleaning is sufficient, but *near white* cleaning may be indicated for highly visible situations, e.g., in parks where pedestrians may be able to view the structure at relatively close range. Conversely, in re-

mote areas, where the structure's members are less visible, brush-off cleaning or, in some cases, no cleaning may be suitable.

As with all other operations, costs will rise proportionately as the number of procedures increase; thus, surface preparation economies can only be obtained by specifying the minimal amount and level of blast cleaning actually needed for the given job.

## SHOP PAINTING

Where painted steels are used, the degree of surface preparation should be adequate for the specified paint system. Selection of a particular paint system is predicated on meeting site conditions and the expected economic life (i.e., before re-painting is required). The shop paint system and the degree of surface preparation must be compatible with the field painting system, and should be specified in the Plans and Specifications.

Designers should also give thought to and specify the materials to be used for piece marking and other identification. Inadequate attention to this seemingly unimportant matter has created problems with paint systems, particularly in unpainted applications where color match is important.

Ultimately, the decision rests on choosing between a higher first cost to obtain a longer life versus a lower first cost that almost always means higher maintenance costs and/or a shorter life. Currently, the trend seems to be toward an improved surface preparation that promises a longer paint life. An accepted reference source to aid in selecting a proper paint system is the SSPC Vol. 2, Table I.<sup>8</sup>

A word of caution must be introduced at this point: It is important not to lose sight of the fact that the shop primer coat is merely a temporary covering; intended only to withstand normal field weather exposure for a short time. If unforeseen circumstances create construction delays and the structure's field painting is held up, one of two courses must be taken: either provide physical protection for the primed steel, or make provisions for the probability that field cleaning and re-priming will be undertaken when work is resumed.

### Painting Faying Surfaces

When the cleaning and painting system is such that bolt values are not reduced, as in the case of zinc-rich paints, consideration should be given to the idea of permitting field contact surfaces to be painted. If this is done it eliminates the need to mask the surfaces of pieces that are to be bolted together at the project site.

## DESIGN DETAILS

A complete change-over from riveted members to welded members requires an overall review and updating of design details. The primary rule of good design is to *specify only those attachments that are structurally required*. Following this, each detail should be examined for 1) simplicity, 2) potential stress risers (avoiding arrangements which restrict the flow of material under

stress), 3) over-welding (whether in weld size, weld length, or penetration requirement), and 4) classification by the proper fatigue category.

The publication *Bridge Fatigue Guide—Design and Details* describes the research and development of the AASHTO fatigue requirements. Details given in AASHTO Fatigue Categories D and E should not be used in areas of high tensile stress or where subjected to high stress reversal. To prevent lamellar tearing,<sup>10</sup> concentration of tensile stress in the thru-thickness direction should be avoided. At points where such stress cannot be avoided, non-destructive testing of the material at those points should be considered.

Research<sup>11</sup> has determined that the resistance of material to brittle fracture can be given in terms of fracture toughness (critical stress intensity factor) and of Charpy V-Notch (CVN) test values. These values, as now listed in the AASHTO Materials Specifications,<sup>12,4</sup> are mandatory requirements for various types of steel when used in members carrying *main tensile* stresses. Fracture toughness requirements depend on yield strength, material thickness, rate of loading, transition temperature, stress level and flow, and crack length.

