# Designing Longitudinal Welds for Bridge Members

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## INTRODUCTION

By far the greatest amount of welding in all structures consists of longitudinal welds used to join components of axially loaded members so that they will act in unison to form effective and stable structural members. Many designers opt to use complete joint penetration groove welds "just to be safe" when other weld types or combinations of welds would be a better choice in terms of limiting residual stress and distortion, avoiding lamellar tears, and reducing the cost of welding and fabrication. Since all properly designed continuous longitudinal welds of good quality have the same static life and same fatigue life, why do designers think it is safer to use complete joint penetration groove welds?

This paper examines the choices available to engineers in designing longitudinal web-to-flange welded joints in I-shape and box-shape members and endeavors to briefly discuss materials, design, and welding information that affects that choice.

## EVOLUTION OF DESIGN

The design and detailing of welded bridges began in the early 1940s as a simple conversion of riveted structural steel details and philosophy to mirror-image weld details. Fillet welded/riveted (converted) details did not work in the bridge fatigue environment. It soon became apparent to bridge engineers that flanges of built-up members should not be constructed in multiple layers like riveted members, but rather of single plates joined by fillet and groove welds, as appropriate. Knowledgeable designers also learned that, to avoid fatigue cracks and brittle fractures, groove welds subject to tension normal to the weld throat must be welded completely through their cross section and have no significant weld defects such as lack of fusion, slag, porosity, or cracks.

For over 30 years, bridge welding specifications have contained general proscriptions against details that concentrate stress in weld and base metal. The AWS *Specifications for Welded Highway and Railway Bridges,* AWS D2.0-56, states in the foreword, "Fatigue testing has demonstrated that any sudden discontinuity of section and stress path is a factor adversely affecting the strength of members subject to cyclic loading. Gradual rather than sudden transitions of sections should be employed and for the same reason groove welds are preferable to fillet welds."[1] This advice continues to be printed in AWS specifications to this day except that the Code now states, ". . .welds in butt joints are preferable to fillet welds."[5] Unfortunately, although well-intentioned, this statement is misleading, widely misunderstood, and often the cause of poor design details. Longitudinal fillet welds are no better or worse than longitudinal groove welds in fatigue.[2,3,4,8] It depends on the quality of the welds and how the fillet or groove welds are loaded. The AWS preference for groove welds in butt joints is based upon fatigue testing of lap welded details done 40 years ago.

Misunderstanding of the historic AWS preference for groove welds over fillet welds has, when applied to longitudinal welds, led to costly overwelding and, in some cases, distortion, and lamellar tearing. Welds should be large enough to carry the stress with appropriate safety factors and large enough to produce welding heat inputs that are sufficient to ensure weld soundness, but groove welding of web-to-flange connections in axially loaded members is not necessarily better than fillet welding and may be worse.[8,9]

## FACTORS AFFECTING FATIGUE PERFORMANCE

Good fatigue life or poor fatigue life depends solely upon the presence or absence of discontinuities normal to applied tensile stress that can significantly amplify localized stresses. Fatigue crack initiation and growth occur when localized tensile stresses at the tip of a flaw cause the metal to deform plastically under the influence of applied stress. Without a stress concentration, there is no plastic deformation and no fatigue cracking under bridge design loads.[2]

Flush-ground complete joint penetration groove welds with no significant internal discontinuities are better in fatigue than any other weld with surface contours, weld defects, or an unwelded portion of the joint that, because of an orientation normal to applied tensile stress, may serve as a stress raiser. However, when the load transmitted between connected parts is shear parallel to the weld throat, as in the case of web-to-flange connections in welded plate girders or box girders, fillet welds perform as well as partial or complete joint penetration groove welds.[8,9]

The effect of weld orientation, surface profile, frequency, size and shape of internal discontinuities, residual stress, and transfer of stress at the termination of welded details is the basis for differences in fatigue performance and the establishment of stress range categories in the AASHTO *Standard Specifications for Highway Bridges.*[4] AASHTO design specifications have until 1987 designated continuous web-to-flange welds in bridge members as Fatigue Category B regardless of whether the welded connection was made using

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two fillet welds, two partial joint penetration groove welds, or one complete joint penetration groove weld. The 1987 AASHTO *Interim Specification* has introduced a new stress range Category B′ which modifies that position somewhat based upon anticipated weld flaws. Research conducted in Japan and England[8,9] is the basis for the new stress range category proposed by Fisher and Keating[8] and adopted by AASHTO.

All continuous sound longitudinal welds attaching components of axially stressed members have equivalent fatigue lives because the unwelded portion of the welded joint, if any, is parallel to, not perpendicular to, the applied tensile stress. Discontinuities normal to applied stress are a function of workmanship and may or may not justify a new stress range Category B′ depending on how carefully workmanship and inspection are controlled.

