BRIDGE FATIGUE GUIDE

DESIGN AND DETAILS

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

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DESIGN AND DETAILS

by

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PREFACE

It is estimated that there are more than one-half million steel highway and railroad bridges in the United States. With only a few exceptions, these structures have performed satisfactorily in every respect and, in most cases, have carried loads far in excess of those for which they were designed.

During the past decade, a great deal of research has been focused on the effects of the repetitive loadings to which highway and railroad bridges are subjected. This work, as well as lessons learned from the relatively few cases of undesirable performance, have led to a better understanding of bridge fatigue behavior and to substantial changes in fatigue provisions of bridge design specifications.

This booklet has been prepared as a guide to the general problem of bridge fatigue and to assist the designer with the selection and design of bridge details that offer superior fatigue strength. It is a revised and expanded version of the earlier AISC booklet *Guide to the 1974 AASHTO Fatigue Specifications*.

Since 1974, portions of the AASHTO fatigue specifications have been adopted by AREA and AISC. Hence the classification of various details and their permissible stress range for specified load cycles is now identical for all three specifications. As a result, the general application of the AASHTO fatigue provisions to the design examples for highway bridges are equally applicable to other structures. Obviously, the loading conditions and design life criteria will differ, depending on the application.

A method for estimating equivalent design life for use with constant cycle fatigue stresses is described for highway bridges. This permits the potential cumulative damage of random truck traffic to be accounted for in design. A comparable approach for railroad bridges can also be found in the Commentary to the AREA Specification, reproduced in Appendix B.

One of the major fatigue problems that has surfaced in recent years is cracking from secondary and displacement-induced stresses. Discussed briefly in the 1974 Guide, this subject is treated in much greater depth in Chapter 5 of this booklet. The problem has developed because many bridges are essentially linear structures and are designed for in-plane loading and deflection of the main girders and the cross-framing. However, even though interaction between the longitudinal and transverse framing does not alter the in-plane behavior of the framing enough to economically justify a space frame analysis, it is of paramount importance to consider the distortions resulting from such interaction. Generally, the effects of secondary and displacement-induced stresses are seen at connections to main members. The severity is often dependent on geometrical conditions which the designer can control. These are discussed at length and recommendations provided as to how the problem may be minimized or avoided. A general procedure for the design of connections to insure the intended performance is provided at the end of Chapter 5.

The direct applicability of fatigue specifications to the main load carrying members has usually been very apparent to bridge designers. As a result, appropriate details have been provided which satisfy the specification requirements. However, the design of secondary members and connections has not always been as obvious. Often these members interact with the main members and receive more numerous cycles of stress with a higher stress range than assumed. Discussion of this problem is provided and recommendations given on the treatment of such components.

A brief discussion of the background and history of fatigue specifications for highway bridges is provided, as well as a summary of the laboratory studies on fatigue that form the rationale for the stress range concept and lead to the current specification provisions.

Article 1.7.2 of the 1977 AASHTO Specifications, and Art. 1.3.13 of the 1977 AREA Specifications and its Commentary, are reproduced in full in Appendix B. They contain the major changes to Art. 1.7.3 of the 1973 AASHTO Specifications and Art. 1.3.13 of the 1976 AREA Specifications. These changes are summarized at the beginning of Appendix B.

The causes of fatigue problems and a number of examples are examined in detail in this booklet. The recommendations provided throughout are intended to aid in minimizing and avoiding these problems in the future. The user is urged to examine in detail the various sections of this booklet throughout the design of a cyclically loaded structure.

This booklet was sponsored by the American Institute of Steel Construction under the auspices of the AISC Committee on Bridges. The author is indebted to the Committee and its Advisors for their many suggestions and advice. The AISC Committee also provided the design examples contained herein.

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May, 1977 John W. Fisher

CONTENTS

CHAPTER 1

DESIGN DETAILS TO OPTIMIZE FATIGUE STRENGTH

The major factors governing fatigue strength are the applied stress range, the number of cycles, and the type of detail. Structural details behave differently because the stress concentration condition changes. The inherent variability of initial discontinuities is also a major factor.

All welding processes introduce small discontinuities in or near the weldment. Although good welding practice will minimize the number and size of these discontinuities, they cannot be eliminated. The fatigue design rules were developed from research on test specimens that contained normal discontinuities. The usual visual inspection of fillet welds and longitudinal groove welds and the nondestructive inspection of transverse groove welds in tension flanges may detect discontinuities that are adequately accounted for in the design provisions for fatigue. In fact most attempts to remove allowable discontinuities from manufacturing and fabrication that are permitted by ASTM and AWS will result in a condition that is worse than the original condition.

For design there are two options available: (1) the choice of a detail (or the severity of the stress concentration introduced by a detail) and (2) limiting the stress range to acceptable levels.

Details that provide the lowest allowable stress range involve connections that experience fatigue crack growth from weld toes and weld ends where there is a high stress concentration. This is true of both fillet and groove welded details. Details which serve the intended function and provide the highest fatigue strength should be considered.

As a general rule, details which involve failure from internal discontinuities, such as porosity, slag inclusion, cold laps, and other comparable conditions, will have a high allowable stress range. This is primarily due to the fact that there is no geometrical stress concentration at such discontinuities other than the effect of the discontinuity itself.

The AASHTO Specifications, in Table 1.7.2A2 (see Appendix B), describe various situations and categories. Similar provisions are provided by AREA in Table 1.3.13C (see Appendix B). A more detailed evaluation of typical welded bridge details for fatigue loading is given in Ref. 1.

The stress cycles for fatigue design stresses are defined by the bridge location and type of member (see Table 1.7.2B). The maximum stress ranges permitted on the bridge for the various stress cycles are listed in Table 1.7.2A1 of the AASHTO Specifications. Tables 1.3.13A and B provide this information in the AREA Specifications. If well defined traffic conditions are known, these should be used in lieu of the stress cycles in Table 1.7.2B or in Table 1.3.13B to determine a suitable design life and the corresponding allowable stress ranges; the use of an equivalent design life is discussed in Chapter 3.

Thus, the designer can, to a large extent, control the type of detail selected and its location in regions of significant cyclic stress. Every attempt should be made to place Category E details in regions of low cyclic stress, so that the member size need not be increased. For example, coverplated beams can have the cover plate termination extended into regions of low stress range.

A wide class of fillet and groove welded details is covered by Category E.[1] However, alternate details which result in higher allowable stress ranges are available and can be used.

For example, transverse or lateral bracing which frames into a girder, as illustrated in Figs. 1 and 2, results in a Category E detail on the flange surface at the weld end. If the stress range is critical, details such as shown in Figs. 3 and 4 will provide much higher allowable stress ranges. Also, as shown in Fig. 5, the detail could be moved to a location where the stress range is smaller.

In many structures it may be possible to omit the lateral bracing system in the high stress range regions of the span. Lateral bracing is not required in highway spans up to 125 ft long (AASHTO) or in railroad deck spans up to 50 ft (AREA). In longer spans, a continuous lateral system may not be required over the full length of the structure.

For a more complete discussion of lateral connection details, see Chapter 6.

If the attachment were bolted to the flange as in Fig. 3, the allowable stress range is increased to Category B on the net section for a bearing-type connection and the gross section for a friction-type connection. This permits a higher allowable stress range than for the welded attachments of Figs. 1 and 2. The reduction in net area will only slightly reduce this increased fatigue strength.

Still another method of increasing the allowable stress range is to use attachments with a "radiused" transition, as illustrated in Fig. 4. The weld ends must also be ground smooth

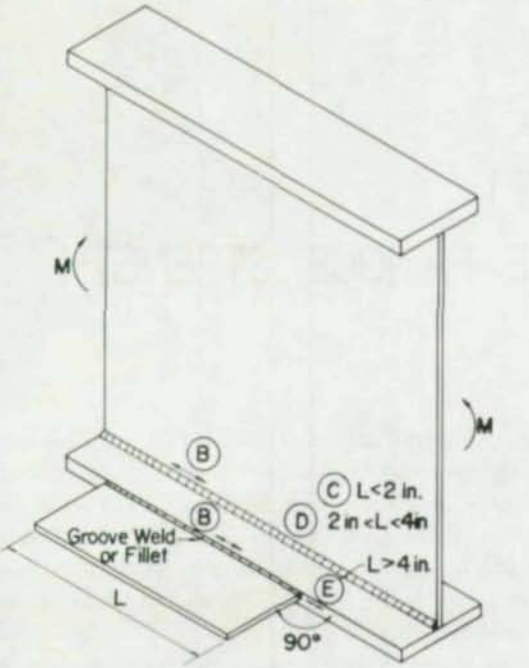


Fig. 1. Bracket attached to flange by longitudinal fillet or groove welds

at the transition radius to accomplish the desired increase. Obviously the machining and grinding required to fabricate such details may be more costly than other methods of satisfying the fatigue provisions. The AASHTO, AREA, and AISC specifications all include radiused transitions in the most recent fatigue specifications.

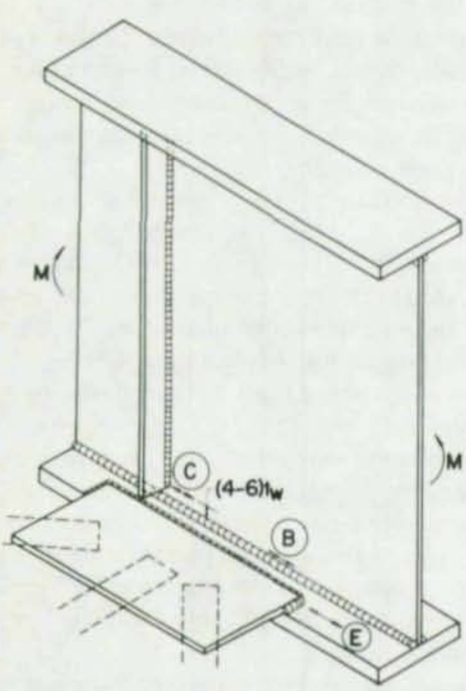


Fig. 2. Bracket attached to flange by fillet welds

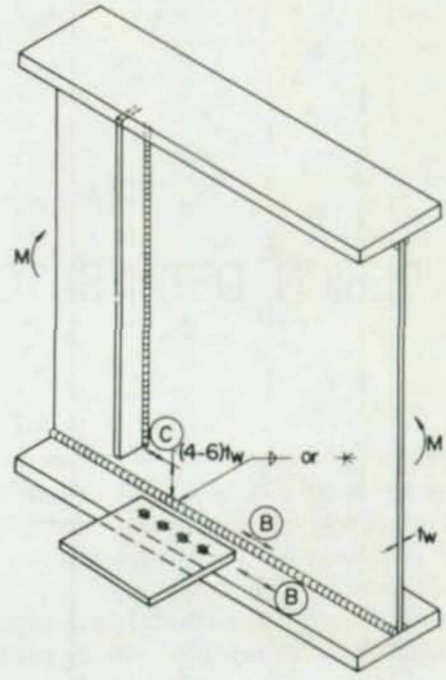
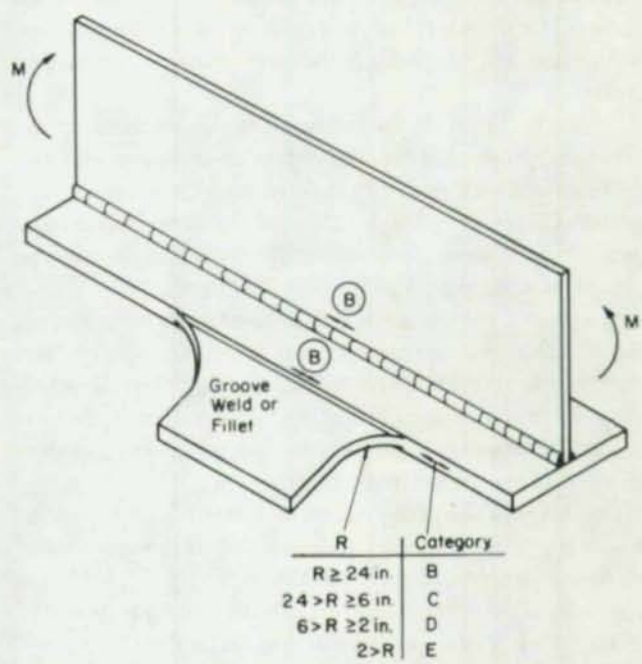


Fig. 3. Bolted flange attachment

If out-of-plane forces are to be resisted, as in curved girder bridges, the transverse stiffeners can be welded to both flanges at cross bracing to assist in resisting these forces, as in Fig. 5. The resulting detail provides a Category C condition at the top of the flange surface which has a stress range from 52% to 100% greater than the flange attachments shown in Figs. 1 and 2, depending on the design life.



Note: The weld end must be ground smooth at the transition radius.

Fig. 4. Radiused transition for welded flange attachments

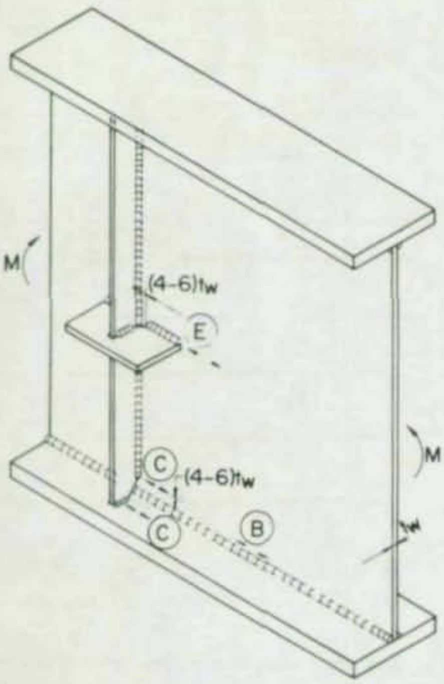


Fig. 5. Bracket attached to beam web in lower stress region

It is important to realize that any detail can be used if it is properly accounted for in the design. The simplest detail consistent with the stress requirements will generally be the most desirable from the standpoint of design, fabrication, and economy.

When transverse and longitudinal stiffeners are used, each provides a weld termination, as is illustrated in Fig. 6. Since

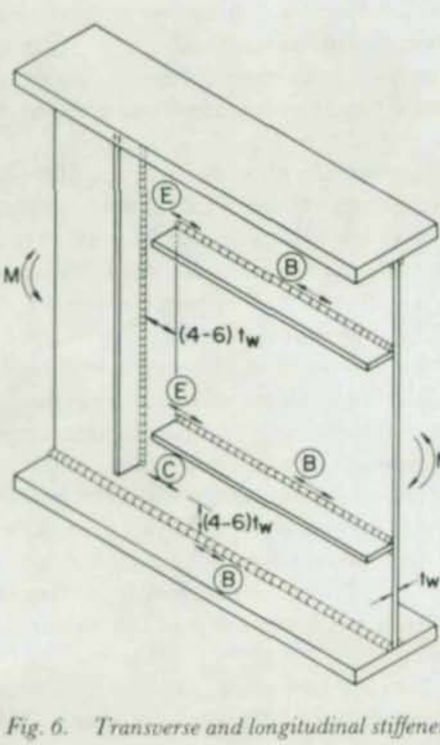


Fig. 6. Transverse and longitudinal stiffeners

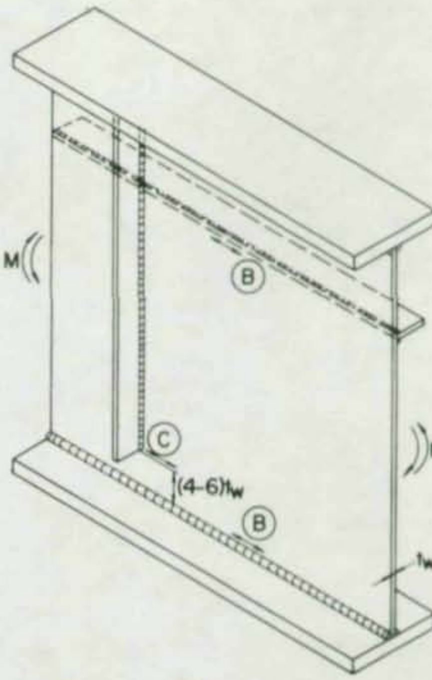


Fig. 7. Transverse and longitudinal stiffeners placed on opposite sides of web

the longitudinal stiffener is a long attachment, the end of the stiffener is governed by the Category E design condition. At other points along the stiffener, Category B is applicable. The transverse stiffener does not provide as severe a condition, because it is much shorter in the direction of applied stress. If both types of stiffeners are needed in an area of stress reversal, the most desirable condition can be achieved by placing the longitudinal stiffener on one side of the web and the transverse stiffener on the other, as in Fig. 7, so that the longitudinal welds can be continuous and the longitudinal stiffener can either be terminated in a region of low stress range or compressive stress, or incorporate a radiused transition at its end.

Fillet welds for transverse stiffeners should be terminated short of the web-to-flange welds by a distance of at least four and up to six times the web thickness, as illustrated in Figs. 2, 3, 5, 6, and 7, and should not be returned around the ends of the stiffener. Failure to terminate stiffener welds a suitable distance above the web-flange connection can result in adverse behavior, due to restraint stresses introduced by weld shrinkage and possible cyclic stresses due to transverse movements during shipping or handling. This is discussed in greater detail later, in the section dealing with secondary stresses (Chapter 5).

Transverse groove welds in regions of cyclic tension or stress reversal are examined by nondestructive inspection, as specified on plans or job specifications, to insure that excessive internal discontinuities are not present. Improvements in fatigue strength can be achieved by removal of the weld reinforcement and by appropriate transitions between plates of different thickness or width, as illustrated in Figs. 8 and 9.

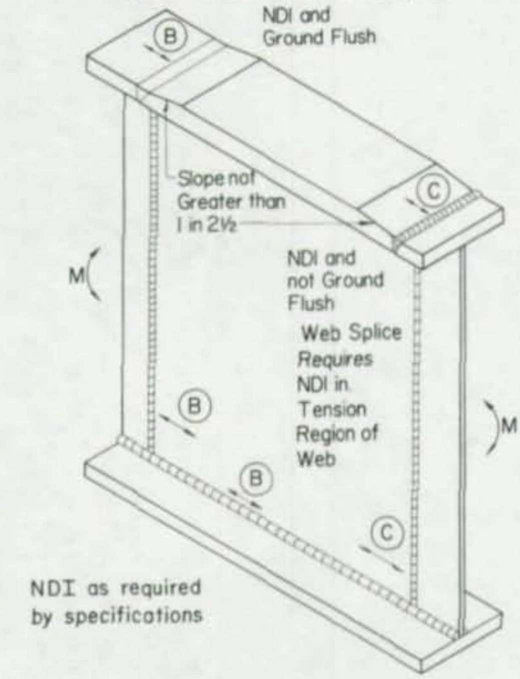


Fig. 8. Transverse groove welds in web and flange

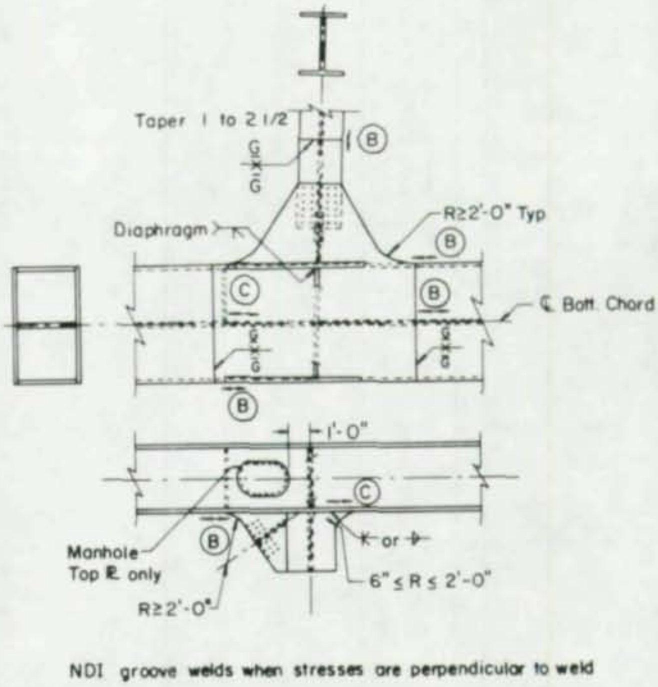


Fig. 9. Gusset connection showing a variety of structural details with a wide range of fatigue strengths

Removal of the weld reinforcement at the groove welded details shown in Figs. 8 and 9 improves the fatigue design condition from Category C to Category B.[15,16]

United States practice does not provide for adjustments in fatigue strength for various sizes of internal discontinuities. Studies in England by Harrison and others[16,18] have shown that fatigue strength can be adjusted to reflect larger internal discontinuities. Currently used weld quality control required by AWS and AASHTO insures that transverse groove welds will achieve the Category C or B design condition. With the reinforcement removed, Category B applies. When the weld reinforcement is left in place, Category C is applicable.