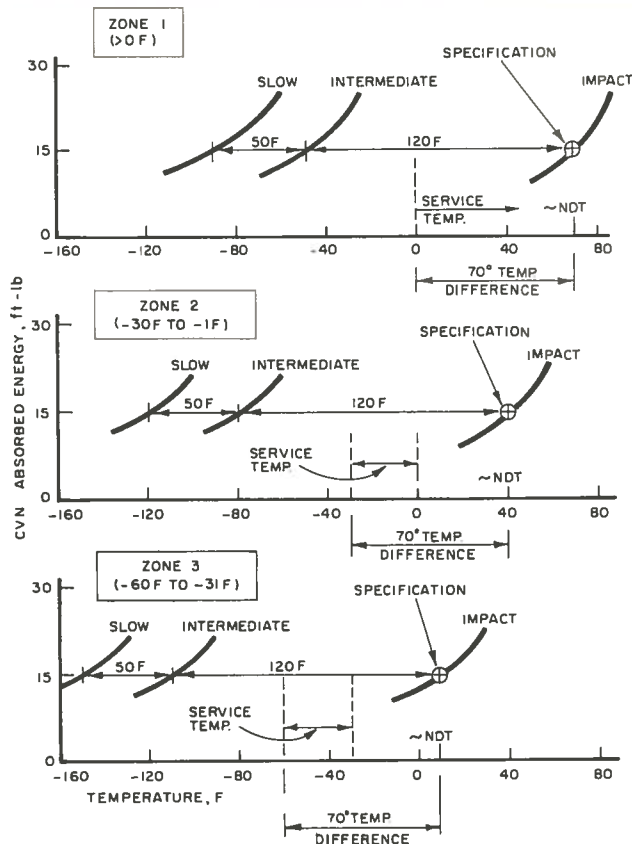
Allowable AASHTO impact values were selected to provide enough material notch toughness to make certain that any fatigue cracks, developed during the design life of components made in accordance with the AASHTO specifications, do not cause unstable fracture of the components. A good rule to follow for an economical design is: Specify only the level of notch toughness required; a greater degree of toughness adds very little to the fatigue life of the structure but will add considerably to material costs.

Increasing notch toughness does not significantly extend the structure's useful life because a fatigue failure may occur regardless of the toughness level. Moreover, to produce material with improved notch toughness may necessitate special processing (e.g., heat treatment) by the steel producer, thereby restricting availability, and imposing size limitations—particularly in the reduction of maximum plate length. Reduced plate lengths result in a need for additional welded splices, and these, in turn, are potential sites where fatigue cracks may initiate and propagate.

For this reason, reduced plate length may increase the probability for cracking. Furthermore, heat treating the base metal to obtain toughness higher than that given in AASHTO usually requires changes in welding parameters in order to ensure proper weldment properties. Because these considerations can have a significant effect on both design and cost of the structure, they should be studied and resolved quite early in the design stage.

As discussed later (see Identification of Material), only those main components which are subjected to tensile stress need to satisfy toughness requirements. For example, tough material can be required for the tension flange of a girder without requiring that the





**Figure 4.** AASHTO material toughness specifications for A36 steel with 65 ksi, or less, yield strength. (Reprinted from AISI publication *The Development of AASHTO Fracture-Toughness Requirements for Bridge Steels* by John M. Barsom of the USS Research Laboratory.)

compression flange also have the same high level of toughness.

An additional guideline is to select the CVN value based on the expected *service* temperature. Sometimes the CVN *test* temperature is erroneously taken to be the service temperature, which over-specifies toughness.

Research<sup>11</sup> has shown that the toughness as measured by the dynamic CVN test at a specified, higher *test* temperature than the *service* temperature, corresponds to the toughness needed for slower highway loading rate applied at the *service* temperature. This difference between test and service temperatures, which is based on "the loading rate temperature shift," is 70° F for material with 65 ksi, or less, yield strength (Fig. 4) and 30° F for material with 90 to 100 ksi yield strength.

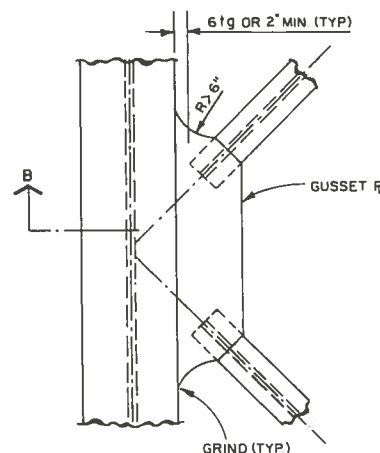
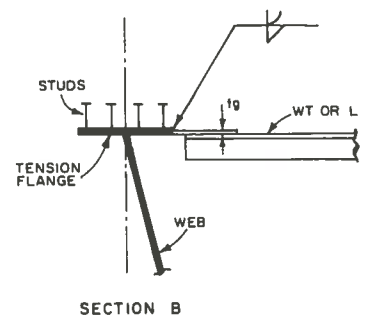
Proper design of details requires control of the stress level. Controlling the size of initial discontinuities is a requirement for proper fabrication practices—particularly those for welding and for oxygen cutting.

