

SECTION IV

Fatigue Design

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1. INTRODUCTION

This section briefly discusses several approaches currently used to design against fatigue, and indicates why the stress-life detail-category approach is preferred for bridge design. Also described are current AASHTO procedures for design and rating of highway bridges, that are based on the stress-life detail-category approach, and the experimental basis for the AASHTO detail categories. The section also presents a proposed new method that more realistically approximates conditions in actual highway bridges.

The second part of this section discusses fabrication practices related to fatigue and good and bad designs for critical details such as longitudinal-stiffener ends, cover-plate ends, splices, and connections. The effects of secondary bending in initiating fatigue cracks is explained. Methods of improving the fatigue performance of new members and of repairing fatigue damage to existing members are described. Such methods as peening, grinding, preloading, TIG remelting, crack-arrest holes, and rewelding are included.

DESIGN APPROACHES

Four fatigue-design approaches are currently in widespread use: 1) the fracture-mechanics approach that is used in a variety of applications, 2) the strain-life approach, widely used in automotive applications, 3) the stress-life reduction-factor approach, widely used in machine design, and 4) the stress-life detail-category approach, generally used for structural applications, such as bridges and buildings.

In the fracture-mechanics approach, the fatigue life of a member is predicted from crack-initiation and crack-propagation data obtained in tests on small specimens.¹ The stress conditions in a small region at the tip of a sharp crack, which change as the crack grows, control fatigue behavior. Several critical assumptions must be made in calculating these stress conditions in fabricated members: 1) the size of any crack, or crack-like imperfection, present before loading, 2) the effect of residual stresses at various stages in the crack growth, and 3) the exact stress concentration factor at various points in the region of the crack due to detail geometry.^{2,3} Because of these assumptions, and other uncertainties pertaining to the basic initiation and propagation data, it is difficult to predict accurately the fatigue behavior of fabricated members by the fracture-mechanics approach.^{2,3}

In the strain-life approach, the fatigue life of a member is predicted from strain vs life data obtained in fatigue tests of small unnotched specimens by relating these data to the cyclic plastic strains at a critical location (usually at a notch) in the member.^{4,5} Strain, rather than stress, is used as the design parameter because the fatigue strain at the critical location (notch) often exceeds the yield strain even though the nominal fatigue stresses are well below the yield stress. Laborious calculations, or direct measurements, are required to trace the variation of strain at a notch subjected to complex variable-amplitude loadings. Furthermore, assumptions and uncertainties similar to those mentioned with regard to the fracture-mechanics approach occur in determining the strain variation and predicting the fatigue life.^{4,5} Therefore, it is difficult to predict accurately the fatigue behavior of fabricated members by the strain-life approach.

In the stress-life reduction-factor approach, a fatigue-strength reduction factor is applied to stress-life unnotched small-specimen data to predict the life of a notched specimen or member.^{6,7} This approach uses stresses to define fatigue behavior even when yielding has occurred at the notch. The reduction factor varies with the size and type of notch, number of cycles, mean stress level, and material.^{6,7} Therefore, selection of the proper reduction factor is critical in accurately predicting the fatigue behavior of a fabricated member by this method.

In the stress-life detail-categories approach, typical structural details are grouped according to severity, and allowable stresses corresponding to various design lives are given for each group. The allowable stresses for each group are obtained by performing fatigue tests on specimens that simulate each type of detail in the group. Thus, the stress concentrations, residual stresses, initial flaws, and material variability (especially near welds) that cause uncertainties in the first three approaches are included directly in the tests. Of course, these quantities vary among individual specimens contributing to the scatter of results. However, this scatter is considered in the design process by using the lower limit of the scatter band. Consequently, the fourth approach is more convenient than the first three for predicting the fatigue behavior of fabricated members and is preferred for structural applications.^{8,9,10}

2. EXPERIMENTAL BASIS FOR AASHTO DETAIL CATEGORIES

The fatigue provisions of the present AASHTO specifications⁸ are based on the stress-life detail-categories design approach. The experimental basis for the selection of detail categories and the assignment of allowable stresses to each category are described in the following paragraphs. The allowable stresses are based primarily on an extensive program of constant-amplitude fatigue tests of simulated bridge members conducted at Lehigh University¹¹ through¹⁷ and are supported by variable amplitude tests conducted at the U.S. Steel Corporation Research Laboratory² and by earlier constant-amplitude tests conducted at the University of Illinois,^{18,19} U.S. Steel Corporation,^{19,20} and elsewhere.¹⁹

AASHTO has classified bridge members into seven detail categories as described in Table I⁸ and illustrated in Figure 1. The fatigue behavior for each category is conservatively represented by the corresponding SN curve in Figure 2. The stress parameter for these curves is stress range; the effects of minimum or mean stress were shown to be small and are neglected.¹¹⁻¹⁶ Tests of simulated bridge members^{2,11} showed that the type (strength) of steel has little effect on fatigue behavior within a practical range of stress parameters. Therefore, the SN curves in Figure 2 apply to all structural steels used in bridges.

The sloping portions of six of the SN curves are parallel and spaced approximately equally. The reciprocal of the slope for these curves, and the stress exponent in the exponential SN relationship given in Equation 2 of Section III, is 3.0. Tests have shown that this value is appropriate for most fabricated members.^{2,11} The seventh SN curve, which applies only for shear stress on the throat of a fillet weld, has a corresponding stress exponent of 5.86. The sloping portion of each curve represents the approximate lower 95 percent confidence limit for 95 percent survival as derived from test results for the most severe detail in that category.^{16,17} An explanation of such a confidence limit is given in Section III of this chapter.

The horizontal portion of each curve represents the best estimate that could be made of the fatigue limit for the most severe detail in that category. This estimate was based on experimental data supplemented by crack-growth threshold calculations.¹⁶ Further experimental studies are being conducted to better establish the fatigue limits for various details.

Tensile residual stresses in welded members can cause fatigue cracking in regions that are subjected only to applied compressive stresses as discussed in Section III. These cracks, however, usually stop growing after they propagate into adjacent regions of zero or compressive residual stresses.¹¹⁻¹⁷ Consequently, the AASHTO fatigue provisions⁸ do not impose stress-range limitations where the stresses are always compressive. In practice, however, caution should be used

before dismissing the possibility of fatigue failures in regions of nominal compressive stresses because unexpected tensile stress components often occur as a result of residual stresses or local triaxial stress conditions.² Such triaxial conditions occur around notches and concentrated loads.

Each of the seven AASHTO detail categories is briefly discussed below. The individual details included in each category are identified and important factors that influence the fatigue strength of each are described. These factors include imperfections normally present in all structural members as a result of manufacturing and fabrication processes.^{3,17} Such imperfections are very difficult to eliminate, and efforts to remove them may cause worse conditions than were originally present.^{3,17} Since the simulated members that were tested to establish allowable fatigue stresses contained such imperfections, the effects of these imperfections have been adequately accounted for.

CATEGORY A

Category A provides the highest fatigue strength attainable for bridge members. It includes plates and shapes with mill surfaces or good-quality flame-cut edges, that is, edges with an average surface roughness²¹ not exceeding¹⁰⁰⁰ micro-inches and no severe gouges. Although the AASHTO fatigue provisions do not mention weathered surfaces, a recent study²² concluded that it is appropriate to include weathering-steel plates and shapes with such surfaces in Category A. The study also indicated that weathering of various types of fabricated details does not change their category classification. Category A does not include plates with sharp nicks or gouges due to handling; such nicks and gouges should be repaired in accordance with ASTM²³ and AASHTO²⁴ specifications.

The allowable SN curve for Category A approximates the lower 95-percent confidence limit from tests conducted at Lehigh University¹¹ on rolled shapes of A36 and A441 steels.²⁵ This curve also conservatively represents data from tests conducted at Lehigh¹² on A514 steel rolled shapes and data from elsewhere on rolled plates and shapes of various steels.¹² The fatigue strength of the rolled plates and shapes was controlled by normal surface imperfections and roughness. Because of the variability in the severity of the imperfections and roughness, the scatter in fatigue results for rolled plates and shapes is greater than the scatter for fabricated details. Tests conducted at Lehigh University¹¹ on welded shapes with good-quality flame-cut edges showed that the failure of such shapes initiated in the flange/web fillet welds rather than at the flame-cut edges. Therefore, good-quality flame-cut edges were included in Category A

CATEGORY B

Category B includes 1) continuous longitudinal* fillet or groove welds, 2) full-penetration transverse* groove welds ground flush, 3) tapered splices with the weld reinforcement removed, 4) 24-inch radius curved transitions for flange plates or groove-welded attachments, and 5) bolted joints. Specific requirements for some of these details are given in Table I.

The allowable SN curve for Category B approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on welded shapes without stiffeners, attachments, or other geometric discontinuities.¹¹ Shapes of A36, A441, and A514 steels²⁵ were included. Longitudinal-fillet-weld data from other sources are conservatively represented by this SN curve.¹¹⁻¹² These data include the variable-amplitude results from a study conducted at the U.S. Steel Research Laboratory.

Since no significant geometric stress raisers were present in the welded shapes, cracking usually initiated at normal internal imperfections, such as gas pockets, in the flange/web fillet welds.¹¹ Generally, the critical imperfections occur near tack welds, stop/start positions, or weld repairs, that are permitted in bridge fabrication.²⁴ Incomplete penetration in continuous longitudinal fillet welds does not affect their fatigue strength because it results in a discontinuity whose plane is parallel with the direction of stress. Such discontinuities do not act as stress raisers and hence have little effect on fatigue behavior.¹³ Terminations or interruptions of longitudinal fillet welds do act as stress raisers and are covered in other detail categories. Manual fillet welds tend to have a lower fatigue strength than automatic or semi-automatic fillet welds, but are included in Category B if they are properly made.¹¹

Longitudinal groove welds are similar to fillet welds and, therefore, are included in Category B.¹³ The assignment to Category B of full-penetration transverse groove welds ground flush is based primarily on fatigue tests conducted at Lehigh University on welded beams with groove-welded flange splices.¹¹ The results for these beams were similar to the results for welded beams without splices. The flange splices had a 1 to 2½ taper in width and were nondestructively inspected to establish weld soundness. Other data on full-penetration groove-welded joints in plates showed that their fatigue strength is controlled by normal internal imperfections, and approaches that of plates with mill surfaces provided that the weld reinforcement is properly ground flush and the weld soundness is established by non-destructive inspection.²⁰ Consequently, to be included in Category B, transverse groove welds must be nondestructively inspected to establish weld soundness.

Joints with a 1 to 2½ taper in either width or thick-

ness were included in Category B primarily on the basis of the Lehigh tests mentioned in the preceding paragraph.¹¹ When these data were interpreted, no tests had been conducted at Lehigh on tapers in thickness,¹² but a few available data from the University of Illinois²⁶ were generally above the allowable SN curve for Category B. More recent studies suggest that a 1 to 2½ taper in thickness is more severe than a similar taper in width because the stress concentration for the thickness taper is about 2.0,²⁷⁻²⁸ while the stress concentration factor for the width taper is only 1.1.¹¹ Nevertheless, the fatigue results from a few recent tests performed at Lehigh on welded girders with a 1 to 2½ thickness taper were above the allowable SN curve for Category B.

Because the original Lehigh Tests¹¹ of welded beams with tapered flange splices showed that the results for A514 steel were below the allowable SN curve for Category B, width tapers in A514- or A517-steel²⁵ plates are prohibited in AASHTO Article 10.18.5.5 on splices.⁸ Curved width transitions, which provided a fatigue strength above the Category B allowable SN curve,¹¹ are permitted with A514 steel, however.⁸ AASHTO gives no special provisions for 1 to 2½ thickness tapers with A514-steel plates, but the above discussion suggests that it would be appropriate to use a fatigue category lower than B for this case.

The welded beams with tapered flange splices that were tested at Lehigh¹¹ also had full-penetration groove-welded flange splices with a curved width transition. The radius of the curve was 24 inches. The transverse groove welds were ground flush and non-destructively inspected to establish weld soundness. The splices with curved width transitions had about the same fatigue strength as the tapered splices if the results for A514 steel were eliminated from both sets of data.¹¹ Therefore, 24-inch-radius curved transitions for flange splices, and attachments or gusset plates that are joined to members by full-penetration longitudinal groove welds, are included in Category B. The ends of the longitudinal groove welds must be ground smooth. For attachments with curved transitions of any radius, the same detail category applies to both longitudinal stresses in the main member and transverse stresses in the attachment.

Twenty-four-inch curved transitions for attachments (Fig. 1) joined to members by longitudinal fillet welds were originally included in Category C.¹³⁻¹⁷ but were moved to a lower category* when later studies showed that there is a high probability of subsurface discontinuities occurring at the roots of the fillet welds in the transition radius near the point of tangency.¹⁴ Such subsurface discontinuities cannot be readily detected by nondestructive inspection. Similar root imperfections could occur in partial-penetration groove welds. Therefore, curved transitions associated with such

*As used here, *transverse* is defined as the direction perpendicular to the main stress in the member, and *longitudinal* is the direction parallel to the main stress.

* Lower category means a category with a lower allowable SN curve.

welds should be treated in the same way as curved transitions associated with fillet welds; this is not clear from the wording of the specifications.⁸

The inclusion of high-strength bolted joints in Category B was based on existing data from several sources.²⁹ These data are specifically for symmetric butt splices (double-lap joints) of two types: 1) slip-resistant (friction) joints and 2) bearing joints. In slip-resistant joints, most of the force is transferred by friction between the contacting surfaces and fatigue cracks usually initiate at the surface in the gross section due to fretting.²⁹ Therefore, fatigue behavior is controlled by gross-section stresses. In bearing joints, a large portion of the force is transferred by bearing and shear of the bolts, and fatigue cracks usually initiate at holes in the net section.²⁹ Therefore, fatigue behavior is controlled by net-section stresses. The available fatigue data are conservatively represented by the allowable SN curve for Category B when the gross-section stresses are used for slip-resistant joints and the net-section stresses are used for bearing joints.²⁹ Consequently, the gross-section and net-section stresses, respectively, must be used in applying the AASHTO specifications⁸ to slip-resistant and bearing joints.

The scatter in the available data for bolted-joints is larger than for most other details because of variations in joint configuration, hole fabrication methods, bolt-tightening techniques, contact-surface conditions, cyclic loading conditions, and other factors. For both types of bolted joints, higher clamping forces, greater coefficients of friction for contact surfaces, and stress reversals (rather than cycles without reversals) tend to result in higher fatigue strengths.²⁹ For bearing joints, drilled, or subpunched and reamed, holes tend to provide higher fatigue strengths than punched holes.¹³

Category B is not intended to apply to 1) single-lap joints, 2) joints subjected to prying action, or 3) joints subjected to direct tension perpendicular to the plate surfaces.²⁹ Significant bending stresses develop in the plates of such joints, but only membrane stresses occur in symmetric butt splices. A truss joint in which an I or box section is connected by gusset plates on two sides can be considered a symmetric butt splice and included in Category B even though the bolts joining each gusset plate are in single shear.

CATEGORY C

Category C includes 1) transverse stiffeners or attachments, 2) full-penetration transverse groove welds with the reinforcement not removed, 3) 6-inch-radius curved transitions for groove-welded attachments, and 4) stud-type shear connectors. Specific requirements for some of these details are given in Table I. The allowable fatigue limit for transverse stiffeners is slightly higher than that for the other details in this category.

The allowable SN curve for Category C approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on welded beams and

girders with transverse stiffeners that were welded to either the web alone or to the web and tension flange.¹² Appropriate data on transverse stiffeners from other sources also fall above this SN curve.¹² Fatigue cracks initiated at the toe of the fillet weld at the end of the stiffener if the stiffener was not welded to the flange, and at the toe of the stiffener/flange fillet weld if it was.¹² The crack initiation was influenced by the stress concentration factor of 2.2 to 4 at these locations²⁷ and sometimes also by the normal weld imperfections. The fatigue strengths of the two types of stiffeners—those welded to the flange and those not—were about the same. When lateral bracing was attached to the stiffeners it did not reduce their fatigue strength.¹²

Bending stress at the crack-initiation location is the stress parameter for the experimental data on stiffeners and for the corresponding allowable SN curve. It was concluded that the presence of shear in combination with this bending stress need not be considered in normal bridge designs because of the shear/moment relationships that exist in such bridges.¹² However, if abnormally high shear stress occurs in combination with abnormally low bending stress at the stiffener, the maximum principal tensile stress should be used as the stress parameter in conjunction with the allowable SN curve.

Transverse attachments, i.e., plates fillet welded to the flange or web perpendicular to the stress, are equivalent to transverse stiffeners and belong in Category C. In fact, any attachment less than 2-inches wide in the direction of stress is similar to a transverse stiffener and is included in Category C.

Test data^{18,19,20} show that weld reinforcement causes a stress concentration that reduces the fatigue strength of full-penetration transverse groove welds. For reinforcement with a 60-degree angle at the toe of the weld, the stress concentration factor was reported to range from 1.3 to 1.8.^{27,30} Available fatigue data showed that Category C is appropriate for such welds, provided that their soundness has been established by nondestructive inspection.¹² One to 2½ tapered transitions in width or thickness are permitted in conjunction with such transverse groove welds because the limited data available for such joints²⁷ were generally above the allowable SN curve for Category C.

If an attachment plate is groove-welded to the edge of the flange of a member, a high stress concentration occurs at the edge of the attachment and reduces the fatigue strength of the member. If a curved transition is provided at the edge of the attachment as illustrated in Case 14 of Figure 1, the stress concentration and the fatigue strength depend on the radius of the transition. As mentioned earlier, tests¹¹ showed that a 24-inch curve transition belongs in Category B; the 6-inch-radius transition was assigned to Category C on the basis of its higher stress concentration factor. Subsequent fatigue tests conducted at Lehigh on 6-inch and 2-inch transitions suggest that this was appropriate.¹⁴ The ends of the longitudinal groove welds must be ground smooth. These welds should be full-penetration

welds, as previously discussed, although this is not specifically stated in the specifications.⁸

Stud-type shear connectors can be considered to be short attachments and, therefore, included in Category C. The results of fatigue tests on plates or beams with studs attached by arc or friction welding³⁰ confirm that it is appropriate to assign such details to Category C. Channel-type shear connectors fillet-welded transversely to the flange are also attachments. They are included in Category C if their width in the direction of the flange stress does not exceed 2 inches; otherwise, they are included in a lower category.

CATEGORY D

Category D includes 1) 4-inch attachments, 2) 2-inch-radius curved transitions for groove or fillet welded attachments, and 3) riveted joints. Specific requirements for some of these details are given in Table I.

The allowable SN curve for Category D approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on beams with 4-inch-long attachments fillet welded to the tension flange.¹² Appropriate data from other sources on fillet- or groove-welded attachments to plates or beams also fall above this allowable SN curve.¹² Two types of attachments were included in the Lehigh¹² data: 1) attachments with welds on longitudinal edges only and 2) attachments with welds on both longitudinal and transverse edges. The former type tended to have a slightly higher fatigue strength.¹² Fatigue cracks initiated at the toes of the transverse welds or the ends of the longitudinal welds, and were controlled by the stress concentrations at these locations.

Fatigue tests were also performed at Lehigh¹² on attachments of other lengths. The results varied with the length of the attachment. These results fell between those for a transverse stiffener, that can be considered equivalent to an attachment of minimum length (in the direction of stress), and those for a partial-length cover plate, which is equivalent to an attachment of maximum length. The fatigue strength decreased with increasing attachment length because more force is developed in longer attachments and this increases the stress concentration caused by the attachment. The 4-inch attachment provides a convenient category about midway between that for cover-plate ends and that for stiffeners. Category D, of course, is conservative for attachments shorter than 4 inches, except that when such shorter attachments exceed 12 times their thickness they are excluded from Category D.^{8,12} This exception governs only attachment plates thinner than 0.333 inch.

Fillet- or groove-welded attachments of any length are included in Category D if they have a curved transition radius of not less than 2 inches and the weld ends are ground smooth.⁸ Such transitions were assigned to Category D on the basis of the reduced stress concentration factor provided by the radius.¹³ Subsequent fatigue tests¹⁴ showed that it is conservative to include

groove-welded attachments with a 2-inch radius in Category D. In fact, data for groove-welded rectangular attachments in which the weld ends had been ground to a radius of 0.2 to 0.4 inch, fell within the allowable SN curve for Category D.

Fatigue data from the University of Alberta³¹ on fillet-welded longitudinal attachments with 4-inch-radius transitions showed that such attachments "provided a fatigue resistance compatible with Category D."¹⁴ On the basis of these and other data,³² it was concluded that fillet-welded attachments with curved transitions of various radii above 2 inches should not be included in Categories B and C, as they formerly were,^{13,17} but could be retained in Category D.¹⁴

Although riveting is no longer used to fabricate new bridges, many riveted bridges are currently (1985) in service. Consequently, AASHTO⁸ includes riveted joints in its classification of details. Available fatigue data^{33,34,35} on riveted symmetric butt splices (double-lap joints) shows that there is an exceptionally large amount of scatter for such data, and that the lower bound SN curve for the data has an unusually flat slope. Consequently, riveted joints have been assigned to the relatively low Category D. Net-section stress is the appropriate stress parameter for such joints because the clamping force provided by rivets is uncertain and cracking often occurs in the net section.

Many different types of riveted details are currently (1985) in service in existing bridges. For example, built-up girders, cover-plate ends, transverse stiffeners, truss gusset-plate details, and transverse attachments are common in riveted, as well as in welded, construction. Although these different types of riveted details may have different fatigue strength, they are all included in Category D.

CATEGORY E

Category E includes 1) ends of cover plates fillet-welded to flanges not greater than 0.8 inch thick, 2) attachments longer than 4 inches, 3) intermittent longitudinal fillet welds, and 4) fillet-welded lap joints. Specific requirements for some of these details are given in Table I. Originally, this category was thought to represent the lower bound of fatigue strengths for fabricated bridge members,^{16,17} but it was later discovered that lower strengths occur.^{13,36}

The allowable SN curve for Category E approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on beams with partial-length cover plates having ends that were either fillet-welded or unwelded.¹¹ Appropriate constant-amplitude fatigue data from other sources on cover-plate ends were shown to be consistent with this SN curve.¹² The results of subsequent variable-amplitude tests conducted at the U.S. Steel Research Laboratory² on cover-plate ends similar to those tested at Lehigh are also consistent with the allowable SN curve.