Longitudinal welds that are parallel to the applied stresses, such as the web-to-flange welds in Figs. 1 through 7, have a high fatigue strength. Both longitudinal fillet and groove welds are Category B details under such circumstances. In both cases the fatigue strength is based on expected internal discontinuities in the web-to-flange connection that are perpendicular to the applied stresses. Examples of discontinuities in these welds may be in the form of porosity, cold laps, slag inclusions, or other conditions. In such welded connections, discontinuities that are parallel to the stress field have no influence on the members' performance. This includes lack of penetration discontinuities in both fillet and partial penetration longitudinal welds, slag inclusions, and other comparable discontinuities. The fatigue strength is governed by discontinuities that are perpendicular to the applied stresses, not by discontinuities that are parallel. Hence, the inspection criteria used for fillet welds is equally applicable to longitudinal groove welds. Generally, this includes a visual inspection, with some magnetic particle examination to determine whether or not there are cracks in the welds. Ultrasonic and radiographic inspections are not necessary for longitudinal welds. Large internal discontinuities that are perpendicular to the applied stresses will not be present, as would be possible with transverse groove welds. In longitudinal groove welds, the maximum discontinuity perpendicular to the applied stress cannot exceed the weld size. In transverse groove welds, lack of penetration, slag inclusions, or other types of discontinuities may result in discontinuities several times larger than the weld cross section, which is why nondestructive inspection is necessary.

Coverplated beams, such as shown in Fig. 10, result in low allowable stress range (Category E) at the cover plate termination. Category B is applicable away from the cover plate end if it is attached by continuous welds. About the same fatigue strength is provided with or without transverse end welds.[2] The end of the longitudinal weld attaching the cover plate to the flange and the toe of the transverse end weld provide comparable conditions. Geometrical changes in the cover plate end have little influence on the fatigue strength. For example, tapering the cover plate width, providing a radius at its end, or other variations as illustrated in Fig. 11 all provide a Category E detail.[3,4] These geometrical variations do not significantly alter the stress concentration at the weld end that is transverse to the applied stresses. Simply altering the shape of the cover plate end does not change this condition by a significant amount.

Backing bars are frequently used when fabricating box girders with single-bevel full penetration groove welds for the

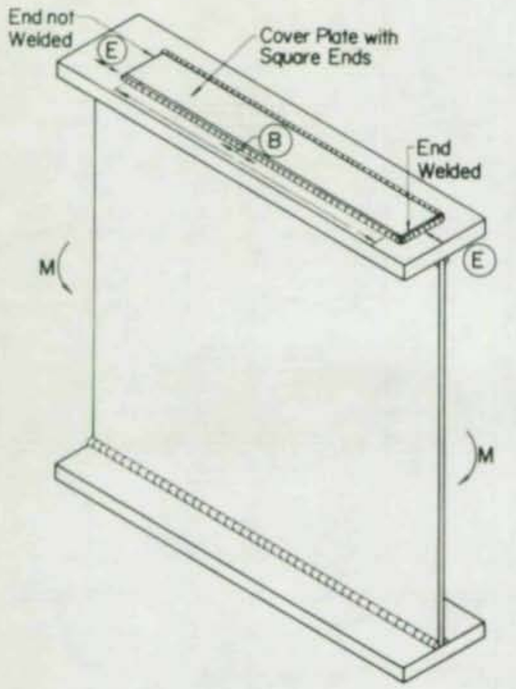


Fig. 10. Coverplated beam showing region of critical fatigue stress

web-to-flange connection. Care should be exercised with the use of such bars. If intermittent fillet welds are used to connect the backing bar to the web and flange plates, they may provide a Category E connection for the tension flange (see Fig. 12a). This conservative treatment of intermittent tack welds reflects the lack of test data on this type of connection. Further research is being planned on this detail.

If backing bars are needed, it is preferable for the connecting welds to be continuous on the tension flange, as illustrated in Fig. 12a, so that Category B is applicable. Alternately, intermittent welds could be used and then removed after completing the joint. Intermittent welds can be used in regions subjected to compression without any adverse effect on the member design. Discontinuous backing bars should not be used.

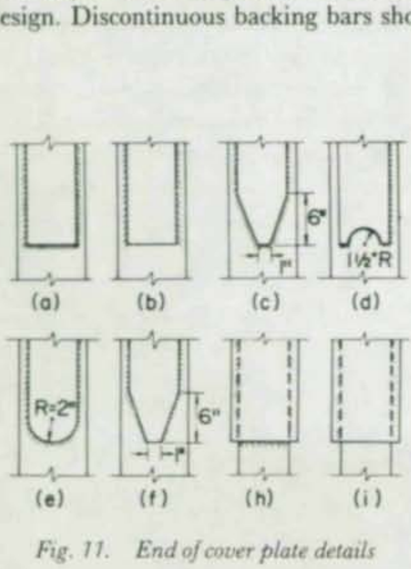


Fig. 11. End of cover plate details

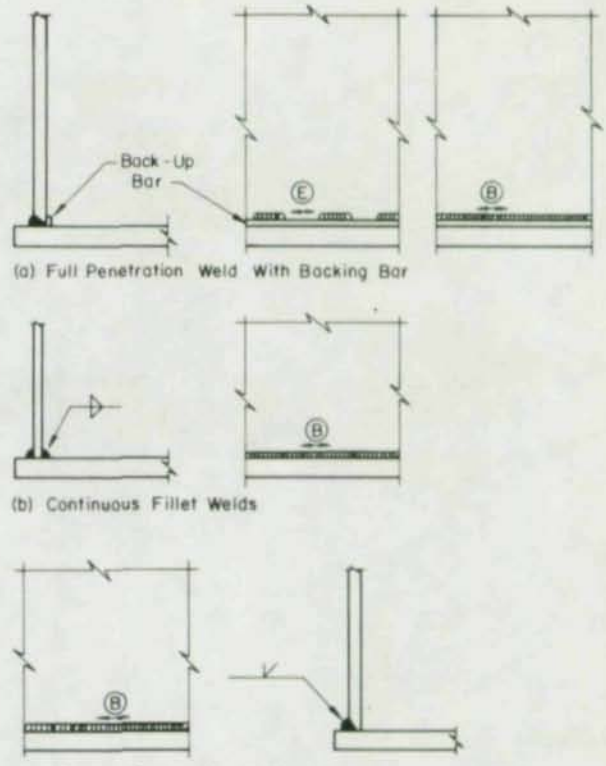


Fig. 12. Cover welds for steel boxes

The problems associated with backing bars can be avoided in some instances, when girders have sufficiently thick webs, by using either a partial penetration groove weld or two fillet welds for the web-to-flange connection (see Figs. 12b and 12c). Both of these joints provide a Category B connection. Fillet welds may not be practicable for many boxes, because access may not permit the placement of some of the inside fillet welds. A single partial penetration groove weld can be provided without requiring a backing bar (see AWS Articles 2.5 and 9.12). It also permits easy access to the box joints, since the welds can be made from outside the boxes. Both partial penetration groove weld and twin fillet weld connections provide a Category B connection, which is the same fatigue strength detail as the full penetration groove weld.

Floor beam or cross-girder to longitudinal girder connections and stringer to full-length cross-girder connections are a category of joints that can provide wide variation in fatigue strength. Floor beams must either pass through the longitudinal girders or be attached to them. Typical examples of floor beam-to-girder connections are shown in Figs. 13 and 14. Among the problems of concern are the continuity of the floor beam flanges, the attachment of the floor beam flanges to the girder flange, termination of the web-to-flange welds, and the attachment or passage of the floor beam compression flange through the girder web.

If the floor beam tension flange is passed over the girder, as illustrated in Fig. 13, wide variation in fatigue strength can result, depending upon how the floor beam web-to-flange

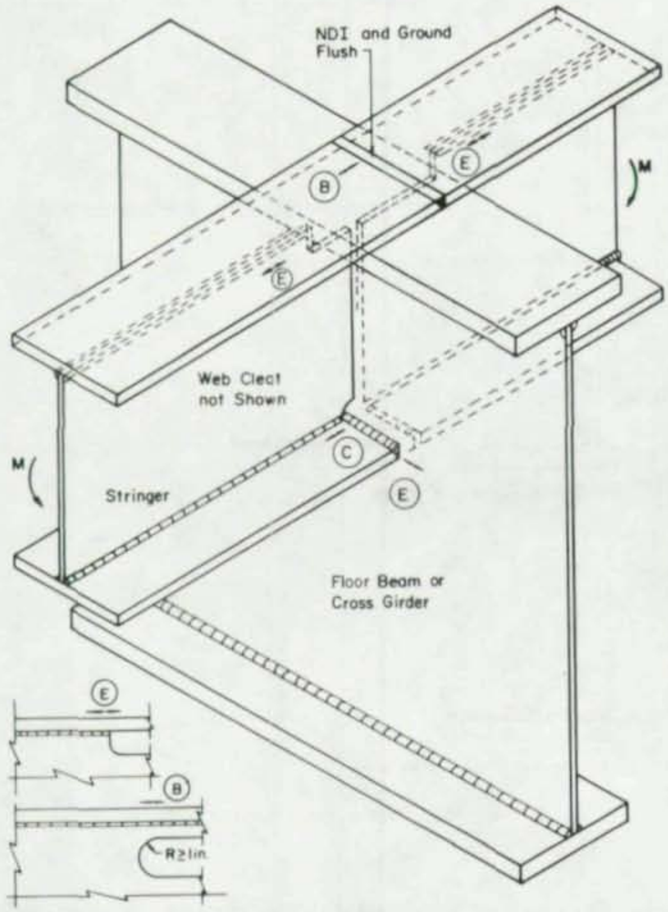


Fig. 13. Floor beam-to-girder connection with continuous floor beam flange

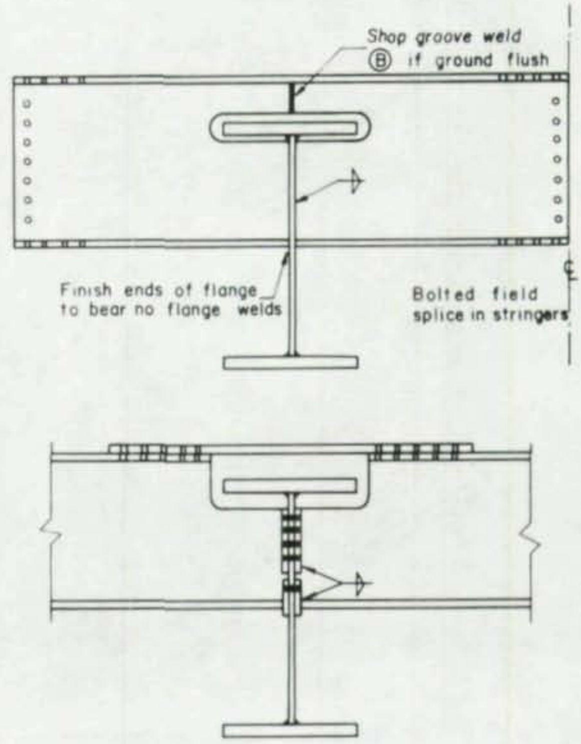


Fig. 14. Stringer-to-floor beam connections with high allowable fatigue strength

connection is treated. The groove weld in the floor beam flange is either Category B or C, depending on whether or not the reinforcement is removed or left intact. A more critical design condition is the termination of the web-to-flange connection. If these welds terminate, a Category E design condition results, as shown in Fig. 13. Such a large reduction in fatigue strength can be avoided by providing continuity in the web-to-flange weld, as shown in the insert. A smooth radiused transition in the floor beam web can be provided at the cutout to accommodate the girder flange.

If the floor beam compression flange is welded to the girder web as shown in Fig. 13, a Category E detail results. This will penalize the fatigue strength if a high tensile or reversal stress range occurs in the girder web over a large number of cycles. A detail with a higher allowable stress range will result if the floor beam flange is passed through a cutout similar to that provided in the floor beam web.

A higher allowable stress range can be achieved in the longitudinal main member if the floor beams are bolted. This may be a more desirable design condition if relatively high stress ranges are present. Figure 14 shows two details that provide Category B design conditions at the stringer-floor beam intersections. In both cases a Category C design condition would result in the floor beam from the web fillet welds or welded shear plate.

If the cyclic stresses in the girder web are not critical and the detail shown in Fig. 13 is used, care should be exercised in the development of the flange-to-web connection. If large floor beam flanges are groove welded to opposite sides of the web plates, shrinkage stresses will be introduced into the web plate which may result in restraint or lamellar tears in the girder web.

CHAPTER 2
DESIGN EXAMPLES

Two design examples are summarized to demonstrate the application of the AASHTO fatigue provisions. One is a simple structure designed by working stress design for stress cycle Case II; the second is a continuous structure proportioned by load factor design for stress cycle Case I.

DESIGN EXAMPLE 1

Design Information:

1. 90-ft simple span; working stress design
2. Composite construction; rolled beam and cover plate
3. 8-in. reinforced concrete slab; $\frac{1}{2}$-in. assumed integral wearing surface not considered for composite properties
4. HS20 loading
5. Dead Load 1: $D_1 = 0.85$ kips/lin. ft (weight of concrete slab) plus estimated weight of steel section
6. Dead Load 2: $D_2 = 0.40$ kips/lin. ft (static loads applied after concrete is cured, carried by composite section)
7. Girder spacing: 8 ft-0 in.
8. Wheel load distribution factor: $S/5.5$
9. Fatigue Case II, AASHTO Article 1.7.2
10. Steel: $F_y = 50$ ksi
11. Concrete: $f'_c = 3{,}000$ psi; $n = 10$

Moment Diagrams (Truck Loading):

See Fig. 15.

Select a Trial Section:

Try W36×170 with cover plate 10 in. x $1\frac{5}{8}$ in.:

Stress checks at point of maximum moment:

Load	Midspan Applicable Moment (kip-ft)	f'_c Concrete, Top (ksi)	f_t Steel, Top (ksi)	f_t Steel, Bottom (ksi)
D_1	1002	N.A.	18.3	12.2
D_2 $(k = 3)$	405	0.142	2.9	4.0
$L + I$ $(k = 1)$	1201	0.626	3.5	10.6
Total	2608	0.768 $(<0.4f'_c = 1.2$ o.k.)	24.7 $(<0.55F_y =$	26.7 27 o.k.)

$\therefore$ W36×170 with cover plate 10 in. x $1\frac{5}{8}$ in. **o.k.**

Determine Cover Plate Length (Non-cyclic Analysis):

Determine theoretical cutoff point of cover plate by finding point at which loads can be carried by concrete and steel section alone.

Stress in bottom fiber of tension flange of rolled beam will probably be the controlling design criterion at the cover plate termination. Therefore, check bottom flange stresses at 10th points:

10th Point	Stresses in Bottom Flange of Steel Beam (ksi) D_1	D_2	$L + I$	Total
5 (Midspan)	20.7	6.7	18.0	45.3
4	19.9	6.4	17.5	43.8
3	17.4	5.6	15.5	38.6
2	13.2	4.3	12.1	29.6
1	7.5	2.4	6.9	16.8
0 (Support)	0	0	0	0

Theoretical cover plate cutoff point (where stress in tension flange equals 27 ksi) is at 16 ft-2 in. from support, based upon straight line interpolation between bottom flange stresses at 1st and 2nd 10th points.

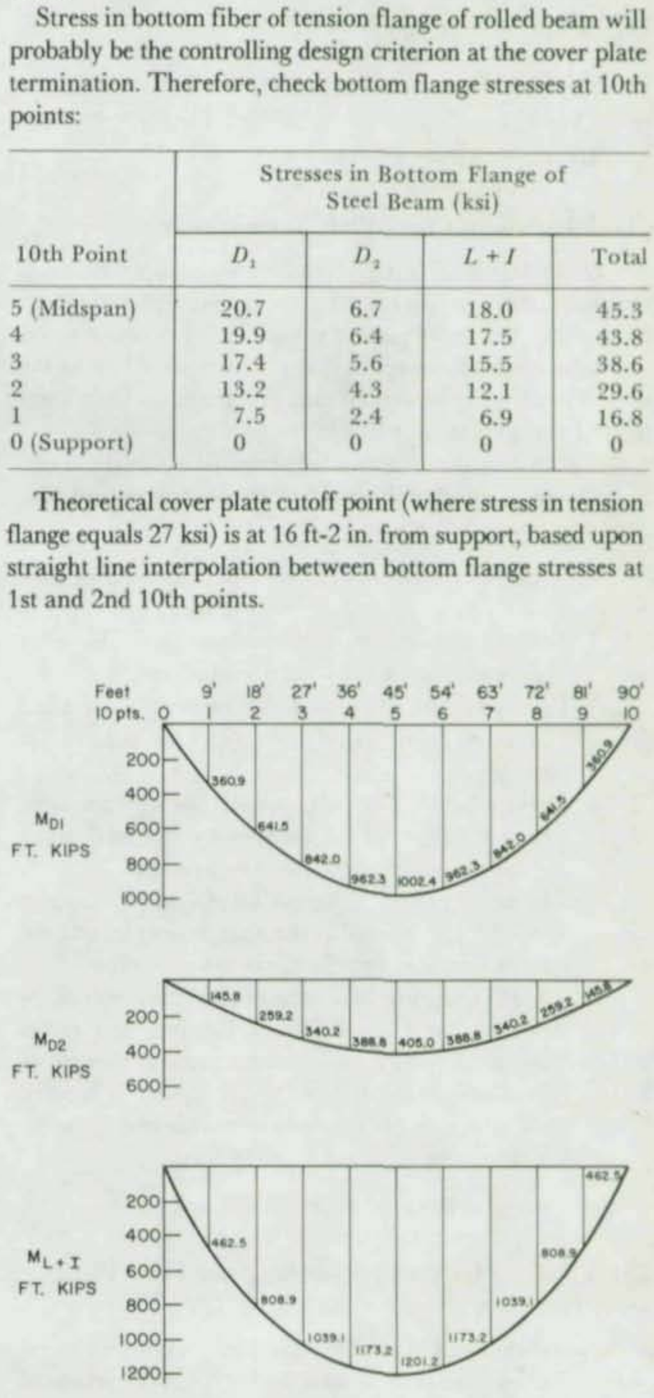


Fig. 15. Moment diagrams (truck loading)

Top steel stress at 16 ft-2 in. from support = 16.0 ksi.

Top concrete stress at 16 ft-2 in. from support = 0.6 ksi.

∴ Bottom steel is controlling stress.

Terminal distance equals 2 times the cover plate width for plates not welded across the ends:

$2 \times 10 = 20$ in. = 1 ft-8 in.

Therefore, cover plate terminates at:

16 ft-2 in. − 1 ft-8 in. = 14 ft-6 in. from each support

Total length of plate equals 61 ft.

AASHTO Article 1.7.12 specifies minimum cover plate length equals $2D + 3$, where D equals depth of beam (ft):

$2D + 3 = 2(3) + 3 = 9 \text{ ft} < 61 \text{ ft}$ **o.k.**

Check Fatigue at Cover Plate Termination:

AASHTO Article 1.7.2 requires a fatigue check of base metal in tension flange (rolled beam) at the ends of a partial length cover plate. The cover plate is square ended, without welds across the ends. However, fatigue calculations would be the same if welds were across the ends or if the cover plate were tapered at the ends.

Calculation of tensile stress in bottom flange of the rolled beam at termination of the cover plate, due to live load plus impact:

1. AASHTO Appendix A indicates truck loading controls.
2. From non-cyclic analysis, the location of the cover plate termination is 14.5 ft from support.
3. From straight line interpolation between 1st and 2nd 10th point moments, $M_{L+I} = 672.1$ kip-ft at 14.5 ft from support.
4. Stress at bottom fiber of bottom flange resulting from M_{L+I} is $f_{L+I} = 10$ ksi, from non-cyclic analysis of the composite section without cover plate.
5. The stress range (f_{sr}) equals live load plus impact stress (f_{L+I}). Note that the span is simple, and the bottom flange is, therefore, always in tension.
6. AASHTO Article 1.7.2 indicates that for a redundant structure, for Case II Truck Loading and Stress Category E (which includes base metal at the ends of partial length cover plates having square or tapered ends, with or without welds across the ends), the allowable range of stress $F_{sr} = 12.5$ ksi.

$$f_{sr} = 10 \text{ ksi} < F_{sr} = 12.5 \text{ ksi } \textbf{o.k.}$$

Check Fatigue for Cover-Plate-to-Beam Fillet Weld Away from End:

If the condition is satisfied at midspan (maximum stress range), the condition will be satisfied for the entire length of the cover plate.