Most State Highway Bridge Engineering Divisions and many consulting engineering firms have developed standard drawings for frequently used details. There are additional details illustrated in other USS publica-

tions.<sup>13,14</sup> But, as noted earlier, no matter what the source may be, every detail used on design plans or on shop detail drawings should be carefully examined. The prudent designer will ask: Are all re-entrant cuts shown coped with a fillet? Are horizontal plates and stiffeners cut back four to six times the web thickness from intersections with vertical stiffeners? Do the welded details represent good welding practice? Are the details merely modified types of riveted details? And so forth.

The designer should be aware of the possible consequences of his detail designs. As an example, lateral connectors to the top flange of a box girder, were designed to be fabricated via a connector plate welded to the edge of the flange plate (Fig. 5). The radius coping of the plate—a means of avoiding stress risers—made this detail extremely expensive and difficult to produce. In all probability, bolting the connector plate to the flange would have proved a simpler and, thereby, cheaper solution.

Beyond the creation of an appropriate design, it is, of course, necessary to control the materials and work so as to make certain that the completed structure is in total conformity with the specified design. The procedures that are to be followed to achieve such control, must be included in job documents, i.e., specifications, special provisions, and as notes on the design drawings.



**Figure 5.** Bottom flange connectors with connector plate welded to the edge of the flange plate—a costly detail.

### III. CONTROL

#### TOLERANCES

A basic requisite for controlling a project is to establish procedures that guarantee adherence to accuracy in all structural components. When generally accepted industry practices are followed in the mills, fabrication shops, and on construction sites, variation from specified dimensions should be within limits regarded as acceptable. As to the magnitude of specific tolerances, ASTM Specification A6,<sup>6</sup> Chapter 5 gives dimensional tolerances for rolled or cast materials including mill camber and sweep.

Shrinkage forces are the principal reason for making it difficult to maintain dimensional accuracy in welded construction. Thus, more specific tolerances are required for welding than was formerly the case with bolted or riveted connections. The AWS Structural Welding Code<sup>3</sup> and the AISC *Quality Criteria and Inspection Standards*<sup>15</sup> contain dimensional tolerances for most types of welded structural members. These, in addition to the tolerances given in ASTM A6, should be adequate for most bridge designs. If, because of special conditions or considerations, closer tolerances are indicated, such particular requirements must be made part of the control documents. Here again, the designer should be aware that as tolerances are made more restrictive, the cost of fabrication increases substantially since extra work such as straightening, planing or other operations will probably be needed.

When considering any dimensional tolerances, it is important to remember the basically significant factor of cumulative effect. Thus, where the eventual overall dimension is critical, the design should include provisions for making a final adjustment, as, for example, by the use of shims, or other means.

In this context, a case in point would be that of a bridge with a concrete deck seated atop steel girders. While the concrete must have sufficient depth to perform its structural functions, the top surface of the concrete must be at the stipulated roadway elevation. However, the elevation of the top of the steel, that supports the concrete, is subject to variations in all of the following:

- elevation of top of pier,
- depth of bearing device,
- depth of girder at pier,
- camber of all girder shipping pieces.

For this situation, the simplest means of obtaining any needed adjustments would consist of a concrete haunch at each girder—making the haunch sufficiently deep to include the maximum total tolerance.

The use of high-strength bolts—in over-size or slotted holes—for field connections provides another means of making adjustments to compensate for the required mill and fabrication tolerances. Such holes, when used in accordance with Article 1.7 of the AASHTO Specifications,<sup>1</sup> will, very often provide economical connections since the higher cost of working to closer tolerances can be avoided. (Because of the high clamping action of high-strength bolts, the allowable stress per bolt is not reduced because of the oversize hole—except in the case of long, slotted holes.)

#### QUALITY CONTROL

Maintaining weld quality at critical locations is yet another significant factor in fabrication control. Standard acceptance requirements can be found in the AWS Code<sup>3</sup> as modified by the AASHTO Specifications for Welding,<sup>2</sup> and AASHTO Fracture Control Plan.<sup>4</sup> Since these requirements generally produce welds with a strength greater than that of the base metal, there are very few occasions when more restrictive acceptance criteria should be specified. There is an AWS Code<sup>3</sup> requirement for visual inspection of all welds for conditions such as cracks, complete fusion with the base metal and proper weld profile. More extensive proof of compliance with the acceptance criteria is usually accomplished by means of non-destructive testing (NDT) of specified welds.