## WELD DESIGN CHOICES

Bridge designers have traditionally been given the choice of making welded attachments by using either fillet or groove welds (Figs. 1a and b). In earlier times there was also a choice of intermittent or continuous welds regardless of type. This choice is no longer available. All welds are required to be continuous to assure good fatigue performance. When designers consider groove welds, they generally think of complete joint penetration welds (CP welds). Complete joint penetration groove welds may be designed as butt joints, tee joints, or corner joints, depending on the load and orientation of parts. In addition to complete joint penetration groove welds, when the stress to be carried is shear between axially loaded parts, the designer should also consider using partial joint penetration groove welds with reinforcing fillets or just fillet welds. The AWS advice that ". . .Welds in butt joints are preferable to fillet welds" refers only to welds stressed normal to the weld throat. Partial joint penetration groove welds stressed normal to the weld throat are not necessarily better than fillet welds.

Think of partial joint penetration groove welds (PP welds) as "inside" fillet welds. Partial joint penetration groove welds have proven to be vastly superior to large fillet welds. All tee and corner groove welds in bridges are required to have reinforcing fillets to improve the contour of the weld surface and, thereby, avoid stress raisers. Partial penetration welds require less weld metal and produce less shrinkage and distortion than fillet welds of equivalent strength. In fact, because of their different effective throats (Figs. 2a and b), partial penetration welds are 1.4 times as strong as fillet welds for an equal weight of weld metal.