In the Lehigh tests,¹¹ the partial-length cover plates were welded to 1) rolled beams, 2) rolled beams with

full-length cover plates, or 3) welded beams. The test specimens included 1) cover plates that were both narrower and wider than the flange, 2) several different types (strengths) of steel, and 3) cover plates that were either $1\frac{1}{2}$ or 2 times the flange thickness. AASHTO Article 10.13.3 presently (1985) limits the thickness of a single cover plate to 2 times the flange thickness and the total thickness of all cover plates to $2\frac{1}{2}$ times the flange thickness. Fatigue cracks initiated at the toes of the transverse welds or the ends of the longitudinal welds and were controlled by the stress concentrations at these locations.

The Lehigh tests showed that, on cover plates wider than the flange, a transverse end weld is required to assure that the fatigue strength does not fall below Category E.¹¹ Therefore, in the previous edition of the AASHTO specifications, a transverse end weld was required when the cover plate was wider than the flange. This provision, however, is not included in the present edition.⁸ For cover plates narrower than the flange, an end weld tends to reduce the fatigue life¹¹ and is not required.⁸

Variations in the end geometry of cover plates, including tapered and rounded ends, were shown to have little effect on fatigue strength.^{17,18,37} Therefore, all such cover-plate ends are included in Category E.

As mentioned earlier, tests¹² showed that the fatigue strength of attachments decreases as attachment length increases, and is equal to the fatigue strength of a cover-plate end when the attachment length exceeds a certain value. This value has not been exactly established, but tests¹² showed it exceeds 8 inches and probably is about 16 to 20 inches. To avoid an excessive number of different categories, however, attachments with lengths exceeding 4 inches have been conservatively included in Category E. Similarly, attachments having curved transitions with a radius less than 2 inches are included in category E. Similarly, attachments having curved transitions with a radius less than 2 inches are included in Category E, even though they have a higher fatigue strength than attachments without a curved transition. The assignment to Category E of attachments with a length greater than 4 inches and a transition radius less than 2 inches applies to both fillet- and groove-welded attachments.

The ends of intermittent longitudinal fillet welds cause stress concentrations somewhat similar to, but probably lower in magnitude than, cover plate ends. Because of the lack of fatigue data on such welds they have been conservatively assigned to Category E.¹³ Fillet-welded lap joints such as illustrated in Case 9 of Figure 1, are also similar to cover-plate ends and are included in Category E.

CATEGORY E'

Category E' includes 1) ends of cover plates fillet-welded to flanges greater than 0.8 inch thick and 2) girder flanges greater than 1-inch thick that pierce through the web of another girder and are fillet welded

to each side of that web. This category represents the lower bound of fatigue strengths for acceptable bridge details. Details with lower fatigue strengths exist but should not be used because such fatigue strengths are too low for most practical bridge applications.

The allowable SN curve for Category E' approximates the lower 95-percent confidence limit from tests conducted at Lehigh University on heavy beams with partial-length cover plates having ends that are either fillet welded or unwelded.^{13,28} Most of the beams were W36x230 rolled shapes, which have a flange thickness of 1.26 inches. Most of the cover plates were 1.25-inches thick. Fatigue cracks initiated at the toes of the transverse welds or ends of the longitudinal welds and were controlled by the stress concentrations at these locations. Apparently, the stress concentrations are higher for these heavy beams than for the lighter beams included in Category E.

The AASHTO specifications⁸ do not specifically mention girder flanges greater than 1-inch thick that pierce through the web of another girder and are fillet welded to each side of that web. However, the results of recent fatigue tests have led Lehigh University to recommend¹⁴ that such details be included in Category E'. Similar details in which the piercing flange is less than 1-inch thick can be thought of as attachment plates and "treated as Category E connections."¹⁴ Similar cross-girder details in which "seal" welds are placed on only one side of the pierced web have very low fatigue strengths and should not be used.^{13,14}

Fatigue tests¹⁴ of plates attached to flange surfaces only by transverse fillet welds showed that such details have fatigue strengths well below the allowable SN curve for Category E'. Therefore, the AASHTO specification⁸ states that "Gusset plates attached to girder flanges with only transverse fillet welds, [are] not recommended." In the tests, fatigue cracking usually initiated at the root of the weld and severed the plate from the flange.¹⁴ In one test, however, the attachment also caused fatigue cracking in the flange.

CATEGORY F

Category F includes only shear stress on the throat of fillet welds. It applies to continuous or intermittent longitudinal or transverse fillet welds. The allowable SN curve for this category was developed¹³ from fatigue tests conducted at the University of Illinois³⁸ during the early 1940s on small fillet welded plate specimens in which the welds were subjected to high shear stresses. Shear stresses, as defined in Category F, usually do not control the fatigue strength of fillet welded details;¹³ instead, the fatigue strength of such details is usually controlled by axial or bending stresses at the toe of transverse welds or ends of longitudinal welds as defined in other categories. The allowable SN curve for Category F has a slope different from that of the SN curves for other categories, but it has been suggested that the same slope could be used.³⁹ Future studies have been suggested to verify this.¹³

3. BRIDGE DESIGN AND RATING METHODS

PRESENT AASHTO METHODS

The present (1985) AASHTO methods for both the design⁸ and rating⁴⁰ of highway bridges are similar. For each of the detail categories discussed previously, the specifications⁸ give allowable stress ranges for each of four design-life categories. The appropriate design-life category for a particular bridge and type of loading (truck or lane) is defined in tables given in the specifications.⁸ The calculated stress range for any detail must be less than the allowable stress range for that detail. These present AASHTO methods are discussed in more detail below.

Allowable Stress Range

Instead of giving allowable SN curves such as those in Figure 2, AASHTO⁸ gives specific values of allowable stress range corresponding to four different design-life categories or points on these curves: 1) 100,000 cycles, 2) 500,000 cycles, 3) 2,000,000 cycles and 4) over 2,000,000 cycles. Stresses for this last category are taken at the fatigue limit or horizontal portion of the curve.

Specific values of the allowable stress ranges given by AASHTO for redundant and nonredundant load path structures are listed in Table II. The values for redundant structures correspond exactly to the allowable SN curves in Figure 2 and are intended for use with structures that have multiple load paths, such as multi-girder bridges or multi-element eye bars. For such structures, fracture of a single element does not cause collapse of the structure.

The allowable stress ranges for nonredundant structures are intended to provide greater safety for those structures that could collapse as a result of the fracture of a single element. Such structures⁸ include "flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, and caps at single or two column bents." The allowable stress ranges for each of the first three design-life categories was obtained by using the redundant allowable stress range from the next longer design-life category.¹⁷ The allowable stress ranges for the last design-life category (over 2,000,000 cycles) are generally the same as, or a few ksi lower than, the corresponding values for redundant structures. Category E' details are not permitted with nonredundant structures because allowable stresses derived as discussed above would be too low for practical usage.¹³

Calculated Stress Range

Generally, only traffic or wind loadings need be considered in calculating the stress range. Main (longitudinal) load carrying members must be checked for two traffic loadings: 1) truck loading and 2) lane loading. Transverse members and details subjected to wheel loads must be checked for truck loading but not lane loading. Wind bracing members must, of course, be checked for wind loading. The stress range calculated for each applicable loading must be less than the cor-

responding allowable stress range unless the calculated stresses are always compressive.

At any location along a beam or girder, the stress range equals the sum of the absolute values of the maximum positive and negative live-load (plus impact) moments divided by the section modulus. Positive moment is defined as moment producing tension in the bottom flange. The maximum live-load moment of either sign is obtained by placing the truck or lane load in the critical position for moment of that sign in accordance with AASHTO⁸ Article 3.11. Lane load is placed on all portions of the member where it contributes to the desired moment and omitted elsewhere.

Dead load does not affect the stress range since it does not cause stress variations. Nevertheless, dead load can affect fatigue calculations because a fatigue check is required only if a portion of the stress cycle is in tension. For continuous-span beams or girders, negative dead-load moments near interior supports are usually larger than the positive live-load moments in this region. Therefore, the bottom portions of the girder are always in compression and need not be fatigue checked. Similarly, the positive dead-load moments near midspans are usually larger than the negative live-load moments in this region. Therefore, the top portions of the girder are always in compression and need not be fatigue checked. In the lengths of the beam or girder between these two regions, tension stresses occur in both the top and bottom portions. Therefore, both portions must be fatigue checked. Dead load does not affect fatigue calculations for simple-span beams and girders because both the live- and dead-load moments are always positive. Consequently, the top portion of the girder is always in compression and need not be fatigue checked.

Chord members in simple- or continuous-span trusses are subjected to stresses similar to those in the flanges of girders, and need to be checked for fatigue in the same way. Web members (diagonals and verticals) in trusses are usually subjected to reversals as the truck passes from one side of the member to the other. Such members must be checked for fatigue.

Generally, the same loadings, lateral-distributions factors, and impact factors are used in calculating the stress range for the fatigue check as are used in the normal strength design. As a result, the calculated stress ranges generally exceed the stress ranges that actually occur in bridges as will be discussed in more detail later.

For one design-life category, namely that for over 2,000,000 cycles applied to longitudinal members, special calculation procedures are used in the fatigue check. Specifically, the stress range is calculated by placing a single truck on the bridge and distributing its weight to the girders as designated in AASHTO Article 3.23 for one traffic lane loading.⁸ This results in the use of a lateral-distribution factor of $S/7$ instead of $S/5.5$,

where S is the average girder spacing in feet. This procedure recognizes that most fatigue stresses are caused by single trucks as discussed in Sections I and II.

Design-Life Categories

The appropriate design-life category for a particular bridge member subjected to traffic loadings can be determined from Table III.⁸ Case I was recently adopted to provide for extremely heavy truck traffic¹⁷ such as that which caused fatigue cracking in the Yellow Mill Pond Bridge.³⁶ Cases II and III were retained from earlier versions of the AASHTO specifications because "no fatigue problems have been experienced with bridges in these categories."¹⁷

The AASHTO specifications do not define which types of trucks should be included in the Average Daily Truck Traffic (ADTT). Presumably, panel, pickup, and other 2-axle/4-wheel trucks should be excluded since they cause little fatigue damage (Section I).

The meaning of the particular design lives specified in Table III is obscure. As discussed later in more detail, these lives do not represent either the total number of trucks expected to pass over the bridge during its life, nor the number of 72-kip trucks expected to pass during this period. Instead, they are artificial numbers corresponding to allowable stress ranges that are expected to provide reasonable designs and that are consistent with a life expectancy of 60 to 70 years.¹⁷

AASHTO specifies that the design lives in the table should be used "unless traffic and loadometer surveys or other considerations indicate otherwise." Because the specified design lives are artificial numbers, however, it is difficult to relate them to such surveys. Some guidance for doing this is given in Reference 17.

AASHTO⁸ specifies that 100,000 cycles of maximum wind loading should be used in the fatigue check unless available information indicates otherwise. Again this design life is an artificial number intended to provide a reasonable design.

Rating of Bridges

To assure the continuing safety of existing bridges, such bridges are periodically inspected and safety rated by procedures given in the AASHTO maintenance manual.⁴⁰ The manual permits the rating of steel bridges by either the Load Factor Method or the Working Stress Method. By either method, each bridge is rated at two load levels that provide different factors of safety against static (nonfatigue) failure: 1) the Operating Rating, corresponding to the maximum load permitted to cross the bridge, and 2) the Inventory Rating, corresponding to the load permitted to cross the bridge on a routine basis.

In the Load Factor Method, fatigue strength does not affect the Operating Rating, but is one of three criteria affecting the Inventory Rating. The rating procedure yields the maximum load (or percentage of the rating vehicle used by a particular state) permitted for fatigue. This maximum load is based on the most critical fatigue detail in the bridge, and is calculated by multiplying the gross weight of the rating vehicle by the ratio of allowable stress range to the stress range

calculated for that vehicle. The methods of calculating the stress range are the same as those used for the design of new bridges. Fatigue is not mentioned in the Working Stress Method, but similar procedures can be used to get a maximum fatigue load for this method.

Comparison with Actual Behavior

The AASHTO⁸ fatigue-design procedures described above were developed before extensive information was available on the fatigue stresses that actually occur in bridges. Therefore, these procedures do not accurately reflect the actual conditions described in Sections I and II. The differences are illustrated in the examples in the next two paragraphs.

First, consider a relatively short simple-span girder bridge for which the fatigue design is governed by truck rather than lane loading. Assume that the average daily truck traffic is 2,000 and that each passage of a truck causes one loading cycle. This traffic will cause 36,500,000 loading cycles in 50 years — a reasonable minimum life for a bridge. According to the FHWA 1970 nationwide loadometer survey presented in Section I, 7.37% of these cycles or 2,670,000 cycles, would exceed a weight of 72 kips, which corresponds to the AASHTO HS20 design truck. The AASHTO specifications⁸ require that main members of a bridge on a major highway with an average daily truck traffic of less than 2,500 be designed for 500,000 stress cycles caused by an HS20 truck (or for 100,000 cycles of lane loading, which was assumed not to govern). As discussed in Section II, field measurements have shown that the actual stress ranges in bridges are usually considerably less than would be calculated for the HS20 truck by present AASHTO procedures. Thus, the AASHTO design conditions for this example combine an artificially high fatigue stress range with an artificially low number of stress cycles to get a reasonable design.

Next, consider a continuous girder bridge with relatively long spans for which lane loading governs the AASHTO fatigue design. In designing this bridge, the loading would first be applied over certain portions of the bridge to obtain the maximum positive moment and then over different portions of the bridge to obtain the maximum negative moment. The sum of the two moments would be used to calculate the design stress range. This type of loading results in the worst possible positive and negative moments and, therefore, is appropriate for static design.

However, this type of loading, which results in large stress ranges, would occur very rarely, if ever. Therefore, it is overly conservative for use in determining the fatigue life of the bridge. Instead, the life would be mainly influenced by truck loadings. Thus, the AASHTO design conditions for this example are far from the conditions that actually affect the fatigue life of the bridge.

Next, consider the present method of including fatigue in the rating process. Not only does this method fail to reflect the actual fatigue conditions in bridges, but it also fails to provide the *type* of information needed to accurately assess the present condition and

estimated remaining life of a bridge. In fact, the meaning of the maximum permissible fatigue loading, which is presented as a percentage of the rating vehicle gross weight, is vague. How many additional repetitions (beyond those that have already been applied) of this loading can be tolerated? More important, how does this loading relate to the actual traffic, which consists of trucks of various weights? How can it be used to estimate the present amount of fatigue damage and the remaining life?

PROPOSED NEW METHODS

Because the AASHTO specifications⁸ do not accurately reflect the fatigue conditions that actually occur in bridges, several new methods of fatigue design have been proposed in recent years.⁴¹⁻⁴⁵ The intent of these new methods has been to utilize more realistic fatigue loading and stress conditions and/or to provide more uniform reliability factors. All of the new methods use the AASHTO detail categories and the fatigue strength information developed at Lehigh University.

One of the proposed new methods,^{39,41} which was developed at the U. S. Steel Research Laboratory, is described in detail under subsequent headings. It accurately reflects actual fatigue conditions as described in Sections I and II, and can be conveniently applied to both the design and rating of bridges. It could be easily incorporated into specifications, and could be tailored to provide any desired degree of safety by simply changing the value of certain key parameters.

In addition to the proposed new methods mentioned above, a method involving modifications of the detail categories has been proposed in Europe. Specifically, the Lehigh fatigue data has been used by the European Convention for Constructural Steelwork* (ECCS) as the basis for a proposed group of 15 detail classes. The allowable SN curves for these classes are equally spaced between the upper- and lower-bound AASHTO curves. The percentage difference between the allowable stress ranges for adjacent classes is about 12 percent compared with about 30 percent for the AASHTO detail categories. This appears to be a greater refinement in detail categories than is justified by the available fatigue data. Nevertheless, the ECCS proposal is being reviewed in this country.¹⁵

Fatigue-Design Truck

The first step in the proposed new method^{39,41} is to select the gross weight for a fatigue-design truck that represents the variable truck weights in actual traffic. This gross weight is less than that of an HS20 truck⁸ because it represents an "average" condition for the actual traffic. Specifically, the weight is selected so that the fatigue damage caused by a given number of passages of this truck is the same as damage caused by an equal number of passages of different-sized trucks in actual traffic.

If the distribution of truck weights (percentages of trucks of different weight) for the traffic under consideration is known, the following formula² is used to calculate the gross weight of the fatigue-design truck:

$$W_F = (\sum \alpha_i W_i^3)^{1/3} \quad (1)$$

In which α_i is the fraction of trucks with a gross weight W_i . As discussed in Section I, a value for W_F of 50 kips would generally be appropriate if the actual weight distribution is not known. This weight is based on distributions from various nationwide surveys and may not be appropriate for locations where extremely heavy truck traffic is expected. It is the bridge designer's responsibility to identify such locations and to obtain the data needed to calculate an appropriate weight from Equation 1. This weight is one of the key parameters that could be changed by specification writers to provide any desired degree of safety.

The appropriate axle spacing and distribution of axle loads for the fatigue-design truck are shown in Figure 4 of Section II. As discussed in this section and elsewhere,⁴⁷ the spacing of 30 ft between main axles is appropriate because most fatigue damage is done by trucks with this spacing.

Lane loading generally need not be considered in fatigue design because the net effect of closely spaced trucks on fatigue is usually small as discussed in Sections I and II and elsewhere.⁴⁷ However, exceptions may exist at locations where special conditions tend to bunch trucks. These conditions may include a steep grade on a two-lane road or a traffic signal. For such situations the bridge designer must use his judgment based on a knowledge of local traffic conditions because no specific criteria for fatigue design are available. The discussion of traffic characteristics given in Section I might provide some help in interpreting local conditions.

Design Stress Range

The next step is to calculate the design stress range caused by the passage of the fatigue-design truck across the bridge in the lane under consideration. First, the moment range is calculated by putting the truck in positions that cause the maximum positive and negative moments at the detail under consideration. Then a lateral-distribution factor is used to calculate the design stress range from the moment range. The lateral-distribution factors given in the AASHTO specifications⁸ are based on severe conditions⁴⁸ that do not occur often as discussed earlier.

Therefore, the lateral-distribution chart⁴⁸ shown in Figure 11 of Section II is used in the new method unless a more refined lateral-distribution analysis is made. This chart, which is explained in detail in Section II, gives the fraction of the *total* truck moment that is carried by an exterior or interior beam or girder. It is based on a single truck at the center of the critical traffic lane since this is the appropriate condition for fatigue design. The lateral-distribution factor for any beam is a function of the lane position ratio, P , which is

*The United States is represented in this international group.

the distance from the exterior beam to the center line of the outer traffic lane divided by the beam spacing. This parameter is important because the bridge cross section tends to act as a semi-rigid unit subject to twisting due to the eccentricity of the load.⁴⁸

The solid lines represent the upper limit for the lateral-distribution factor, and can be used to make an initial conservative fatigue check. If this check is satisfactory, no further check is required. Otherwise, a second check can be made using the more precise, less conservative, dashed lines, which depend on the beam span and moment of inertia. When P exceeds 0.5, the solid horizontal line applies to both interior and exterior beams, but below 0.5 it applies only to interior beams. When P is less than 0.5 the exterior beams are governed by the solid sloping line.

As discussed in Section II, the impact factor defined in AASHTO Article 3.8⁸ is generally appropriate for fatigue design and should be applied to both the positive and negative moments.⁴⁹ This factor may not always be conservative for cantilever (suspended span) girder bridges, but alternative factors are not available.⁴⁹

In line with the present AASHTO fatigue specifications, no fatigue check is required if the stress due to combined dead, live and impact loading is compressive for all trucks in the traffic. This occurs when the dead-load compressive stress exceeds twice the live-load (plus impact) tensile stress caused by the fatigue-design truck because this truck has about $\frac{1}{2}$ the weight of the largest truck in the traffic³⁹ as will be discussed in more detail later.

Allowable Stress Range

The next step is to compare the design stress range with the allowable SN curve for the detail under consideration. The AASHTO⁸ detail categories are used. The sloping portions of the allowable SN curves, which are given in Figure 3, are generally identical with those in Figure 2 except that the slope of the curve for Category F has been made equal to that of the others as suggested in Reference 39. Therefore, all curves are defined by the equation

$$N = \frac{A}{F_{sr}} \quad (2)$$

Values of the constant A are given in Table III for the various detail categories.

The horizontal portions of the SN curves in Figure 3 are drawn at stress ranges equal to $\frac{1}{2}$ the constant-amplitude fatigue limits, which correspond to the horizontal lines in Figure 2. Truck-weight distributions given in Section I, and other information,³⁹ indicate that the weight of the heaviest truck in normal traffic is typically about 100 kips, twice the weight of the fatigue-design truck. (Special-permit trucks with more axles could be heavier without causing higher fatigue stresses.) The level of these horizontal lines is one of the key parameters that could be changed to provide any degree of safety desired by specification writers.

When the design stress range is below the appropri-

ate horizontal line in Figure 3, all of the stress ranges in a spectrum corresponding to typical traffic are below the constant-amplitude fatigue limit. Therefore, the fatigue life is infinite and no further fatigue check is required. This would often be the case in practical designs. In exceptional cases, where the weight of the heaviest truck is more than twice that of the fatigue-design truck, the horizontal lines in Figure 3 should be lowered accordingly. The bridge designer is responsible for identifying and accounting for such conditions.

If a greater degree of safety for nonredundant structures is desired, this could be done by using lower allowable SN curves in a manner similar to that in the present AASHTO specifications.⁸ It could also be done by requiring that the design stress ranges for all details be below the horizontal lines in Figure 3, or perhaps below a lower set of such lines.

Design Life

If the design stress range is above the appropriate horizontal line in Figure 3, the minimum life of the bridge must be estimated from

$$L = \frac{N}{365TP} \quad (3)$$

in which L is the life in years, N is the number of cycles determined from the allowable SN curve, T is the average daily truck traffic, and P is the number of stress cycles per passage of a truck. A minimum life of 50 years is proposed, but in some cases a higher value may be desirable. The minimum life is another value that could be changed by specification writers.