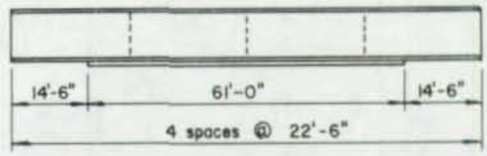


Fig. 16. Geometry of beam and location of diaphragms

Live load plus impact stress at the cover plate bottom fiber:

$$f_{L+I} = f_{sr} = 10.6 \text{ ksi}$$

(Note this is a slight simplification; a more elaborate analysis could be used to compute the actual stress range at the fillet weld location. However, if the condition is satisfied at the bottom fiber, the condition will be satisfied at the location of the fillet weld.)

AASHTO Article 1.7.2 indicates that for a redundant structure, for *Case II Truck Loading and Stress Category B* (which includes continuous fillet welds parallel to the direction of applied stress), the allowable range of stress $F_{sr} = 27.5$ ksi.

$$f_{sr} = 10.6 \text{ ksi} < F_{sr} = 27.5 \text{ ksi } \textbf{o.k.}$$

Check Fatigue at Diaphragm Details:

Per AASHTO Article 1.7.17, diaphragms are to be located at the ends of the span and at intervals not to exceed 25 ft. A uniform spacing is chosen at 22 ft-6 in. (See Fig. 16.)

Assume stiffener plates are used on both sides of the web for attaching channel (18 in. typical) diaphragms, tight fit. (See Fig. 17.)

Fatigue check of bottom flange at the toe of the transverse stiffener:

1. Live load plus impact stress at the top of the tension flange at midspan = 9.4 ksi, from straight line slope calculations based upon the previously calculated midspan live load plus impact stress values (see table of stress checks at point of maximum moment). Therefore, f_{sr} equals $f_{L+I} = 9.4$ ksi.

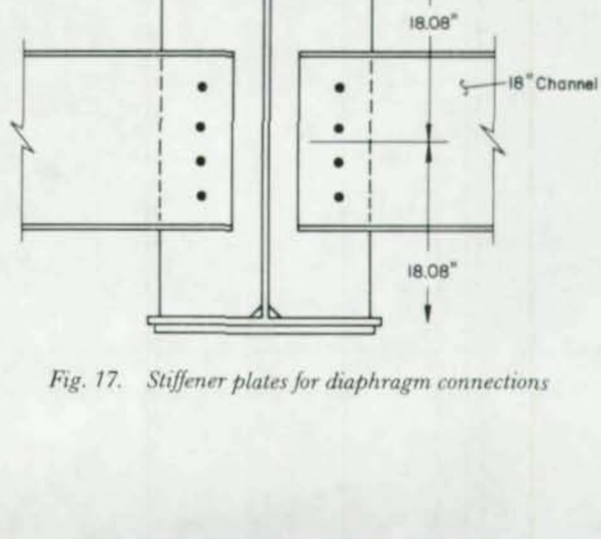


Fig. 17. Stiffener plates for diaphragm connections

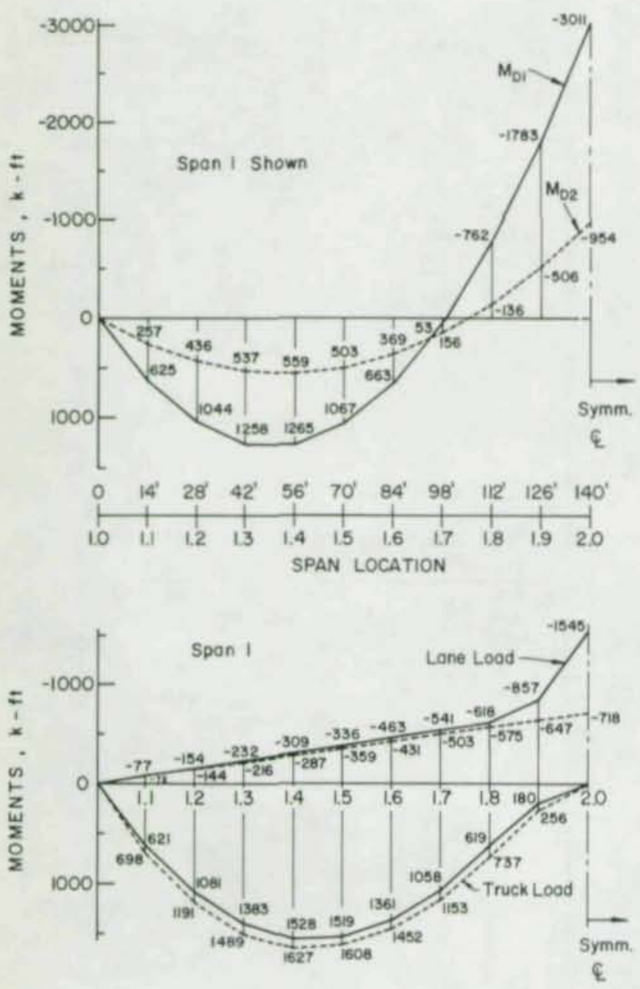


Fig. 18. Moment diagrams (S/5.5)

2. AASHTO Article 1.7.2 indicates that for a redundant structure, for Case II Truck Loading (500,000 cycles) and for Category C (which includes flexural stress at the toe of transverse stiffener welds on girder webs or flanges), the allowable range of stress F_{sr} = 19.0 ksi (Table 1.7.2B).

$f_{sr} = 9.4$ ksi $< F_{sr} = 19.0$ ksi **o.k.**

Notes: 1. The top flange is always in compression; therefore, a fatigue check of the top flange at the shear connectors is not necessary.

2. The span is less than 125 ft-0 in.; therefore, no lateral bracing is required.

DESIGN EXAMPLE 2

Design Information:

1. Two span continuous 140-ft—140-ft; load factor design—redundant construction
2. Unshored composite construction (positive moment region only); welded plate girder
3. 8-in. reinforced concrete slab; ½-in. assumed integral wearing surface not considered for composite properties
4. HS20 loading
5. Dead Load 1: D_1 = 0.85 kips/lin. ft (weight of concrete slab) plus estimated weight of steel section
6. Dead Load 2: D_2 = 0.40 kips/lin. ft (static loads applied after concrete has cured, carried by composite section)
7. Girder spacing: 8 ft-0 in.
8. Wheel load distribution factors:
   $S/5.5$ (non-cyclical and 2,000,000 or less cyclical loads)
   $S/7.0$ (over 2,000,000 cyclical loads)
9. Fatigue Case I, AASHTO Article 1.7.2
10. Steel: ASTM A588, F_y = 50 ksi
11. Concrete: f'_c = 3,000 psi; n = 10
12. Bolted field splice at 1.71 point of Span 1 and 2.29 point of Span 2.

Design for Non-cyclical Loading:

The dead load and live load moments are as shown in Fig. 18. The live load moments are based on a wheel load distribution factor of $S/5.5$ for the initial non-cyclical design.

Figure 19 shows the load factor girder design which satisfies the moment and shear criteria for dead and non-cyclical live loading. The remainder of this design example will involve analysis of the girder and redesign where necessary to insure compliance with all AASHTO fatigue specifications.

Design for Cyclical Loading:

This design example is for a heavily traveled arterial and is to be designed for Case I, Article 1.7.2, where the design life number of cycles is 2,000,000 for truck loading, or 500,000 cycles of stress for lane loading, considering multiple lanes loaded. Members must also be investigated for $S/7$ for one traffic lane loaded. Longitudinal members should be checked for and satisfy the three criteria. The values of allowable stress range for redundant structures should be used in this case, as a continuous multiple girder bridge is considered redundant.

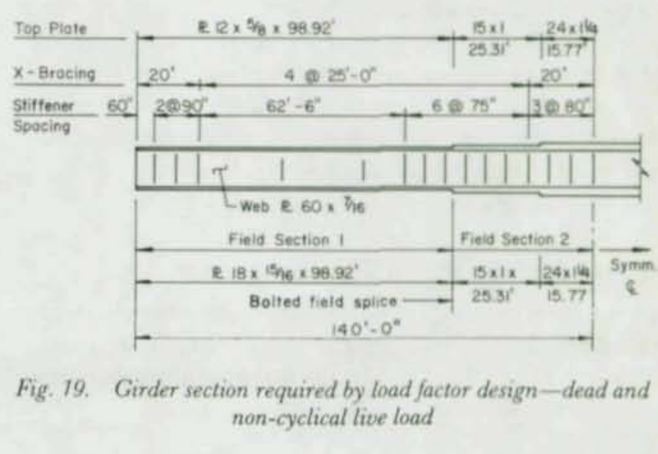


Fig. 19. Girder section required by load factor design—dead and non-cyclical live load

Field testing has indicated that a wheel load distribution factor of $S/7.0$, rather than $S/5.5$, more accurately reflects measured stress values. In some instances, this may still be conservative. This also recognizes the fact that for fatigue the vast majority of loading cycles are caused by a heavy truck in one lane only, rather than from heavy trucks in adjacent lanes. The probability of having two lanes loaded simultaneously with heavy trucks, each producing a maximum live load moment coincidentally, is extremely small. Stress history measurements have included stress cycles under a variety of load conditions and have indicated that $S/7$ was acceptable even for infrequent simultaneous loading because of impact, variation in gross vehicle weight, and other related factors.[26]

Therefore, for fatigue analysis for Case I, *a wheel load* distribution factor of $S/5.5$ (multiple lanes loaded) is used for live loading where the design life number of cycles is 2,000,000 for truck loading and 500,000 for lane loading. In addition, a footnote to Article 1.7.2B indicates that members must also be investigated for over 2,000,000 stress cycles produced by a single truck placed on the bridge (for a multiple girder structure, a wheel load distribution factor of $S/7$ is used).

Combined Stress Diagrams:

It is convenient to plot the combined stress diagrams, i.e., the sum of the dead and live load stresses for both top and bottom flanges. This will facilitate the fatigue analysis of the girder for items such as studs welded to top flange, transverse stiffeners welded to web, the bolted field splice, fillet weld of web to flange, and lateral bracing. Note that the live load and impact moments due to truck loading are used here, since the allowable stress range in fatigue, considering the number of cycles, is much lower for truck loading than for lane loading and governs this design. Generally, both lane loading and truck loading must be checked to determine which governs. The combined stress diagrams for top and bottom flanges of Span 1 are shown in Fig. 20 for the $S/5.5$ distribution factor. Applicable values for a distribution factor of $S/7$ are obtained by decreasing the values shown by the ratio 5.5.7.

The combined stress diagrams are constructed by plotting the combined maximum and combined minimum stresses at each point of concern on the girder. The difference between maximum and minimum stress at a point reflects the live load and impact stress range, F_{sr}. The stress range may be computed directly from positive and negative lane load moments or positive and negative truck load moments. However, it is necessary to know whether the stress range is one of tension-to-tension or reversal, in which case the fatigue criteria apply. The fatigue criteria do not apply for a range from compression-to-compression.

Fatigue Analysis of Stud Type Shear Connectors:

The length of top flange over which stud-type shear connectors are installed is governed in fatigue by Stress Category C (Table 1.7.2A2). This permits the following allowable stress range for truck loading (Table 1.7.2A1):

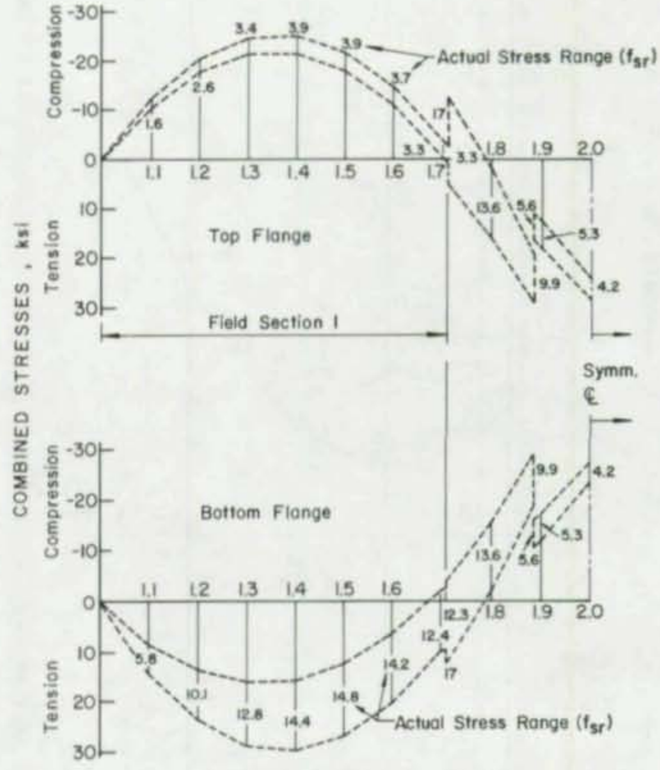


Fig. 20. Combined stress diagrams (S/5.5)

1. Multiple lanes loaded (2,000,000 cycles with $S/5.5$):

   $F_{sr} = 13$ ksi

2. Single lane loaded (over 2,000,000 cycles with $S/7$):

   $F_{sr} = 10$ ksi

This example problem is composite in the positive moment region (Field Section 1). By inspection of the top flange combined stress diagram, the maximum stress range in the composite region is 3.9 ksi. This range is always in compression and, therefore, fatigue is not of concern. Since the negative moment region (Field Section 2) is non-composite by design choice, no stud shear connectors are required there.

If the longitudinal slab reinforcing in the negative moment region was assumed to act compositely with the main girders (AASHTO Art. 1.7.63), shear studs would have been required in this region (Art. 1.7.48) and the fatigue design specifications applied. In this case, the combined stress diagram would have to be computed on the basis of the composite section properties of the reinforcing and the main girder, and the top flange stress range kept within the allowable.

Fatigue Analysis of Toe of Transverse Stiffener Welds:

Where transverse stiffeners are attached to the web with fillet welds, the allowable stress range at the toe of the fillet is governed in fatigue by Stress Category C (Table 1.7.2A2).

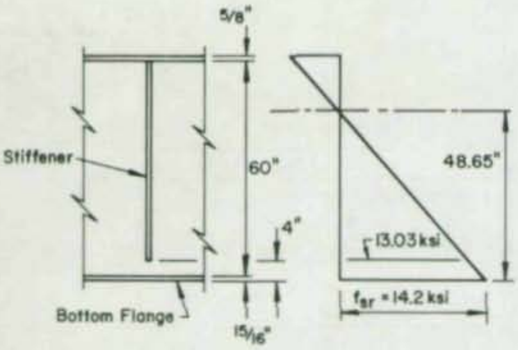


Fig. 21. Stress range at end of stiffener near 1.6 tenth point

This permits the following allowable stress ranges for truck loading:

1. Multiple lanes loaded (2,000,000 cycles with $S/5.5$):

   $F_{sr} = 13$ ksi (Table 1.7.2A1)

2. Single lane loaded (over 2,000,000 cycles with $S/7$):

   $F_{sr} = 12$ ksi (Table 1.7.2A1, footnote)

Before analyzing the combined stress diagram, first consider which of the above criteria will govern. To obtain the actual stress range for single lane loaded, multiply the actual stress for multiple lane loaded by $(S/7.0 \div S/5.5)$ or 5.5/7.0. The allowable stress for single lane loaded is 12/13 of the allowable stress range for multiple lanes loaded. Note that the reduction is greater for the actual stress range than for the allowable stress range. Therefore, by inspection, the multiple lane loaded condition governs ($F_{sr} = 13$ ksi).

Transverse stiffener welds must be terminated short of the web-to-flange welds by a distance four to six times the web thickness (see discussion of stiffener details in Chapter 1). The critical point in fatigue is the toe of the fillet weld connecting the stiffener to the web.

A group of transverse stiffeners begins near the 6th tenth point of Span 1 (1.6 point) and continues to the interior support (2.0 point). First, for Field Section 1, note from the combined stress diagram for the bottom flange that the maximum stress range over this region is 14.2 ksi at the 1.6 tenth point. Compute the stress range at this location as follows:

Distance from extreme fiber of bottom flange to location under consideration (see Fig. 21):

Use 6 times thickness of web plus thickness of bottom flange plate:

$(6 \times {}^{7}\!/_{16}) + {}^{3}\!/_{8}$ weld $+ {}^{15}\!/_{16}$ bottom flange $= 3.94$ (say 4.0 in.)

$$\text{Stress range } f_{sr} = 14.2\,\frac{(48.65 - 4.0)}{48.65} = 13.03 \text{ ksi}$$

Therefore, the actual stress range is close to the allowable stress range of 13 ksi at the location in question. No increase in bottom flange size is required.

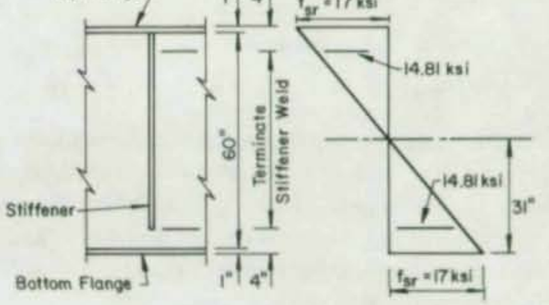


Fig. 22. Stress range at end of stiffener near 1.71 tenth point

Inspection of the combined stress diagram for the top and bottom flange plates in Field Section 2 indicates a stress range of 17.0 ksi in both top and bottom plates at the 1.71 point. Use the moment at this point in lieu of the moment at the actual stiffener location, as they are approximately the same. This is the point at which non-composite action is assumed and the resulting section properties are significantly decreased.

Distance from the extreme fiber to location under consideration (see Fig. 22):

$(6 \times {}^{7}\!/_{16}) + {}^{3}\!/_{8} + 1.0 = 4.0$ in.

$$\text{Stress range } f_{sr} = 17.0\,\frac{(31.0 - 4.0)}{(31.0)} = 14.81 \text{ ksi}$$

Therefore, the actual stress range exceeds the allowable stress range of 13.0 ksi at the location in question, and both top and bottom flange plate sizes must be increased.

Redesign Field Section 2:

$F_{sr} = 13$ ksi (at point of stiffener weld termination approximately 4 in. from the outer flange surfaces)

At 1.71 point:

The stress range for the flanges necessary to insure that the stiffener weld complies with the 13 ksi allowable is:

$$F_{sr} = 13.0\,\frac{(31.0)}{(31.0 - 4.0)} = 14.93 \text{ ksi}$$

Try top and bottom flanges 18×1.00 in.:

$$S_t = S_b = \frac{41{,}360}{31.0} = 1{,}334 \text{ in.}^3$$

Top flange stress range:

$$f_{max} = \frac{(136 + 1{,}125)(12)}{1{,}334} = -\,11.34 \text{ ksi}$$

$$f_{min} = -\,\frac{(136 - 508)(12)}{1{,}334} = +\,3.35 \text{ ksi}$$

$f_{sr} = |3.35 - (-\,11.34)| = 14.69 < 14.93$ ksi **o.k.**

Bottom flange stress:

f_{sr} = same as top flange stress ∴ **o.k.**

Area of flange plates required = 18 × 1.0 = 18 in.^{2}

Note that since the top flange plate is primarily in compression, the stiffener should not be cut short, but should be left to bear on the top flange plate. However, it should still be coped as described previously, and the weld terminated six times the web thickness away from the flange-to-web fillet weld.

At 1.8, 1.89, 1.9 and 2.0 points:

The allowable stress range at all these points is within the allowable stress range; therefore, no increase in plate size required at these points. Figure 23 shows the area of flange required at each tenth point, as required by fatigue criteria (O) or by basic flexural criteria (Δ). A solution to this example for Field Section 2 is two plates as shown in Fig. 23, or one plate 24 × 1¼ continuous over the interior support. An economic evaluation should be made to determine whether or not a splice is justified.

Use plates spliced as shown in Fig. 23.

Fatigue Analysis for Lateral Bracing Connections:

Lateral bracing may be attached to the girder in several different ways. A simple and inexpensive detail would be a horizontal plate welded to the web at a calculated distance up from the bottom flange. This distance would be the location where the calculated stress range in the web is less than the allowable stress range for this detail. The allowable stress range for a plate greater than 12 times the plate thickness or greater than 4 in. is Stress Category E (Table 1.7.2A2). This permits the following allowable stress ranges for truck loading:

1. Multiple lanes loaded (2,000,000 cycles with $S/5.5$):

   $F_{sr} = 8$ ksi

2. Single lane loaded (over 2,000,000 cycles with $S/7$):

   $F_{sr} = 5$ ksi

The single lane loaded with allowable stress range of $F_{sr} = 5$ ksi governs, since $(5.5/7.0)(8) > 5$.