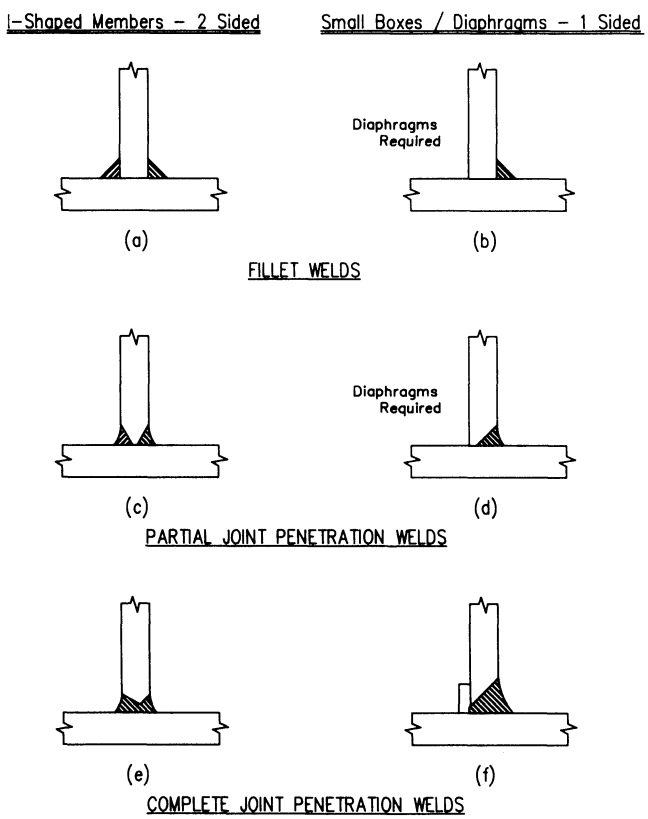


*Fig. 1a. Longitudinal weld design choices—tee welds.*

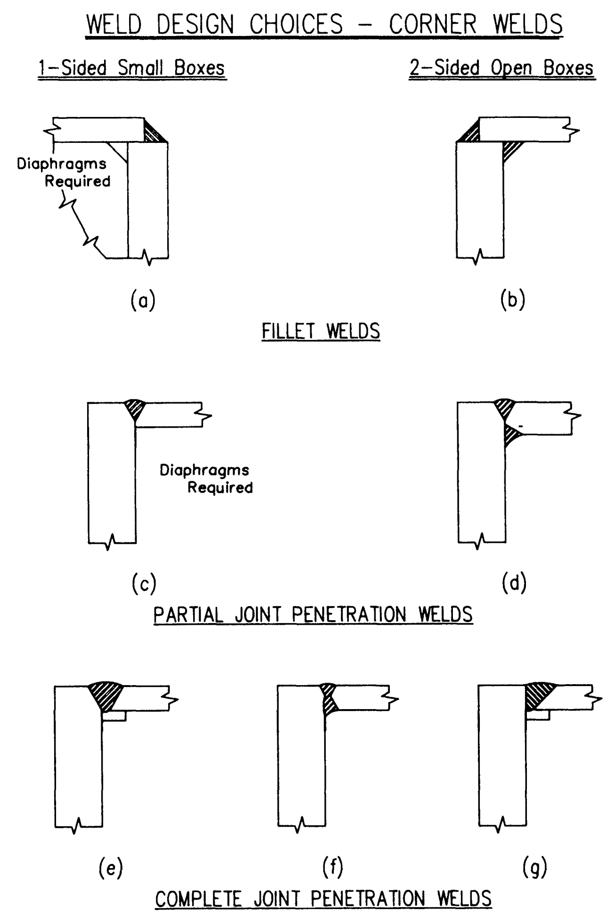


*Fig. 1b. Longitudinal weld design choices—corner welds.*

## DESIGN RELATED DISCONTINUITIES AND FATIGUE PERFORMANCE BASED ON WELD TYPE

It is the nature of fillet welds and partial joint penetration groove welds to have an unwelded portion of the joint between the welds. Even complete joint penetration groove welds with fused steel backing have an unwelded portion of the joint (Fig. 3). Any unwelded area will produce a serious stress concentration if stress is applied normal to the plane of the unwelded segment. However, when the stress to be transmitted is shear parallel to the weld throat, the unwelded area between longitudinal fillet or groove welds or adjacent to welds with backing concentrates no applied stress and has no adverse effect on fatigue life. Partial penetration welds are "inside" fillet welds. They, like fillet welds, transmit shear stresses on the weld throat and are oblivious to unwelded portions of the joint in between or beside the weld. Welded beams with cover plates survive millions of cycles of design load without distress, provided the fillet welds are subject only to shear. When fatigue cracks occur, they always initiate at the termination of the cover plate or at a significant flaw in the weld. Cracking at the end of cover plates is caused by yield point residual stresses at the weld termination plus the effect of dynamic stress concentrated by the abrupt change in contour at the end of the cover plate. Cover plates attached full length have no fatigue cracking problems if the welds are sound.

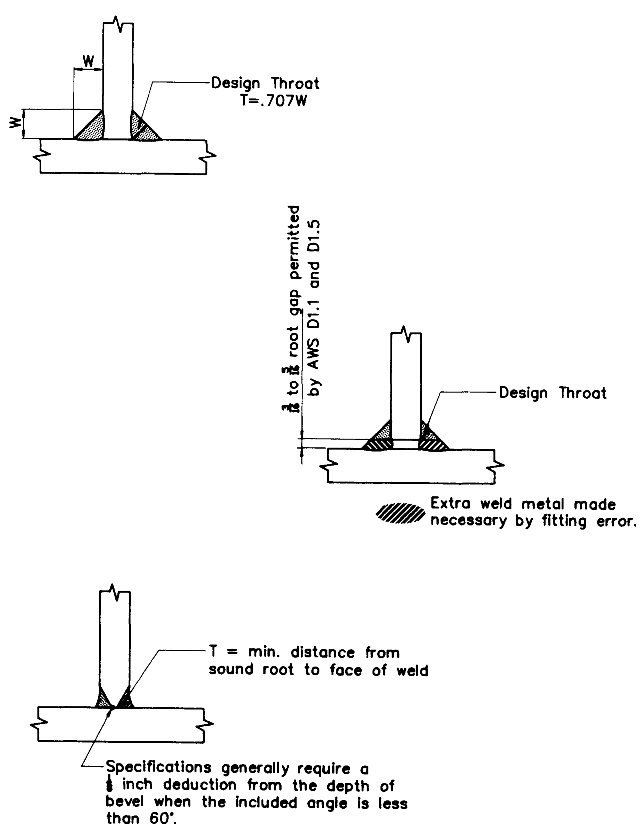


*Fig. 2a. Effective weld throats.*

Fatigue testing of axially loaded box members joined at the corners by one-sided fillet, J-groove, bevel groove and complete joint penetration groove welds has revealed that