For bridges on two-lane highways, T is the total truck traffic volume in both directions and includes all trucks except panel, pickup, and other 2-axle/4-wheel trucks. For bridges on highways of more than two lanes, T is the total truck traffic in all lanes in one direction. In both cases, this is conservative because only part of the traffic actually travels in the lane under consideration. The rest travels in the adjacent lane or lanes and causes less fatigue damage than assumed.

If the expected truck-traffic volume is known, it should be used; otherwise, the Design Daily Truck Traffic values listed in Table IV of Section I would generally be appropriate, as discussed in that section. However, it is the bridge designer's responsibility to identify extreme conditions that would require the use of higher traffic volumes.

The design values listed in the table were developed from nationwide traffic surveys and represent very heavy traffic. For bridges on major rural highways, the proposed design value⁵⁰ is 400 times the number of lanes, that is, 800 for bridges on two-lane highways, 1600 for bridges on four-lane highways, and 2400 for bridges on six-lane highways. For bridges on two-lane highways, this value represents the traffic in both directions, and for highways of more than two lanes, the value represents the traffic in one direction. Thus, bridges on multi-lane highways are designed for higher traffic volumes per lane than bridges on two-lane highways. This is consistent with observed traffic volumes

from nationwide traffic surveys. For bridges on major urban highways, the proposed design value is 600 times the number of lanes.

Proposed⁴⁷ design values of P , the number of stress cycles per truck passage, are given in Table V of Section II and are explained in that section. Generally, the design value is 2 for very short spans that are affected by individual axle loads and 1 for longer spans, but there are exceptions. (Table V of Section II contains an error. The number of stress cycles per truck passage for continuous-span girders near interior supports should be 2.0 rather than 1.0 when the span is below 40 ft.)

Rating

The proposed new method is well suited to the evaluation or rating⁴⁰ of existing bridges because it provides a realistic evaluation of the remaining life and can account for future changes in traffic volume or truck-weight distributions. Generally, the procedures de-

scribed in the preceding paragraphs are used in evaluating existing bridges as well as in designing new bridges. Of course, estimates of past and future traffic volume are used to select an appropriate value of T .

The effect of changes in the volume or weight distribution of the truck traffic during the life of the bridge can be calculated from

$$\sum \frac{T_n L_n}{N_n} = \frac{1}{365P} \quad (4)$$

in which the subscript n denotes a period of constant volume and distribution of traffic. Any number of periods of constant traffic can be used. L for the last period is the unknown in Equation 4.

Fatigue considerations do not restrict the weight of the trucks that are permitted to cross the bridge. If this weight is restricted by other considerations, the restricted weight should be used in estimating the remaining fatigue life.

4. DESIGN AND FABRICATION PRINCIPLES

FABRICATION PRACTICE RELATED TO FATIGUE OF NON-FRACTURE-CRITICAL MEMBERS

Normal fabrication practice for welded, bolted, or riveted steel highway bridges is specified in Division II, Section 10 of the AASHTO specifications.⁸ In addition, fabrication practice for welded bridges is covered in more detail in the AWS specifications⁵¹ as modified by AASHTO,²⁴ and the repairs permitted on steel plates before fabrication are described in an ASTM standard.²³ Several aspects of normal fabrication practice that relate to fatigue behavior are discussed in this article. Special fabrication procedures that are required for fracture-critical members⁵² are discussed later under fracture-critical members.

Plate Surfaces and Edges

Injurious imperfections, such as surface or edge pits, nicks or gouges, should be repaired as specified by ASTM A6²³ before rolled plates or shapes are shipped from the steel producer to the fabricator. Similar imperfections that result from handling during fabrication should be repaired in the same way to assure that the material will provide a fatigue strength consistent with Category A.

The welding specifications^{24,51} give extensive provisions defining when and how internal discontinuities exposed at cut edges must be repaired. These provisions apply to internal discontinuities that were present before the plate was cut and are parallel with the uncut plate surfaces. Discontinuities less than 1-inch long need not be repaired regardless of their depths. Longer discontinuities over 1/8-inch deep must be repaired by grinding or by gouging and welding. Discontinuities deeper than 1 inch are gouged and welded only

to a depth of 1 inch. The unwelded portion that is deeper than 1 inch, and any internal discontinuity that does not extend to within 1 inch of a cut edge, is permitted to remain unless the total area of all such discontinuities exceeds 4% of the plate surface area. If this limit is exceeded, the plate is rejected. Although they may be fairly large, the internal discontinuities permitted to remain have little effect on fatigue strength if the plate is stressed in its plane so that the discontinuity is parallel with the direction of stress. If the plate is stressed in the through-thickness direction, however, such discontinuities greatly reduce fatigue strength.

The welding specifications^{24,51} require that the surface roughness of oxygen-cut (flame-cut) edges not exceed 1000 microinches²¹ for plates up to 4-inches thick and 2000 microinches for thicker plates. Surface roughness exceeding these limits and occasional shallow notches or gouges must be repaired by proper machining or grinding.^{24,51} Within certain depth limits, deeper notches can be repaired by welding.^{24,51} Oxygen-cut edges with depths beyond these limits are unacceptable and cannot be repaired.^{24,51} Corners of oxygen-cut edges on main stress-carrying members must be provided with 1/16-inch chamfers.²⁴ This removes shallow notches that tend to occur along such corners and can initiate fatigue cracks. Thus, fabrication practice is consistent with Category A for oxygen-cut plates up to 4-inches thick, but not for thicker plates. To assure that these thicker plates meet the fatigue-strength requirements for Category A, the engineer must specify on the contract drawings that oxygen-cut edges must be ground to a maximum surface roughness of 1000 microinches. Otherwise, the thicker plates could probably be included in Category B, although no fatigue

data are available to verify this.

Surface requirements for sheared edges are not mentioned in the fatigue or fabrication provisions of the AASHTO specifications,^{8,24} but such edges should meet the same limitations on notches and surface roughness as oxygen-cut edges. To assure this, the engineer must include an appropriate note on the contract drawings.

Groove Welds

Articles 2.5, 9.17, and 9.21 of the welding specifications^{24,51} require that all transverse groove welds, except unstressed welds specifically identified on the drawings, have complete joint penetration and be fully inspected by radiographic or ultrasonic procedures. Thus, such welds qualify for Category B if the contract drawings show that the welds must be ground flush, and for Category C if not. To assure complete penetration, groove welds welded from one side must be provided with a steel backing (backup bar) or another type of backing demonstrated to be satisfactory by qualification tests, and groove welds welded from both sides must be back gouged.⁵¹

The designer should be cautious in permitting partial-penetration transverse groove welds in components intended for purely architectural purposes because such components may actually carry stress and cause fatigue cracking. For example, fatigue cracks have been caused by incomplete penetration in transverse groove-welded splices in longitudinal stiffeners in positive-moment regions where they were used for architectural reasons alone.¹⁷

The welding specifications^{24,51} permit partial-penetration groove welds for longitudinal joints, such as flange/web joints in plate or box girders, and such welds need not be fully inspected by radiographic or ultrasonic procedures. Thus, fabrication practice for such welds is consistent with Category B.

The welding specifications^{24,51} also permit partial-penetration longitudinal groove welds joining an attachment to a member. If such attachments are provided with a transition radius of 6 inches or more, and are intended for inclusion in Categories B or C, however, these groove welds should have complete joint penetration. To assure this, an appropriate note should be made on the plans.

The welding specifications require that steel backing (backup bars) used for transverse groove welds must be removed and that the joints must be ground smooth. This is necessary to avoid a stress concentration not anticipated in fatigue Categories B and C.

Steel backing used for longitudinal groove welds need not be removed, but is subject to the same fatigue provisions as the member.^{24,51} The backing must be continuous or must be spliced by complete-penetration groove welds before being attached to the base metal. This is necessary to avoid the high stress concentration that would occur at a gap in a discontinuous backing.⁵² Such a stress concentration is not anticipated in the fatigue provisions.⁸ A similar stress raiser occurs where a backing bar is butted against a diaphragm or

other transverse plate.¹⁴ Although this detail is not specifically prohibited in the welding specifications,^{24,51} it should be avoided because it can cause early fatigue cracking.^{17,53}

Welds used to attach steel backing to base metal in longitudinal joints must be continuous for the length of the backing.²⁴ This practice avoids the stress raisers that occur at the ends of intermittent welds.¹⁷ However, stress raisers do occur at the ends of continuous welds used to attach backing for joints that do not extend the entire length of the member, such as those for longitudinal attachments. The effects of such stress raisers on fatigue behavior has not been investigated.

Glass tape backing can produce a very smooth surface,¹⁴ but can be used instead of steel backing only if demonstrated as acceptable by weld-joint qualification procedures.^{24,51} No fatigue data are available on joints made with such tape.¹⁴

Complete-penetration groove welds in prequalified T or corner joints must be reinforced by fillet welds,⁵¹ which provide a better contour at the corners of the intersecting plates. These fillets improve the fatigue behavior of such joints in resisting lateral-bending or axial-tension stresses applied to the stem of the T or to either of the corner plates. As discussed later, lateral-bending stresses often occur due to secondary distortions, especially twisting of flanges.

Prohibited Welds

According to the welding specifications,^{24,51} no welds are permitted on the work except 1) welds detailed on the approved shop plans, 2) welds used in the repair of base metal or welds as authorized by the applicable codes,^{23,24} and 3) other welds approved by the responsible engineer. This provision is necessary to avoid fatigue failures initiated by welds unknown to the engineer.

Certain types of welds, that have low fatigue strengths, are specifically prohibited in bridges:^{24,51} 1) butt joints not fully welded throughout the cross section, 2) intermittent groove welds, 3) intermittent fillet welds except those used to attach stiffeners (presumably vertical stiffener only) to webs, and 4) plug and slot welds on primary tension members.

Workmanship

The welding specifications^{24,51} include several requirements for good workmanship that are very helpful in avoiding fatigue failures. Temporary welds, including tack welds, are subject to the same welding procedure requirements as final welds. Tack welds not incorporated into final welds must be removed in such a manner that the base metal is not nicked or undercut. Other temporary welds must be removed when required by the engineer and the surface must be finished flush. In bridge construction, extension bars and runoff plates must be removed, and the ends of the welds made smooth and flush with the edges of abutting parts. Cracks and blemishes caused by arc strikes must be ground to a smooth contour. The importance of properly repairing areas damaged by fabrication procedures, such as tack welding, is illustrated by the

results of a recent fatigue test on an electrosag-welded girder; early fatigue cracking initiated at a gouge caused by knocking off a tack-welded strongback.²⁷

The welding specifications^{24,51} require that abutting parts of butt joints be carefully aligned, but permit a misalignment not exceeding the lesser of 1/8 inch, or 10% of the thickness of the thinner part, when bending due to misalignment is restrained as it in butt joints of girder flanges. Fatigue tests have shown that misalignment has a large detrimental effect if the bending it causes is not restrained, but only a minor effect if it is restrained.⁵⁴

Bolted Joints

Only high-strength bolts of ASTM A325 or A490 steels²⁵ are permitted in bolted joints subjected to fatigue loadings.⁸ Regardless of whether the bolts are used in slip-resistant (friction) or bearing joints they must be tightened sufficiently to produce a bolt tension at least 70% of the specified minimum tensile strength of the bolt.²⁹ The AASHTO construction provisions⁸ give procedures for bolt tightening and inspection to assure that the required minimum bolt tension is achieved. In addition, AASHTO specifies⁸ nine different classes of surface conditions that are acceptable for the contact surfaces of slip-resistant joints; the class must be shown on the plans for all slip-resistant joints. These requirements for bolt tension and contact surface conditions assure that slip-resistant joints develop sufficient friction to provide a fatigue strength consistent with Category B. Contact surfaces for bearing joints need not satisfy the requirements of these classes, but must be free of loose scale, burrs, dirt, and other foreign material not including paint.⁸

The construction provisions⁸ require the use of sub-punched (or subdrilled) and reamed holes with certain fabrication procedures and joint details, but permit the use of full-size punched or drilled holes with others. In all cases, however, the holes must be clean cut without ragged edges or burrs, and must be properly aligned.⁸ These provisions help to prevent premature fatigue cracks from initiating at the holes.

Contract Plans

As indicated in the preceding paragraphs, many of the requirements for various fatigue detail categories are consistent with normal fabrication practice and need not be specified on the design or working drawings. Other requirements, including the following, must be specifically stated on the plans: 1) the removal of reinforcement by proper grinding for transverse groove welds intended for Category B, 2) use of complete-penetration longitudinal groove welds with attachments having transition radii of 6 inches or more and intended for Categories B or C, 3) grinding of the weld ends at transition radii for details in various categories, and 4) the finishing to a surface roughness not exceeding 1000 microinches²¹ for oxygen-cut edges of plates thicker than 4 inches intended for Category A.

Although not required, notes pertaining to some specified fabrication requirements may help to prevent these requirements from being overlooked and possibly

causing fatigue failures. A few examples of such notes follow: 1) No welds are permitted except as shown on these plans or approved by the engineer, 2) all groove welds must have complete joint penetration except as shown on these drawings, 3) all tack welds not incorporated into final welds must be removed and the base metal ground flush.

FABRICATION PRACTICE RELATED TO FATIGUE OF FRACTURE-CRITICAL MEMBERS

Steel tension members or tension components of members whose failure would be expected to result in collapse of the bridge are considered to be fracture critical. AASHTO requires special procedures for the fabrication and inspection of such members.⁵² Special welding procedures are specified. Weld repairs of base metal at the producing mill are not permitted. Repair welding during fabrication generally requires prior approval by the engineer and is tightly controlled. Extensive nondestructive testing by qualified personnel is required.

These special fabrication and inspection procedures assure high-quality fabrication and thereby improve the safety of the member with respect to fatigue, as well as to brittle fracture. Furthermore, fracture-critical members are designed for the allowable stress ranges for nonredundant members which are well below those for redundant members. This combination of high-quality fabrication and low allowable stress ranges greatly reduces the risks of a fatigue failure in a fracture-critical member.

DETAILS NOT COVERED BY AASHTO

Three simple details not specifically covered by AASHTO,⁸ but used in more complex bridge details, are discussed in this section: cope holes, cruciform joints, and bolted attachments.

Cope Holes

Cope holes are often used, at such locations as butt splices in beams, where fillet or groove welds intersect. The purpose of such holes is to provide access for welding and to avoid 1) possible weld imperfections at a location that is difficult to weld properly, and 2) high shrinkage strains that tend to occur at weld intersections. Fatigue cracks have initiated at weld intersections in highway bridges.⁵⁵ A cope hole can also prevent a crack that initiates in one weld from propagating into an adjacent perpendicular plate. For example, a cope hole in a longitudinal stiffener can prevent cracks that initiate at a welded joint between this stiffener and a transverse stiffener from propagating into the girder web.

AASHTO⁸ does not specifically cover cope holes. If a cope interrupts a longitudinal fillet or groove weld, it might be classified as a weld end and included in Category E. However, available fatigue data indicate that this is too conservative. Specifically, the results of 136 tests^{18,19,26} conducted at the University of Illinois on butt-spliced welded beams with cope hole interrupting the flange/web welds are consistent with Category C

(Fig. 4). The proposed European fatigue code⁴⁶ includes cope holes (interrupting longitudinal flange/web type welds) together with several AASHTO Category C details, such as transverse stiffeners and attachments (thicker than ½ inch) in its Class 72; the allowable SN curve for Class 72 is 20 percent below that for Category C.

Cope holes that interrupt longitudinal welds should have good-quality oxygen-cut or machined edges. Nicks should be repaired by grinding. To be consistent with the test results cited above, the cope holes should be semicircular and have a radius not exceeding 1 inch. Preferably, the cope hole should be cut before the longitudinal weld is made.

Cope holes that interrupt transverse welds, such as the fillet welds between a transverse stiffener and the tension flange, provide the beneficial effect of eliminating a weld intersection without producing a stress raiser in the longitudinal direction. Such cope holes can be conveniently made by chamfering a plate corner to produce a triangular hole. It has been recommended that the cope distance be 2 inches or 4 to 6 times the web (or main plate) thickness, whichever is larger.¹⁷

Cruciform Joints

Fillet- or groove-welded cruciform joints are not specifically covered by the AASHTO specifications,⁸ but occur in several bridge details. In such a joint, the axial stress in a main plate is transmitted through an interrupting plate that is connected to the main plate by fillet or groove welds (Fig. 5a). This joint differs from a transverse-attachment, or stiffener, detail in which the main plate is not interrupted (Fig. 5b).

If complete-penetration groove welds are used in the cruciform joint, fatigue cracking initiates at the toe of the welds, and the fatigue behavior of the joint is similar to that of a transverse stiffener or attachment. Thus, it is appropriate to include this joint in Category C. The proposed European code⁴⁶ does specifically include such joints together with transverse stiffeners and attachments (thicker than ½ inch) in Class 72.

If fillet welds are used, however, a large notch perpendicular to the direction of stress is created by the lack of fusion. The controlling fatigue strength of the joint can be determined either by cracking initiated at the root of the weld as a result of the notch or by cracking across the throat of the weld.⁵⁶ Both types must be checked. The first type of cracking is related to axial stress in the loaded plates, and the fatigue strength of the joint may be considerably below that for Category C.⁵⁶ The proposed European code⁴⁶ includes this case in Class 57, which is slightly above Category E. The second type of cracking is controlled by shear in the welds as defined by AASHTO Category F. The slope of the SN curve for this category is different from that of the other categories. In contrast, the proposed European code⁴⁶ uses an SN curve that has the same slope as the others, but is 10 percent below the SN curve for AASHTO Category E'.

This type of joint involves through-thickness stresses in the interrupting plate. Such stresses are

undesirable because the plate may contain laminations that could cause early cracking due to weld shrinkage or fatigue stresses. However, if the interrupting plate is a narrow plate, such as a stiffener, any laminations that exist are likely to be exposed during cutting.

Bolted Attachments

While bolted attachments are not specifically mentioned in the AASHTO fatigue provisions, they are generally presumed to be covered by the provisions for bolted slip-resistant or bearing connections, that are based entirely on data for symmetric butt joints (double-lap joints). However, the behavior of high-strength bolted attachments differs from that of high-strength bolted butt joints in two ways.

First, the stresses in bolted attachments, as well as in welded attachments,⁵³ are controlled by compatibility requirements. Specifically, the elongation of the attachment (minus any slippage in the bolted detail) must equal the elongation in the main plate between the attachment ends. The stresses in the bolted butt joint, on the other hand, are not controlled by compatibility requirements. Instead, all of the force in the main plate must be transmitted through the splice plates.

Second, the bolted joint can be readily classified as either a slip-resistant or bearing joint, depending on how many bolts are used to carry the desired joint force, but the bolted attachment cannot be so readily classified as a slip-resistant or bearing detail with respect to the stress in the main plate. This is because the attachment is not designed to carry any specific force in that direction, and the force that actually is carried cannot be accurately computed. Therefore, it is somewhat confusing to apply the AASHTO⁸ bolted-connection provisions to bolted attachments.

For bolted attachments, as well as welded attachments,⁵³ the force in the attachment, and the fatigue strength of the detail, depends on the rigidity of the attachment plate or angle. Rigid attachment plates or angles cause high forces and low fatigue strengths. However, microslippage that occurs between the attachments and main plates, especially at the attachment ends, greatly reduces the stress concentrations at these ends.²⁹ Consequently, bolted attachments have much higher fatigue strengths than similar welded attachments. Therefore, it is generally reasonable to assign bolted attachments to Category B, based on the nominal (gross section) longitudinal stress in the main (web or flange) plate. By using the nominal stress as the governing stress, the necessity for classifying the detail as a slip-resistant or bearing connection is eliminated. However, since various types of bolted attachments have not been fatigue tested, it would be prudent to minimize geometric stress concentrations, or use a lower fatigue category, for particularly severe bolted attachment details. For example, a curved transition could be provided for lateral-bracing gusset plates bolted to girder flanges.

SECONDARY BENDING

Secondary bending results from either 1) partial fixity

at beam or truss joints that are assumed to be pinned, or 2) distortions of various members of the bridge, especially bracing members.¹⁷ Secondary bending stresses usually have little effect on the static strength of the bridge and are not calculated in the design. They can, however, cause fatigue cracking in either secondary bracing members or main members. In fact, many of the fatigue cracks that have occurred in actual bridges have resulted from secondary bending.

Since secondary bending stresses are not usually calculated, the normal AASHTO fatigue-design procedures⁸ cannot be applied directly. Instead, the bridge design must be systematically reviewed, after the framing plan has been completed, to identify and correct potential fatigue problems due to secondary bending. To provide guidance for such a review, the relevant general principles are discussed in this section and applied to various specific details in the next section.

Partial End Fixity

The behavior of a typical "pinned" joint connecting a beam to a girder web is illustrated in Figure 6.²⁹ The moment/rotation relationship for the joint is given by the solid curved line, which is specifically for a simple web connection. (A double seat-angle connection to the beam flanges would provide a stiffer joint represented by the dashed curved line.) The relationship between the end moment and end rotation for the beam is represented by the solid straight line. The intersection of the straight and curved lines defines the actual end moment and rotation for the case under consideration. A stiffer connection type results in a higher end moment and lower rotation.

The end moment can cause fatigue cracking in 1) the connecting angle or plate, 2) the beam itself, or 3) the bolts (or weld) attaching the connecting angle to the girder web. Reducing joint stiffness improves fatigue behavior with respect to all three failure modes. This can be accomplished by using 1) the most flexible type of connection (simple web connection), 2) the smallest connecting angle thickness consistent with static design, and 3) the minimum number of bolts necessary to carry the shear.

Cracking in the connecting angle can be further minimized using a large gage for the outstanding leg (distance from angle corner to first row of rivets) in the top third of the beam; a minimum value of

$$g = \sqrt{\frac{Lt}{12}} \quad (5)$$

is recommended.¹⁷ In this equation, g is the gage in the top third of the beam depth (and also in the bottom third if tensile stresses can develop in that region), L is the span in inches, and t is the thickness of the angle in inches.