Four non-destructive testing methods are presented in the AWS Code: 1) *radiographic* and 2) *ultrasonic* (UT)—to detect internal discontinuities; 3) *magnetic particle*—to detect discontinuities at or near the surface; and 4) *dye penetrant*—to detect surface discontinuities.

As compared to radiography, ultrasonic inspection is a poor substitute for evaluating the porosity in welds. On the other hand, the ultrasonic technique is far superior to radiography for detecting crack-type discontinuities. This clearly shows that the two testing methods complement one another, and can be used jointly for rigorous weld testing. As a matter of fact, both techniques are required by AASHTO for inspecting fracture-critical members.<sup>4</sup> Even when a structure is not fabricated to this Specification, radiographic along with ultrasonic examination may be worthwhile for critical members. Generally, however, either testing method is adequate when properly applied, if due consideration is given to thickness and joint configuration.

There are certain circumstances when it may be impossible to use radiography because of physical limitations on the placement of the film—hence, it cannot be used for corner or tee joints, or for joints of some complexity. Further, the radiographic technique cannot determine the depth of an internal discontinuity, requires expensive film, needs isolated areas during filming, and costs about twice as much as UT for the same weld inspection. Nonetheless, it offers the advantage of a readily available and permanent film record. UT, however, depends largely on the operator's skill to make a full weld inspection. While currently available UT processes do not make a permanent record of the inspection, ongoing research suggests future progress in this area.

The Structural Welding Code<sup>3</sup> does not specify which welds are to be inspected by the four non-destructive testing methods. This decision is left to the Engineer, since it is assumed that he knows and will designate

only those structural joints that are critical, highly restrained or highly stressed. The AASHTO Specifications for Welding<sup>2</sup> and the Fracture Control Plan<sup>4</sup> guide the designer by identifying the main member joints that should be tested by NDT methods; this is also done by the AREA Specifications.<sup>5</sup> Again, since unnecessary inspections simply add to costs, requirements for NDT should be specified only for welds at critical locations. For projects where the AASHTO or AREA Specifications do not apply, the project documents must specify the precise locations that require weld testing.

In all probability, the most effective quality control program consists of continuous inspection and/or checking throughout the entire work progression. Obviously, such a procedural approach calls for a staff with special competence: a thorough familiarity with 1) the requirements of the contract documents and, 2) the practical, mutually acceptable methods for verifying compliance. For example, the contractor must know when the owner's representatives will be available to inspect operations and which of the operations are to be witnessed by these inspectors. Similarly, the owner's inspectors must be kept current with the shop's scheduling of qualification, fabrication and shipping so that erection procedures may be planned. Such pre-planning and exchange of information will, of course, contribute considerably to the successful completion of any project.

The discussion in the following section is concerned with some of the more common shop operations. A basic familiarity with these operations is essential to exercise proper work control.



## IV. OPERATIONS

### MATERIAL IDENTIFICATION

At the outset, the first questions relating to a fabricator's operations concern the means by which the various grades of steel are identified, handled and properly assembled. In broad terms, fabricators will follow the procedures listed below; specific details will vary, however, because of the variations in the individual practices of each shop.

- 1) The fabricator will prepare bills for the required material in terms of size, length, quantity and specification requirements. Often known as "advance bills," they are consolidated into mill orders as purchasing documents to the producing mills. The *line or item number*, on any given mill order becomes the identifying mark for that material.
- 2) ASTM Specification A6<sup>6</sup> Requirements for Delivery, states that the mill must place its heat number, identification number, and size, including length, on the material. This information plus the customer's purchase order number appears on the mill test reports and bills of lading. For most structural steel specifications, except A36, the mill is also required to mark the specification number on the material along with its color code.
- 3) When material arrives at the fabricator's receiving yard, the "item number" — (see item 1 above) — is marked directly on the material and this is checked against the information on the mill order as well as with the information marked directly on the material by the mill. Before the material is stored, receiving yard personnel will usually mark the shop order number on the material.
- 4) By the time the preceding steps have been taken, shop detail drawings as well as bills of material for each drawing have been prepared. The item number by which the material was mill ordered is added to the proper line of each drawing's bill of material — often called the shop bill. Where the material is ordered from a warehouse or taken from stock, it must be completely identified.
- 5) When material is ordered into the shop — by item number for detail drawing number — from stor-

age, fabrication does not begin until the material markings are again checked against the information shown on the drawing's bill of material.