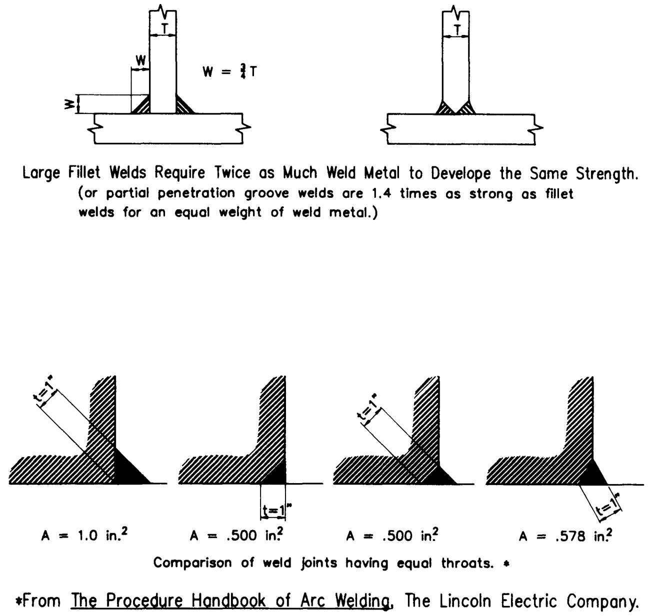


*Fig. 2b. Weld metal requirements.*

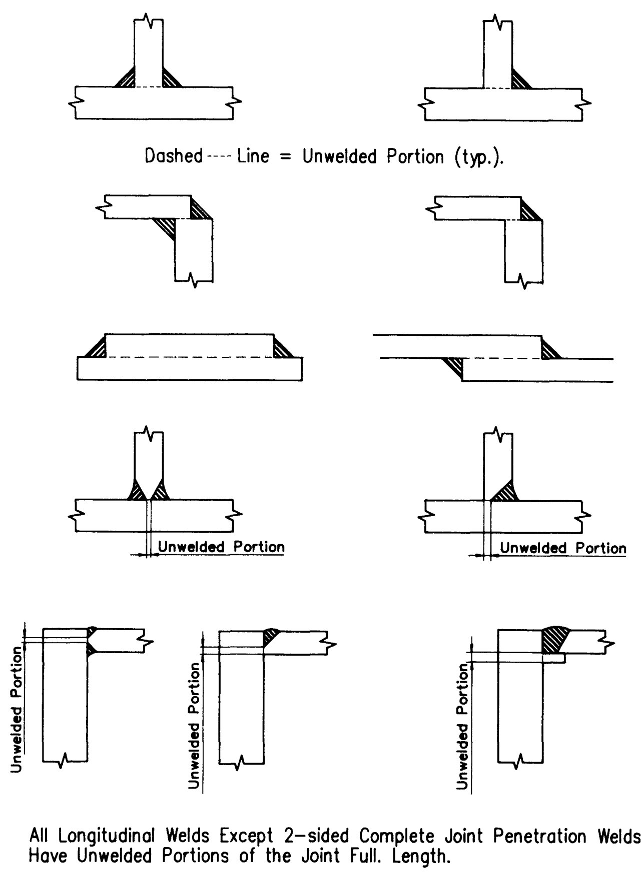


*Figure 3*

all have equivalent fatigue lives when they have equivalent weld soundness and surface roughness.[9] The unwelded portion of the joint parallel to the applied stress has no effect on fatigue strength.

The fatigue life of longitudinal welds in bridges is based upon the soundness of the welds, i.e., freedom from internal defects and excessive surface roughness.[9] Since there is no relationship between the severity or frequency of weld defects and the type of prequalified detail of welded joint, all properly designed continuous longitudinal welds should be considered equivalent in fatigue. Weld type does not predispose differences in surface roughness or weld defects.

Many competent bridge designers believe that complete joint penetration groove welds are inherently better in fatigue in all cases because they are stronger. The new AASHTO stress range Category B′ appears to add weight to that argument, but it is not true for longitudinal welds in axially loaded or bending members. NCHRP Report 286[8] demonstrates that fillet welds are equivalent to complete joint penetration groove welds used to make tee and corner welds in box members. Unfortunately, the presence of "blow holes" in partial joint penetration groove welds led researchers of the Honshu-Shikoku Bridge Authority[9] to conclude that partial penetration welds were slightly less desirable than other welds because of a propensity for weld defects.

It is also unfortunate that research in Japan, testing steels with unusually high strength ($\sigma_{ys}$ = 112 and 125 ksi), under conditions of high stress range (18.6 to 33.4 ksi) and fast loading (120 to 300 cpm), resulted in modifications of AASHTO stress range categories that have been used successfully for years. The SMAW and GMAW procedures used in the research do not reflect procedures used to construct welded bridge members under provisions of existing Codes.[5] These tests of high strength steels do not accurately predict the performance of 50 ksi AASHTO steels subject to real bridge loads. Most states do not permit the use of manual SMAW electrodes to make continuous longitudinal welds, thereby introducing frequent surface flaws, and most states do not use the GMAW process because of concern for internal flaws. When welding is done by properly controlled SAW procedures, all longitudinal weld types perform essentially the same, as AASHTO stress range Category B has properly reflected. Weld joints with fused steel backing may or may not have reduced fatigue life, depending on the quality of workmanship. The key to success or failure of a welded joint continues to be the presence or absence of significant workmanship or design-related discontinuities normal to applied tensile stress. The changes in stress range categories for longitudinal welds are based upon the quality of workmanship and not the type of joint.