Cracking in the beam itself usually occurs when the bending strength of the beam has been greatly reduced by coping the flanges to facilitate the connection. Therefore, avoiding such copes, or suitably strengthening coped beams, prevents such crackings.¹⁷

Fatigue failures of the bolts attaching the angle to the

girder web result from direct tension loads caused by the end moment. To avoid such failures, the bolts must be properly tightened since this reduces the variation of stress caused in the bolt by the cyclic tension loads.²⁹

The end moments that develop due to partial fixity in "pinned" truss joints are similar to those in "pinned" beam joints. However, the joint rotations that must be accommodated in truss joints result from member shortening rather than lateral loading on the members and, therefore, are much smaller.

Member Distortions

Distortions of various members in a bridge can cause lateral bending of webs and gusset plates. Usually the lateral bending in the web results from twisting of the flange, lateral movement of the flange, or out-of-plane distortion of the web. Lateral bending in gusset plates usually results from out-of-plane movements imposed on these plates by the members connected to them. Several specific cases are illustrated in Figure 7; most of these are discussed in detail in Reference 17.

Cross bracing (and to a lesser extent, diaphragms) between adjacent girders cause out-of-plane movements in the girder webs when the girders deflect different amounts. Similarly, traffic loadings can cause lateral bracing members to impose out-of-plane movements on the horizontal gusset plates to which they are attached, even though such bracing is designed only to resist lateral buckling and/or wind loading. The out-of-plane movements caused by both types of bracing are usually much greater in curved and skewed bridges than in straight bridges. Vibration of lateral bracing excited by traffic loadings can also cause out-of-plane movements and fatigue cracking.¹⁷ Horizontal traffic loadings, especially on curves, can cause lateral movements of the flanges of floor beams supporting the deck.

As illustrated in Figure 7, lateral forces or moments imposed at locations away from the girder supports can usually be accommodated by twisting of the cross section as a whole without the development of large lateral bending stresses in the web. At supports where twisting of the cross section is prevented, however, large lateral bending stresses can occur in the web, especially if a portion of the web is restrained by stiffeners, connection plates, or connection angles, so that all of the imposed rotation must be accommodated in a short length of the web. The magnitude of the stress varies inversely with the gap distance between the flange/web weld and the end of the stiffener, connection plate, or connection angle weld. Thus, lateral bending stresses in the web can be minimized by providing an adequate gap. A minimum gap of 4 inches has been recommended.¹⁷ Alternatively, lateral bending stresses in the web can be eliminated by welding the stiffener or connection plate to the flange.

Similarly, the lateral bending stresses caused in bracing gusset plates were thought to vary inversely with the gap between the end of the bracing and the girder web or flange to which the gusset is attached. However, a recent finite-element study, and fatigue tests of web gusset plates, suggest that the lateral bending stresses

imposed by the bracing are small and are not greatly affected by the gap.¹⁴

The lateral-bending fatigue strength depends on the type of fillet provided at the intersection of the web and flange or of the gusset plate and web. For rolled shapes, which have a smooth generous fillet, the fatigue strength approaches that of Category A.¹⁷ For fillet welded or complete-penetration groove welded flange/web or gusset/web joints, the fatigue strength is probably equal to that of Category C.¹⁷ Partial-penetration groove welds, such as are used at the corners of box girders, usually do not have a fillet and often have a lack of fusion that is equivalent to a crack at the corner. Consequently, such joints have a low lateral-bending fatigue strength and are particularly susceptible to secondary bending problems.

GOOD AND BAD DESIGNS FOR COMMON TYPES OF DETAILS

Table V identifies the particular details that must be checked for fatigue in five common types of beam and girder bridges. Typical cross sections and details for such bridge types are shown in Volume II of this Handbook and in other U. S. Steel publications.^{57,58} Since plate girder bridges framed with cross beams and stringers are not covered in these publications, a typical cross section for such a bridge is shown in Figure 8.

Suggested good designs for the details used in these types of bridges are discussed in the following paragraphs. Poor designs for these details, and other details that have caused fatigue problems, are also discussed. For some types of details, several different designs, of increasing cost but decreasing fatigue severity, are suggested so that the designer can select the lowest cost detail consistent with his needs. Generally, a detail design should be chosen that provides sufficient fatigue strength so that the girder cross section need not be increased to accommodate fatigue.

The suggested designs are based on the current (1984) AASHTO fatigue provisions⁸ supplemented by other current information. Judgment is necessarily involved in applying the AASHTO provisions to many of the details. Other details, such as cope holes, are not specifically covered by AASHTO⁸. In such cases, however, the basis for the suggested design is indicated. In the figures that illustrate suggested designs the noted fatigue categories relate to the effects of the detail on the main longitudinal members.

During recent years, several details have been reclassified into new AASHTO categories. For example, fillet-welded attachments with a transition radius exceeding 2 inches were recently moved to Category D from higher categories. Generally, the original classifications were based on judgment supported by limited test data on related details. Because of the complex nature of fatigue, such reclassifications are inevitable, and will probably continue in the future as more information is obtained. Similarly, some of the suggestions made herein may require modification as more information becomes available in the future.

Stiffeners

Suggested stiffener details are shown in Figure 9. Transverse intermediate and bearing stiffeners should be attached to the girder web by fillet welds. Preferably, these welds should be continuous, but the specifications^{24,51} permit intermittent welds for intermediate stiffeners. Bearing stiffeners should be tight fitted (milled) or complete-penetration welded to the bottom flange as specified in AASHTO⁸ Article 10.34.6.1 to provide a satisfactory transfer of the bearing force into the stiffeners. Generally, bearing stiffeners are also fitted to the top flange. Intermediate stiffeners should be tight fitted to the compression flange as specified in AASHTO⁸ Articles 10.34.4.6 and 10.48.5.5, but are generally stopped a short distance from the tension flange to avoid the cost of tight fitting. To minimize fabrication costs, the stiffeners are not welded to either flange unless they are used as attachments, which will be discussed later. To reduce the chances of fatigue cracks occurring at the ends of the stiffener welds due to twisting or lateral movements of the flanges during shipping or handling,¹⁷ a gap of 4 to 6 times the web thickness should be provided between the end of the stiffener weld and the near edge of the flange/web welds as specified in AASHTO⁸ Articles 10.34.4.9 and 10.48.5.5. Such a gap is needed even when the stiffener is "tight fitted" to the flange because the fitting tolerance⁵¹ is sufficient to permit significant lateral bending of the web.¹⁷ Transverse stiffeners have been widely fatigue tested and can be classified as Category C.

When longitudinal stiffeners are terminated in contraflexure regions that are subjected to stress reversals, the ends must be designed to resist fatigue. Four different longitudinal stiffener end details are suggested in Figure 9. Only the first of these stiffener details, the Category E detail, has been fatigue tested as such. The classification of the other three is based on the general category of fillet or groove welded attachments with transition radii. The last two details, which are classified as Categories C and B, require that a short distance near the end of the stiffener be complete-penetration groove welded rather than fillet welded. A suggested procedure for fabricating these two details is illustrated in Figure 10 and described below.

First, bevel the portion of the stiffener that will be groove welded as required by the welding specifications.⁵¹ A notch will exist where the bevel is stopped at Location A in Figure 9. This notch should be ground or gouged to a smooth rounded contour to avoid weld flaws at that location. Thus, the end of the groove weld will be similar to the end of a weld repair and should not cause a fatigue problem.

Second, cut the end of the stiffener to the proper radius leaving enough material to permit a satisfactory groove weld where the cut edge becomes tangent to the web. Third, manually groove weld the end portion, starting at the end of the bevel and moving toward the end of the stiffener. Do not use a back-up bar; instead, back gouge as required by the AWS specifications⁵¹ to

achieve a satisfactory complete-penetration groove weld.

Fourth, place submerged arc fillet welds on both sides of the stiffener for its entire length, including the end portions already groove welded. This will provide fillet weld reinforcement to the groove welds, as required⁵¹ for prequalified T joints, and should result in a smaller stress raiser than would result if the fillet welds were stopped where the groove welds started.

Fifth, grind the weld and stiffener material to a smooth contour where the radiused stiffener end becomes tangent to the web. By groove welding this region before the fillet welds are placed, the lack of fusion and weld imperfections that would otherwise occur at this critical location¹⁴ are avoided. A minor stress raiser is expected to occur where the groove weld ends, but is not expected to be critical because it is away from the geometric stress raiser at the end of the stiffener. Nevertheless, it would be desirable to perform fatigue tests on this detail design to verify its fatigue strength.

Figure 9 shows an experimental stiffener end detail consisting of a smooth round hole at the end of a fillet welded longitudinal stiffener. This detail would be easy and cheap to fabricate; the hole could be drilled before the stiffener is attached. The detail has not been used in practice or fatigue tested, but would probably qualify as a Category B or C detail. The hole should reduce the stress concentration at the end of the stiffener in the same way that a circular hole reduces the stress concentration at the end of an existing fatigue crack. As discussed under "Repair of Fatigue Cracks," a hole is widely used to arrest the growth of existing fatigue cracks.

Longitudinal stiffeners are sometimes shop spliced before they are attached to the web. Such splices should be complete-penetration groove welded and nondestructively inspected as shown in Figure 11. Such splices qualify for Category C and may be upgraded to Category B by grinding off the weld reinforcement.

As illustrated in Figure 11, longitudinal stiffeners generally should be terminated at both sides of bolted field splices in girder webs. (There is generally little need to make the stiffener continuous across the splice.) If this is done, the appropriate fatigue category for the detail depends on the stiffener end design as discussed earlier. However, if necessary to provide continuity, two plates can be bolted to both ends of the stiffener. Such a detail has not been fatigue tested, but presumably would qualify as a Category B double-lap bolted joint.

Most intersections between transverse and longitudinal stiffeners occur in compression regions and need not be designed to resist fatigue. However, some intersections occur near points of contraflexure and are subjected to stress reversals, so they must be designed to resist fatigue. Several designs for stiffener intersections are suggested in Figure 11. The simplest approach is to place the two types of stiffeners on opposite sides of the web as shown in Detail 1.

If the stiffeners are placed on the same side of the web, the longitudinal stiffener may be cut at its intersections with transverse stiffeners as specified in AASHTO⁸ Article 10.34.5.4 without significantly reducing the effectiveness in resisting web buckling. The interruption in the longitudinal stiffener, however, causes a stress raiser that affects fatigue behavior even if the stiffener is not considered to be carrying stress. Therefore, the fatigue strength of the intersection will generally be controlled by the design chosen for the stiffener end as illustrated in Details 2 and 3.

Sometimes the transverse and longitudinal stiffeners are welded together as illustrated in Details 4 and 5. Detail 4 qualifies as a Category C detail and may be cheaper to fabricate than Detail 3, which requires the somewhat involved procedure discussed earlier. In Detail 4, the longitudinal stiffener is continuous and interrupts the transverse stiffener. However, the transverse stiffener is fillet welded to the longitudinal stiffener to assure that it is fully effective as a transverse stiffener. A chamfer is provided to avoid the intersection of these fillet welds with the fillet welds joining the transverse and longitudinal stiffeners to the web. This chamfer, and the lack of fusion in the fillet welds joining the transverse stiffener to the longitudinal stiffener, do not reduce the fatigue strength of the detail because of their orientation with respect to the longitudinal stresses in the web and stiffener. Although this type of detail has not been fatigue tested, it is essentially the same as a stiffener welded to the tension flange, and therefore, qualifies as a Category C detail.

In Detail 5, the longitudinal stiffener is interrupted by the transverse stiffener. If properly fabricated, this detail could qualify for Category C, but it has two important disadvantages when compared with Detail 4. First, it is probably more expensive to fabricate, because it requires complete-penetration nondestructively inspected groove welds, instead of fillet welds, joining the stiffeners. Complete-penetration groove welds are required because the stress in the longitudinal stiffener must be transferred through the transverse stiffener as discussed under "Cruciform Joints." Second there is less certainty about its fatigue strength, because the effect of the chamfers in the longitudinal stiffeners cannot be established precisely. (These chamfers are necessary to avoid the intersection of the groove welds with the fillet welds joining the longitudinal and transverse stiffeners to the web.) Fatigue cracks have initiated from such an intersection in an actual bridge.⁵⁵ Probably, these chamfers have a fatigue strength similar to that of the semicircular cope holes discussed earlier, but this has not been verified by fatigue tests.

Some of the stiffener details discussed previously apply to composite box girders as well as plate girders. Other stiffener details that apply only to composite box girders are suggested in Figure 12. The transverse web stiffener details shown in Figure 12 are similar to those for plate girders and qualify for Category C.

Bottom-flange stiffener arrangements are shown in

Details 3, 4, 5, and 6. AASHTO⁸ Article 10.39.4.4.7 specifies that the transverse stiffeners need not be attached to the flange, but must be attached to the web and longitudinal stiffener in a manner sufficient to resist a specified force. Details 3, 4, and 5 are controlled by the transverse fillet welds joining the transverse stiffener to the longitudinal stiffener and to the web or web stiffener. Although none of the details have been fatigue tested, they clearly qualify for Category C unless the transverse stiffener is thicker than 2 inches. If significant secondary bending stresses were to occur in the girder webs as a result of distortions of the cross section, the ends of the welds attaching the transverse stiffeners to the webs would be Category E details. However, AASHTO⁸ Article 10.39.3.2.1 indicates that such stresses generally need not be considered.

Detail 6, which has not been fatigue tested, is limited to Category D if the width of the transverse stiffener flange does not exceed 4 inches, and to Category E or lower if it does. The attachment of the transverse stiffener flange to the longitudinal stiffener flange is similar to the attachment of a plate to a girder flange by transverse welds¹⁴ alone and should be avoided if these welds are spaced more than a few inches apart.

Splices

Flange, web, and cover plates are often spliced to provide transitions in thickness and/or width. Such splices must be made before the plates are joined to other components to form built-up members.⁵¹ Several such splices are illustrated in Figure 13. All have 1 to 2½ tapers as specified in AASHTO⁸ Article 10.18.5.5, and qualify for Category B if the weld reinforcement is properly ground off and for Category C if it is not. Most of these types of splices have been adequately fatigue tested, but splices with a thickness taper and the weld reinforcement in place have been subjected to only very limited testing. Furthermore, poor weld contours in combination with the taper could cause higher than expected stress concentrations. Consequently, it is preferable to avoid such splices even though they are permitted by AASHTO.⁸ A 24-inch-radius curve can be used instead of the taper for width transitions, but fabricators generally prefer the taper. However, the curved transition must be used with A514 steel.⁸ Thickness tapers can be made by 1) chamfering the thicker plate, 2) sloping the weld surface, or 3) a combination of the two.⁵¹ Chamfering, which is usually done by oxygen cutting, adds to joint preparation costs, but reduces the amount of weld metal required.

The weld intersection resulting from a spliced flange fillet welded to a web, has not caused fatigue problems in either laboratory fatigue tests or actual bridges. The inner surface of the splice weld should be ground smooth where the fillet welds cross the splice, but a cope hole should not be provided in the web. Similar intersections between web splice welds and longitudinal stiffener fillet welds are not expected to cause fatigue problems.

Welded butt splices may be used to join portions of long girders or beams either in the shop or field. Bolted

butt splices are used primarily in the field. Suggested designs for such splices are given in Figure 13. Detail 1 uses a semi-circular cope to avoid intersecting welds and to permit weld passes to be made on the under side of the flange. As discussed under "Cope Holes," the cope should be made before the welds, and should have a good-quality oxygen-cut or machined edge. As discussed earlier, this type of splice has been fatigue tested and shown to be consistent with Category C even though it is not specifically covered by AASHTO⁸. Detail 2 uses a circular hole, which may provide a lower stress concentration than the semicircular hole, but this type of splice has not been fatigue tested.

The bolted splice shown in Detail 3, is in Category B based on gross-section stresses if it is designed as a slip-resistant (friction) joint, and based on net-section stresses if it is designed as a bearing joint. Although few bolted beam or girder butt splices have been fatigue tested, such joints are similar to symmetric butt splices which have been widely fatigue tested. If the flanges on the two sides of the joint have different widths, the wider flange should be tapered at a 1 to 2½ slope to meet the narrower flange.

Cover Plates

Four designs for cover-plate ends are suggested in Figure 14. Details 1 and 2, which are typical of present practice, must be assigned to Category E' if the flange thickness exceeds 0.8 inch, and to Category E if it does not. Category E' details are not permitted in non-redundant bridge members. For Detail 1, the transverse end weld may be omitted if the development length from the theoretical end to the actual end is increased to 2 times the cover-plate width as specified in AASHTO Article 10.13.4. However, the end weld is required in Detail 2.

Details 3 and 4 are experimental details that are expected to provide a better fatigue strength. In Detail 3, the cover-plate end is chamfered to a 1 to 3 slope, fillet welded, and carefully ground to this same slope. The results of fatigue tests conducted at the University of Maryland⁵⁹ on 28 cover-plate ends of this design were consistent with Category C. Detail 4 is expected to provide a fatigue strength consistent with Category B, which includes bolted joints. The cover plate should extend a minimum of 1½ times its width beyond the theoretical end, but the longitudinal welds should be stopped at the theoretical end. The bolted end extension should be designed as a slip-resistant (friction) joint to carry the computed force in the cover plate at its theoretical end. Only two fatigue tests have been performed on this type of detail;⁶⁰ the results are consistent with Category B. On the basis of these two results the detail was used on a bridge in Holland.⁶⁰

Diaphragms, Simple Cross Beams, and Brackets

Suggested connection designs for diaphragms, simple cross beams, and brackets are shown in Figure 15. In the diaphragm connections in Details 1, 2, and 3, connection plates or angles are welded or bolted to the main-beam web, and are bolted to the diaphragm web. To minimize lateral bending stresses in the beam web, a

minimum gap of 4 inches should be provided between the connection plate or angle and the nearest flange, or the connection plate should be welded to the flange. The connection plates, of course, are similar to stiffeners and are Category C details with respect to the beams. Although Detail 2 has not been fatigue tested, it should qualify for Category B if the angle is attached to the beam web by properly tightened high-strength bolts as discussed earlier. The diaphragms in Detail 3 can be attached to the connection plates by fillet welds or bolts.

Details 4 and 5 show suggested connections of transverse floor beams to longitudinal girders when the deck is composite or noncomposite. In both cases, the floor beam is bolted to a transverse bearing or intermediate stiffener. In Detail 5, a short horizontal stiffener is fillet welded to the floor beam web to replace some of the bending strength lost by coping the top flange. Such a stiffener is not needed when the deck is composite with the floor beam because the deck strengthens the coped beam.¹⁷

A similar bolted connection between a transverse floor beam and longitudinal stringers is suggested in Detail 6. Because its bottom flange need not be coped, the stringer generally has adequate bending strength without a web stiffener even if the deck is not composite. To minimize the partial fixity at the end of the stringers, the gage distance to the first row of bolts should be consistent with Equation 5.

When the cross beams are attached to stiffeners, these stiffeners should be welded to both flanges to minimize lateral bending of the web. These welds should preferably be stopped short of the flange edges to avoid possible craters. Chamfers should be provided to avoid weld intersections.

Detail 7 shows stringers placed on top of a floor beam and bracket. The splice plate connecting the top flanges of the floor beam and bracket should not be connected to the girder flange; otherwise, relative movements of the girder flanges and stringers can cause high in-plane bending stresses and fatigue cracking in the splice plate.¹⁷ Alternatively, such fatigue cracking can be prevented by framing the stringers into the floor beam and bracket and embedding the top flanges of the girder, floor beam, and bracket in the slab.¹⁷

Cross Frames

Suggested designs for cross frames for plate-girder and composite box-girder bridges are shown in Figure 16. To minimize lateral bending stresses in a plate girder web, the stiffeners to which the cross frames are attached should be fillet welded to both flanges. As shown in Details 1 and 2 of Figure 16, these welds should preferably be stopped short of the flange edges to avoid possible craters at the edge. Chamfers should be provided at the top and bottom of the stiffener to accommodate the flange/web fillet welds and to avoid an intersection between these welds and the stiffener/web and stiffener/flange welds. The chamfer should be a minimum of 1 inch, but 2 inches would usually be preferable.

Many times, the stress in intermediate cross frames is assumed to be small and is not computed. Therefore, the connections of cross frame members to the stiffeners are usually not designed for fatigue and can be made by either high-strength bolts or fillet welds. However, intermediate cross frames in curved-girder bridges,⁶¹ intermediate cross frames in substringer bridges, and end cross frames in all bridges are designed as main members and must be fatigue checked in the usual way. Consequently, such frames might require high-strength bolted connections.

Intermediate cross frames (or diaphragms) and struts are not required in composite box-girder bridges, according to AASHTO Articles 10.39.6.2 and 10.51.6, but they are often used to stiffen the cross section to facilitate handling and erection. If such cross frames are not removed, their effect on the fatigue strength of the box girders must be considered. After the slab has been placed, the box girder has a high torsional rigidity. Consequently, it was formerly believed that the stresses in the cross bracing members will be small, and the stiffeners to which they are attached need not be welded to the flanges. Later, it was demonstrated that fatigue cracks developed in the webs of some in-service box girder bridges at the toe of the fillet welds connecting stiffener to the web, at cross frame locations. Therefore, it is recommended that the web stiffeners be welded to the flanges at cross frame locations.

Lateral Bracing

Until recently, AASHTO required that bottom-flange lateral bracing be used to carry wind loadings in plate girder bridges when the span length exceeded 125 ft. Now, however, AASHTO^a does not specify any particular span length beyond which bottom-flange lateral bracing is required. Since lateral bracing attachments tend to be severe fatigue details, and have caused cracking in actual bridges, lateral bracing should be avoided unless definitely needed.

Several suggested lateral bracing attachments are shown in Figure 17. These attachments must be fatigue checked with respect to both the longitudinal stresses in the main member due to traffic loadings, and the transverse stresses applied to the gusset (attachment) plate by wind forces in the bracing. In addition, connections of the bracing members to the gusset plate must also be fatigue checked for wind loadings. These connections can be made either by bolting, which qualifies for Category B, or fillet welding, which qualifies for Category E. Since the wind can blow from either side of the bridge, the bracing connections and the transverse gusset plate stresses are subjected to complete reversals in two-girder bridges. Such reversals can cause relatively high applied stress ranges, but the allowable stress ranges are also relatively high because only 100,000 cycles of wind loading need be considered.⁸ For multi-girder bridges, two lines of lateral bracing are usually provided. Each line carries wind from only one direction so that reversals do not occur.