For example, the location at which lateral bracing can be attached at the 1.71 point is found as follows:

The stress range at the 1.71 point, using the redesigned flange plate of 18 × 1.0, is:

$f_{sr} = 14.69\ (5.5/7) = 11.54$ ksi (single lane loaded)

By proportion, letting y equal the distance above the bottom flange at which lateral bracing may be attached (see Fig. 24):

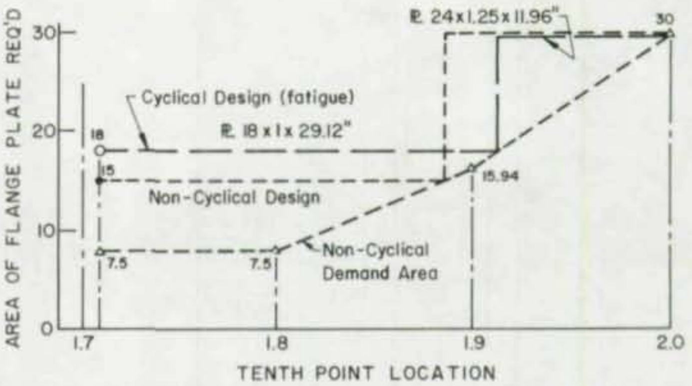


Fig. 23. Flange redesign

$$\frac{y}{(11.54-5)} = \frac{31.0}{11.54}$$

$y = 17.6$ in. (above the bottom of the flange to the point on the web where $F_{sr} = 5$ ksi)

For some details, this may be too far above the bottom flange, and another detail should be chosen. By using a 6-in. radius and making the connection to the web, the allowable stress range increases to 10 ksi. Here the detail could be placed 4.2 in. above the bottom of the flange, based on a similar method of calculation. Another option would be to bolt a suitable detail to either the flange or web, in which case Category B would be applicable and the member would be satisfactory. The actual choice of details is an economic decision. In some cases it may be more economical to add material and decrease the stress range; in other cases special details which satisfy the criteria without changing material sizes may be the better choice.

Analysis of Fatigue for Bolted Field Splice:

Proper sizing of the field splice material will satisfy Category B (Table 1.7.2A1).

Fatigue Analysis of Flange/Web Fillet Weld:

This condition is satisfied by inspection for the redesigned girder. All areas of the girder were analyzed for fatigue and

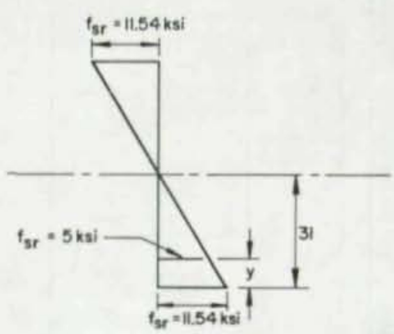


Fig. 24. Position on girder web where category E is satisfied

redesigned where necessary. In the area of the girder where no stiffeners or stud shear connectors are required (Field Section 1), the stress range on the original design was well within the 18 ksi stress range allowed by Category B (2,000,000 cycles, $S/5.5$ governs), Table 1.7.2A1.

Fatigue Analysis for Lane Loading:

Table 1.7.2B indicates that stress ranges caused by lane loading must also be investigated for 500,000 cycles (Case I). For Category C, $F_{sr} = 19$ ksi for this case. Since the live load moments due to truck loading are only significantly exceeded by the live load moments due to lane loading between the 1.9 and 2.0 tenth points, only this area need be checked.

Top flange: $F_{sr} = 19$ ksi

Bottom flange: $F_{sr} = 19$ ksi

At 2.0 tenth point:

Section properties computed are:

$$S_t = S_b = \frac{64{,}156 \text{ in.}^4}{31.25 \text{ in.}} = 2{,}053 \text{ in.}^3$$

We need only to consider the range of live load moments due to lane loading to determine the stress range:

$$f_{sr} = \frac{(1{,}545 - 0)(12)}{2{,}053} = 9.0 \text{ ksi} < 19 \text{ ksi o.k.}$$

General Comments:

This example is for a homogeneous girder of A588 steel. In hybrid designs, the designer should investigate using steel with a lower yield point in any flange plate where the flange size was increased due to fatigue requirements.

In load factor design, when a flange plate size is increased due to fatigue requirements, the ultimate moment capacity of the girder is increased. Under certain conditions this may reduce the required shear capacity of the web. This would permit larger stiffener spacing, possibly leading to fewer stiffeners and a more economical design.

CHAPTER 3
STRESS CYCLES FOR DESIGN

Overall, the history of both highway and railroad bridges has been quite satisfactory. The failures that have occurred pointed out the importance of properly considering in design and fabrication the factors that influence the fatigue strength of steel bridge structures. Some fatigue crack growth has occurred in a few bridge structures and components. The possibility of fatigue cracking under relatively high stress range conditions was demonstrated by the coverplated steel beam bridges of the AASHO Road Test.[6] More recently, cracks were observed in a coverplated bridge located on an interstate highway which carried an unusually high volume of heavy truck traffic,[7] causing large numbers of cyclic stress.

Fatigue cracks have been observed in other structures and their occurrence usually resulted from conditions that were not accounted for in design. These conditions have included: tack welds that were not incorporated into final welds, but were used during fabrication as means of temporary attachment; the addition of welded plates or attachments without considering their reduction in fatigue strength; unaccounted for out-of-plane displacement induced stresses; and details which changed the structures' behavior, such as connections which provided fixity when simple supports were assumed in the design. Many of these latter types of failures have been due to oversights in either design or fabrication and account for most of the adverse behavior experienced.

Early fatigue specifications in the United States originated from railway bridge design, which required reductions in allowable stress when members were subjected to load reversals.[8] During the 1940's both AREA and AASHO adopted the AWS bridge specifications for welded structures. These provided for three load cycle conditions: 100,000; 600,000; and 2,000,000. Allowable stresses were expressed in terms of the maximum stress and varied with the stress ratio R, defined as the algebraic ratio of minimum and maximum stress. These provisions were based on available test data, mainly on small plate specimens, and 2,000,000 cycles was generally assumed to be the run-out or infinite life condition.[9]

Little change in these provisions occurred until 1965, when new steel bridge fatigue provisions were adopted by AASHO. These provisions were developed from accumulated data from a variety of sources and a reexamination of older test data. Various types of conditions and details were divided into nine different classifications for fatigue lives of 100,000; 500,000; and 2,000,000 cycles. The allowable fatigue stress was still expressed in terms of the maximum stress, with provisions for stress ratio and steel strength. In the 1965 provisions, some details and members were permitted higher allowable stresses for high strength steels, whereas other details were not permitted such increases.

Minor changes were introduced as further data became available and the data base increased. Many of the early fatigue studies were carried out on A7 and A36 steels, while more recent studies were concentrated on higher strength steels. Because of this, some differences attributed to steel strength were more likely due to changes in welding techniques and improved experimental procedures, rather than the yield point of the material. Many past studies did not provide for an experiment design that would permit a statistical evaluation. Hence, it was not possible to provide a statistical analysis of the design factors that influence fatigue strength and determine their significance. Duplication was rare, critical variables were not controlled systematically, and the experimental error was not defined.

In order to overcome these limitations, the National Cooperative Highway Research Program supported a comprehensive study on "The Effect of Weldments on the Fatigue Strength of Steel Beams" at Lehigh University.[2,10] These studies used statistically designed experimental programs under controlled conditions, so that analysis of the data could reveal the significance of the parameters believed to be important in fatigue behavior.

These studies and other work available in the literature permitted a comprehensive specification to be developed.[10] These provisions were first adopted by AASHTO in 1973 and issued as *Interim Specifications—1974*. Revisions have been made in 1975, 1976, and 1977. Following is a brief description of the laboratory studies and criteria used to establish the current AASHTO Fatigue Tables 1.7.2A1, 1.7.2A2, and 1.7.2B, shown in Appendix B.

Experience with actual highway bridge structures in the United States has demonstrated that fatigue crack growth can occur when a bridge is subject to extremely high volumes of truck traffic.[7,11] This behavior is related to the fact that 2,000,000 cycles of loading does not correspond to a fatigue limit or crack growth threshold for some structural details, as was previously assumed in various specifications.[2,10] Fatigue damage in some cases can occur from many cycles of low stress range.

A reevaluation of the design life provisions was necessary to prevent occurrences on other bridges located on extremely heavily traveled arteries. Furthermore, studies on some transverse members subjected to wheel loadings suggested that higher stress ranges occur a larger number of times than was observed in main longitudinal members.

In order to develop a relationship between the design stress range and the actual truck traffic for the extremely heavily traveled artery, bridge lives were estimated from laboratory tests, assuming that damage accumulated in a linear fashion as suggested by Miner.[12] The applicability of this procedure was subsequently verified by extensive studies of beams under random variable stress cycles.[20,21] The fatigue studies used to develop design stress range values[2,10] have shown that the fatigue life, N_i, is related to the applied stress range S_{ri} as follows:

$$N_i = A S_{ri}^{-3} \tag{1}$$

where A is a function of the fatigue behavior of a detail. The design stress ranges are represented by nearly parallel stress-life curves. Throughout the nation, load-stress history measurements indicate that the measured stress ranges are always less than the design stress range, due to such factors as differences in load distribution, impact, actual truck loadings, etc.[13,14] Consequently, for fatigue design the actual stress range produced by vehicles similar to the design truck is a factor α (less than one) times the design stress range.

The relationship between gross vehicle weight (GVW) and stress range can be considered linear, and is usually constant for similar vehicles.[13,14] Hence, the relationship between actual stress range and (GVW) can be expressed as:

$$S_r = \alpha\beta(GVW) \tag{2}$$

where β is the elastic constant relating load and stress to a particular location on the structure. Miner's linear fatigue damage equation, $\Sigma n_i/N_i = 1$, yields the following relationship when expressed in terms of Eqs. (1) and (2):

$$\frac{(\alpha\beta)^3}{A} \sum n_i (GVW)_i^{\,3} = 1 \tag{3}$$

where n_i is the number of occurrences of $(GVW)_i$. When expressed in terms of frequency of occurrence of (GVW), (see Fig. 25), Eq. (3) yields:

$$\frac{(\alpha\beta)^3}{A} (GVW)_D^{\,3} (ADTT)(D_L) \sum \gamma_i \phi_i^{\,3} = 1 \tag{4}$$

where

$(ADTT)$ = average daily truck traffic
D_L = design life in days
ϕ_i = ratio of actual vehicle weight to design vehicle weight, $(GVW)_i/(GVW)_D$
γ_i = fraction of $(ADTT)$ for $(GVW)_i$

The summation in Eq. (4) is a function of the vehicle weight distribution and was determined from the 1970 FHWA loadometer survey (see Fig. 25). Figure 26 shows $\gamma_i \phi_i^{\,3}$ plotted

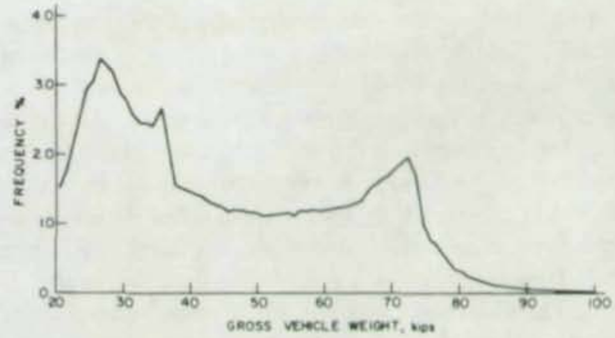


Fig. 25. Gross vehicle weight distribution from 1970 FHWA Nationwide Loadometer Survey

as a function of GVW. The sharp rise of the curve as the design load is approached indicates that most fatigue damage is likely to result from vehicles near the design vehicle weight. The summation of $\gamma_i \phi_i^{\,3}$ in Eq. (4) for all vehicles in the loadometer survey is about 0.35. If all vehicles in excess of 20 kips are assumed to cause damage, Eq. (4) can be conservatively expressed as:

$$\frac{\alpha^3}{A} [\beta (GVW)_D]^3 (ADTT)(D_L)(0.35) = 1 \tag{5}$$

The term $\beta(GVW)_D$ is the design stress range. Since design stress range can be determined from Eq. (1) for a specified number of constant stress cycles, N, the following ratio between the total number of trucks and constant stress cycles results:

$$\frac{(ADTT)D_L}{N} = \frac{2.85}{\alpha^3} \tag{6}$$

The factor α is the ratio of the actual stress range due to the passage of a design vehicle and the design stress range, and is less than one. Conservative values of α of about 0.8 for transverse members and 0.7 for longitudinal members were determined from field tests[11,13,14] and used to derive the ($ADTT$) found in Table 1.7.2B of the AASHTO Specifications.

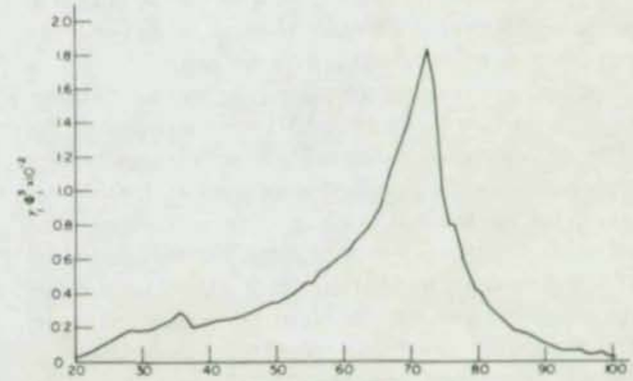


Fig. 26. Probable damage caused by various truck weights

All available studies indicate that most of the stress cycles caused by vehicle traffic are below the fatigue crack growth threshold (i.e., the actual stress range is less than the stress range which will propagate a crack from an initial discontinuity for the category corresponding to more than 2,000,000 stress cycles). No damage is believed to be caused by stresses below the fatigue crack growth threshold unless the maximum stress range in the variable stress spectrum exceeds the fatigue limit. Hence, the actual $\Sigma \gamma_i \phi_i^3$ is less than the value 0.35 used. The differences between actual stress cycles and the design condition also indicates that a transverse lateral wheel load distribution factor of $S/7$ is reasonable, especially for fatigue design of longer span steel I-beam bridges with a concrete floor, when both lane and truck loading must be examined. It reflects the fact that traffic induced stresses are caused primarily by single traffic lane loading.

When the few known fatigue cracks in bridges are compared with the $(ADTT)$ and observed stress history measurements, most of the damage appears to be caused by the heavier trucks. Only 10% to 15% of the $(ADTT)$ appears to result in stresses causing crack growth. This condition is only true for the most severe design details, such as coverplated beams and attachments which have terminating weld toes. Most other details have much higher fatigue crack growth thresholds and no crack growth is likely under any loading condition, unless some unusual condition exists. Transverse members which receive loads directly from individual wheels experience proportionately more cycles of loading.

The stress cycle tables recognize the increased stress cycles to which transverse members will be subjected. Experience with a few bridges indicated that a greater possibility for fatigue cracking existed, and conservative provisions were developed pending further studies which could provide more rational values.

The minimum life expectancy under the worst possible combination of loading cycles and the resulting stress range is between 60 and 70 years if all stress cycles are assumed to cause damage. Obviously, the minimum life is even greater, since many stress cycles are below the fatigue crack growth threshold and cause no damage at all. Since highway bridges are subjected to both deterioration and obsolescence, 60 to 70 years seems a reasonable life to anticipate should fatigue be a controlling factor. For the vast majority of bridges and their components, no crack growth is expected at all.

Experience with existing structures indicated that the design conditions used for Cases II and III were satisfactory. No fatigue problems have been experienced with bridges in these categories. Hence, the previously used stress cycle table was retained for longitudinal bridge members unless extreme numbers of truck passages were expected. Further load history studies will no doubt lead to refinement and better estimates of the ratio α of actual stress range to the design stress range, including the transverse distribution effect and its relationship to the vehicle weight distribution. Most highway structures are not subjected to the extreme volumes of truck traffic indicated by Case I. Therefore, the designer should not unduly penalize the fatigue design of a structure by using Case I, unless it appears to be warranted by traffic projections.

This section has described the assumptions used to develop the AASHTO stress cycle table for the design of highway bridge structures (see Table 1.7.2B). It is apparent that average conditions were used and assumed to apply to all highway bridges. If well defined traffic conditions are known, these can be used to determine a suitable design life using the method developed. For example, if an analysis indicates that the ratio α of actual stress range due to the passage of a design vehicle to the design stress range is 0.5 and an $(ADTT)$ of 3,000 is expected with the same vehicle weight distribution shown in Fig. 26, Eq. (6) could be used to estimate the required constant stress cycles. For a 60-year life this would yield:

$$N = \frac{(ADTT)(D_L)\alpha^3}{2.85} \tag{7}$$

$$= \frac{3{,}000(365)(60)(0.5)^3}{2.85}$$

$$= 2{,}882{,}000 \text{ cycles}$$

Hence, fatigue design could be based on the stress ranges corresponding to this life, using the plots given in Fig. 30 (see Chapter 4). This results in stress ranges of 7.1 ksi for Category E, 8.9 ksi for Category D, 12 ksi for Category C, 16 ksi for Category B, and 24 ksi for Category A.

It is also apparent that the stress cycles for design will be substantially different for railroad and mass transit bridge structures. Comparable design cycles can be developed based on span length, stress cycles per train, frequency of trains, type of member, and other conditions. These lead to design conditions that can be placed into a table analogous to Table 1.7.2B of the AASHTO Specifications. Such a table has been developed for railroad bridges in the AREA Specifications (see Table 1.3.13A in Appendix B).

AASHTO also adopted material toughness provisions in 1974 which insure adequate performance providing fatigue crack growth does not occur.[23]

Three primary factors control the susceptibility of a structure to brittle fracture. These are material toughness, flaw size, and stress level.[22,23]

Concern with nonredundant members, i.e., single box girder, two plate girder, or truss systems, etc., where failure of a single element could cause collapse of the structure, resulted in the adoption of a greater factor of safety for these types of structures in 1977, i.e., to further minimize the possibility of fatigue crack growth, the allowable stress range has been reduced for nonredundant members. This was accomplished by shifting one range of loading cycles for fatigue design, which results in a reduction in allowable stress range. Although a completely rational explanation cannot be supplied, the very restrictive stress range that results for certain categories will require the designer to investigate details that provide less reduction in fatigue strength.[48]

CHAPTER 4
STRESS RANGE CONCEPT

The fatigue strength of a particular structural joint has been evaluated in the past by tests on specimens that simulated the prototype connection, or on smaller connections which were similar. Only approximate design relationships were developed, because of the limitations of the test data.[15,16] Often many variables were introduced into the experiment with a limited number of specimens, which made it impossible to clearly establish the significance of stress conditions, details, type of steels, and quality of fabrication.

A substantial amount of experimental data has been developed on steel beams since 1967, under the sponsorship of the National Cooperative Highway Research Program (Project 12-7), which has shown that the most important factors that govern the fatigue strength are the stress range and the type of detail.[2,10] Stress range means that only the live load and impact stresses need to be considered when designing for fatigue. These findings were observed to be applicable to every beam and detail examined. Beam specimens were used to overcome some of the limitations of smaller simulated specimens. These beam tests and other work available in the literature were used to develop a comprehensive specification based on stress range alone.

A brief summary of some of the test data is given here to demonstrate that stress range and type of detail are the two factors which are most likely to govern the fatigue strength.

INITIAL DISCONTINUITIES

Two types of welded plate girder details examined in the laboratory are reviewed in this brief summary: (1) the welded plate girder without attachments and (2) beams with welded cover plates. Test data has demonstrated that all fatigue cracks commence at some initial discontinuity in the weldment, or at the weld periphery, and grow perpendicular to the applied stresses. In the welded plate girder without attachments, most laboratory fatigue cracks were observed to originate in the web-to-flange fillet welds at internal discontinuities such as porosity (gas pockets), incomplete fusion, or trapped slag. It should be noted that these discontinuities are always present, independent of the welding process and techniques used during fabrication. Identical behavior has been observed in the laboratory for longitudinal groove welds with either incomplete or complete fusion.[16]

The coverplated beam provides a structural detail in which crack growth starts at the weld periphery, where small sharp discontinuities exist at the toes of fillet and groove welds made by conventional welding processes.[3,17] The fatigue crack in a coverplated beam, with or without transverse fillet welds, forms from these micro-discontinuities perpendicular to the applied stress.

References 2 and 10 contain a number of photographs of fatigue cracks. These photographs illustrate the various types of discontinuities that exist in structural joints. Under large cyclic stresses these discontinuities grow and eventually result in failure. The test data are described in the following discussion of fatigue strength.

FATIGUE STRENGTH

The test data for the welded plate girder without attachments and coverplated beams are summarized in Fig. 27. Stress range is plotted as a function of cyclic life for several different levels of minimum stress on a log-log scale. It is visually apparent that stress range accounted for the fatigue strength for both structural details, i.e., minimum or maximum stress did not have a significant influence on the fatigue behavior. The ratio of minimum to maximum stress, R, did not affect the stress range to cycle life relationship. The coverplated beam results included wide cover plates, thick cover plates, and cover plates on both rolled and welded beams.

No significant difference was observed for either the welded girder or coverplated beam that could be attributed to the type of steel when a given detail was subjected to the same stress range conditions. This is readily demonstrated in Fig. 28, where the results are plotted for three grades of structural steel with yield stress ranging from 36 ksi to 100 ksi, representing the extremes generally used in bridge construction.