Some engineers reading NCHRP Report 286 and results of the fatigue tests conducted by the Honshu-Shikoku Bridge Authority[8,9] appear to have concluded that fillet welds are less subject to weld defects than partial joint penetration groove welds because there is more access for welding. This is not so. There is a big difference between theoretical weld access and actual weld access due largely to root conditions that vary because of fit-up tolerances and errors. If there is already sufficient access for proper welding, more access doesn't improve the weld; it just concentrates residual stress and wastes money. Groove weld joint details should require the minimum amount of weld metal consistent with dimensions that permit access for welding. Fit-up openings in the weld root increase access, but they may also increase the difficulty of welding for other reasons.

## SIMILARITIES BETWEEN FILLET AND GROOVE WELDS

Bridge welding specifications require that all mill scale be removed from web-to-flange welds. Thus the surface on which welds are deposited is essentially the same whether the weld is deposited as a fillet or groove weld. There is no metallurgical difference between a fillet weld and a groove weld made by the same process under the same procedure controls. This is clearly stated in the AASHTO/AWS Bridge Welding Code, Sect. 5.5.3.

The preparation and assembly for welding of butt, tee, and corner joints provide essentially the same welding conditions for fillet and groove welds under the provisions of AWS and AASHTO Codes.[5,6] The exception is that certain complete joint penetration groove welds may be made from one side using fused steel or unfused nonmetallic backing. Fused steel backing has been used for many years. When used properly, it is very effective, but when used improperly, it may cause cracking. Unfused nonmetallic backing is currently the subject of an NCHRP research project[7] designed to identify or develop and verify the performance of suitable unfused weld backing for bridge applications.

Both fused steel backing and unfused nonmetallic backing must permit the repair of all portions of a weld from the outside (weld side) of the joint. Welds may be made from one side using fused steel backing just for convenience where there is no chance that root discontinuities might initiate fatigue cracks. One-sided welds without fused steel backing are presently prohibited by the AASHTO/AWS Bridge Welding Code, paragraph 9.12, unless the joint using unfused backing is qualified by test.

The advantage of fused steel backing is that it is easily attached by tack welding and facilitates the positioning of the parts to be joined. When tack welding is done inside the joint and the tack welds will be remelted by subsequent submerged arc welds, tack welds can be made without preheat and the quality of the tack welds has no effect on final weld performance. When tack welding of backing is done outside the joint, the quality of the tack welds and the heat affected zones they create are critical to fatigue performance. Extreme care must be exercised in preheating and welding. External welds must be continuous, and preheats higher than required by Table 4.2 of the Codes[5,6] may be required to avoid cracking. If external tack welds were made without preheat, remove the temporary tack welds and ⅛ in. of base metal by grinding to ensure that both the weld and hardened heat affected zones are no longer present to initiate fatigue cracks. Do not tack weld outside the joint unless absolutely necessary.

The 1987 AASHTO Interim Specifications[4] separate longitudinal complete joint penetration tee and corner welds into stress range Categories B and B′, depending on whether or not fused steel backing is removed after welding and the root of the weld finished by grinding to remove surface defects. For two million cycles of load, the stress range for redundant members is 18 ksi for welds without backing and 14.5 ksi for joints with backing in place. Proportional reductions in stress are specified for nonredundant fracture critical members. These reductions in stress are specified for partial joint penetration groove welds and complete joint penetration groove welds with backing.

Complete joint penetration longitudinal welds made using fused steel backing as described in AWS Joint Designations B-U2, C-U2, B-U4 or TC-U4,[5] i.e., all joints with fused steel backing, are similar to fillet welds and partial penetration welds in the sense that all incorporate a continuous longitudinal discontinuity (Fig. 3), which, if parallel to the applied stress, will have no adverse effect on fatigue life. The Codes[5,6] specify that all fused backing normal to computed stress shall be removed and the joint ground smooth. Steel backing need not be removed from welds parallel to the applied stress, from tee welds in compression or from compression welds in columns. Unfused backing that produces a smooth weld root should not have to be removed. However, until effective unfused backing materials and methods are identified and verified by research,[7] unfused backing must be qualified by test.

The appropriate stress range category for longitudinal welds made with fused steel backing depends on the quality of the workmanship regardless of whether the backing is removed or not. Flaws created by careless backing removal may make the situation worse, not better. It is difficult to conceive how backing can be removed properly without damage within a small box member.

Welded plate girders constructed in the traditional I-shape may have web-to-flange connections made by fillet welding, partial joint penetration groove welding, or complete joint penetration groove welding (Fig. 1a). Welding should be done from both sides of the web and the weld sizes on each side should be balanced unless there is a very good and unusual reason not to.

Economies are not achieved by making fillet and groove welds of minimal size. Imprudent reductions in size of longitudinal welds often result in greater costs for repair, handling and maintenance. It is far better to make generously proportioned, sound welds that do not require repair and any special care or handling. Automatic submerged arc welding produces the best weld quality and lowest weld cost.