From a fatigue standpoint, lateral bracing can gen-

erally be attached to the flange more effectively than to the web. Bolted and welded attachments to the flange are suggested in Details 1 and 2 of Figure 17. The bolted detail is provided with a transition radius to minimize the geometric stress raiser at the projecting edges of the gusset plate as suggested earlier. Also, as noted, this detail has not been fatigue tested.

Detail 2, which is complete-penetration groove welded and has a 6-inch transition radius, qualifies for Category C with respect to both longitudinal and transverse stresses. The weld must be non-destructively inspected if it is subjected to transverse stresses. As illustrated, the ends of the transition radii must be ground smooth, but the weld reinforcement that exists if both the gusset plate and flange are the same thickness need not be removed. If the gusset plate is thinner than the flange, the groove weld must be reinforced with fillets as required for prequalified groove-welded T joints.⁵¹ This type of welded detail had been adequately fatigue tested.

Detail 2 could be upgraded to Category B for both longitudinal and transverse stresses if 1) the gusset plate is the same thickness as the flange, 2) the weld reinforcement is ground off, and 3) the transition radius is increased to 24 inches. On the other hand, Detail 2, must be downgraded to Category D for both longitudinal and transverse stresses if the gusset plate is thinner than the flange and is connected by fillet welds instead of groove welds. If no transition radius is provided, it must be further downgraded to Category E for the longitudinal stresses. None of these modified details, however, has been fatigue tested. A gusset plate fillet welded to the flange surface, as illustrated in Detail 3, has a very low fatigue strength and is not recommended.

Lateral bracing attachments to the web are usually made at transverse stiffener locations where cross frames are also attached. Details 4, 5, 6, and 7 illustrate such attachments, which may be controlled by either the ends of the gusset plate or the gusset/stiffener intersection. Gusset-plate ends are similar to stiffener ends. Consequently, the end detail necessary to achieve the desired fatigue category can be selected from Figure 9. However, since the transverse stiffener prevents the detail from achieving a classification higher than Category C, there is no advantage to choosing an end detail with a category higher than this.

Detail 4 utilizes a continuous gusset plate that interrupts the transverse stiffener, which is fillet welded to the gusset. Since this intersection is essentially the same as a transverse stiffener welded to a tension flange, there is little doubt that it qualifies for Category C. Similarly, the gusset-plate ends are expected to qualify for Category C as discussed under "Stiffeners." Therefore, Detail 4 is classified as a Category C detail although it has not been specifically fatigue tested.

Detail 5 utilizes an interrupted gusset plate that is fillet welded to both the web and transverse stiffener. The ends are provided with a 2-inch radius that is ground smooth near the point of tangency. Thus, the

ends are expected to fit into Category D. The attachment is slotted to fit over the stiffener and then fillet welded to it. As discussed under "Cruciform Joints," the classification of this type of fillet-welded joint, which is adversely affected by lack of fusion perpendicular to the direction of stress, is uncertain. Since the stress in this fillet-welded joint is probably less than the stress at the gusset-plate ends, however, the joint is not expected to be more critical than these ends. Similarly, the cope holes are not expected to be more critical than the ends. Therefore, Detail 5 is expected to be a Category D detail, but there is considerably more uncertainty about it than about Detail 4.

Recently, details with a cutout around the stiffener, like that in Detail 6, have been widely used in bridges, and have been fatigue tested.¹⁴ However, the tested details had square ends that controlled the fatigue strength at values consistent with Category E. Consequently, the classification of the cutout itself has not been established. Probably, it is between that of a small semi-circular cope hole (Category C) and that of an interrupted fillet weld (Category E). Thus, Category D seems to be an appropriate classification for Detail 6, but uncertainty remains about this classification.

Detail 7 is a bolted attachment with a cutout around the stiffener. Although this detail may qualify as a Category B detail, it has been shown as Category C because of uncertainty about the effect of the cutout. It has not been fatigue tested.

Continuous Cross Beams

Intersections between longitudinal and transverse bending members sometimes must be made in such a way that bending continuity is provided in both directions. For example, intersections between longitudinal beams or girders and transverse rigid frames are sometimes made in this way. Such intersections are very complicated¹⁷ and often have very low fatigue strengths. Furthermore, a large number of different intersection details are possible. In most cases, however, the flange of one member either passes through a cutout in the web of the other or is welded to both sides of the web.

In Detail 1 of Figure 18, which was proposed by the Federal Highway Administration,¹³ cross beams are butted against opposite sides of the main girder web. The webs of the cross beams are attached to the web of the main girder by bolted connection angles. Flange splice plates are passed through cutouts in the girder web and are bolted to the top and bottom flanges of both cross beams to provide continuity in that direction.

The cutouts have rounded ends and good-quality oxygen-cut or machined edges to improve fatigue strength. Although most of the moment in the cross beams will pass through the bolted flange splices, some will pass through the web connections. Therefore, the bolts that pass through the girder web will be loaded in direct tension and are subject to prying action.²⁹ Fatigue criteria for such joints are given in Reference 29. Although this type of detail has not been fatigue tested, it is expected to qualify for Category B or C in

both directions.

Fatigue tests¹⁴ have shown that the attachment of cross beam flanges to girder webs by groove welds or fillet welds, as illustrated in Detail 2, results in very low fatigue strengths (Category E or lower) with respect to the stresses in the girder. Cross beam flanges that are passed through a girder web, but are seal welded to that web as illustrated in Detail 3, have even lower fatigue strengths¹⁴ and should not be used.

Trusses

A typical high-strength bolted truss joint is illustrated in Detail 1 of Figure 19. Gusset plates with appropriate curved transitions are provided on both sides of the member so that the joint behaves like the symmetric butt joints on which the AASHTO⁸ fatigue provisions for bolted connections are based. Therefore, such joints qualify for Category B.

When a gusset plate is welded to one truss member and bolted to others, as illustrated in Detail 2, its fatigue behavior is similar to that described previously for lateral bracing attachments. The appropriate fatigue category, for the truss member to which the gusset plate is welded, depends on the transition radius and type of weld.

Attachment of the floor beams to the truss members involves many of the different types of attachments that have been classified by AASHTO⁸ and discussed previously.

TREATMENTS THAT IMPROVE FATIGUE PERFORMANCE

Many different treatments have been applied to various details in an attempt to improve their fatigue performance. A comprehensive assessment of such treatments is given in Reference 62. The application of several of the more practical treatments to new bridge members is discussed in this section. The application of some of these treatments, in the repair of cracked members in service, is discussed in the next section. Because these treatments are not presently recognized in the AASHTO specifications,⁸ they cannot be used to qualify a detail that would not otherwise satisfy the specifications. However, they could be used to provide an extra margin of safety for key members, and to lengthen the actual fatigue life of the bridge.

Peening

Peening has been used successfully to improve fatigue strength in many different applications. To be effective, the peening should be uniform in intensity and coverage. Three types of peening have been used:^{62,63} 1) shot peening⁶⁴ in which pellets are shot at the surface, 2) single point peening^{13,65} in which a single 1/2-inch diameter rod is applied pneumatically and 3) multiple point peening^{65,66} in which 0.08-inch-diameter rods are applied pneumatically. In all of these methods, the peening cold works the surface and causes a thin layer of compressive residual stresses that are balanced by tensile residual stresses below this layer. The peening also closes shallow surface imperfections or cracks.

The surface compressive residual stresses are

superimposed on the applied stresses and thereby improve the fatigue strength as discussed in Section III. The improvement is greatest when the applied tensile stresses, both constant and cyclic, are low enough so that the net cyclic stresses are always in compression. High tensile dead load stresses tend to reduce the effectiveness of peening unless it is done while the dead load stresses are being applied. The tensile residual stresses below the surface can be detrimental to crack extension behavior if the crack front resides or propagates into this region.

For bridge applications, peening is most often applied to the toes of transverse or longitudinal welds, and to groove welds with the reinforcement in place. For such details, increases in fatigue strength (at 2,000,000 cycles) of about 20 to 200 percent have been reported.⁶² For fillet-welded details, the improvement that can be achieved by peening the toe is often limited by root cracking.¹³ Root cracking is normally less critical than toe cracking but becomes more critical when the toe is improved. Peening requires a lesser degree of operator skill, and is generally cheaper, than the other treatments. Of the three peening methods, the single point method is generally preferable with respect to both cost and effectiveness.⁶²

TIG Remelting

The gas tungsten arc (TIG) welding process can be used to remelt the toe of a previously deposited fillet weld and thereby eliminate shallow imperfections that often occur at that location. TIG remelting is generally regarded^{13,62} as the most reliable treatment for improving the fatigue strength of fillet weld details, but requires greater operator skill and is more costly than peening. In fact, it is estimated that the cost of TIG remelting is about 3 times that of single point peening. Usually, it is necessary to remove mill scale by sand blasting before TIG remelting¹³ and to use appropriate procedures^{13,67} to help avoid weld craters at critical locations.

Since the improvement due to this treatment is caused by the removal of imperfections, it is not significantly affected by dead load stresses, but it may be limited by root cracking. Increases in fatigue strength (at 2,000,000 cycles) of 40 to 250 percent and 15 to 35 percent, respectively, have been reported⁶² for transverse and longitudinal fillet welds. Fisher¹³ indicated that fillet welded details, such as cover-plate ends, can be improved by one AASHTO detail category (from E to D, etc.) by TIG remelting.

Grinding

Grinding to a smooth contour is a requirement for several types of details covered in the AASHTO specifications.⁸ These grinding requirements are discussed elsewhere; only grinding intended to improve other types of details, and not covered by AASHTO, is discussed here. Specifically, the discussion covers grinding to remove imperfections and to improve the contour at the toes of fillet welds.

Grinding can be done with either a rotary disc (typically a 4-inch disc with a 60-150 grit) or a conical burring

bit. Burr grinding is preferred for treating fillet weld toes because disc grinding tends to be erratic and can cause worse conditions than existed before grinding.⁶² Some investigators used three successive 30-200 grit polishes after burr grinding to further improve the surface. The cost of burr grinding with and without polishing is estimated⁶² to be 3 to 4 and 12 times, respectively, that of single point peening.

Fisher¹³ applied burr grinding without subsequent polishing to cover-plate ends, and did not achieve a significant improvement in fatigue strength. Other investigators were able to achieve 40 to 200 percent improvements in the fatigue strengths of various fillet welded details by burr grinding either with or without polishing⁶². Because of its higher cost and less reliable results, however, burr grinding is a less attractive alternative than peening or TIG remelting for improving fillet welded details.

Spot Heating

As discussed in Section III, welding or flame cutting can cause compressive residual stresses at the end of a previously placed longitudinal fillet weld and thereby improve fatigue strength. A similar beneficial effect can be achieved by spot heating. As mentioned under Peening, the beneficial effects of compressive residual stresses are greatest when the applied tensile stresses are low enough so that the net cyclic stresses are always compressive.^{63,68} Improvements of 70 to 200 percent in the fatigue strength (at 2,000,000 cycles) have been achieved with spot heating. However, data on this treatment are very limited, and detailed procedures are not available for applying the treatment to various detail geometries. Therefore, it is less attractive than the treatments discussed previously.

REPAIR OF FATIGUE CRACKS

Selecting the best method for repairing a fatigue crack in a bridge member depends primarily on the crack's size and location. Large cracks may require major repairs such as replacing members, adding bracing to redistribute load, or adding bolted splice plates. In certain cases, it may be desirable to leave the crack unaltered and merely monitor its future growth.⁶⁹ Several different methods are available for repairing or arresting cracks that fall between these two extremes. These methods are discussed here.

Grinding

Steel producers are permitted²³ to remove surface or edge imperfections up to 1/8 inch deep by grinding without replacing the removed metal. Edge or surface fatigue cracks (probably initiated by a nick) not exceeding this depth can easily be repaired in the same way. In fact, even deeper edge cracks could be safely removed by grinding provided that the ground area is well faired with gentle changes in contour. Grind marks perpendicular to the direction of stress should be avoided. Fatigue cracks at the toe of a fillet weld are more difficult to remove successfully by grinding¹³ because more abrupt changes in contour are required at the weld.

Peening

Single point peening has been used successfully to repair fatigue cracks up to 1/8-inch deep at the toe of a fillet weld.¹³ It is a simple, effective, and economical way of making repairs, but should be used with caution unless there is reliable evidence to show that the cracks are not deeper than 1/8 inch and that the peening was uniform in coverage and severity to provide compressive stresses along the entire front of a 1/8-inch deep crack. Otherwise, a buried crack will remain and severely limit the remaining fatigue life.

TIG Remelting

Gas tungsten arc remelting has been shown¹³ to be effective in removing cracks up to 3/16 inch deep at the toe of a fillet weld. Again, caution is needed to avoid buried cracks.

Rewelding

Larger fatigue cracks can often be repaired in the same way that unacceptable welds and internal imperfections are repaired during the original fabrication. The AWS specifications⁵¹ cover such repairs. First, the crack is completely removed by air carbon arc gouging, oxygen gouging, chipping, grinding, or machining. It may sometimes be desirable to use dye-penetrant or magnetic-particle inspection to assure that the crack has been completely removed. Next the gouge is re-welded to its original contour. Subsequent grinding to a smooth contour may sometimes be desirable.

Generally this is the most reliable method of repairing a fatigue crack because the crack can be fully removed and the repaired region restored to its original condition. The repair will extend the remaining fatigue life of the detail, but will not always fully restore the original life because of the effects of accumulated cycles outside of the repaired region. Treatments such as peening, TIG remelting, and grinding can be used after rewelding to further extend the remaining life of the detail. Residual stresses caused by extensive rewelding on an existing bridge could affect the fatigue strength of adjacent details and should be considered in selecting an appropriate repair method. An NCHRP study (Project 12-27) is currently developing detailed guidelines for the weld repair of large cracks in existing bridges.

Drilling Holes

The growth of full-thickness fatigue cracks in steel plates can be arrested by drilling holes at the crack ends. This technique has been successfully used in many different applications including bridges.^{13,14,69} The purpose of the hole is to reduce the very high stress intensity that occurs at the crack tip. Therefore, it is essential that the hole include the crack tip. Since the actual end of a fatigue crack is difficult to detect visually, it is suggested the the near edge of the hole be placed at the apparent crack end to assure that the actual end will be within the hole. Also, it is advisable to dye-penetrant inspect the hole surface to verify that the crack tip has been removed. Hole diameters between 0.5 and 1.0 inch have been used.^{13,14,69}

The fatigue category for a circular hole in a plate

generally ranges from B to D depending on the smoothness of the hole edges.⁶⁹ A carefully reamed hole qualifies as Category B.⁶⁹ Since it is not important that the hole be precisely circular, hand filing can be used if needed to improve smoothness.

The fatigue strength of a crack with circular holes at both ends is less than that of a single circular hole and depends on the length between the outer edges of the holes. If this length is below a limiting value, L_{LIMIT} , further cracking will not occur. The following equation is proposed to define the limiting length

$$L_{LIMIT} = \frac{200}{S_r^2} \text{ for } L_{LIMIT} \geq 1 \text{ inch} \quad (6)$$

where L_{LIMIT} is in inches and S_r is the applied stress range in ksi. The actual maximum stress range occurring in the bridge, rather than an artificially high design value, should be used in this equation. The equation is derived in Appendix A from a stress-intensity threshold developed¹⁴ from a rather limited number of data and therefore should be regarded as approximate. The equation implies a fatigue limit of 14 ksi for a single 1-inch-diameter hole; this is slightly below the fatigue

limit of 16 ksi for Category B.

High-strength bolts with washers can be placed in the drilled holes and tightened by the turn-of-nut method further to reduce the possibility of cracking.^{14,69} This produces compressive stresses around the hole, but makes it more difficult to inspect for new cracks. The nonburr side of the washer should be placed against the plate to avoid cracks initiated by the burr⁶⁹.

Replacing Rivets

The fatigue life of riveted joints in existing bridges can be considerably extended by merely replacing some of the rivets with high strength bolts. The maximum extension can be achieved by replacing all rivets, and repairing all observed cracks in the joint plates. However, life extensions of 2 to 6 times can be obtained by merely replacing rivets at locations where cracks can be observed in the adjacent plate material.⁷⁰ With this approach, cracks in the plates need not be repaired unless they extend more than 1 inch beyond a rivet head. The bolts should be tightened by the turn-of-nut method as specified for bolted joints. Washers under the turning elements should be placed with the nonburr side against the plate.⁶⁹

5. SUMMARY

Four currently used fatigue-design approaches are described briefly. It is concluded that the stress-life detail-category approach is most convenient for structural applications. In this approach, which is utilized in the AASHTO specifications, typical structural details are grouped according to severity, and allowable stresses corresponding to various design lives are given for each group. The experimental basis for the AASHTO detail categories is discussed in depth. Application of the AASHTO procedures in the design and rating of highway bridges is described, and a proposed new method that more accurately approximates conditions in actual bridges is presented.

Several fabrication practices affect fatigue performance; these practices are identified and discussed. Secondary bending can occur in bridges as a result either of partial end fixity of joints that are assumed to be pinned, or of distortions of various members, especially bracing members. Secondary bending stresses are usually not calculated in design, but can cause fatigue

cracking as discussed in this section.

Suggested good designs are presented for details for 1) stiffeners, 2) splices, 3) cover plates, 4) diaphragms, simple cross beams, and brackets, 5) cross frames, 6) lateral bracing, 7) continuous cross beams, and 8) trusses. Also, poor designs that have caused fatigue cracking are discussed. Some of the discussed details utilize cope holes, cruciform joints, and bolted attachments, that are not specifically covered by AASHTO. Consequently, information is presented on the fatigue strengths of these three simple details.

Several treatments that have been shown to be effective in improving the fatigue performance of new bridge members are described: 1) peening, 2) grinding, 3) TIG remelting, and 4) spot heating. Methods of repairing or arresting fatigue cracks in existing bridge members are also discussed. These include 1) grinding, 2) peening, 3) TIG remelting, 4) rewelding, 5) drilling holes, and 6) replacing rivets with high strength bolts.

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7. TABLES

TABLE I
AASHTO DETAIL CATEGORIES

General Condition	Situation	Kind of Stress	Stress Category (See Table II)	Illustrative Example (See Figure 1)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1,000 or less.	T or Rev ^a	A	1, 2
Built-Up Members	Base metal and weld metal in members without attachments, built-up plates, or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3, 4, 5, 7
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends			
	(a) Flange thickness < 0.8 in.	T or Rev	E	7
	(b) Flange thickness > 0.8 in.	T or Rev	E'	7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by nondestructive inspection.	T or Rev	B	8, 10, 14
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no greater than 1 to 2½, with grinding in the direction of applied stress, and weld soundness established by nondestructive inspection.	T or Rev	B	11, 12
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8, 10, 11, 12, 14
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	13
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, is greater than 12 times the plate thickness or greater than 4 inches long.	T or Rev	E	13
	Base metal at details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length when weld soundness transverse to the direction of stress is established by nondestructive inspection.			
	(a) When provided with transition radius equal to or greater than 24 in. and weld end ground smooth	T or Rev	B	14
	(b) When provided with transition radius less than 24 in. but not less than 6 in. and weld end ground smooth	T or Rev	C	14
	(c) When provided with transition radius less than 6 in. but not less than 2 in. and weld end ground smooth	T or Rev	D	14
	(d) When provided with transition radius between 0 in. and 2 in.	T or Rev	E	14

**TABLE I (cont'd.)
AASHTO DETAIL CATEGORIES**

General Condition	Situation	Kind of Stress	Stress Category (See Table II)	Illustrative Example (See Figure 1)
Fillet ^b Welded Connections	Base metal at intermittent fillet welds	T or Rev	E	—
	Base metal adjacent to fillet welded attachments with length L, in direction of stress less than 2 in. and stud-type shear connectors	T or Rev	C	13, 15, 16, 17
	Base metal at details attached by fillet welds with detail length, L, in direction of stress between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev	D	13, 15, 16
	Base metal at attachment details with detail length, L, in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in.	T or Rev	E	7, 9, 13, 16
	Base metal at details attached by fillet welds regardless of length in direction of stress (shear stress on the throat of fillet welds governed by stress category F)			
	(a) When provided with transition radius equal to or greater than 2 in. and weld end ground smooth	T or Rev	D	14
	(b) When provided with transition radius between 0 and 2 in.	T or Rev	E	14
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connected material.	T or Rev	B	18
	Base metal at net section of high-strength bolted bearing-type connections	T or Rev	B	18
	Base metal at net section of riveted connections	T or Rev	D	18
Fillet Welds	Shear Stress on throat of fillet welds	Shear	F	9

^a"T" signifies range in tensile stress only; "Rev" signifies a range of stress involving both tension and compression during a stress cycle.

^bGusset plates attached to girder flanges with only transverse fillet welds, not recommended.

TABLE II
AASHTO ALLOWABLE FATIGUE STRESS RANGES

Redundant Load Path Structures*				
Category See Table I	Allowable Range of Stress, F_{sr} (ksi)^a			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For 2,000,000 Cycles
A	60	36	24	24
B	45	27.5	18	16
C	32	19	13	10
				12 ^b
D	27	16	10	7
E	21	12.5	8	5
E'	16	9.4	5.8	2.6
F	15	12	9	8
Nonredundant Load Path Structures				
Category See Table I	Allowable Range of Stress, F_{sr} (ksi)^a			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For 2,000,000 Cycles
A	36	24	24	24
B	27.5	18	16	16
C	19	13	10	9
			12 ^b	11 ^b
D	16	10	7	5
E ^c	12.5	8	5	2.5
F	12	9	8	7

*Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

^aThe range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

^bFor transverse stiffener welds on girder webs or flanges.

^cPartial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures.

TABLE III
AASHTO DESIGN-LIFE CATEGORIES

Main (Longitudinal) Load Carrying Members				
Type of Road	Case	ADTT^a	Truck Loading	Lane Loading^b
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	2,000,000 ^c	500,000
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000
Transverse Members and Details Subjected to Wheel Loads				
Type of Road	Case	ADTT^a	Truck Loading	
Freeways, Expressways, Major Highways, and Streets	I	2,500 or more	over 2,000,000	
Freeways, Expressways, Major Highways, and Streets	II	less than 2,500	2,000,000	
Other Highways and Streets	III	—	500,000	

^a Average Daily Truck Traffic (one direction).