The data plotted in Figs. 27 and 28 show clearly that stress range is the critical stress variable for all structural steels. The results also confirm the significance of the type of detail. The coverplated beam only provided about 45% of the fatigue strength of the welded plate girder without attachments.

Studies on other details have also confirmed that stress range alone is the only significant factor for designing a given detail against fatigue. Results on beams with transverse stiffeners, attachments, and transverse groove welds have also demon-

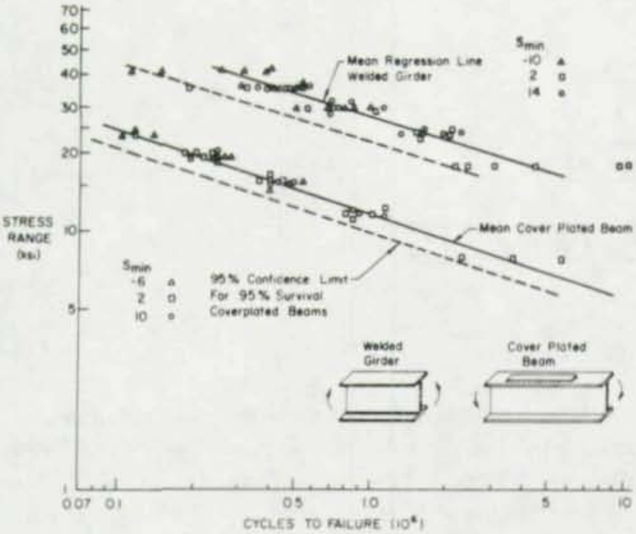


Fig. 27. Effect of minimum stress and stress range on the cycle life for the welded end of coverplated beams and plain welded beams

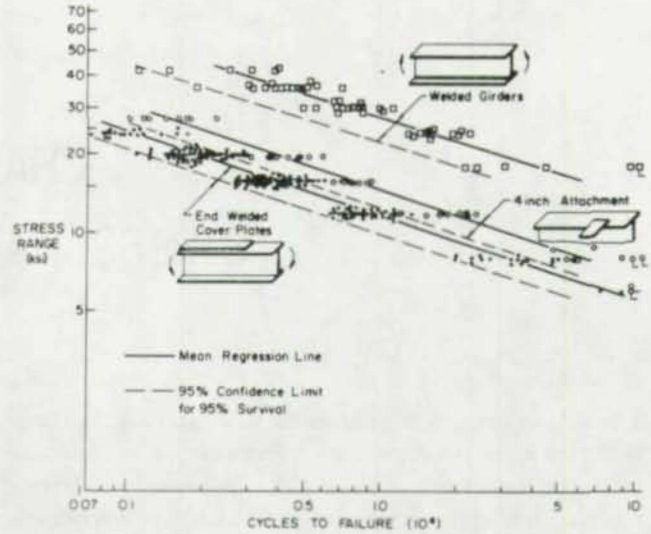


Fig. 29. Comparison of short welded attachments with coverplated and plain welded beams

strated that minimum stress and type of steel are not critical factors.[2,10] Groove welded splices at flange width transitions in A514 steel were more severely affected by the straight tapered transition. This led to the requirement for a curved transition for A514/A517 steel.

In a transverse groove weld with the reinforcement left in place, the stress concentration at the weld toe, with its associated small micro-discontinuities, is usually more severe than nominal internal discontinuities. However, if lack of penetration, slag inclusions, or other internal discontinuities are large in size, crack growth will become more critical at the internal location.[15,16,18] In bridge construction, transverse groove welds that are subjected to tension or reversal of stress are generally nondestructively tested to prevent large internal discontinuities from occurring. Also, the weld reinforcement is often removed, so that the weld toe is not critical and a high fatigue strength results.

All evidence indicates that the termination of groove and fillet welds provides a more critical crack growth condition than internal discontinuities in the weld. This is illustrated in Fig. 29 where the test data for three typical welded details are summarized. The welded detail with the highest fatigue strength is the welded beam without attachments. The same strength was observed in groove welded flange splices.[2] In these flange splice details, cracks normally grow from internal discontinuities that are perpendicular to the applied stresses. The other two details shown in Fig. 29 are a short attachment (4 in. long) and the coverplated beam. Both fatigue strength relationships were defined by cracks that formed at the end of the attachment at their weld toes. When the attachments were very short, as with a transverse stiffener, the fatigue strength approached the strength of a welded plate girder.[10] For an attachment 4 in. long, Fig. 29 shows that the fatigue strength is about midway between the upper bound (welded beam) and the lower bound (coverplated beam). Attachments longer than 4 in. quickly approach the lower bound condition given by the coverplated beam.

The stress range values given in Table 1.7.2A1 were derived from the 95% confidence limits for 95% survival. Rolled beams were used for Category A, welded plate girders for Category B, stiffeners and short 2-in. attachments for Category C, 4-in. attachments for Category D, and coverplated beams for Category E. The stress range cycle life relationships are plotted in Fig. 30 for each design category. After 2,000,000 cycles, the stress range approaches the crack growth threshold level for the various details and becomes a constant value. For more than 2,000,000 cycles, the fact that transverse stiffeners are less severe than a 2-in. attachment is reflected by an allowable stress range of 12 ksi, which appears to be representative of the threshold level for this design condition.

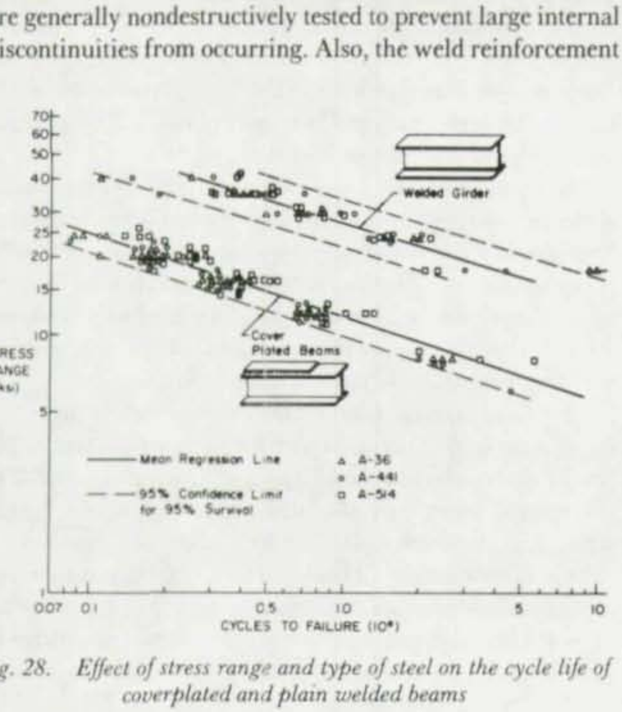


Fig. 28. Effect of stress range and type of steel on the cycle life of coverplated and plain welded beams

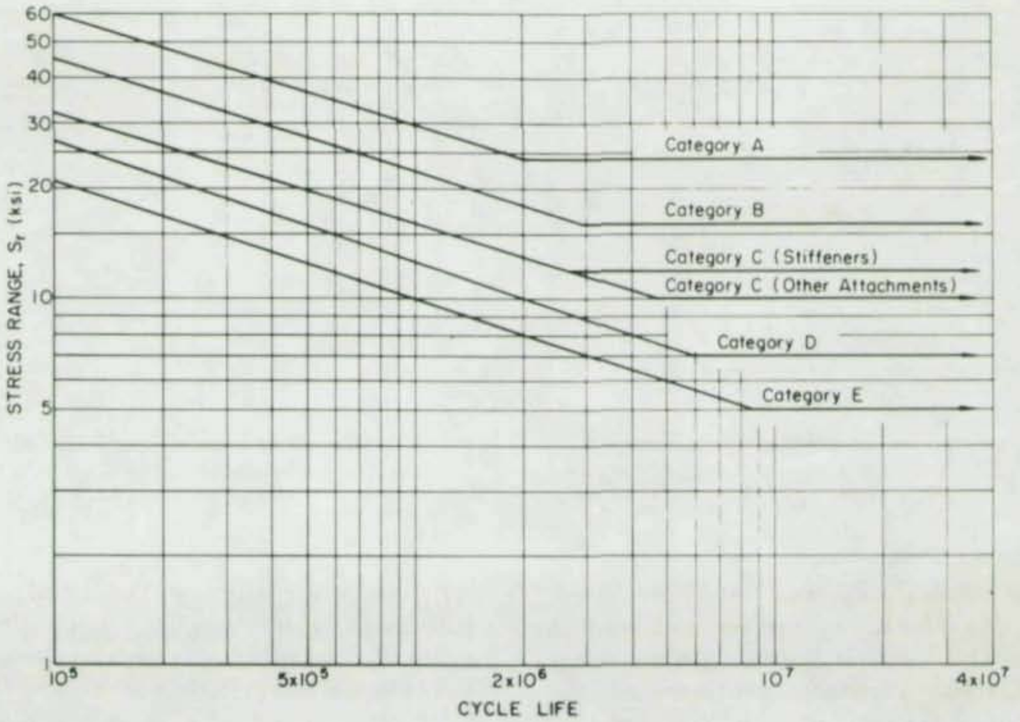


Fig. 30. Design stress range curves for Categories A to E

RESIDUAL STRESSES

All welding processes result in high tensile residual stresses, which are at or near the yield point in the weldment and base metal adjacent to it. These occur as the weld shrinks upon cooling. Thus, in the initial stages of fatigue crack growth in an as-welded structure, most of the fatigue life occurs in regions of high tensile residual stress. Under cyclic loading, the material at or near the initial discontinuity will be subjected to a fully effective cyclic stress, even in cases of stress reversal. This is the major reason why stress range alone is the variable describing the fatigue behavior of welded joints. As a result, the stress ratio, R, does not play a significant role when describing the fatigue strength of welded details, because the maximum stress at the point of fatigue crack initiation and growth is almost always at the yield point. Most of the fatigue life is exhausted by the time the fatigue crack propogates out of this high tensile residual zone.

An examination of the available data has shown that cracks have grown in the tensile residual stress zones of beam flanges subjected to cyclic compression alone.[5,6] However, these studies also showed that the crack arrested as it grew into adjacent compressive residual stress regions. No beams lost load carrying capability as a result of compression flange cracks unless out-of-plane bending stresses were introduced.

The existence of small cracks confined to the tensile residual stress regions of components subjected to compression alone is analogous to the compression splice proportioned to carry only part of the member's strength, with the balance of this force resisted in bearing.

As a result of this behavior, the fatigue design criteria is limited to regions subjected to tension or stress reversal. If the member is subjected to stress reversal, fatigue must be considered no matter how small the tension component of stress range is, since the crack generated in a tensile residual stress zone could be propagated to failure with very small components of the tension portion of the stress cycle.

It is apparent that residual stresses play an important role in both the formation of cracks from discontinuities that reside in the tensile residual stress zone and the arrest of cracks as they grow into a compression residual stress zone of a member subjected to compression alone.

VARIABLE STRESS CYCLES

The most widely used method to account for cumulative damage is the Miner hypothesis.[12] Variable stress cycle damage is accumulated in proportion to the relative frequency of occurrence of each level of stress range. Other methods have been proposed, but Miner's hypothesis is among the simplest.

In order to evaluate the significance of random variable stress cycles and assess the applicability of cumulative damage criteria such as Miner's Rule or the RMS (root-mean-square) procedure, a program of study was undertaken in 1971 under the sponsorship of the National Cooperative Highway Research Program (Project 12-12).[20,21] The study was carried out at the Research Laboratory of U. S. Steel Corporation on beams identical in geometry to those tested on Project 12-7 at Lehigh University.[2]

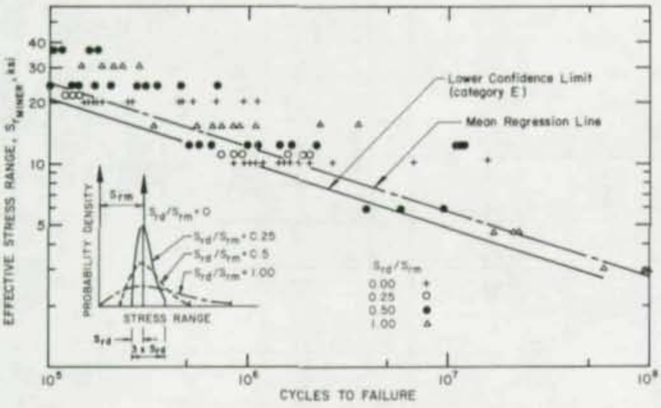


Fig. 31. Comparison of variable load tests with mean and lower confidence limit for Category E using Miner's Rule (Ref. 21)

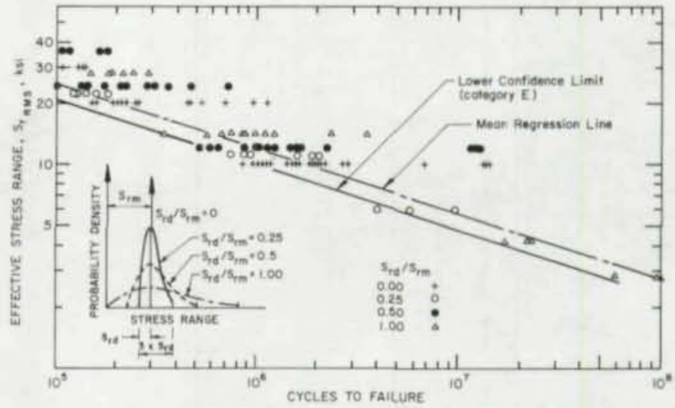


Fig. 32. Comparison of variable load tests with mean and lower confidence limit for Category E using RMS effective stress range (Ref. 21)

The results of this study indicated that Miner's linear damage hypothesis and the RMS stress range both provided a means of relating random variable stress cycles to constant cycle data.[20,21] An effective stress range can be developed using Miner's linear fatigue damage relationship $\Sigma n_i / N_i = 1$ together with Eq. (1) (see Chapter 3) as:

$$S_{r(MINER)} = [\sum \gamma_i S_{ri}{}^3]^{1/3} \qquad (8)$$

where γ_i is the frequency of occurrence of stress range S_{ri}.

The RMS stress range for a variable stress spectrum can be defined as:

$$S_{r(RMS)} = [\sum \gamma_i S_{ri}{}^2]^{1/2} \qquad (9)$$

The results of coverplated beams tested under variable cyclic loading are plotted in Figs. 31 and 32 and compared with the mean and lower confidence limit given in Fig. 29 for constant cycle loading. Equation (8) was used to determine an effective Miner's stress range for the variable stress spectrum for the points plotted in Fig. 31, and Eq. (9) was used to determine an effective RMS stress range for the test points plotted in Fig. 32. The variable stress spectrums conformed to a Rayleigh distribution as shown schematically in Figs. 31 and 32. It is apparent that Miner's linear damage relationship and the RMS stress range both provided good methods of transforming the variable stress spectrum into an equivalent effective stress range. A second factor is also apparent at the lower levels of effective stress range. Several tests were conducted with an effective stress range below the constant cycle fatigue limit. Some cycles in the stress spectrum exceeded the constant cycle fatigue limit and this apparently caused all stress cycles to contribute to fatigue damage. The plotted points are seen to fall between the confidence limits. Hence, if no crack growth can be tolerated and extreme life is required, all stress cycles should be less than the fatigue limit.

CURRENT RESEARCH

Considerable research is underway in the United States and abroad on structural fatigue. Studies are continuing on the high cycle fatigue behavior of the lower fatigue strength details, variable stress cycles, curved girder details, methods to retrofit or repair fatigue-damaged members, the effect of environmental conditions, and other related problem areas.

Studies on full scale welded bridge details, completed in 1976, indicated that full sized coverplated beams have less fatigue strength than implied by Category E.[24] A comparison of this test data with results of studies on several bridges that experienced fatigue cracking shows reasonable agreement with the laboratory findings and field experience.[25]

Work currently underway on NCHRP Project 12-15(2) will provide a more comprehensive data base on full scale beams, so that an appropriate design category can be provided in the near future.

Stress history studies are continuing or are planned, so that the stress spectrum can be better defined for both highway and railroad structures. Most of the studies have focused on bridges of short or medium span length. The behavior of larger span bridges is now under study. Field measurements are also being made to help evaluate methods of retrofitting and upgrading older bridges.

CHAPTER 5
CONNECTIONS — FATIGUE CONSIDERATIONS AS A RESULT OF SECONDARY STRESSES

The influence of connecting various components to the main load-carrying members was discussed in detail in Chapters 1 and 2. Various design details were examined and their effect on the fatigue strength of the member was noted and dealt with. The stress range resulting from the design load was used to control the detail selection and location.

In addition to control over the detail selection and the allowable stress range applicable to it, other factors must be taken into account if the possibility of fatigue cracking is to be minimized or eliminated. The interaction of the components of a structure in response to live load creates secondary stresses at the connections. These secondary stresses must be provided for, in addition to the computed primary stresses, when bridge connections are being designed. In this Chapter, the effect of secondary stress conditions not directly accounted for in the design are discussed, and recommendations are provided to minimize their effect.

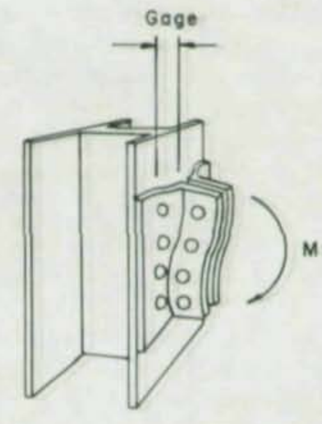


Fig. 33. Deformation of standard beam connection

RESTRAINT AT SIMPLE END CONNECTIONS

Many simple framing connections that fasten beam or girder ends, such as stringer-to-floor-beam connections, are usually considered to be completely flexible and to carry shear only.

However, no practical riveted, bolted, or welded connection can be completely flexible. Some resisting moment, or end restraint, is always developed as the connected parts resist the rotation of the end of the beam. Its magnitude depends on the relative flexibility of the connection and the connected parts. End rotation at a simple framing connection causes distortion to occur, as illustrated in Fig. 33. Usually the upper part of the connection is in tension while the lower part is compressed against the member to which it is connected. This end rotation is accommodated by the distortion of the angles and deformation in the fasteners (bolts or welds). Typical moment-rotation curves for several end connections are shown in Fig. 34.[27,28] These curves demonstrate the end restraint offered by various bolted connections. Two undesirable conditions can develop at "simple end" connections. One results from deformation of the components, as shown schematically in Fig. 33. The second can result from the restraint and resulting end moment.

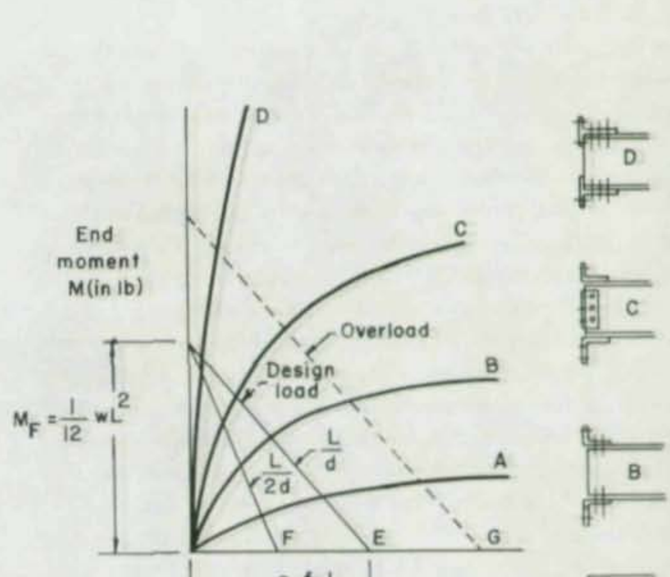


Fig. 34. Typical moment-rotation curves and beam lines

Problems with Distortion of Simple End Connection Components

Usually, simple connections have good rotation capacity and are able to accommodate to the imposed end rotation without distress, so that the assumed beam capacity is developed. In most static loading cases, whatever end restraint develops could be considered beneficial, because it increases the resisting capacity of the members. However, under cyclic loading this restraint may result in fatigue damage and cracking of the connected parts. Since the connected parts must deform to accommodate the end rotation, cyclic stresses will result in the deformed angle or plate. If these stresses are large enough, eventual cracking is inevitable. Because a rational analysis is not always possible in connections, it is very difficult to predict accurately the stress range that occurs as the end connection is repeatedly deformed.

Only after many years of service did the railroads experience cracking in riveted web-connection angles as a result of end rotation. The deformation resulted in cracks in the angle, as shown in Fig. 35. Studies were made by Wilson[29,30] and rules were suggested for the gage for fasteners in the outstanding legs to assure the necessary flexibility to reduce the range of stress and minimize restraint. This rule is in use today in the AREA Specifications[31] as

$$g = \sqrt{\frac{Lt}{8}} \tag{10}$$

where L = span length, t = connection angle thickness, and g = gage of the fasteners in the outstanding legs of the upper third of the member depth.