If the design of the structure is such that significant out-of-plane bending stresses are imposed on the web, the web should be expected to crack regardless of the type of weld used to join web to flange. Fillet welded connections subject to out-of-plane bending might possibly crack slightly sooner than partial or complete joint penetration groove welds.

Fillet welds are the absolute best choice for routine web-to-flange connections when there is access to both sides of the web and shear stresses permit the weld size to be ⅝ in. or less. Fillet welds are best because they are very effective and represent the least cost. When fillet welds will not do, partial joint penetration groove welds are next best. Complete joint penetration groove welds should be used only where there are unusual applied stresses, concentrated loads, or significant stresses not parallel to the weld throat. When CP welds are required in this latter condition, they should be made by welding from both sides of the web, back-gouging to sound weld metal before welding the second side. A CP weld that employs steel backing creates concern about workmanship and is not appropriate when there is access to weld from both sides of the joint.

The AASHTO Bridge Specifications[4] do not mandate specific requirements for the design of web-to-flange welds in I-shape members. However, they do specify requirements for the same weld in composite box girders. The engineer should design longitudinal welds that join web to flange in I-shape members observing the following guidelines.

1. Use fillet welds on each side of the web until the design fillet weld exceeds ⅝ in. (⅝ in. is not an absolute maximum, but it is a good point to consider the use of partial penetration welds. Remember, actual fillet weld sizes may have to be increased due to fit-up errors.)
2. Use partial joint penetration groove welds with reinforcing fillets, as required by the Bridge Welding Code,[6] on each side of the web when fillet welds would be too large.
3. Use complete joint penetration groove welds produced by welding from both sides of the web when design stress will not permit option 1 or option 2. Welds requiring backing should not be considered.

The weld metal used to join web to flange should conform to the provisions of Sect. 4.1 of the Bridge Welding Code[6] and need not match the base metal in strength since the welds are loaded only in shear. The reinforcing fillets required by the Codes[5,6] are specified to be ¼ the thickness of the web but not more than ⅜ in. in size.

## DESIGN CHOICES FOR BOX MEMBERS

Any discussion of the design of longitudinal welds required to join web to flange in box members should first consider the type of box member to be constructed. Box girder highway bridges are generally built utilizing large open box sections designed for composite action with the concrete bridge deck. Some railroads prefer large closed box girders they call "unitized" girders that can be erected quickly as single erection pieces supporting ties and rails directly on the top flange. These heavy box girders are particularly useful in situations demanding quick bridge replacement.

Truss bridges are generally built using small box members that are bolted together at connection points to form continuous top and bottom chords separated by vertical and diagonal "web" members. Small and large boxes are also used in the construction of arch bridges, towers, and bents.

Large boxes should be considered separately from small boxes because they are very different in most cases. The designation "large" indicates that there is adequate room for welders and inspectors to enter the box, as necessary, to perform their respective tasks. When constructing "small" boxes, you may be able to look inside, but you generally cannot get inside.

Box members should also be broken down into "open" and "closed" depending on whether or not you can see inside the box from the side, top, or bottom. Large, open, composite box members have excellent access for welding during fabrication and erection, but permanent access to the inside of the box is generally precluded by placement of the concrete bridge deck. Small, closed boxes generally allow no access for longitudinal welding on the inside except that two corners may be welded on the inside before the box is closed.

Designers should make every possible effort to ensure that welders are not required to work inside closed boxes unless they are very large. Welding and inspection cannot be done properly where there is inadequate room to move, where there is poor lighting and ventilation, or where high preheating temperatures make working uncomfortable, dangerous, or both.

The longitudinal welds that join web to flange in box members may be either fillet welds, partial joint penetration groove welds, or complete joint penetration groove welds, just as in I-shape members. The major difference is that in small, straight box members, longitudinal welds need not be placed on both sides of the webs to avoid a concentration of tensile stress at the weld root, and CP welds are not required either as long as a sufficient number of rigid internal diaphragms are used. At NYSDOT it was common to space diaphragms at about 12-ft centers. This facilitated fit-up and took care of all forces that might warp the box cross section. It permitted all welding to be done from the outside. Since diaphragms were attached by high strength bolts, box members designed and constructed using only continuous longitudinal welds were considered stress range Category B.

AASHTO[4] gives no specific instructions for the design of web-to-flange connections in small box members but states in Sect. 10.39.5, under the heading "Composite Box Girders": "The total effective thickness of the web-flange welds shall not be less than the thickness of the web. If fillet welds are used, they shall be on both sides of the connecting flange or web plate." Since most, if not all, composite box girders are constructed as open boxes with access for men and equipment inside the box during welding, this is good advice. However, if this requirement for weld size and symmetry is applied to all box members, some superior methods of designing and constructing welded box members are eliminated.