^b Longitudinal members should also be checked for truck loading.

^c Members shall also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 3.23.2 for one traffic lane loading.

TABLE IV
FATIGUE EQUATIONS AND CONSTANTS FOR PROPOSED NEW METHOD

Category (1)	F_{srL} in kips per square inch (2)	Constant A (3)
A	12	240×10^8
B	8	105×10^8
C (stiffeners)	6	37×10^8
C (other attachments)	5	37×10^8
D	3.5	20×10^8
E	2.5	10×10^8
E'	1.3	4×10^8
F	4	10×10^8

Note: $N = A/F_{sr}^3$; N = estimated minimum number of stress cycles to failure; F_{sr} = design stress range based on W_F , in kips per square inch; F_{srL} = maximum allowable stress range for infinite fatigue life, in kips per square inch; A = constant listed herein.

TABLE V
DETAILS THAT MUST BE FATIGUE CHECKED

Type of Detail	Type of Bridge				
	Plate Girder				Box Girder
	Rolled Beam	Multi Girder	Sub-stringers	Cross Beams	
Web/Flange Weld	—	A	A	A	A
Transverse Stiffener (Intermediate; Bearing)	NM/S	S	S	S	S
Longitudinal Stiffener (End; Intersection with Transverse Stiffener; Long. Weld)	—	S	S	S	S
Butt Splice	S	S	S	S	S
Cover Plate (End; Longitudinal Weld)	S	—	—	—	—
Bracket for Overhang or Side-walk (Attachment; Connections)	S	S	S	S	S
Diaphragm (Attachment)	A	—	—	—	S
Cross Frame (Attachment; Frame; Connections)	—	A	A	A	S
Cross Beam	—	—	—	A	—
Lateral Bracing (Attachment; Frame)	S	S	S	S	S
Shear Studs	NM/C	NM/C	NM/C	NM/C	NM/C

Symbols:

A = always; S = sometimes; NM = negative moment region only; C = composite.

Notes:

- 1) Attachment means that the effect of the attachment on the main member must be checked.
- 2) Frame means that the secondary frame itself must be checked.
- 3) Connections means that the connection between the secondary frame or member must be checked.

8. FIGURES

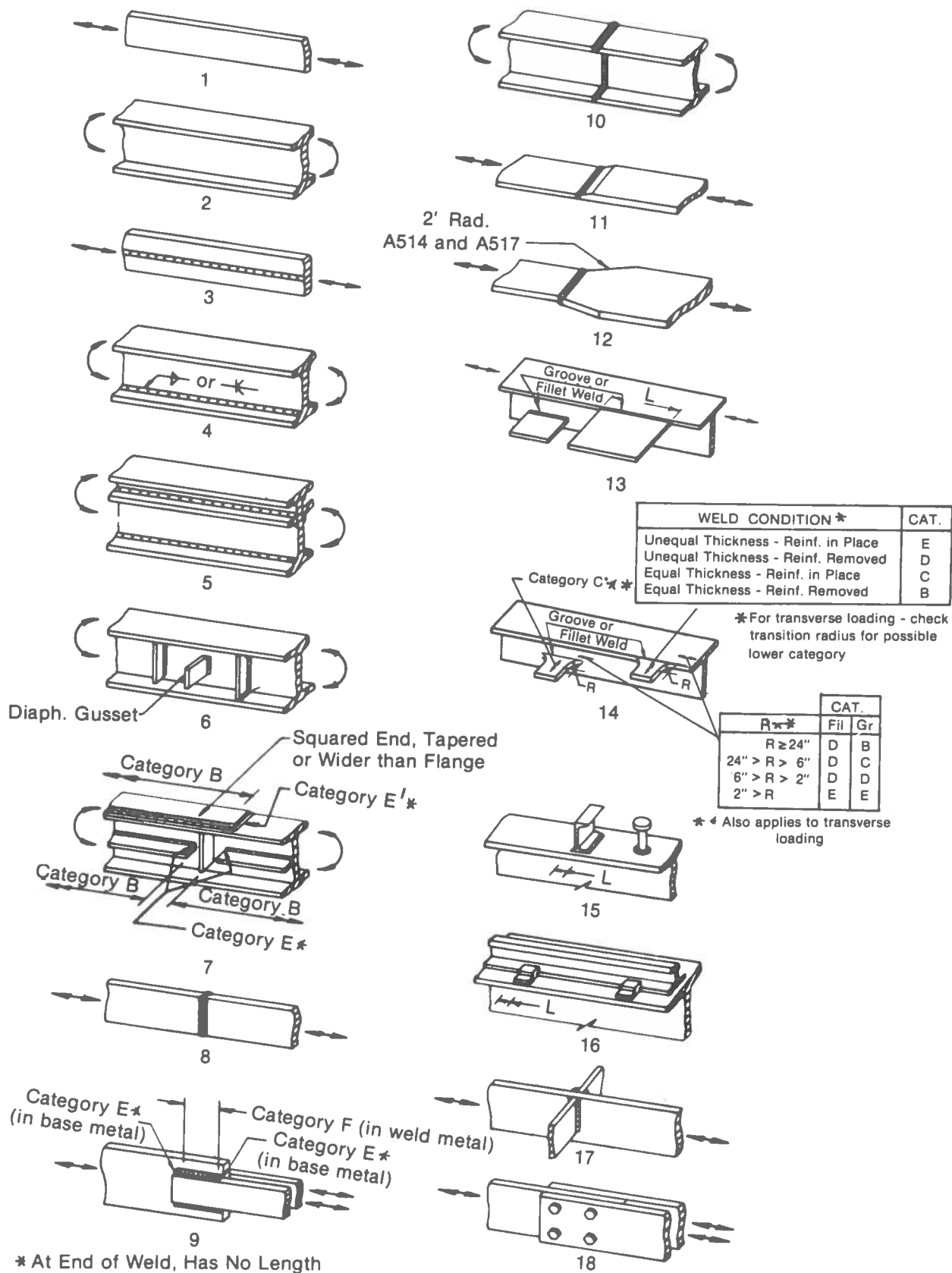


Figure 1. AASHTO detail categories. Reprinted from AASHTO "Standard Specifications for Highway Bridges" 13th Edition, 1983.

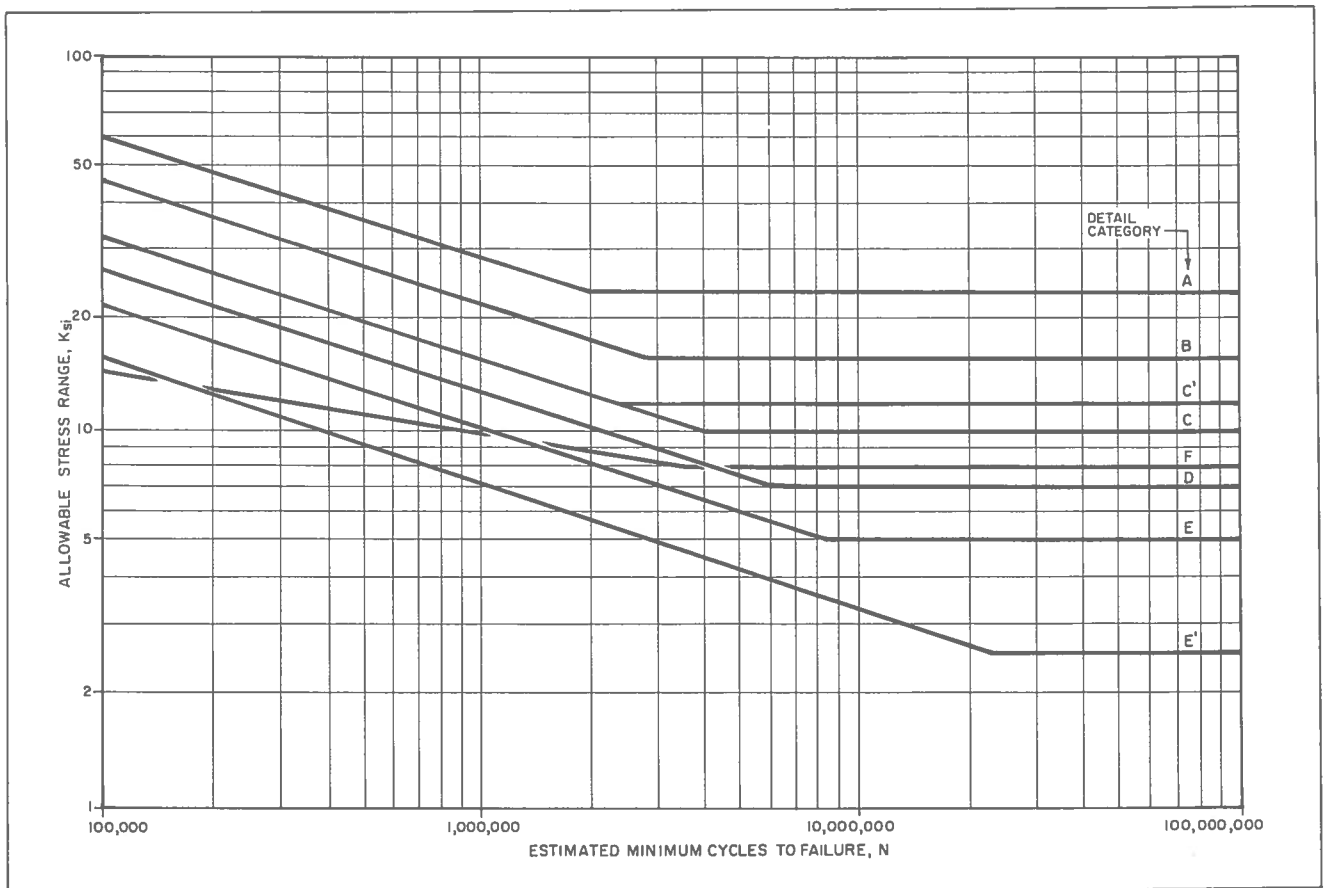


Figure 2. AASHTO allowable SN curves for redundant members.

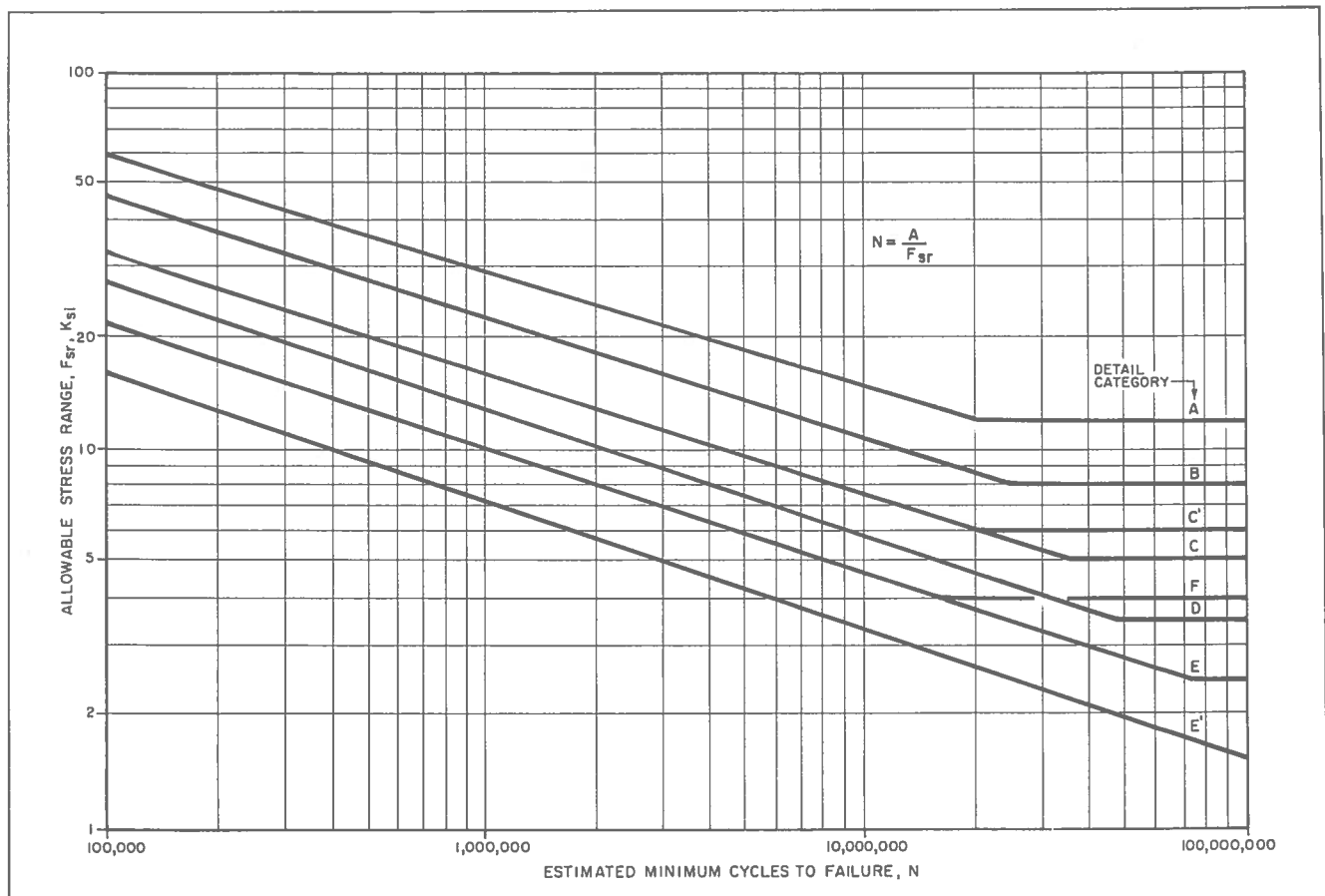


Figure 3. Allowable SN curves for proposed new method.

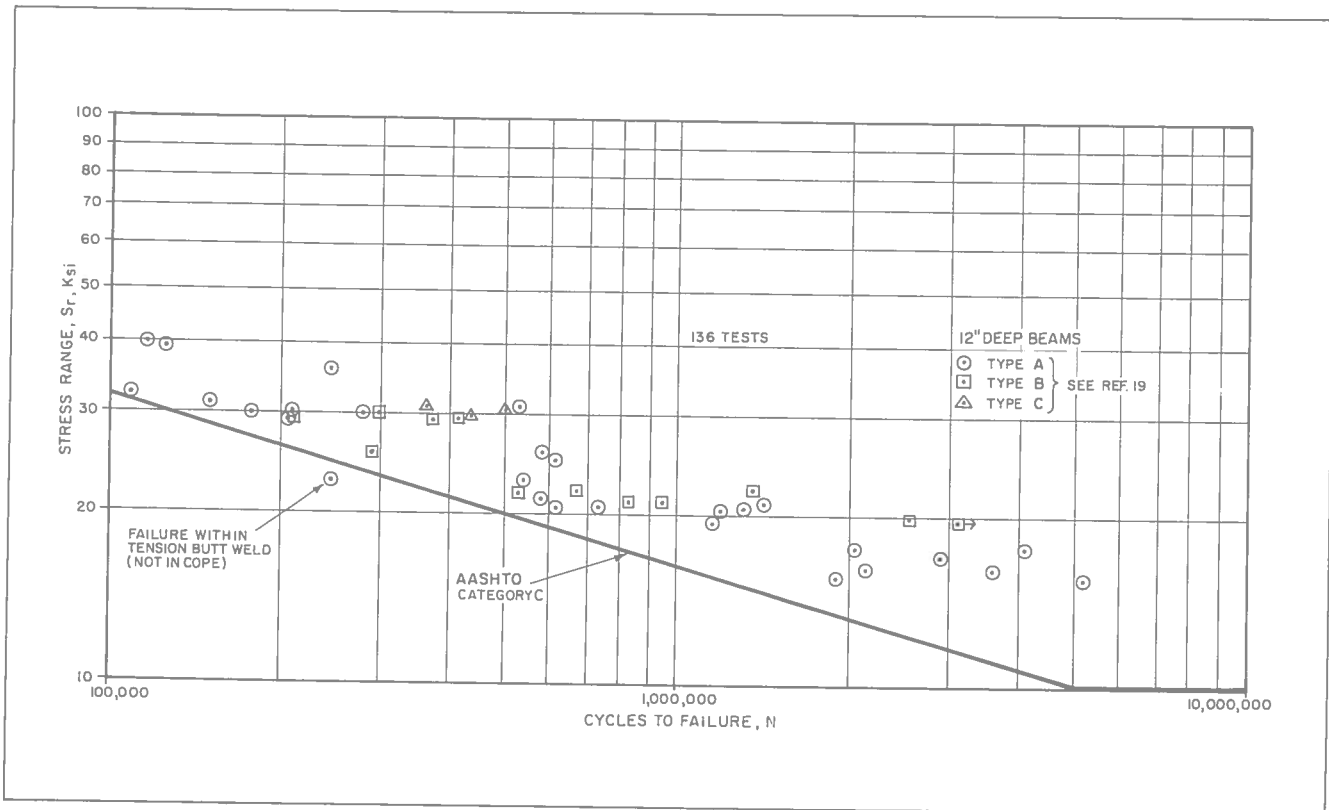


Figure 4. SN curve for cope holes.

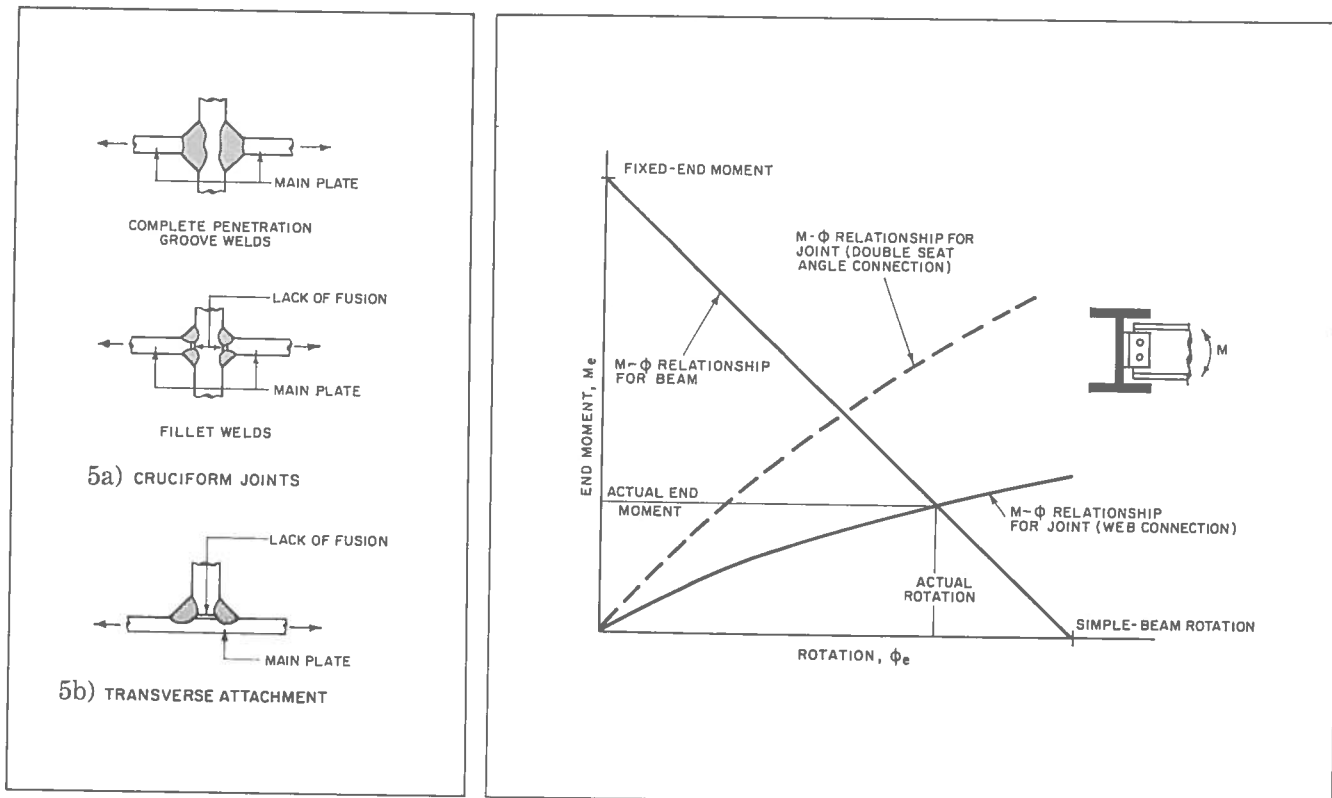


Figure 5. Comparison of cruciform joints with transverse attachments.

Figure 6. Behavior of simple-beam end connections.