Wilson derived this rule by considering the end rotation that occurred in a stringer during passage of a train. The results of fatigue tests and Wilson's analysis were used to provide the criteria for the necessary gage.[30]

It should be noted that cracking usually will occur in the top of the connection angle, as the distortion shown in Fig. 33 places the upper portion of the angle in tension. However, occasionally the bottom of the connection angle will also crack, because the flexibility of the floor beam will cause the angle to open up at the bottom or because the "compression" side does not come into contact with the member it frames into, as a result of thermal effects or construction conditions.

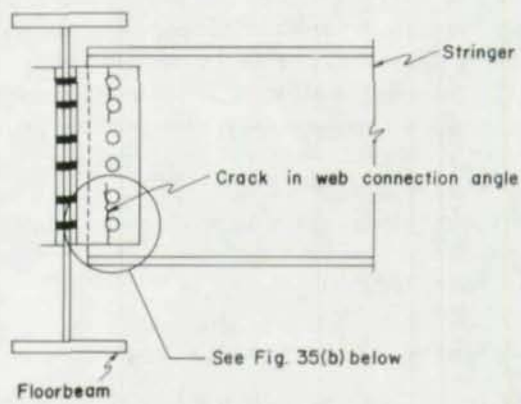
A few riveted highway bridges have also experienced distress in stringer-to-floor-beam connections, as is illustrated in Fig. 36. The prying of the outstanding legs on the rivet heads of the stringer-to-floor-beam connection has caused fatigue cracks to develop in the rivet under the head, which propagate until failure occurs. An analysis indicated that unintended continuity of the stringers existed as a result of the end connection and restraint to the top flange from the continuous composite slab. This placed the neutral axis near the top of the end connection and resulted in the bottom of the connection being in tension during part of the stress cycle. Hence, even though the stringers were assumed to be simply supported in the design, they in fact acted as though they were continuous.

This indicates that consideration must be given to both "tension" and "compression" sides, as both are potential areas for cracking. This depends on where the center of rotation is, how the parts are connected, how flexible the support members are, and what part of the connection can go into tension during deformation.

In the two cases discussed, the restraint developed at the end connection caused distress because the connected parts were distorted to accommodate end rotation. This resulted in cyclic flexing of the web connection angle and the coupled prying of a rivet or bolt head. Neither of these cases satisfied the requirement of Eq. (10).

Problems with Restraint

Since some resisting moment occurs at every "simple" end connection,[27,28] the use of a very low fatigue strength detail in highly stressed areas (i.e., a detail which permits only a low allowable stress range or, conversely, a detail which will fail in a low number of cycles if subjected to a high stress range)



(a)

(b)

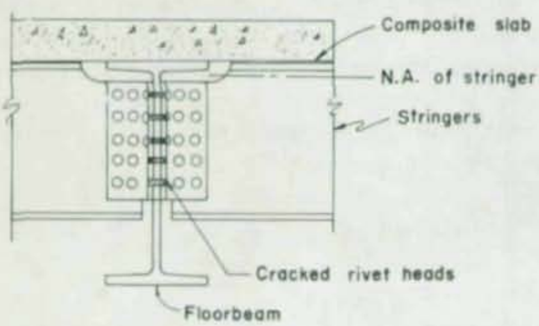
Fig. 35. Fatigue crack in standard connection angle

will invariably lead to cracking from distortion and restraint. An example is the case shown in Fig. 37, where a stringer web is welded to a floor beam connection plate. Restrained rotation at the ends of the simple span stringers has resulted in high bending stresses at the ends of the web shear connection. Fillet welds connect the beam web to a transverse stiffener plate which is, in turn, welded to the floor beam web and compression flange. Restraint from the connection caused crack growth in the stiffener plate, as illustrated; other comparable details have exhibited cracking in the beam web as well. Weld terminations occur at the top and bottom of the connection in the highest stressed regions.

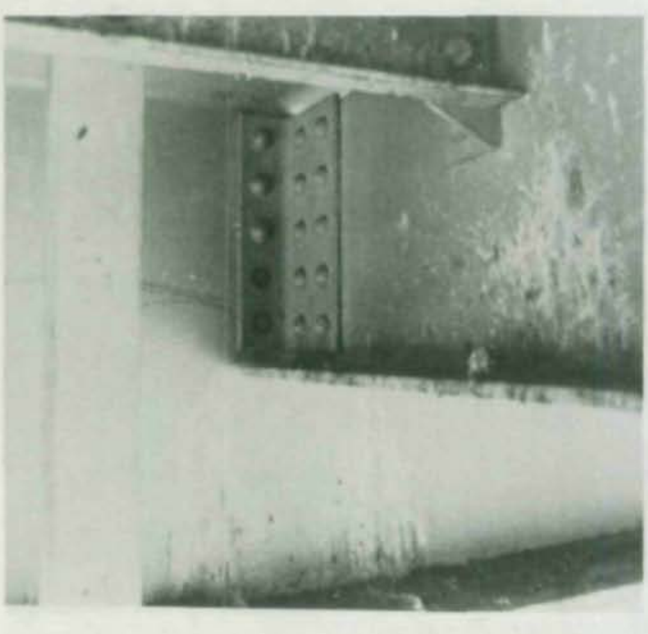
Insufficient bending capacity in the connected material is another reason for very high stresses when restraint occurs. If the flange of the stringer is assumed to have the same area as the web, the section modulus, S_x, of the stringer is approximately $7td^2/6$. However, by removing both flanges to provide a lap splice of the web and connection plate, the section modulus is reduced to less than $td^2/6$. Hence, resistance to bending has been reduced by about 85%, which substantially increases the stress range. In the actual structure, the slab continuity will obviously reduce the stresses and minimize displacement because of partial composite action, but this is often not enough to prevent fatigue crack growth.

A related problem can develop at bolted connections. Such an example is shown in Fig. 38. Here, cracking has developed at the end of a stringer which was high-strength-bolted at a "simple" web angle connection to the floor beam. Crack growth has developed from the nominal "compression" end of the connection, where the stringer flange was coped to provide clearance for the floor beam flange. Since only one flange was coped, the bending resistance of the cross section was reduced to about 30% of the original bending resistance.

Because of the cope, the connection end restraint caused a bending stress range to occur at the cope that was more than three times greater than it would have been with the flange intact. Cracking developed because stringer continuity and differential floor beam deflection developed tension during part of the stress cycle, and also because tensile stresses near the

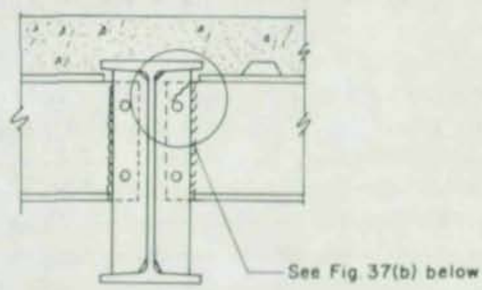


(a)



(b)

Fig. 36. Cracked rivet heads from cyclic end rotation



(a)

(b)

Fig. 37. Crack in stiffener plate of stringer-to-floor-beam connection

yield point due to flame cutting would exist at the point of crack initiation. Reference 2 has demonstrated that a flame-cut edge parallel to the stress is comparable to Category B details. The stress concentration at the cope[10] is likely to reduce this to Category C. After the crack has propagated through the zone of tensile residual stress, the principal tensile stress resulting from the end shear and restraining moment is large enough to continue propagating the crack.

The cases cited have shown that the restraining moment at simple end connections can result in crack growth. Large bending stress ranges were developed in both the welded and high-strength-bolted end connections because the cross-section resistance was substantially reduced.

Summary and Recommendations for Simple Connections Subjected to Cyclical Loads

If a beam is designed as simply supported, the end connection should preferably have minimum restraint. However, although welded lap connections such as that shown in Fig. 37 do not develop restraint much greater than is developed in bolted bearing-type web connections or seated beam connections, such lap connections should not be used for cyclically loaded beams because they have weld terminations in undesirable regions.

A bolted connection can result in unusually high bending stress ranges in the web if the flanges are removed near the connection. Where it is anticipated that high cyclical stresses will occur, Fig. 39 shows the preferable way of connecting simply supported end members. If it is necessary to cope a flange in order to provide clearance, then an auxiliary flange should be welded to the web, or web shear plates should be added, to increase the bending resistance of the cross section. Figure 39 shows these two possible solutions. The bending resistance of the member should not be permitted to decrease by more than 50%. This will normally provide adequate fatigue strength. If an auxiliary flange plate equal in size to the flange is used, this criteria will be satisfied.

A number of steps can be taken to minimize the possibility of cracking in the angle legs. The angles should be as thin as practicable, consistent with the reaction shear requirements, and the gage of the outstanding legs should be large, particularly near the ends of the connections where the deformation is greatest. The AREA rule developed by Wilson[30]

$$g = \sqrt{\frac{Lt}{8}} \tag{10}$$

for the top one-third of the member depth provides a large gage in the tension region of the outstanding legs of a simple web angle connection. A gage large enough to satisfy the requirements for railroad service will not be required for highway bridges, as less end rotation should occur. Based on the same criteria used by Wilson, a gage of $\sqrt{Lt/12}$ would appear reasonable for most highway bridge structures. If an analysis indicates that tension can be present in the "compression" region during the stress cycle, this criterion should be provided at both ends of the connection. Figure 40 shows a suggested fastener configuration that should minimize the possibility of cracking end connection angles.

Fig. 38. Cracked web at simple stringer end connection from restraint and reduced bending resistance

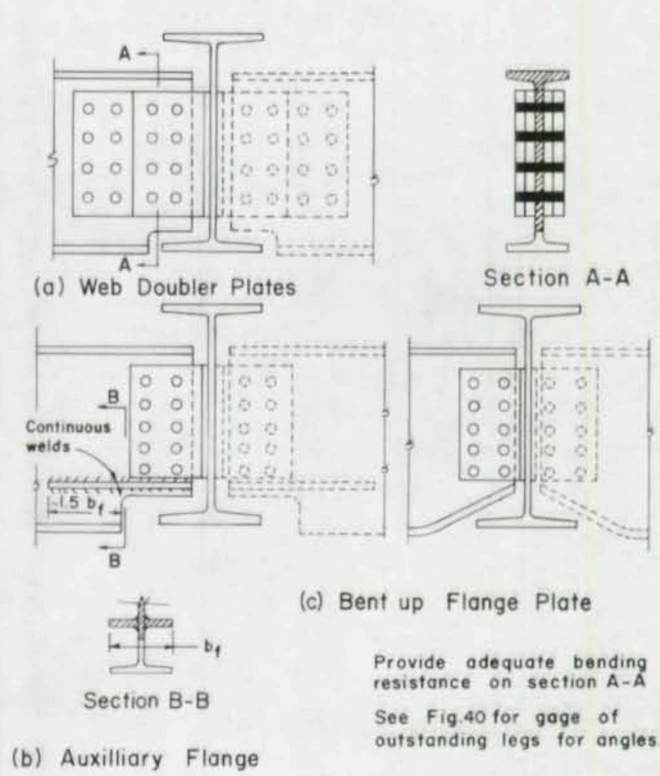


Fig. 39. Recommended connections where stringer flanges must clear floor beams (see Fig. 45a for stringer connection)

Properly tightened high-strength bolts loaded in shear are not subject to fatigue failure. Properly tightened high-strength bolts loaded in tension will seldom have the bolt crack within the grip below the nut, providing prying action has been taken into account. The 1976 RCRBSJ specification for A325 and A490 bolts[32] provides design criteria for this type of cyclic loading. Of major importance is the level of fastener preload. If the bolts are not properly tightened, any cyclic loading (with or without prying) can cause a significant variation in the cyclic stress in the threaded region. If the bolt is tightened at or above the minimum preload, the threaded area will not experience the large variation of stress that would occur in a fastener with little or no initial tension. Hence, any bolt subjected to cyclic tension from direct loading or from distortion of the connected parts should be tightened to at least the minimum preload specified.

DISPLACEMENTS AT STRINGER CONNECTIONS AND WEB BRACKETS

Stresses that are not directly related to the primary calculated stresses, nor to the restraining characteristics of the member, are often introduced into a structural detail. These stresses are most often produced by unaccounted for out-of-plane displacements. Cracks formed as a result of these displacements generally do not adversely influence the member's strength, nor the performance of the structure. They may create maintenance problems and, if not treated, may eventually lead to more serious fatigue damage.

Flange Twisting and/or Lateral Movement

Flanges of stringers are often subject to rotations which result from the deflection of ties, in the case of railroad or mass transit bridges, or the deformation of flexible decks of highway bridges. At end connections, angles and brackets are attached to the web. These attachments are usually detailed with small gaps between the top of the attachment and the stringer top flange. Thus, the central portion of the web is strongly restrained, which forces reverse curvature to occur in the web above the attachment, below the twisting flange.

Figure 41 shows an example of web cracking that occurred in the web gap at a simple framing connection. End connection angles bolted to a rolled beam web resulted in a ¾-in. unstiffened portion of the web. Deflection of ties placed near the end connection caused relatively small out-of-plane displacements to concentrate in this space where bending stiffness is small.

The out-of-plane bending moment in the web at the top of the attachment is given by:

$$M = \frac{4EI\theta}{L} + \frac{6EI\Delta}{L^2} \tag{11}$$

where the displacements θ and Δ are defined in Fig. 42 as the rotation of the flange relative to the web and the out-of-plane movement of the web. This of course assumes a fixed end condition of the web at the connection. Considering a unit strip of web, the resulting stresses in the web are:

$$\sigma = \frac{M}{S} = \frac{tE}{2L}\left[4\theta + \frac{6\Delta}{L}\right] \tag{12}$$

It is apparent that the out-of-plane web bending stress is very sensitive to the unrestrained length of web, L, and the mode of displacement that is imposed. For a given displacement and rotation, if the length L is doubled, stress due to rotation is decreased 50% and stress due to displacement is decreased 75%. If θ and Δ are free to increase when L is increased, stresses will also be increased. Of course, a cycle of stress occurs with the passage of each wheel load.

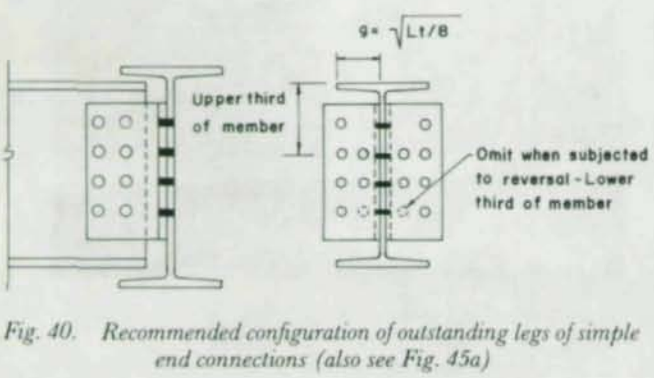


Fig. 40. Recommended configuration of outstanding legs of simple end connections (also see Fig. 45a)

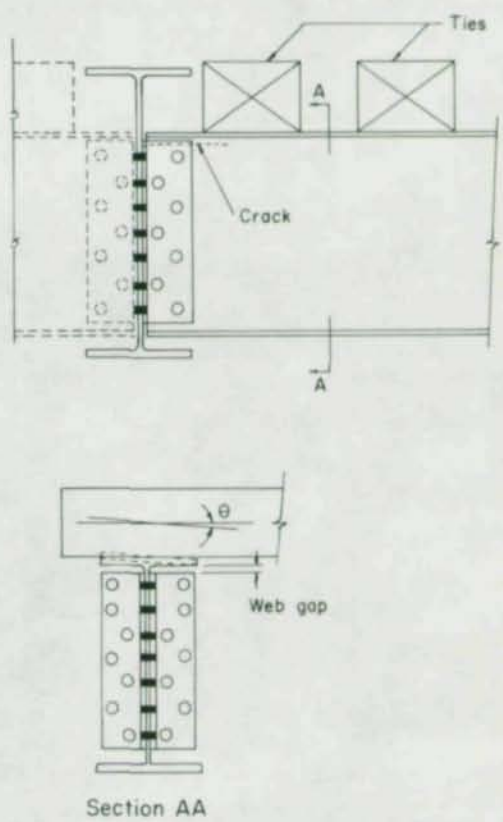


Fig. 41. Schematic of web cracking from flange rotation

Similar difficulties have been experienced when stringers rest on floor beams or floor beam trusses. Brackets attaching the floor beam to the stringer web may cause out-of-plane movement to concentrate in the stringer web as a result of deflection of the slab, shortening of the compression flange of the floor beam, or lateral movement of the slab. Figure 43 shows a cracked stringer web above a bracket. Web bending stresses at the web-flange fillet have caused cracking to occur at the restraint point.

The problem of out-of-plane displacement can be particularly acute when significant horizontal forces develop in structures on curves. These forces can cause additional flange movement relative to the web.

When very large or unrestricted deformations occur, it may not be possible to provide enough length of unrestrained web to accommodate the forced rotations and displacements and, at the same time, satisfy the requirements of Eq. (12) at an acceptable level of stress. A case of this type is illustrated in Fig. 44, where end diaphragms frame into the stringer web. Here a flexible slab has induced large flange rotations and a 6-in. gap was inadequate to prevent cracking at the end of connection angles.

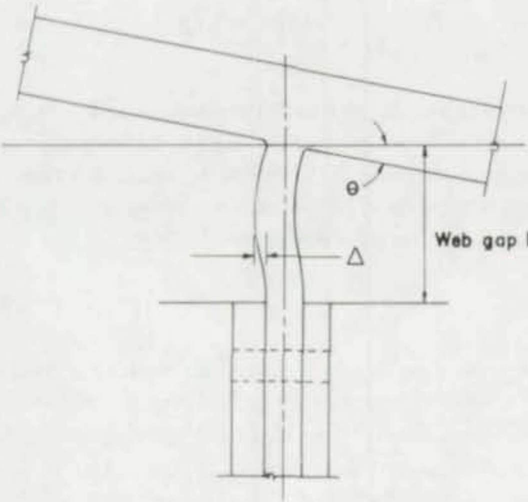
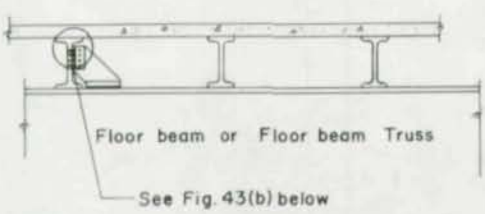
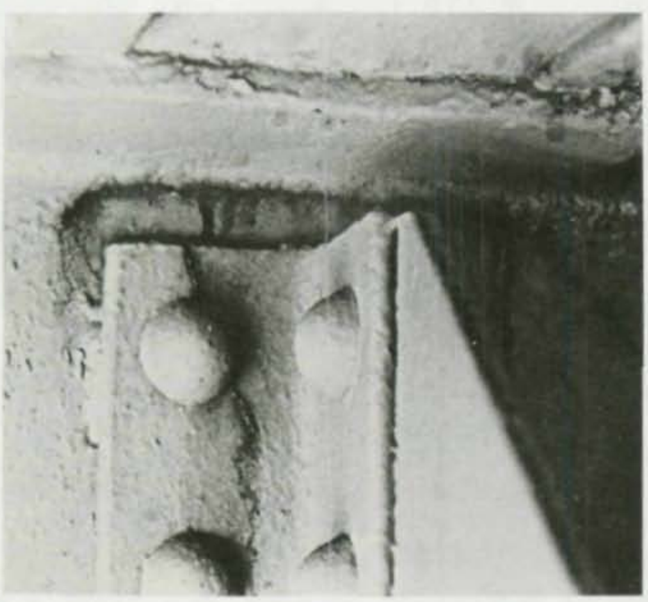


Fig. 42. Schematic of out-of-plane deformation in gap length

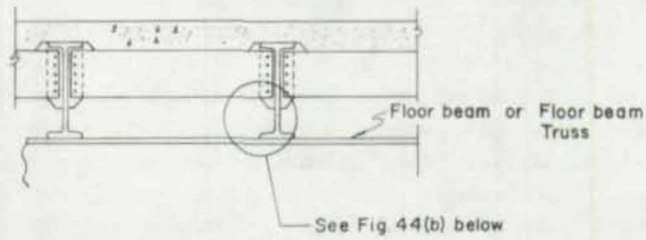


(a)



(b)

Fig. 43. Crack in stringer web above a web bracket



(a)

(b)

Fig. 44. Web cracking at cut-short web stiffener

Recommendations to Minimize the Effects of Out-of-Plane Movement at Stringer Connections

To overcome the possibility of cracking as illustrated in Figs. 41 and 43, a liberal "gap" dimension should be provided, so that the deformations are not forced to occur within short lengths, causing high localized strains and high out-of-plane web bending stresses. The required gap distance to keep the stress range within tolerable limits is dependent on the web thickness and the magnitude of the displacements θ and Δ. For estimating fatigue performance, the unrestrained region is comparable to details of Category C for a welded stringer and Category A for a rolled beam. If it is possible to estimate the magnitude of θ and Δ, these values should be used to establish the required length of gap. Since deformations will occur with the passage of each axle load, most structures will need to meet the allowable stress range requirements for 2,000,000 stress cycles, although heavily traveled bridges should be designed for over 2,000,000 stress cycles.