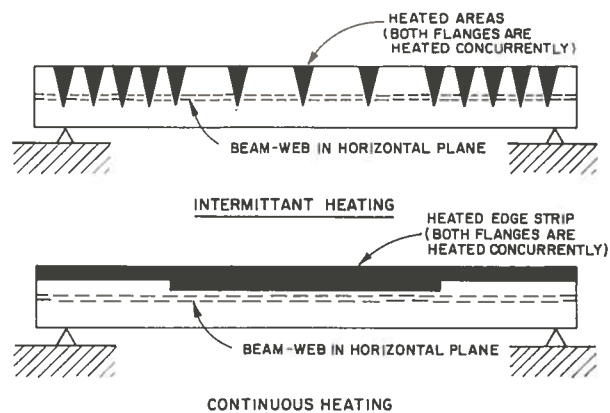
- 6) When main material is to be cut into smaller segments, each of the resulting pieces is marked with the shop order number, the detail drawing number, and the shipping piece number. Again, every piece of detail material, such as stiffeners, splice plates, and small attachments, is marked with the shop order number, the detail sheet number, and an assembly mark. The assembly mark is the same as that shown on the detail drawing for that detail piece.

To summarize: the mill order item number forms the link between the material as received from the mill and the material as listed on the approved detail drawing bill of material. The detail drawing number, shipping piece number, and assembly mark then act as the link between the material as fabricated and the material listed on the detail drawing.

This material identification procedure is more or less routine for most shops; it is an effective system that makes it possible to segregate material that meets special requirements such as toughness or any variety of strength levels. It also provides a key for tracing either a specific heat number, or, in some cases, a group of heat numbers if more than one heat is needed to make up an item. There are other available procedures for tracing a heat number for all main material. This is provided on specific request.

### BENDING AND STRAIGHTENING

While being subjected to a series of manufacturing processes, the steel member being worked may develop some random twists and bends. Frequently, these must be removed, particularly if the piece is to be welded, since the greater the uniformity of the fit-up, the more economical will be the weld. (There are, of course, other reasons for straightening material. Some designs may require bending or forming the material into certain shapes, such as camber or horizontal curvature for beams.)



**Figure 6.** Intermittent and continuous heat application zones on flanges for straightening or curving beams and girders.

Bending or straightening rolls, and a variety of press types are commonly used for “cold” forming, i.e., at room temperature. In this context, it may be noted that forming limitations derive from the thickness of the steel. As a general rule, for shop equipment (e.g., rolls, presses, shears and punches) the forming limitations will be more restrictive as the yield strength of the material increases—though not in direct proportion.

Among the various widely used methods of straightening or curving beams and girders is one that involves gas heating localized areas or edge strips of the flanges. The procedures and precautions for heat straightening and curving are specified in the AASHTO Standard Specifications for Highway Bridges.<sup>1</sup> Because of its inherent economies, heat curving is being used more frequently by highway bridge fabricators for producing horizontally curved girders. The most practical and economical processes used by fabricators are either the continuous or intermittent application of heat to the girder flanges (Fig. 6) to induce residual curvature after cooling. Either type of heating can be done without special equipment. Generally, however, intermittent V-type heating does not produce as severe a curvature as continuous heating. For small radii, it may be necessary to use the continuous heating operation.

In both methods, heat curving is performed with the beam web lying in a horizontal position and the beam itself simply supported at or near its ends. By this procedure, the effects of dead load assist in inducing curvature during the cooling process. The distance between supports used during heat curving horizontally positioned girders should be spaced so that the dead weight bending stress does not exceed the basic allowable stress of the material.

Note that each of the above methods will leave a pattern of residual tensile stress along the heated flange edges—as a result of heating and cooling. However, the presence of such stresses is anticipated in the design provisions and should not be a factor in selecting or rejecting a properly performed method.

To assist steel fabricators in making decisions on choosing a heating method, U.S. Steel has developed two publications: “Fabrication Aids for Continuously Heat-Curved Girders”<sup>16</sup> and “Fabrication Aids for Girders Curved with V-Heats”.<sup>17</sup> In these publications charts are provided which relate temperature, radius of curvature, girder geometry and support conditions. With proper use, these charts can assist the fabricator by showing the heat-curving conditions that will produce a beam curvature close to the desired curvature.