Figures 2a and 2b show 13 different methods of welding the corners of large and small box members. Not all are recommended, but they have been used. When there is access for welding inside large boxes, the welds joining web to flange should conform to AASHTO,[4] Sect. 10.39.5. The welds should be balanced on each side of the web using two fillet welds, two partial joint penetration groove welds with reinforcing fillets, or, in some very unusual cases, a complete joint penetration groove weld with reinforcing fillets. Groove welds requiring backing should be avoided.

Balanced welds give stiffness to the box corners and prevent the concentration of stress in an unwelded portion of the weld root. The Bridge Welding Code[6] states:

> **9.11 Corner and T-Joints**
>
> 9.11.1 Corner and T-joints that are to be subjected to bending about an axis parallel to the joint shall have their welds arranged to avoid concentration of tensile stress at the root of any weld. (This refers to forces tending to rotate the connected parts about the weld axis creating tension on the weld throat and does not refer to bridge members in axial bending.)
>
> 9.11.2 Corner and T-joints parallel to the direction of computed stress between components of built-up members designed for axial stress need not be complete joint penetration groove welds. Fillet welds or a combination of partial joint penetration welds and reinforcing fillet welds may be used.

Section 9.11 covers all web-to-flange joints in I-shape and box-shape members, large and small. It parallels AASHTO[4] 10.39.5 and Sect. 9.12 of the Bridge Welding Code[6] that provide no weld shall be made that may be subjected to a concentration of stress in an unwelded portion of the weld joint. Concentration of stress may be avoided in longitudinal welds by balancing the welds on each side of the web or by fixing parts against rotation by use of frequent diaphragms. Figures 1a (a,c,e,f) and 1b (b,d,e,f,g) show balanced or full cross section welding. Figures 1a (b,d) and 1b (a,c) show the one-sided weld plus diaphragm method. Since diaphragms are generally necessary and desirable for all box construction, it is appropriate to detail a few extra diaphragms so that small boxes can be welded completely from the outside without wasting weld metal.

The design of corner welds and spacing of diaphragms should consider all possible torsional loads that may warp the box and concentrate stress in the root of one-sided tee or corner welds. Torsional stresses may result from fabrication, erection, or service loads. Consideration of torsional stress should include an estimate of the maximum stress, the stress range, and the number of cycles that will be applied. Obviously, significant fatigue stresses are more important than one-time-only erection stresses.

Most small straight boxes have no applied torsional stresses. These boxes are covered by 9.11.2[6] and can be welded effectively using one-sided fillet welds for small members and partial joint penetration groove welds with reinforcing fillets for almost all others. Rarely will applied stress require complete joint penetration groove welds if diaphragms are used.

Significant torsional forces encountered in large curved box girders and in girders subject to torsional displacements due to loads in adjacent spans may justify complete joint penetration groove welds at box corners even though rigid diaphragms and cross frames are used at points of concentrated load. All other web-to-flange connections should be made using fillet or partial joint penetration groove welds, in that order. Fillet welds are only effective when the web is set back from the flange edge a sufficient distance to permit welding on each side.

## LAMINATIONS AND LAMELLAR TEARING

Design of longitudinal web-to-flange welds should consider inadvertent base metal separation that might result from the weld design or selection of the welding process. Avoid unnecessarily large welds of all types when not justified by calculated stress; avoid large fillet welds; use ductile low yield stress weld metal when the design permits, and avoid hydrogen. These provisions and the quality of the welds are much more important to good fatigue life than the type of longitudinal weld.

## WELD DISCONTINUITIES

Five major types of defects may be found in welds: (1) inclusions (generally nonmetallic); (2) gas defects (spherical or elongated "piping" porosity); (3) lack of fusion (LOF); (4) lack of penetration (LOP) (Since PP welds require a specific depth of fusion to meet design requirements, failure to penetrate and fuse to the specified root depth is considered a weld defect in this case.); and (5) cracks. The type, size, and frequency of weld discontinuities is not related in any way to a specific type of web-to-flange weld design. It is not the joint; it is the quality of the weld itself. The problem may, of course, be the welders. Weld type, size, and strength can affect lamellar tearing. The types of weld discontinuities and the frequency of discontinuities is completely dependent upon the thoroughness of preparation for welding, selection of the welding process, and control of the welding procedure.

## COST COMPARISONS BASED ON WELD TYPE

The cost of a welded product is, in general terms, the sum of the cost of labor, materials, equipment, and overhead.