In many structures, if the actual magnitude of θ and Δ is restricted, but difficult to establish with assurance, a gap length of about 4 in. is recommended, as illustrated in Fig. 45a. When deformation is not self-limiting, as for example by simple span end rotation of a floor beam, resistance to the deformation should be provided. As noted earlier, requirements for restraint to deformation can be particularly demanding on curves or where significant horizontal forces are developed. Figure 45b shows one possibility. A second angle placed on each side of the stringer web, with outstanding legs welded to the stringer flange, will provide sufficient out-of-plane resistance. Care should be exercised to insure that the ties are placed over the outstanding leg, otherwise a severe condition can develop in the stringer flange. Consideration should also be given to the displacements at the adjacent tie.

A similar condition can develop at the stringer connections at floor beam roadway relief joints, as was shown in Fig. 44. Sufficient resistance could be provided by extending the stringer connection and attaching it to the flanges, as shown in Fig. 46, which prevents out-of-plane movement in the stringer web.

The coped diaphragm in Fig. 46a is not sensitive to the problem shown in Fig. 38. Continuity across the stringer connection is provided by the slab, and this keeps the neutral axis near the cope. Because the bending stress range at the cope is small when compared to the condition shown in Fig. 38, it is not necessary to reinforce the cope shown in Fig. 46a. This has been verified by successful performance in the field at numerous comparable conditions. If shear connectors are not installed to insure composite action, the coped top flange is a weak point, and the alternate detail shown in Fig. 46b should be used. In this detail, only the connected side of the flange should be cut and chipped away, so that adequate bending resistance will be available in the diaphragms.

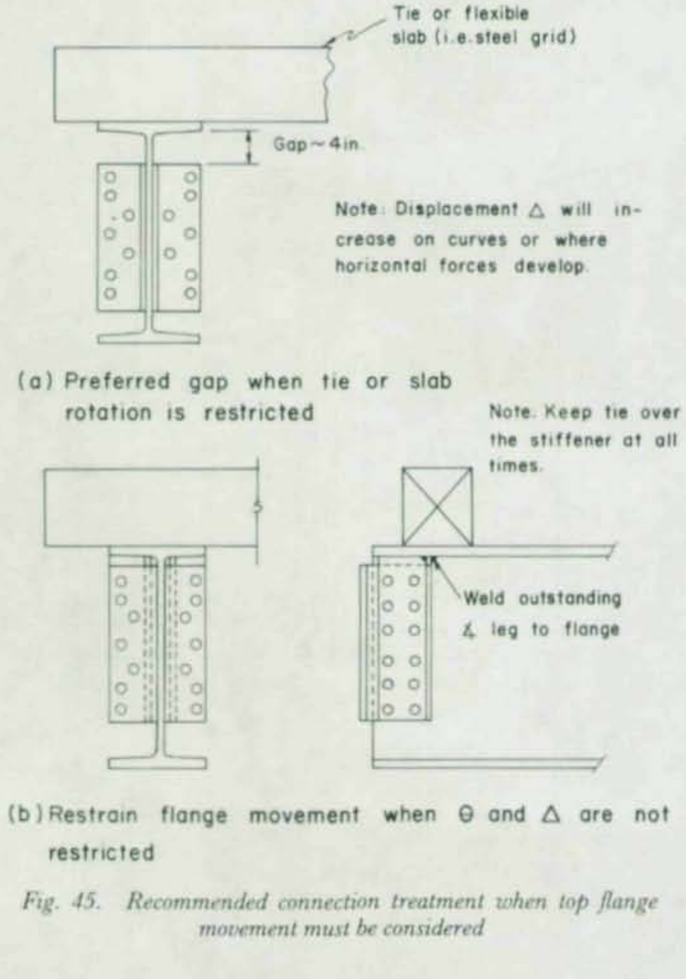


Fig. 45. Recommended connection treatment when top flange movement must be considered

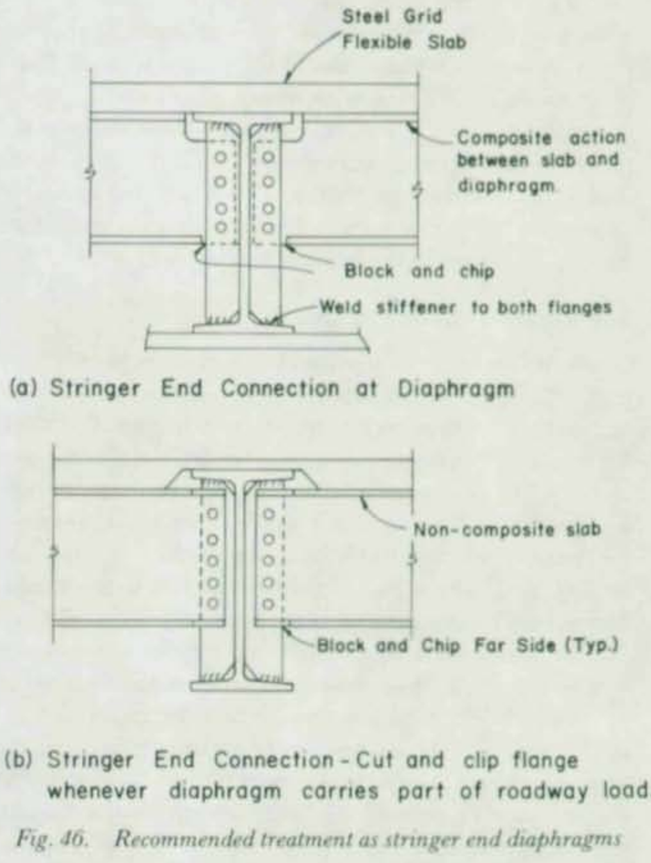


Fig. 46. Recommended treatment as stringer end diaphragms

It should also be noted that care must be exercised when making blocks, copes, and cuts in the beam. Since these are made by torch, the cutting operation is usually made from two perpendicular directions, starting from each end and meeting in the re-entrant radius. Hence, the potential for gouges and other undesirable conditions is great. Should inadvertent overruns or notches be made, they must be ground smooth.

OUT-OF-PLANE DISPLACEMENTS AT FLOOR BEAM CONNECTION PLATES AND TRANSVERSE STIFFENERS

In recent years, some web cracking has developed at the ends of transverse stiffeners and floor beam connecting plates. These cracks have resulted from cyclic stresses caused by out-of-plane movements not generally considered in design.

Such fatigue cracks tend to occur in the girder web, in the region of the gap between the end of a transverse stiffener or connection plate and the tension flange of the girder. Past bridge specifications have required that the distance from the end of a transverse stiffener to the tension flange be not more than four times the web thickness, to prevent web crippling under the stiffener, where the unsupported web acts as a column in resisting the vertical component of tension field action. These provisions followed the recommendations of Basler and Thurlimann,[19] which were based upon ultimate strength laboratory tests of plate girders. However, this criterion for a minimum gap distance was too conservative, because it ignored the restraint of the web-to-flange fillet welds and was based on states of stress that develop near ultimate load, rather than under service conditions.

Because of this gap restriction, a severe condition may develop from out-of-plane movement of the web at floor-beam-to-main-girder connections and other locations where movement develops in small gap regions caused by forces not generally considered in design, nor within the scope of the Basler and Thurlimann studies. This has often resulted in fatigue cracks forming at the end of the stiffener or connection plate, as discussed in the following sections.

End Rotation of Floor Beams

Girder bridges with transverse floor beams commonly have the floor beams framing into the main girder webs. The connection between the floor beam and girder is generally bolted. This joint utilizes a connection plate welded to the girder web and compression flange. The floor beam is bolted to the connection plate as illustrated in Fig. 47a. Rotation of the end of the floor beam is transferred into the girder through the connection plate. The plate has not been welded to the tension flange, because this detail was not permitted until 1974.

Since the connection plate stiffens the girder web to which it is attached, the floor beam end rotation has resulted in out-of-plane deformation in the web near the tension flanges. In the negative moment regions of continuous girders, the tension flange of the girder is prevented from moving by the slab in which it is embedded or connected. In positive moment regions, the tension flange is restrained near the end supports. Hence, the web may be forced out-of-plane in the gap region (see Fig. 47b), in the positive moment regions near end supports and in the negative moment regions where the connection plates are not attached to the tension flanges, but where these flanges are restrained by the slab. Cracks have been observed in both of these locations. Generally, floor beams in positive

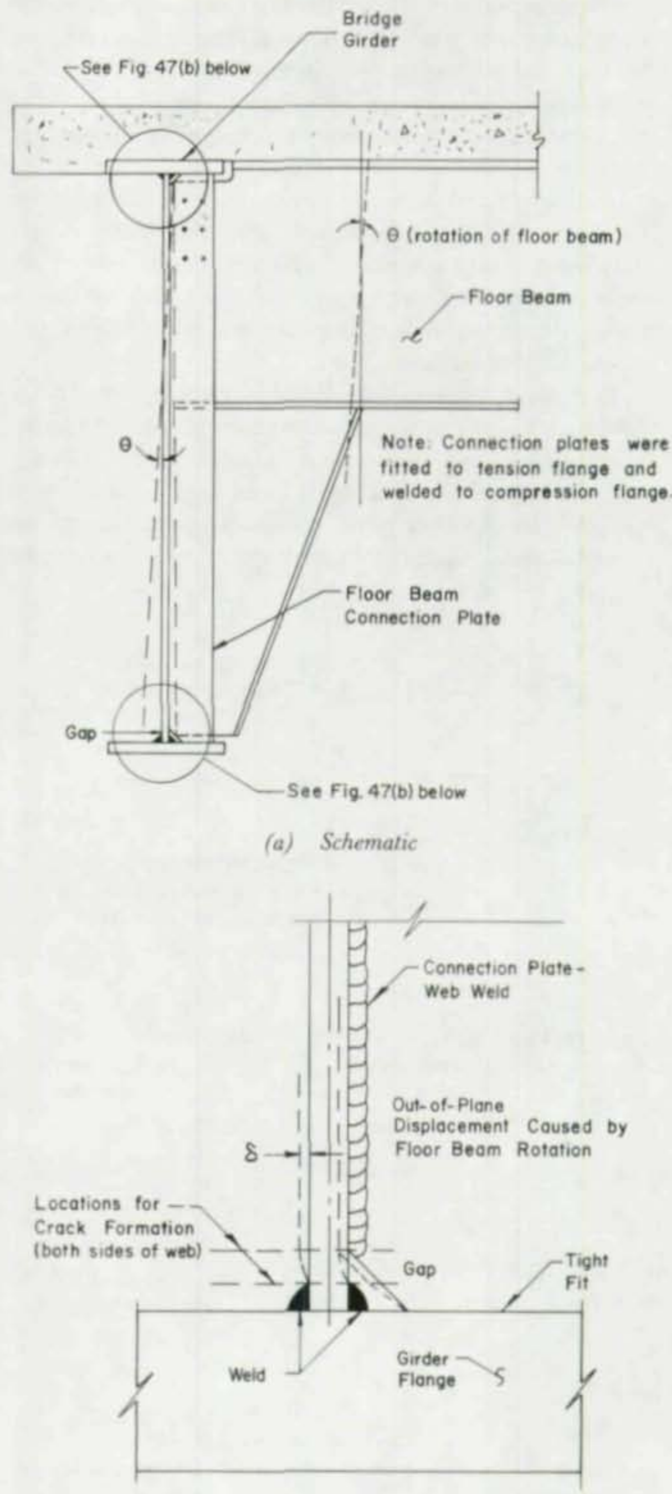


Fig. 47. Floor beam rotation at girder

moment regions that are not near the end reactions do not develop as much out-of-plane flexing of the web, or a relative displacement in the gap region, due to the ability of the entire girder to accommodate this rotation by deforming with the floor beam. Whether or not this region can become a problem depends on the lateral stiffness of the girder flange.

There have been several instances where the usual distance of $^1/_2$-in. to 2 in. between stiffener weld termination and the web-to-flange weld has resulted in very large web bending stresses and caused fatigue crack growth, as illustrated in Fig. 48. The problem with out-of-plane movement in the web gap can be particularly acute with skewed bridges. The floor beams are usually perpendicular to the main girders, which may cause even larger deformation to occur in the web gap region because of additional end rotations due to differential vertical movement of the girders at each end of the floor beam.

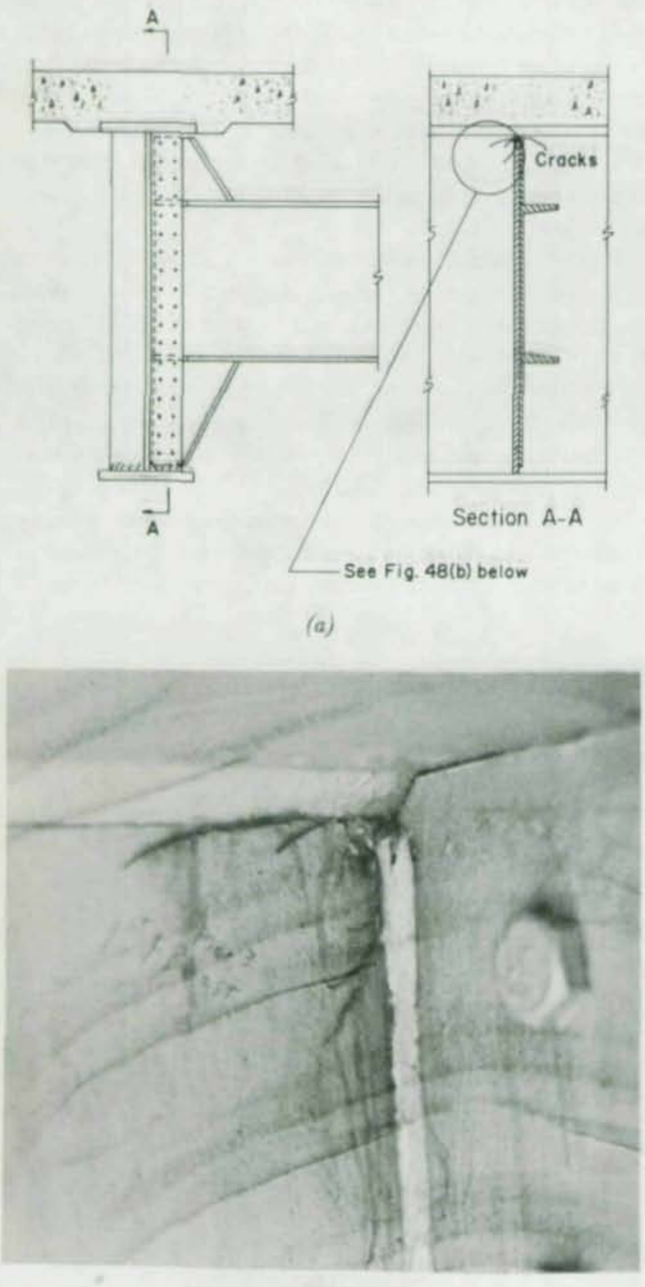


Fig. 48. Cracking in web gap at floor beam-to-girder connection plate in negative moment region where connection plates were not welded to tension flange

The severity of the out-of-plane movement can be seen from Eq. (12). Considering only the out-of-plane movement, Δ, and assuming the web gap and web thickness are $^1/_2$-in., an out-of-plane movement of only 0.0001 in. will result in a web bending stress of 18 ksi.

Sometimes, when a very small gap has been provided, the web cannot move appreciably out-of-plane relative to the flange. This can be beneficial, providing sufficient web-connection weld is available to resist the resulting forces. Where this is not the case, the weld cracks and the connection plate is peeled away from the web. Eventually this also results in web cracking as a gap develops when the plate is pulled away from the web.

Also, extending the stiffener welds all the way to the girder web-to-flange weld may create undesirable conditions because of weld shrinkage. With thick flanges, the restraint to contraction is great and the strain will tend to concentrate in the small gap. This results in high tensile residual stresses which exist prior to any out-of-plane bending stress. On occasion these conditions have resulted in above average initial discontinuities and cracking.

Transverse Stiffeners

Cyclic out-of-plane bending stresses can also occur during handling or shipping as a result of the relative rotation between the web and the flange. The extensiveness of crack formation depends on the girder size, how the girder is handled, what method of shipment is used, the trip length, the degree of cyclic swaying motion that develops while in transit (due to the manner in which the girders were supported and to roadbed conditions), and other factors that result in repeated cycles of out-of-plane web bending stress. Several instances of unsatisfactory performance and web cracking have occurred from these causes.

Figure 49 shows a schematic of a cracked web at the end of a cut-short stiffener. The cracks have formed at the weld toes

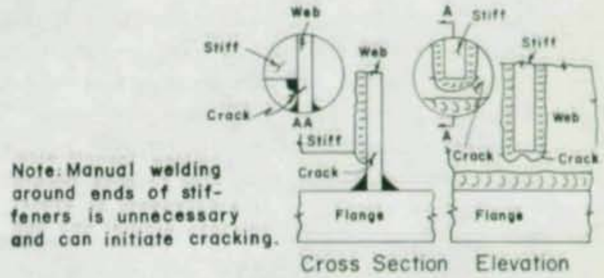


Fig. 49. Schematic of crack formation at end of transverse stiffener

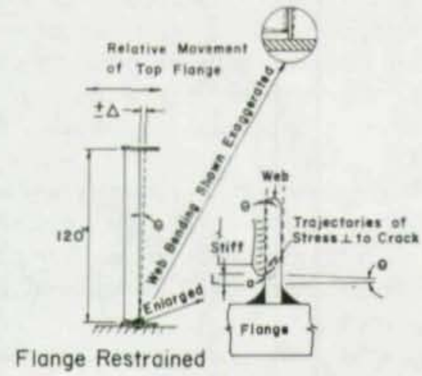


Fig. 50. Schematic showing deformation of web between end of stiffener and flange when lateral sway occurs

at the end of the stiffener. Figure 50 illustrates the type of deformation that will result from sway of the girders on rail cars or trucks or differential rocking of rail cars as they move through switchyards or over poorly aligned or superelevated track. This is especially true of long girders which extend over several car lengths, and can occur with girders that are well supported and secured. Certain handling procedures can also result in twisting of flanges. Near support points, twisting of the flange may occur during handling and shipping.

Once the girder is placed in the structure, the gap regions at vertical stiffeners are not susceptible to out-of-plane movements unless lateral bracing or diaphragms frame into the stiffener. Laboratory studies[33,34,35,36] have shown that pumping, due to out-of-straightness of the web in the middle of panels between transverse stiffeners and to the movement of thin webs under service loads, will not result in flange cracking when slender webs are designed according to the current specifications.

To minimize the possibility of web cracking due to shipping or handling, consideration should be given to terminating the stiffeners a greater distance away from the web-to-flange welds, in order to increase the web gap at the end of the stiffener. Between four and six times the web thickness should be used. This will not influence the service behavior of the girder.

In addition to the cut-short stiffener terminating four to six times the web thickness away from the web-to-flange weld, *provisions should be made to minimize the effects of temporary* loading conditions which could result in relative rotation between web and flange. This can be accomplished by driving steel wedges under stiffeners at support points, or by using short bearing stiffeners, i.e., stiffeners properly fitted to the flange and extending some distance up the web.

It should be noted that "tight fit" stiffeners, which allow a $^1/_{16}$-in. gap according to AWS, will permit approximately the same relative rotation to occur as will cut-short stiffeners. Hence, even with "tight fit" stiffeners web-to-stiffener welds should terminate four to six times the web thickness above the web-to-flange weld.

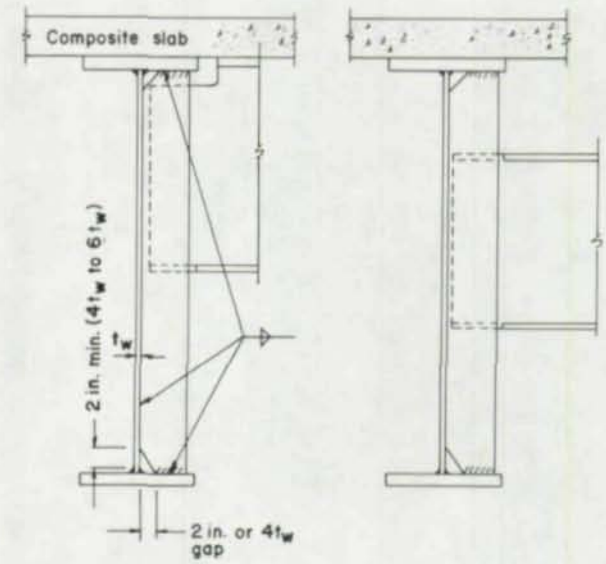
When one-sided stiffeners are used on very large girders, consideration should be given to the use of partial length shipping stiffeners at the support points. These can be attached to the web and fitted to the bottom flange. The objective of these stiffeners is to provide support during transportation.