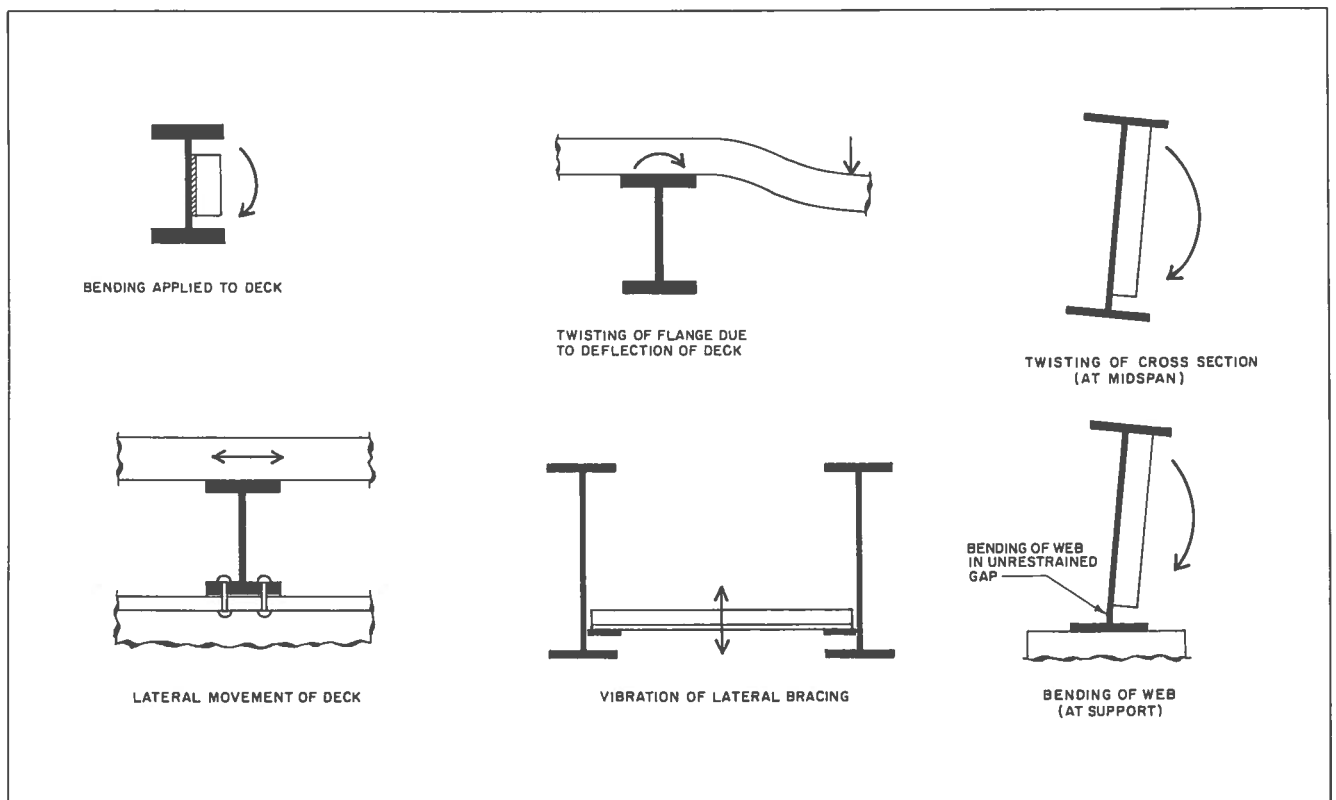


Figure 7. Lateral bending of web.

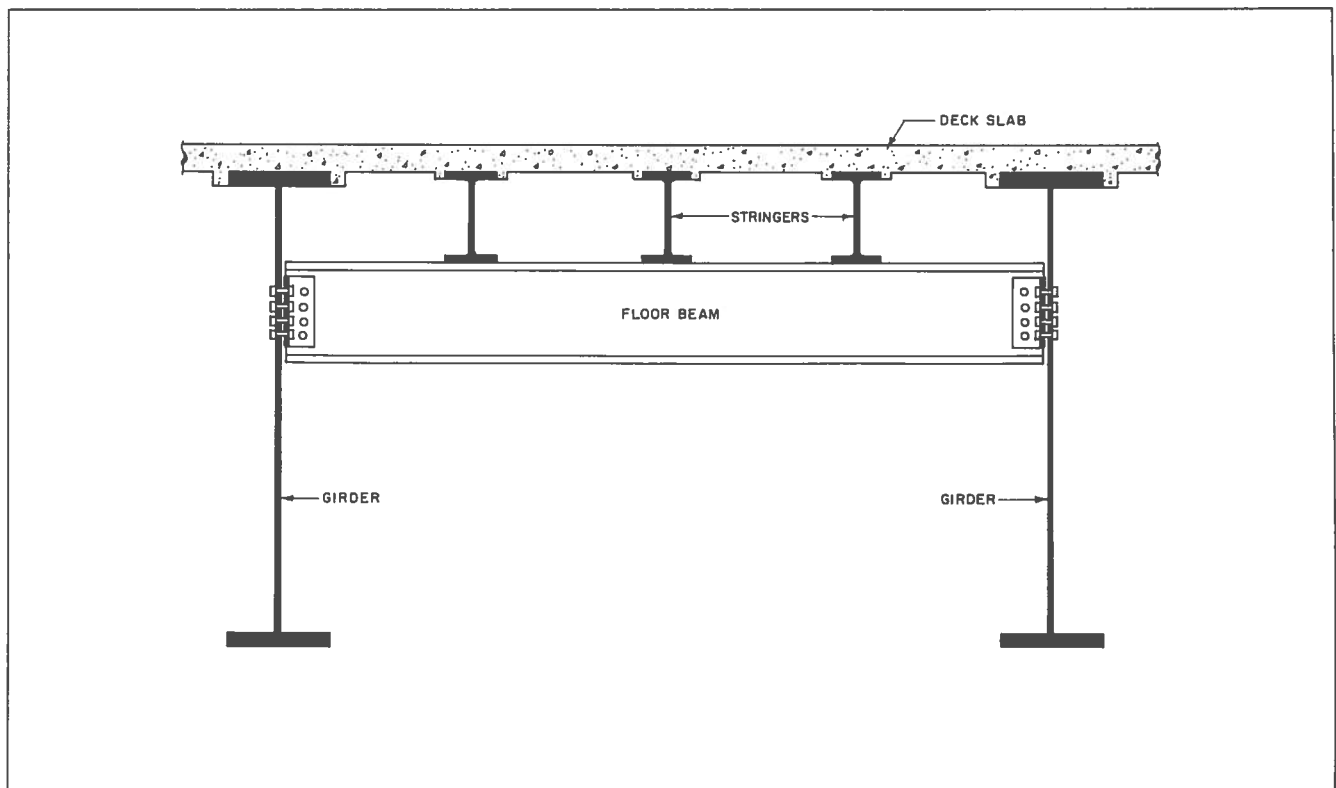


Figure 8. Cross-section for plate girder bridge with floor beams and stringers.

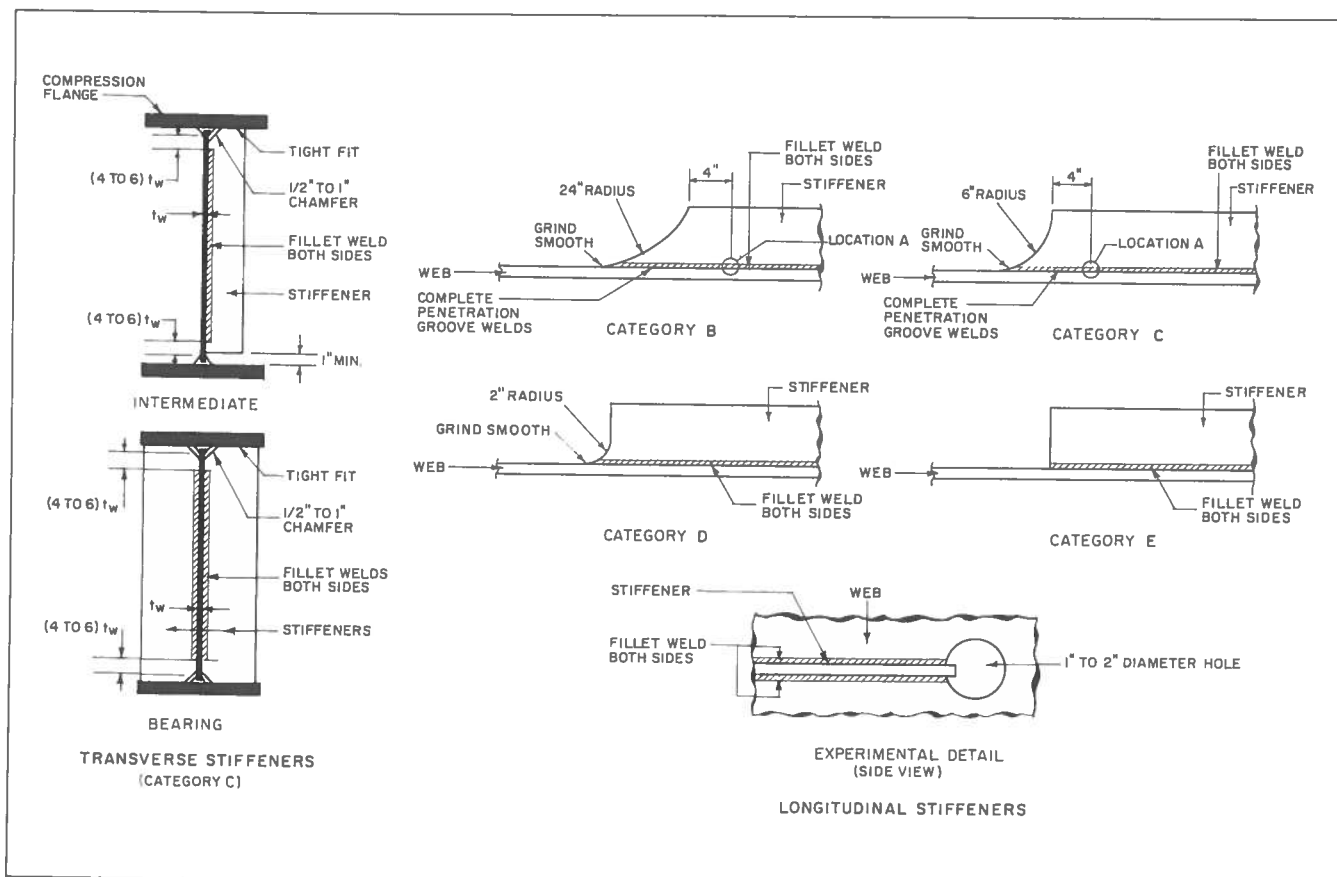


Figure 9. Plate girder stiffeners.

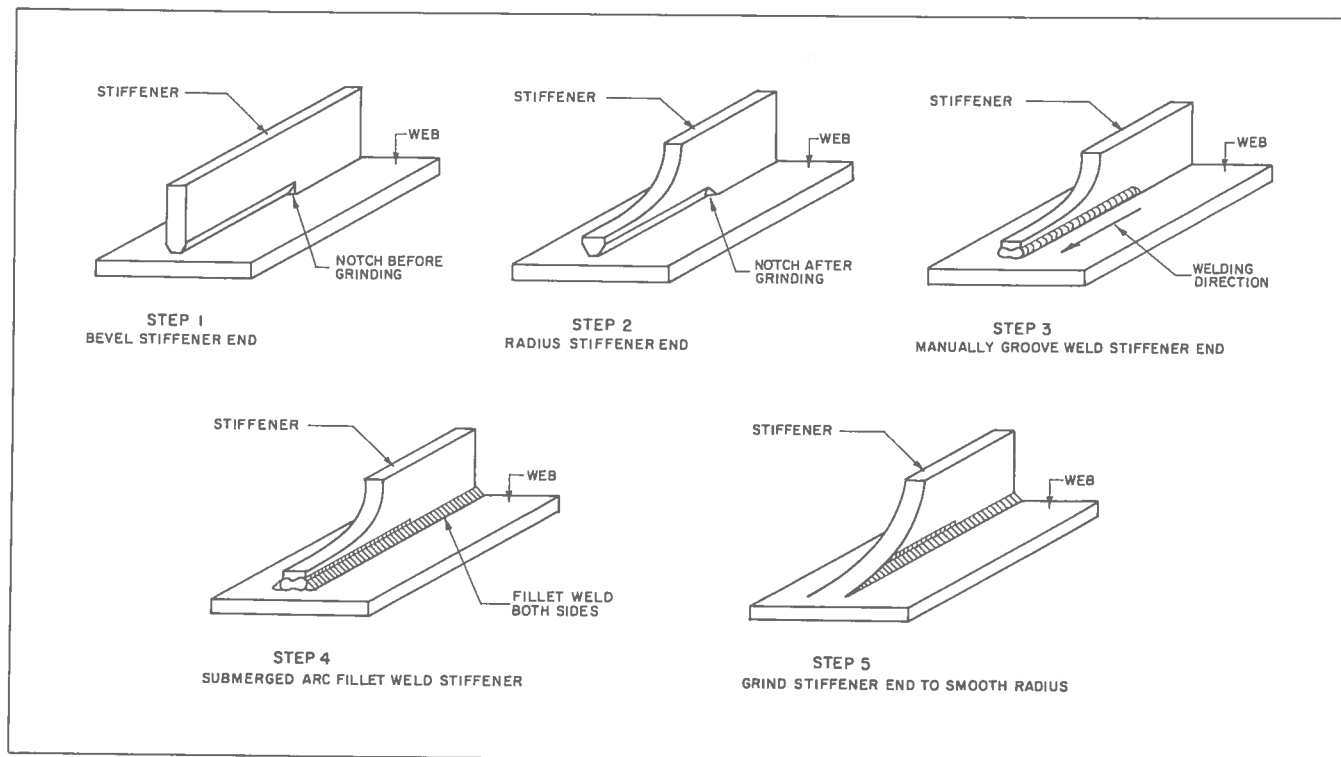


Figure 10. Fabrication procedure of Category C stiffener end.

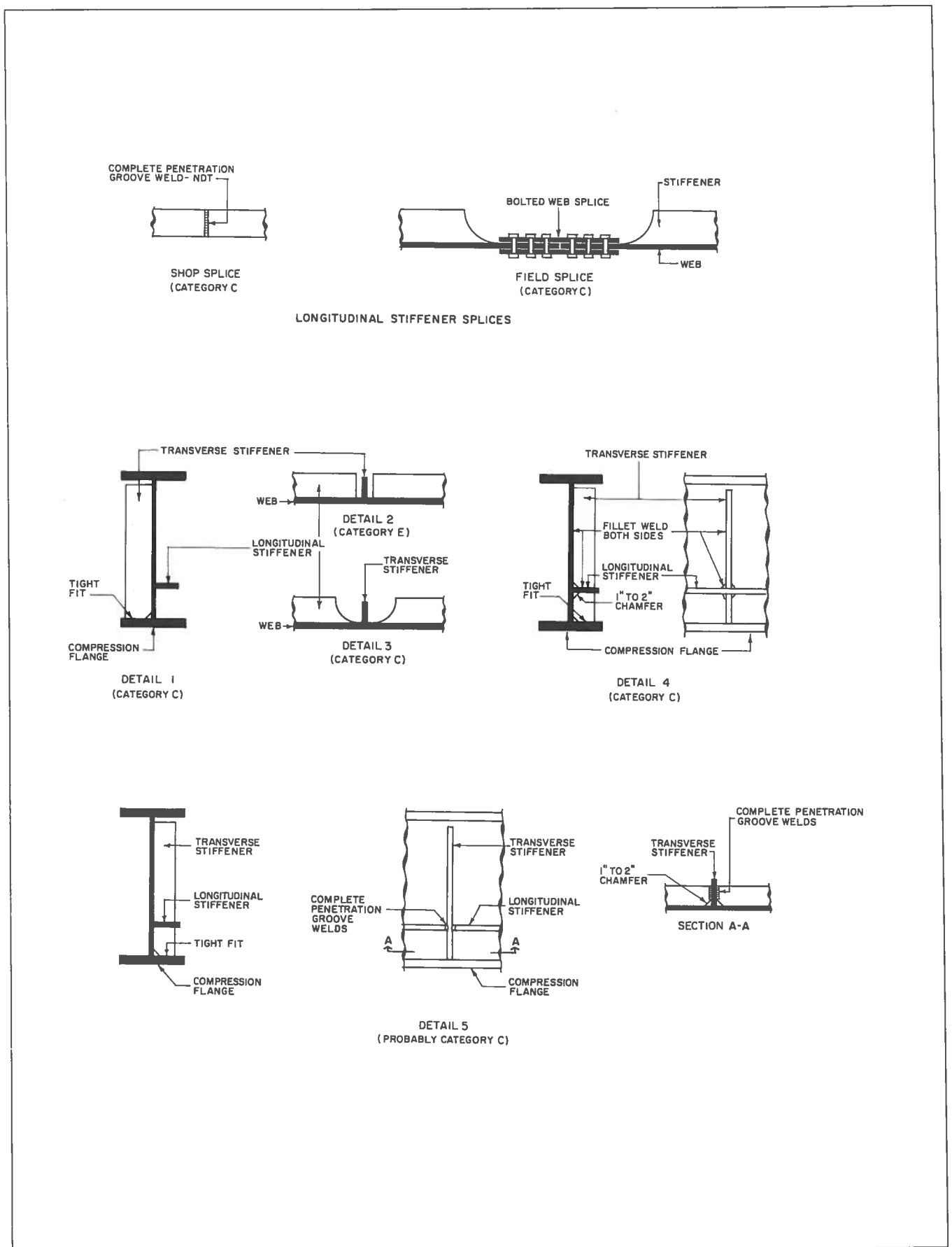


Figure 11. Stiffener splices and intersections.

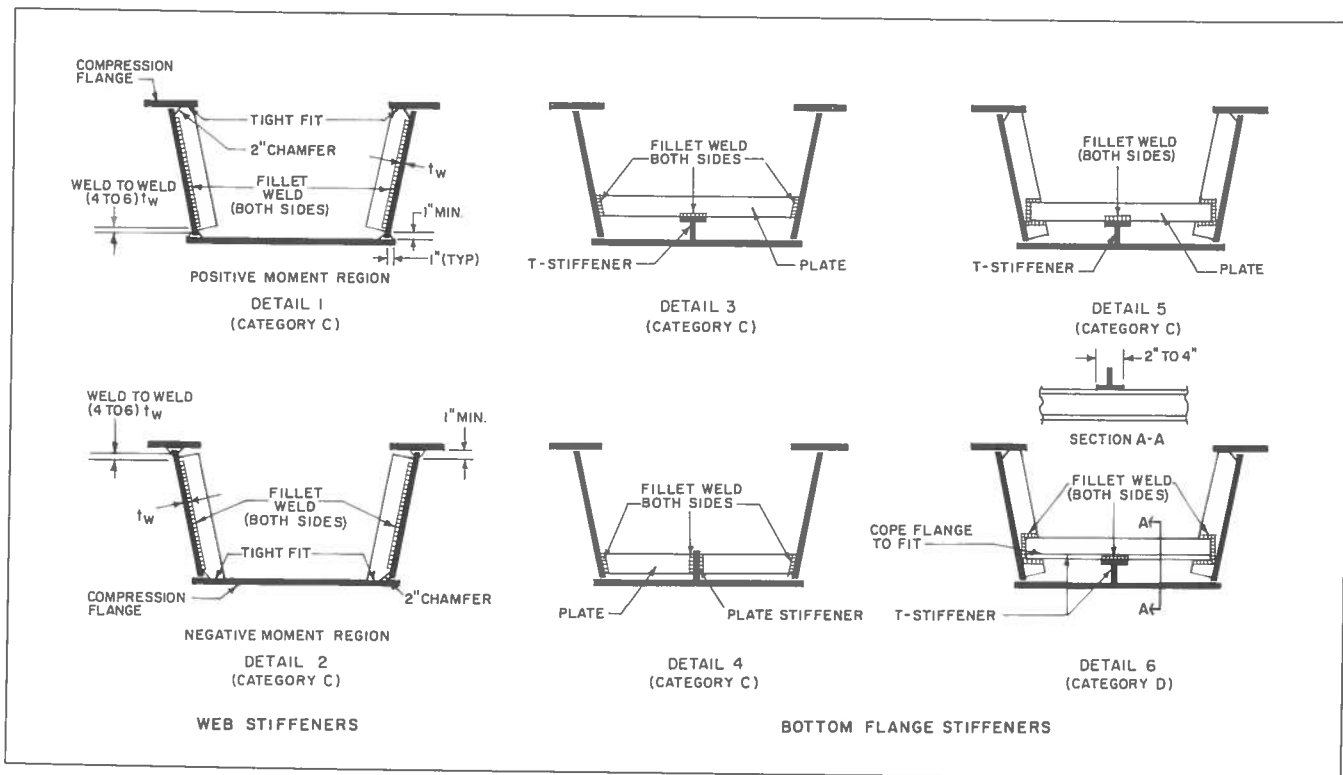


Figure 12. Composite box girder stiffeners.

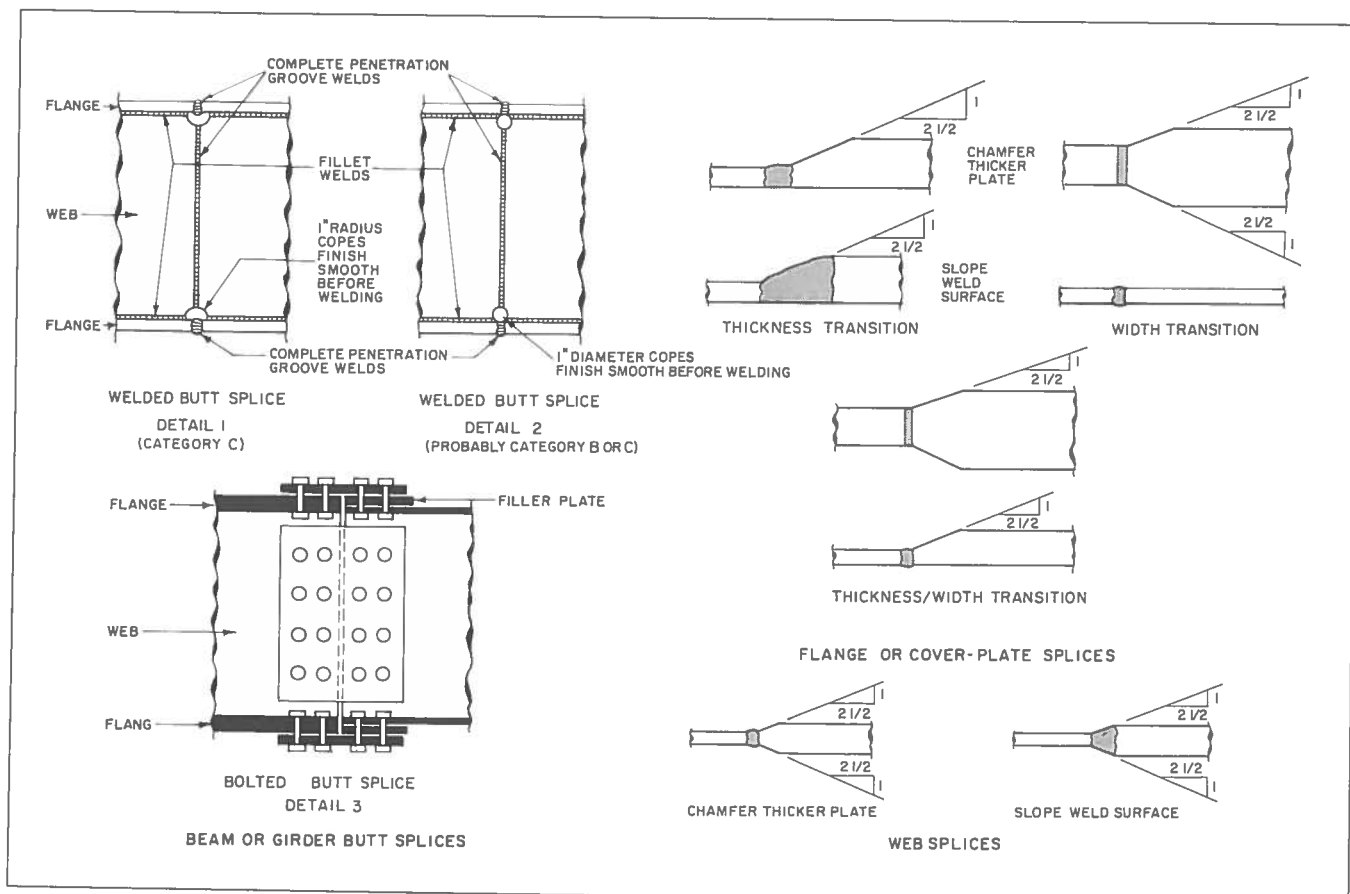


Figure 13. Splices.