ASTM A514 or A709 GR100 and other quenched and tempered steels may be straightened by means of locally applied heat only if the procedure is rigidly controlled. In such cases, however, the Engineer must grant prior approval for the procedure. The AASHTO Specifications<sup>1</sup> prohibit heat curving or heat cambering of these steel grades to avoid any negative effect on their mechanical properties.

### EDGE PREPARATION

Another important operation consists of cutting material to a desired length and/or width. One reason for performing this in the shop is that material is frequently ordered from the mill in multiples of the needed widths and/or lengths. Second, since edges usually have to be straightened and prepared for welding, the cutting operations performed in the shop are often more economical than when done in the mill.

Oxygen cutting, by means of an oxygen-fed torch, is, perhaps, the most versatile method used for such work. The torch can be 1) portable, 2) mechanically guided, 3) manually, automatically or program controlled for movement, and 4) mounted vertically or at an angle for bevel cuts. Further, torches can be 1) used singly, 2) gang-mounted for simultaneous, multiple cutting (stripping) of wide plates, or 3) mounted in sets of two or three to cut and single or double bevel an edge simultaneously. Additionally, edges may be cut to a curve to produce such features as cambered webs, curved girder flanges, and web holes of various shapes. The resulting oxygen-cut edge normally meets a minimum roughness requirement of the American National Standards Institute (ANSI) 1000 micro-inches and has a minimal amount of slag to be removed from its corners. While light “knocking off” of the corners is often specified, this is done primarily for the purpose of permitting better paint adherence rather than for eliminating potential stress raisers. For material that will remain unpainted, such corner preparation is not needed. In virtually all cases, it is best to leave the methods of corner preparation up to the shop personnel.

A frequently asked question concerns the relative merits of ordering a UM (universal mill) plate in a desired width as against ordering a wide plate that will be cut—in the shop—into a number of plates of the desired widths. At the present time, the mills produce UM plates with rounded corners, and with sufficient straightness to meet most requirements. Often, longer

lengths are available in UM, but, just as often, the desired widths and thicknesses may not be available. Cut plates usually are a bit straighter than UM plate—to such a degree that they can be transverse groove-welded to adjacent plates before cutting. This can provide more economical welding but the corners of the cut edges must be cleaned of slag, etc.

To summarize: Unless the edge must be of a specific type, the plate type ordered should be left to the fabricator's option.

Beams and channels are cut to length with band saws; high-speed friction saws are used for beams up to 120 lb/ft; and cold saws are used for beams up to 750 lb/ft. (Cutting with friction saws will produce edge burrs that require removal.) Cutting by these means will produce ends within close dimensional tolerances and with a surface smoothness of about ANSI 700 to 800 micro-inches. Unless otherwise specified by the designer, the cut ends will not require further milling or finishing.

### PREPARATION OF BOLT HOLES

The preparation of holes is an important operation, and requires careful attention on the Engineer's part. Integral to this procedure are the techniques of shop assembly and reaming (SA & R) for multiple-ply holes, or reaming to a templet (RT) of non-assembled material. While specifications for such preparation and assembly were originally developed to accommodate riveted construction, the methods were not changed significantly with the introduction of high-strength bolts—now in common use both for shop and field connections. Specified 1) sub-punching or sub-drilling of holes (or sub-burning when allowed), 2) reaming, and 3) shop assembly for reaming, are all additional shop operations which, obviously, increase shop costs. A modern substitute for such multiple operations is available in numerically controlled (N/C) drilling equipment that can provide full-size drilled holes that are positioned with a guaranteed degree of accuracy.

AASHTO Specifications<sup>1</sup> were revised in 1975 to include the N/C drilling operation and a reduced shop assembly requirement. While the older procedures produced customized, match-marked members, the introduction of N/C equipment has resulted in standard, interchangeable members with better-shaped holes that have little chance of initiating cracks. Truss gusset plates can now be readily fabricated with proper geometric angles for field assembling members with cambered lengths that greatly reduce secondary stresses.