When comparing weld types, the most significant differences in cost are based upon two principal factors, the cost of welding and the cost of preparing the joint for welding. The rates charged for labor, equipment, and overhead are the same regardless of weld type. The base metal costs the same. Welding and cutting or gouging consumable costs are proportional to the amount of work, preparing for welding, and actual welding that must be done. The cost of preparing for welding by cleaning surfaces and preheating is about the same. The difference in the actual cost of welding each type of longitudinal weld is, therefore, based only upon differences in the required volume of weld metal and differences in the extent and difficulty of doing the work required to complete the welds.

*Weld Volume.* Fillet welds theoretically require twice the volume of weld metal that partial joint penetration welds need to carry the same load under ideal conditions. Small fillet welds and PP welds up to $\frac{5}{16}$ in. in size are detailed to provide sufficient welding heat input to ensure weld soundness. Shear stresses rarely control the size of these small welds. Large fillet welds, which are rarely used in good designs, actually require twice as much weld metal as PP welds of the same strength.

*Beveling and Backing Costs.* There are no separate assembly or beveling costs for fillet welding. Partial joint penetration groove welds require two extra oxygen cuts full length of the member to prepare a typical web-to-flange tee joint for welding. The same is true for each complete joint penetration double bevel tee weld. CP joints using fused steel backing require only one extra beveling cut but necessitate joining continuous backing and tack welding it in place. This is more work than preparing a double bevel CP weld without backing.

*Difficulty Costs.* Fillet welding is the easiest weld to complete. Partial joint penetration groove welding requires extra care in welding, and positioning is often required to ensure fusion at the root. CP welding without backing necessitates the joint be back-gouged to sound metal before welding the second side. When CP welding with backing, where back-gouging is not required, extra care is needed to ensure fusion in the root without melting through the backing bar.

When small fillet welds meet all design and weld soundness requirements, they are the most cost efficient weld type. At some point as sizes increase, large fillet welds become wasteful of weld metal. Really large fillet welds create distortion and cracking problems in the base metal. They should not be used no matter how cheap weld metal appears to be. Partial joint penetration groove welds are better for structural reasons than large fillet welds and are less costly when equivalent fillet weld sizes would be required to exceed $\frac{5}{8}$ in. When complete joint penetration groove welds are required by design, CP welds made from two sides are considerably less costly than CP welds made with backing. Complete joint penetration double bevel groove welds require less weld metal and less work. In addition, CP welds without backing may be subjected to higher stress ranges under the design provisions of AASHTO.[4] Higher allowable stress ranges can reduce weld and base metal costs considerably. When there is access to weld from only one side, CP welds with backing must be used. They cost almost twice as much as CP welds without backing, depending on joint design. The engineer can reduce weld volume costs by qualifying joints with smaller included angles and root openings. The Codes[5,6] contain provisions for qualifying nonstandard details of welded joints.

Longitudinal weld types listed in order of increasing costs are: fillet welds, partial joint penetration groove welds, complete joint penetration groove welds without backing, and complete joint penetration groove welds with backing. This comparison is true for both tee and corner welds and remains the same even when slightly different extra work cost units and factors are used.

## CONCLUSIONS AND RECOMMENDATIONS

Many aspects of bridge design and construction that affects the choice of longitudinal weld type and influences the design of those welds have been examined. Careful examination has confirmed that all properly designed continuous welds of good quality have essentially the same static life and the same fatigue life regardless of type. Since performance is not determined by weld type, economy of cost and effort should be the major determinants in the selection of the type of weld to use. Because the cost of longitudinal welds represents such a large portion of total bridge welding costs, the relative cost of such welds has a major role in determining the economic competitiveness of steel bridges. Use of complete joint penetration groove welds where fillet or partial joint penetration groove welds would suffice does nothing to increase safety but, instead, significantly increases welding and fabrication costs and exacerbates decline in the use of welded steel structures.

The type of longitudinal weld used in each bridge application should be selected based upon the arrangement of parts required to construct individual members, the stress to be carried, and the relative cost of weld joint types. CP welds must be used when calculated stress demands full section strength and where there is an applied tensile stress normal to the weld throat. Where stresses permit, small and medium size fillet welds 5/16 in. to 1/2 in. in size are by far the best for all web-to-flange connections. When stresses require fillet welds greater than 5/8 in. in size, partial joint penetration groove welds or CP welds should be used, as necessary. When complete joint penetration welds are required, double bevel CP welds are the best. CP welds with backing should only be used when there is no access to weld from the second side.

We must focus our efforts to ensure that all welds meet specified weld quality standards. The presence or absence of cracks and other significant discontinuities normal to applied tensile stress determines fatigue life and safety. The choice of longitudinal weld type, assuming the design is correct and proper materials have been specified, is less important to bridge safety than the quality of workmanship incorporated in welding and fabrication.

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