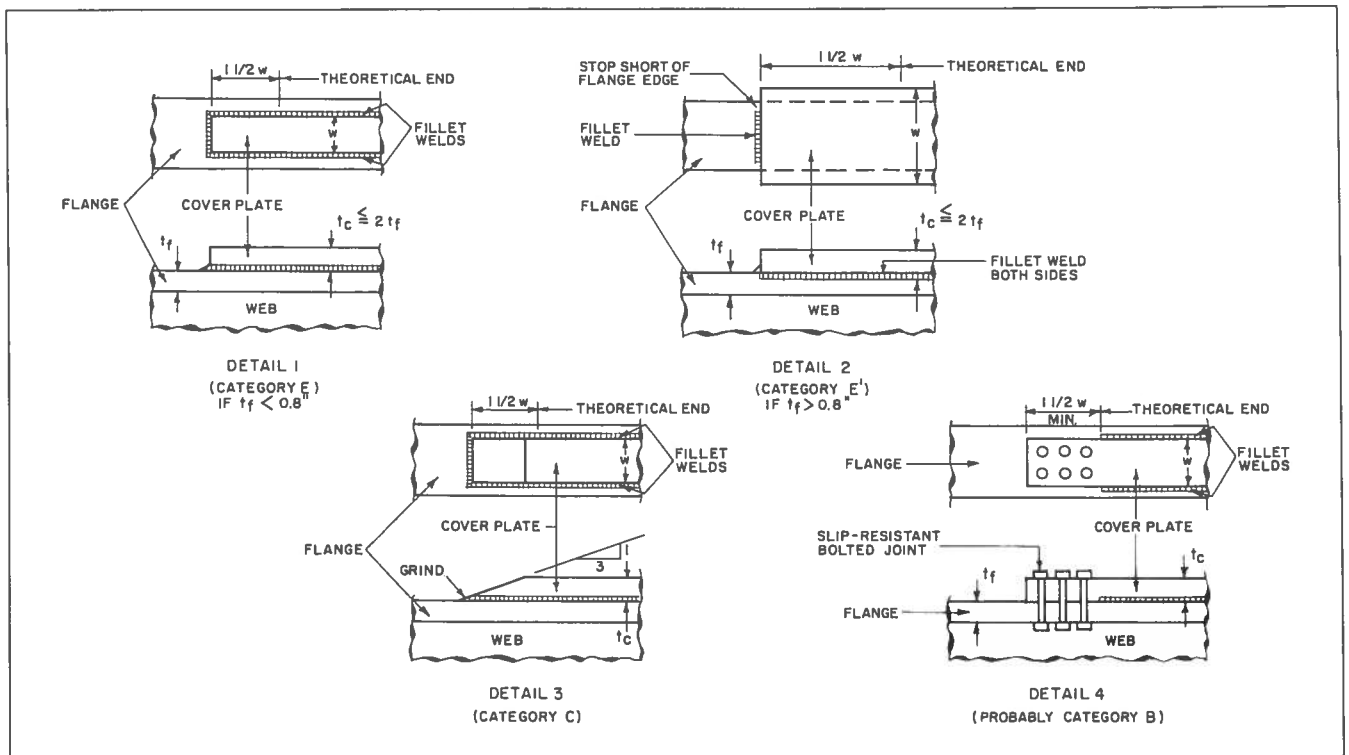


Figure 14. Cover plates.

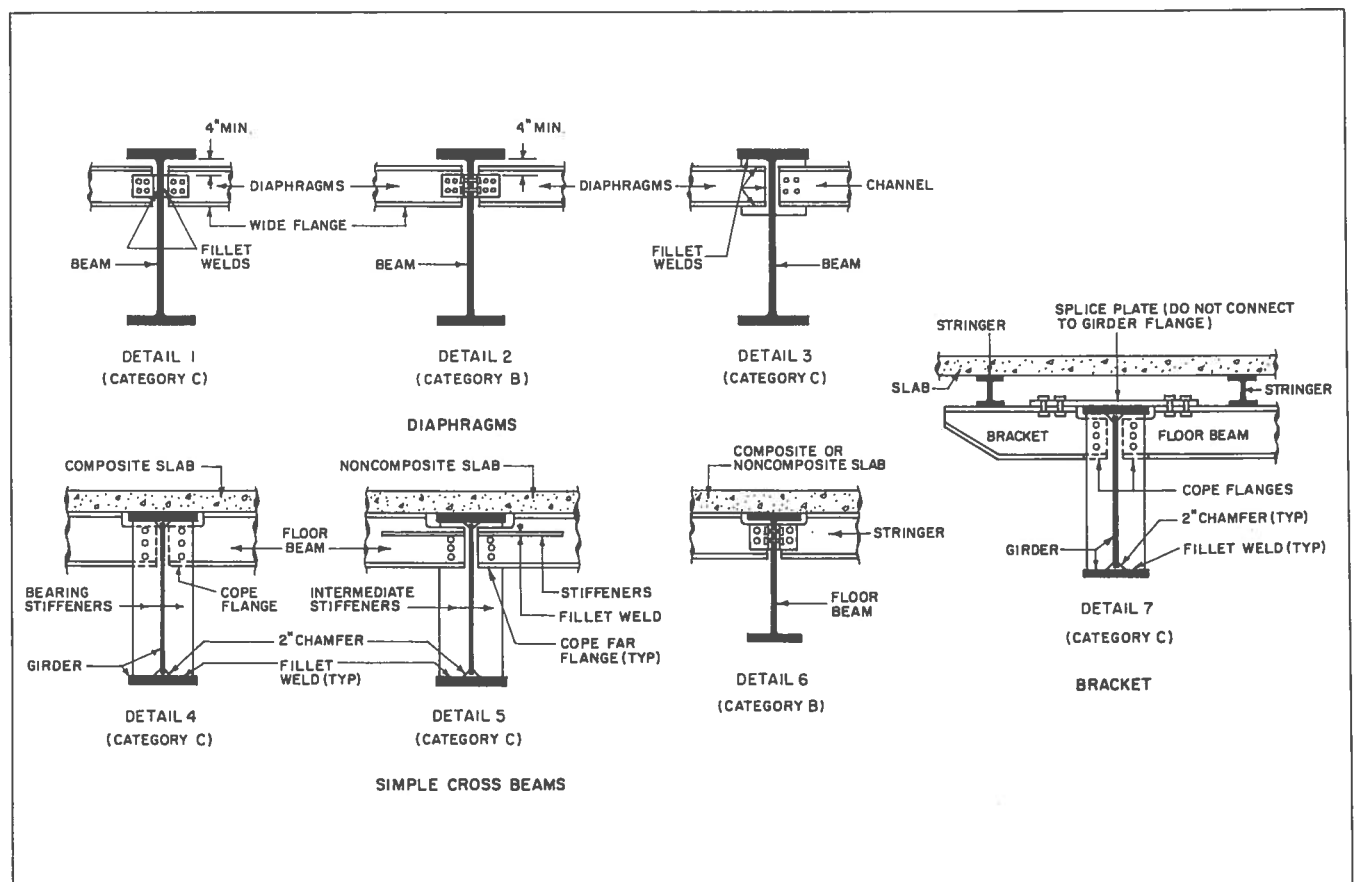


Figure 15. Diaphragms, simple cross beams, and brackets.

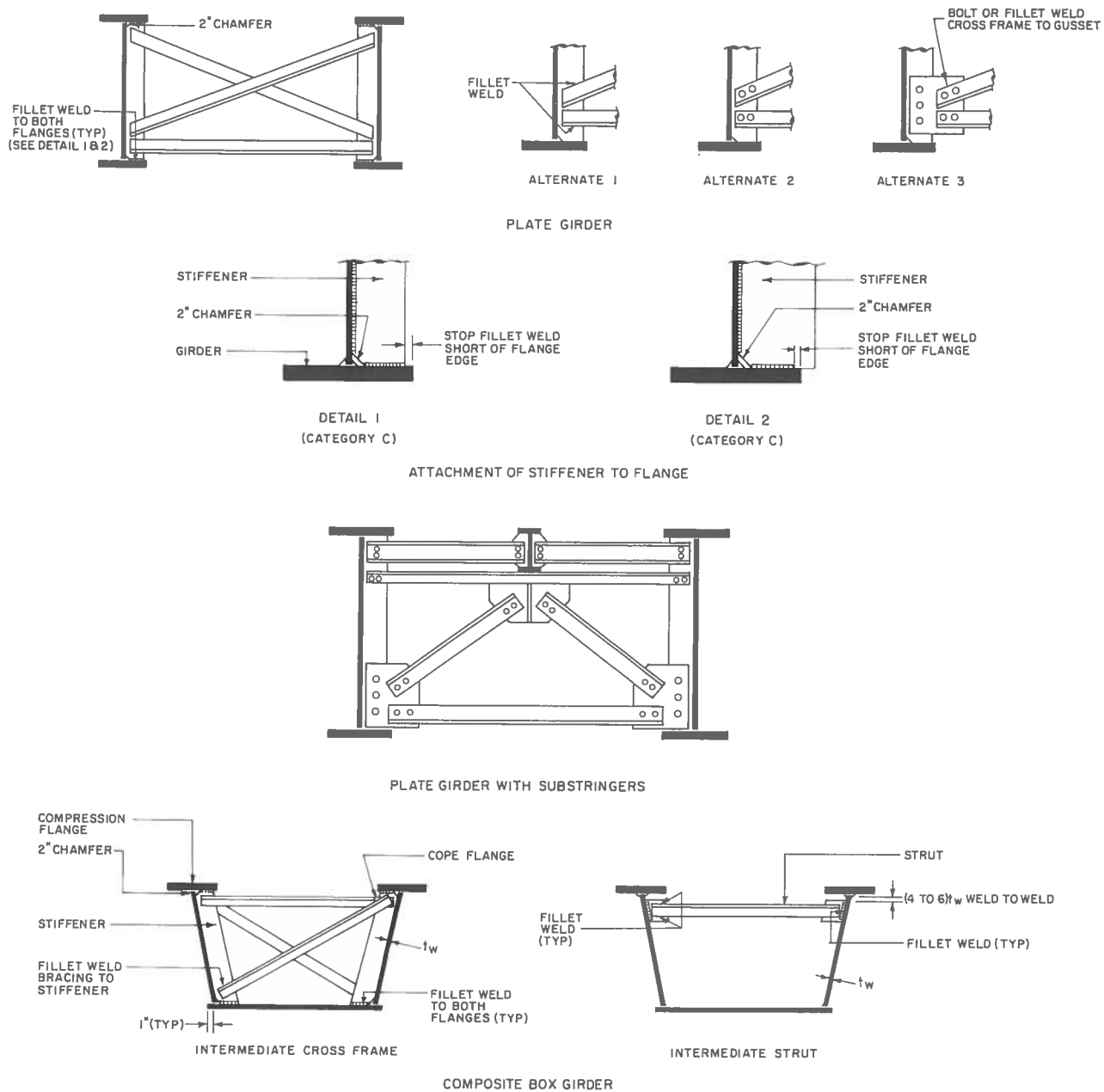
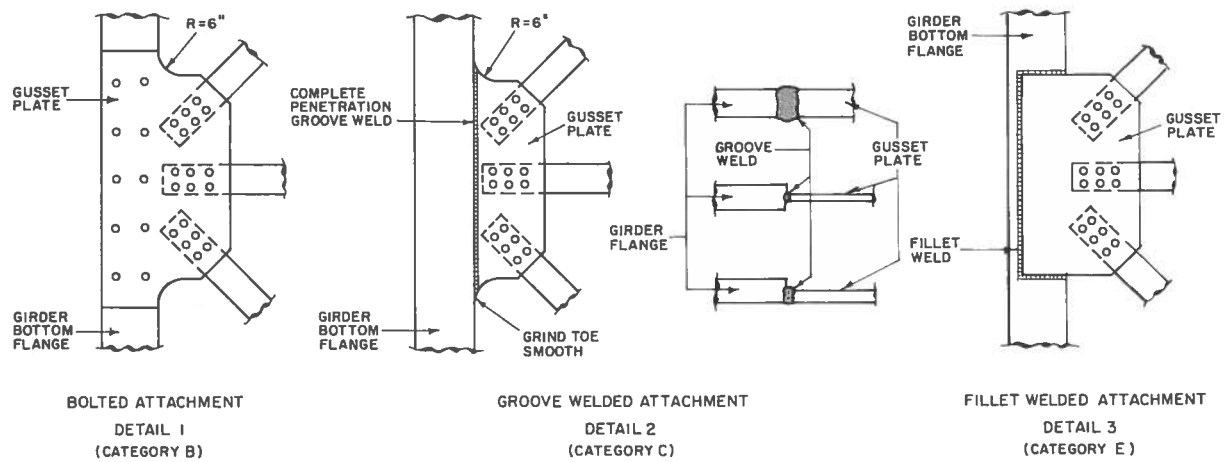
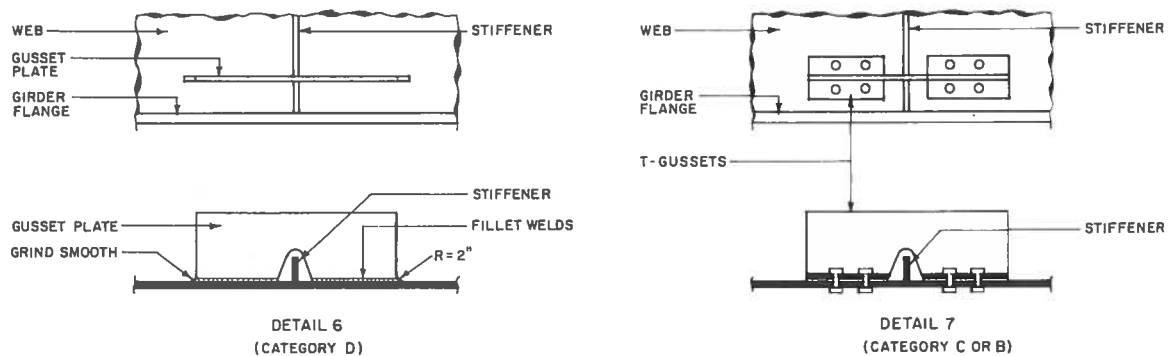
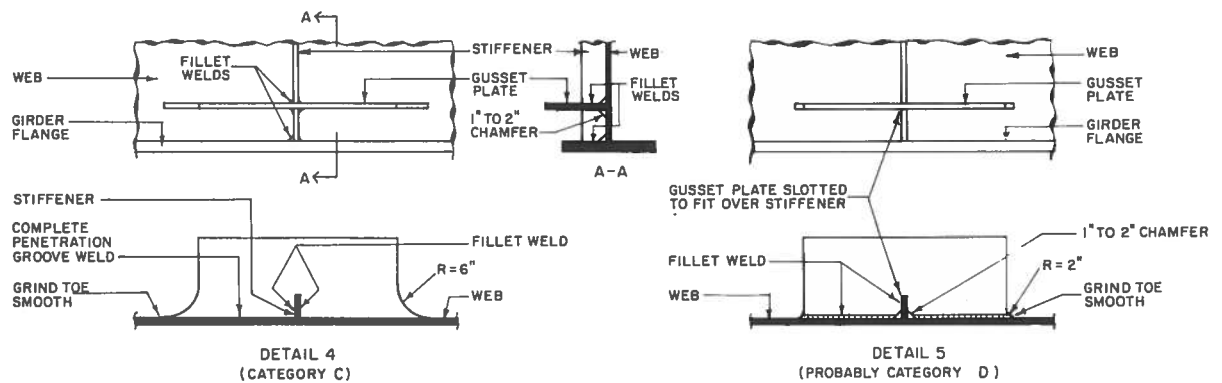


Figure 16. Cross frames.



FLANGE ATTACHMENTS



WEB ATTACHMENTS

Figure 17. Lateral bracing attachments.

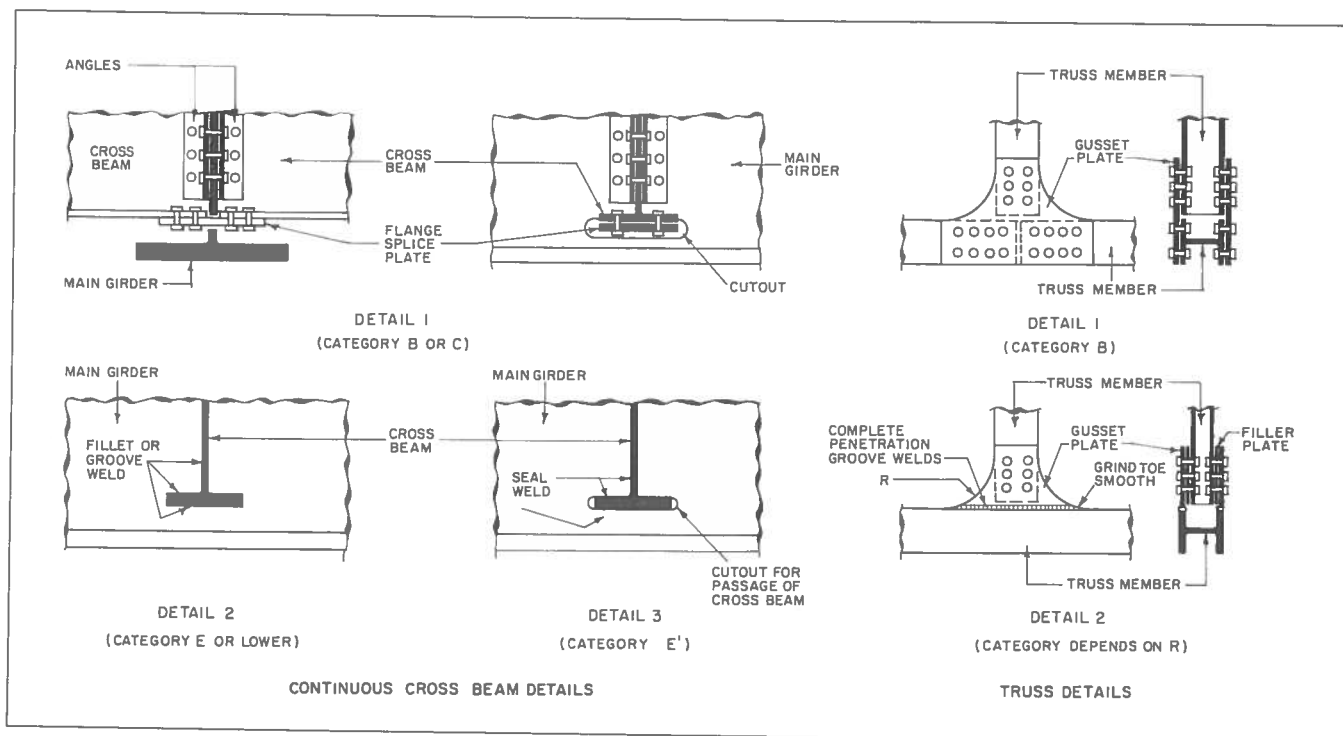


Figure 18. Continuous cross beam and truss details.

9. APPENDIX

(A) CRITERION FOR CRACK ARREST HOLES

Further growth of an existing fatigue crack can be arrested by placing circular holes at both ends if the original crack length is not too large. A criterion for the limiting crack length, below which further cracking will not occur, is derived below from a stress-intensity criterion originally proposed by Barsom^{1,3,71} and modified by Fisher¹⁴ to account for roughness on the surface of a fabricated hole.

Let

- a = one-half the length between outer edges of the two holes
- L = length between outer edges of the two holes
- r = radius of the holes
- S_r = stress range
- S_y = yield stress
- ΔK = stress-intensity range

$$\text{Modified criterion: } \frac{\Delta K}{\sqrt{r}} < 4 \sqrt{S_y}$$

$$\text{Definition of stress intensity: } \Delta K = S_r \sqrt{\pi a}$$

$$S_r \sqrt{\frac{\pi a}{r}} < 4 \sqrt{S_y}$$

$$S_r^2 \frac{\pi a}{r} < 16 S_y$$

$$a_{\text{LIMIT}} = \frac{16 S_y r}{\pi (S_r)^2}$$

Let

$$a = L/2, S_y = 36 \text{ ksi}, r = 0.5 \text{ inch (size of tested holes)}$$

$$L_{\text{LIMIT}} = \frac{16(36)(.5)}{\pi (S_r)^2}$$

$$L_{\text{LIMIT}} = \frac{183.3}{(S_r)^2}$$

$$\text{Round to } L_{\text{LIMIT}} = \frac{200}{(S_r)^2}$$

If this criterion is applied to a single 1-inch-diameter hole, the limiting stress range below which cracking would not occur (fatigue limit) is obtained by setting

$$1 = \frac{200}{(S_r)^2}$$

and solving for

$$S_r = \sqrt{200} = 14.1 \text{ ksi.}$$

This value is slightly below the fatigue limit of 16 ksi for a Category B detail and thus is reasonable for a single hole.