### MOVING MATERIAL

One of the principal requisites of a well-run shop is a safe, efficient and flexible system for handling and moving material. Overhead, gantry, and jib cranes, rail-mounted dollies, positioners of various kinds, conveyor tables, railroad cars, and straddle carriers are some of

the usual types of equipment for moving material. Lifting, turning, moving, or fitting often requires a number of temporary attachments; often referred to as fitting aids, these attachments are particularly useful for components of welded members. AWS Structural Welding Code<sup>3</sup> regulations cover the application and removal of temporary welds for these aids. Since additional restrictions placed on the use of such aids increase fabrication costs, this should be avoided wherever possible. If, however, special restrictions should be necessary, these must be included in the project plans or specifications.

Obviously, eliminating operations that involve needless moving of material will make for greater economy. Reducing shop assembly to a minimum of check assemblies through the use of N/C drilling equipment is one example of how to avoid material moving and cut costs.

### WELDING

After the material has been prepared and fitted, the next shop operation is welding. As discussed previously (see Welded Joints), manual shielded metal arc welding (SMAW), submerged arc welding (SAW), and flux-cored arc welding (FCAW) are the more common processes used in bridge construction. Each procedure may be "pre-qualified" if it adheres to the AWS Structural Welding Code<sup>3</sup> bridge construction requirements for joint design, workmanship, and technique (as modified by the AASHTO Welding Specifications<sup>2</sup>). When a procedure is pre-qualified, it may be used without performing additional qualification tests. This avoids unnecessary operations, and reduces costs. Welding procedures for fracture-critical members are not pre-qualified.

As the name implies, manual shielded metal-arc welding is entirely a hand operation using 14- or 18-inch long electrodes. Only a minimum amount of equipment is needed, and it is best suited for situations involving considerable movement; also for welds in lengths less than about 2-ft long and/or thicknesses of  $\frac{3}{8}$ -inches. The process can be used in all positions for  $\frac{3}{16}$ -inch fillet or groove welds as long as 10 ft. The other processes, submerged arc welding (SAW) and flux cored-arc welding (FCAW), are used as semi-automatic operations with the electrode in coil form, thereby eliminating the need for frequent electrode changes, and thus gaining more efficient use of the welder's time. Moreover, since the electrode feed rate is mechanized, this permits the use of higher welding currents that produce a higher deposition rate. Note that these processes are better suited for heavier and longer welds than is the manual shielded metal-arc process. However, their high deposition rate limits use of these processes to the down-hand (flat) or horizontal positions.

There are a number of other acceptable welding processes that are particularly useful in certain situations; these include gas metal-arc (GMAW), electroslag (ESW), and electrogas (EGW). The Engineer should be

aware of improvements in welding techniques, and of new processes introduced.

For the most part, all processes produce satisfactory welds. This being so, it becomes yet another instance where the fabricator—with his capabilities and expertise—should be entrusted with the task of choosing a process subject, of course, to the purchaser's approval.

### SURFACE CLEANING

Until recently, hand and/or power cleaning were specified as sufficient surface preparation for oil-base paints. Grinding, to remove all mill scale, was the common method used to prepare surfaces for welding.

But, because of the movement away from oil-base paints, and the increasing use of alkyd, zinc-rich and epoxy paint systems, specifications are more often calling for blast cleaned surfaces. The degrees of blast cleaning, normally required, are "commercial" or "near-white" as defined by the Steel Structures Painting Council (SSPC);<sup>8</sup> see also Surface Preparation (p. 11). The mounting demand for blast cleaning has

prompted the concept of performing this operation on plate material *before* the start of fabrication. This operation, commonly referred to as *pre-blasting*, can be used to good advantage when the fabricator's facilities are suitable for the procedure.

To reduce the amount of follow-up blast cleaning prior to shop painting, some shops, when permitted by the purchaser, have been pre-treating the surfaces immediately after blast cleaning. Obviously, the pre-treatment coating should be compatible with the specified paint system and need not be removed. A vinyl wash, or wash primer, similar to the military specification, MIL-P15328B, or a weld-through zinc-rich shop primer, is often used. Such pre-treatment coatings are not harmful to the welding of attachments and are also useful as anti-rust protection for field contact surfaces. Pre-blasting, and pre-treatment coatings, may also reduce the costs for other operations such as edge preparation and joint preparation for welding. Full consideration should be given to requests for the use of these methods.

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