Summary and Recommendations for Floor Beam Connection Plates and Transverse Stiffeners

One method of preventing or minimizing local displacement of the web, as shown in Figs. 47a and 47b, is to weld the connection plate to the flanges. Figures 51a and 51b illustrate this application. Obviously the stresses in the flange must be selected so as to avoid fatigue failure from the weld toe. The design stress range must satisfy the applicable stress range provided for Category C. The composite action of the floor beam and slab in highway bridges causes the neutral axis of the floor beam to be at or near the coped flange shown in Fig. 51a. Hence, no significant bending stresses develop in the cope region and the problem illustrated in Fig. 38 should not develop.

Bolting a connection angle to the web is another viable alternative. However, care should also be exercised to provide the proper web gap length; as Fig. 41 has demonstrated, cracking can develop from bolted connections as well. This is *particularly true of welded girders, where the web-flange fillet* provides the same condition as a welded connection plate. Without proper web gap length, cracking can occur with either bolted or welded connection plates.

Figure 52 shows a recommended floor-beam-to-girder connection for a through plate girder railroad bridge. The floor beam bracket connection plates and intermediate stiffeners



(a) Floorbeam acting composite with slab. (b) Floorbeam without composite action.

Fig. 51. Recommended floor beam connection plates for negative moment regions and near end supports

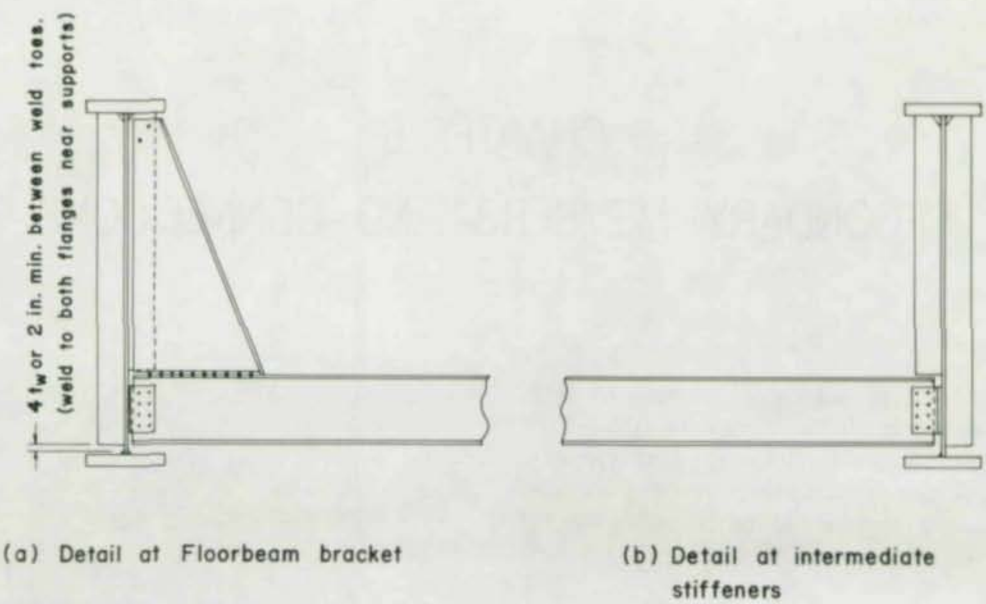


Fig. 52. Suggested floor-beam-to-girder connections

have a large gap (about 6 in.) between their ends and the floor beam end connections. The large gaps and the continuity of the exterior stiffeners insure small out-of-plane movements at the end of the floor beam connection plates and intermediate stiffeners. Near the supports, for a distance at least equal to the girder depth, the outside stiffeners should be welded to both top and bottom flanges. This will prevent any significant web bending stress from developing at the stiffener ends, as the flange is anchored and prevented from moving laterally.

SUGGESTED GENERAL PROCEDURE FOR DESIGN OF CONNECTIONS

1. After completion of the framing system, a systematic study should be made to discover all possible differential movements, both deflection and rotation. This requires a deliberate training in visualization of structure action.
2. Details of connections between members subjected to these differential movements should be examined carefully. It is axiomatic that accommodation of the movement will be made at the point of weakest resistance to bending. One of the following procedures should be considered:
   a. Detail the connections to provide a flexible point that can deflect without being overstressed, or
   b. Eliminate the flexible points and design the connections to withstand the forces that would occur if the connection were rigid.
3. After completion of details, study areas of high secondary stress for stress raisers, i.e., welds, copes, etc.
4. Consideration should be given to vibration of long slender bracing members and the connections detailed to avoid high cyclic stresses. (See Chapters 6 and 7.)

CHAPTER 6
SECONDARY MEMBERS AND CONNECTIONS

Diaphragms, lateral bracing, and other attachments are used extensively in bridges. Generally these members are considered to play minor or secondary roles in the structure's performance. Often, significant problems have developed because they were not properly considered in the structure's overall behavior.

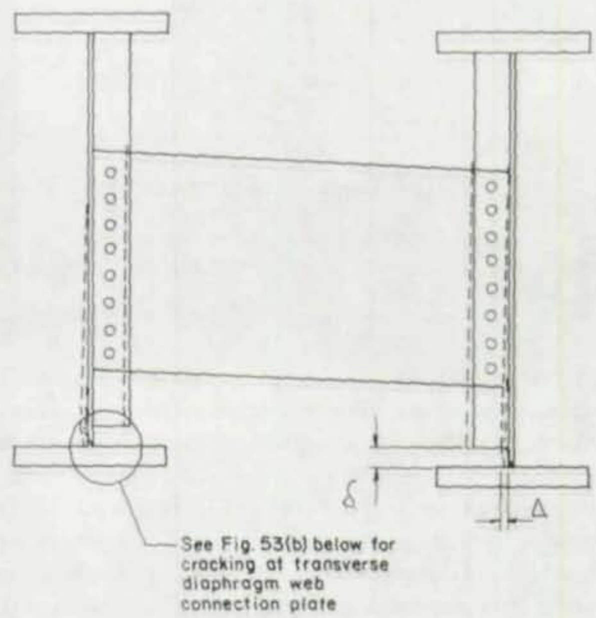
DISPLACEMENTS AT DIAPHRAGMS AND CROSS-FRAMES

Diaphragms and cross-frames are frequently used in multiple beam bridges to assist in erection and construction, as well as to distribute loads laterally in the structural system. In recent years it has been the practice to connect the diaphragms and cross-frames to the longitudinal members by means of a connection plate that is shop welded to the girder web and compression flange, and field bolted to the cross members. Adjacent to the tension flange, the connection plate is either fitted or cut short.

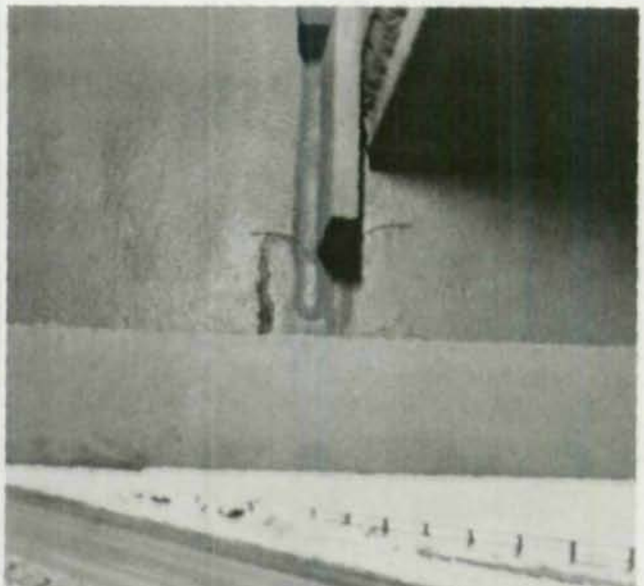
As the structure is loaded, the longitudinal girders may deform different amounts at the cross section where the diaphragms and cross-frames are installed. This can result in the secondary member displacing the girder webs out-of-plane, as shown schematically in Fig. 53a. The differential beam displacement, δ, can be relatively large in skewed bridges which have diaphragms or cross-frames perpendicular to the longitudinal members. Of course, the magnitude of the out-of-plane web displacement, Δ, is related to the relative girder displacement and the lateral bending resistance of the girder flange. Near supports where one member is deflected and the other is not, relatively large deformations can be introduced into the girder web.

Figure 53b shows the cracking that developed in the girder web at an intermediate diaphragm in a skewed railroad bridge. The mechanism of fatigue crack formation is similar to that at the ends of floor beams. The gap provided was 1 to 2 in. between the end of the connection plate weld and the web-flange welds. In many instances this type of cracking developed in only a few years, because the out-of-plane web bending stresses were very large, particularly in skewed bridges.

To some extent the problems which have developed at floor beam and diaphragm connection plates have arisen because transverse welding of attachments to tension flanges was prohibited. This requirement was intended to reduce the likelihood of fatigue crack growth in the flange. Recent research has shown that the restriction of transverse welding on



(a) Relative deformation



(b) Cracking at transverse diaphragm web connection plate (skewed bridge)

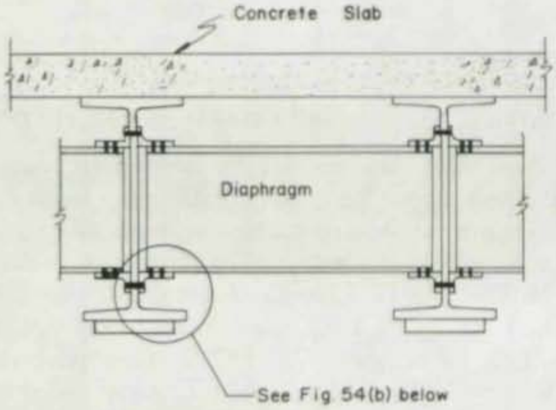
Fig. 53. Deformation in multi-beam bridge

the tension flanges is no longer required if the appropriate stress category is used.[10] This is true because the stress range at the web stiffener welds in close proximity to the flange provides a potential for crack growth in the web that is similar to the potential in the flange. Such web cracks quickly propagate into the flange.

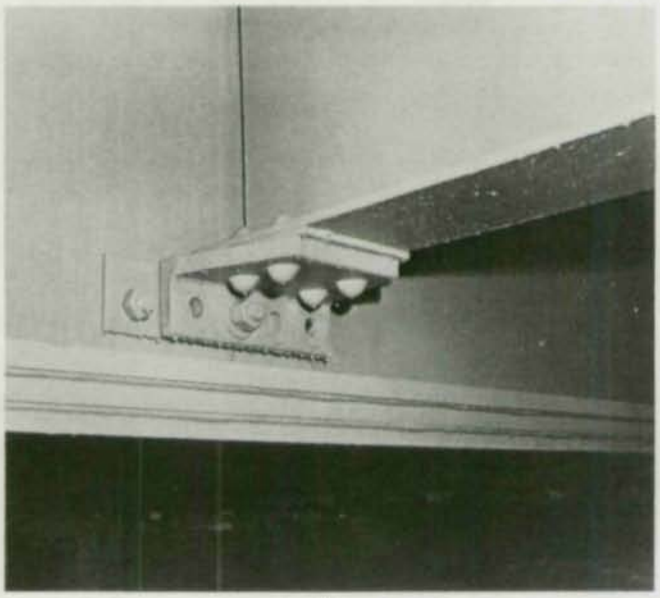
A related problem is the prying introduced in a bolted diaphragm between highway bridge stringers, as shown in Fig. 54. Here, top and bottom seat angles were used to connect the diaphragms to the stringers. The relative movement between adjacent stringers caused cyclic prying forces to develop on the bolts in the upstanding legs and eventually cracked the bolt head off. This demonstrates the need to minimize the prying action on high-strength bolts and to assure that they are properly tightened.

In addition to the potential out-of-plane displacements that may occur in the girder webs at diaphragms and cross-frames, consideration must also be given to the details within the cross-frame and its connection to the girders. Low fatigue strength details should be avoided, because often the stresses are not well defined in the cross-frames. Furthermore, measurements on both highway and railroad structures have shown that cross-frames and diaphragms usually experience a larger number of cycles of stresses from service loading than the main members.

Figure 55a shows a typical cross-frame in a single track railroad bridge. Gusset plates, groove welded to the transverse stiffeners, provide a Category E detail at the groove weld ends. Since the transverse stiffener is welded to the compression flange, the forces tending to cause out-of-plane flexure are not imposed upon the webs. However, the diaphragm force will be resisted by the beam action of the stiffener. The stresses are sufficiently large to have resulted in fatigue crack growth at the weld termination at the end of the gusset, as illustrated in Fig. 55b. This detail corresponds to a Category E design condition. Note that the fatigue crack penetrated to the hole. In this situation, most of the cracks formed at the upper end of the top gusset because a higher stress range condition existed

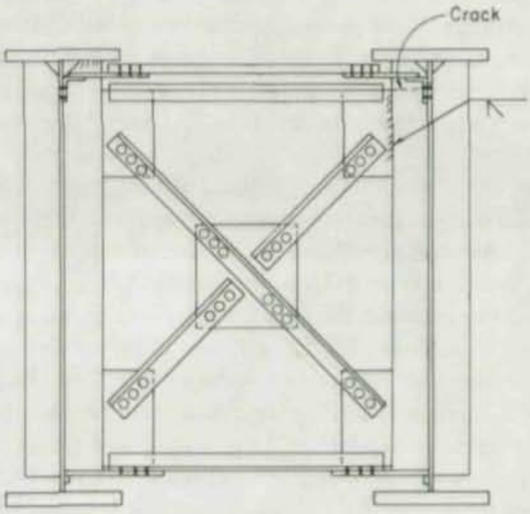


(a)

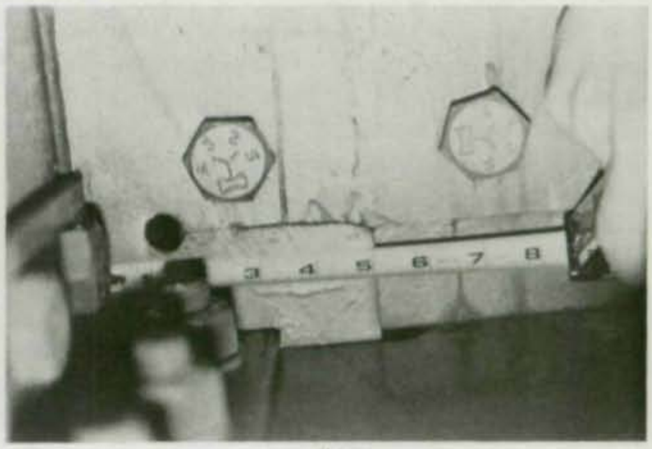


(b)

Fig. 54. Diaphragm with missing bolts in end connection



(a)



(b)

Fig. 55. Cross-frame detail which has developed fatigue crack growth

in this area. However, a few cracks also occurred at the bottom of the top gusset, where stresses are relatively high as well. No cracks were observed next to the tension flange.

SUMMARY AND RECOMMENDATIONS FOR CROSS-FRAMES AND DIAPHRAGMS

Cross-frames and diaphragms are subjected to the same frequency of stress cycles as the main members that they connect. Often the cross-frames are subjected to relatively large forces as a result of deformation of the ties or slab, the Vierendeel truss action of the cross section, or the shear developed in the cross-frame as a result of differential deformation between the main members.

Two factors should be considered when designing cross-frames and diaphragms. One is consideration of the out-of-plane movements that develop between the end of the cross-frame or diaphragm connection plates and the girder flanges. In non-skewed bridges, the transverse stiffeners and connection plates should be terminated at least four to six times the web thickness above the tension flange. Near supports, the connection plates should preferably be welded to both tension and compression flanges. Figure 56 shows schematically the transverse stiffener and connection plates for two typical right angle bridge cross sections (see Chapter 7 for skewed bridges).

The second major factor to consider is the probable forces that will occur in the cross-frame and connection plates under traffic. Because significant forces are developed in these components, it is desirable to provide high fatigue strength details. For example, the gusset plates could be bolted to the transverse stiffener, or the stiffener made larger, so the cross-frame could be attached as shown in Fig. 56. Both the bolted connection and the larger stiffener provide much higher fatigue strength details than the groove welded gusset. If the gusset is welded to the transverse stiffener, radiused ends can be used to improve the fatigue strength, as provided in Fig. 1.7.2, Example 14, of the AASHTO Specifications or in Fig. 1.3.13, Example 19, of the AREA Specifications.

When cross-frames are used in skewed or curved girder bridges, the connection plates should be welded to both flanges, so that web deformation is prevented in the space beyond the end of the stiffener (see Fig. 62).

The alternate recommendation is to soften the connection by increasing the web gap between the end of the connection plate and the flange. Where estimates of the floor beam end rotation or the relative girder deflection can be made, and the out-of-plane deformation Δ evaluated, Eq. (12) can be used to determine the required gap length. Such gaps will provide minimum restraint to the end connection; thus, blocking the floor beam flange is not as critical.

If the connection is softened to permit the web to "breathe", it should be realized that the connection plate may lose its effectiveness as a web stiffener. Also, it should be noted that a full depth stiffener should not be attached on the opposite side of the web.

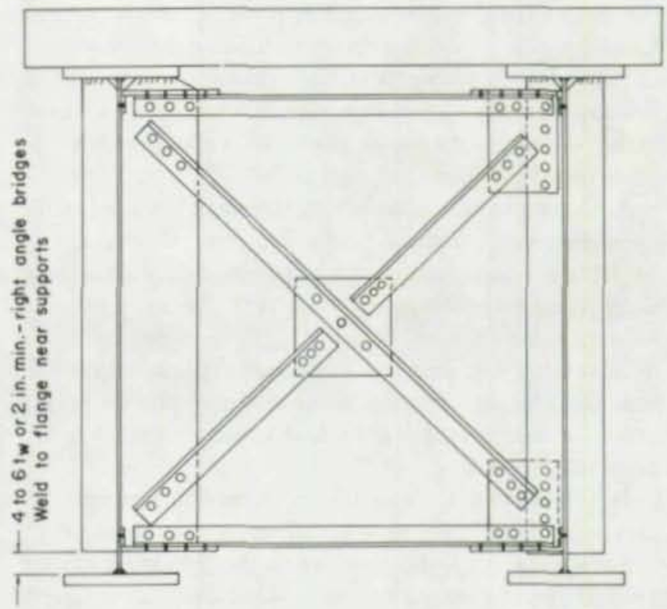


Fig. 56. Cross-frame details with improved fatigue strength for straight right-angle bridges

LATERAL BRACING AND LATERAL GUSSET PLATES

The lower lateral bracing in bridge structures is primarily used to resist lateral forces due to wind or live loading and lateral movement. An upper lateral system is used in some structures to provide lateral stability to compression flanges as well. The AASHTO Specifications do not require lateral bracing in spans up to 125 ft long, with concrete slab or other floor of equivalent rigidity (Art. 1.7.17). When spans exceed 125 ft, Art. 1.7.17 indicates a system of lateral bracing must be provided near the bottom flange. The AREA Specifications (Art. 1.11.2) require bottom lateral bracing in all spans except deck spans less than 50 ft long. Revisions relaxing this requirement under certain design conditions are being processed for adoption in 1978.

Unfortunately, lateral bracing systems usually require that lateral gussets be attached to the web or flange. Such details have low fatigue resistance and are comparable to Category E details.

It is generally accepted that concrete deck highway bridges do not require bottom lateral bracing in spans up to 125 ft. However, some engineers question the need for lateral bracing in spans up to 175 ft. In Ontario, the current practice is to use lateral bracing only in highway spans that exceed 150 ft. Furthermore, it is not clear that a continuous lateral system is always needed. Perhaps only selected portions of longer span structures require lateral bracing when the concrete slab or a top lateral system can be used to provide lateral rigidity.

Depending on the type of lateral system and diaphragms used, the lateral bracing may interact with the main load-carrying members under live load. Interaction results in a reduction in the stresses in the main girders, and the lateral bracing system is subjected to the same stress cycle frequency as the girders to which it is attached. When the lateral system interacts with the main girders, the live load stresses must be included in the investigation of the lateral bracing connections for fatigue performance, using the same criteria as applied to the girders. Often the lateral bracing system has been designed for wind loading only. The cycle frequency and permissible stress range may not impose as much restriction on the connection design.

A lateral system that does not interact with main load-carrying members under traffic will not be as sensitive to the connections used at the ends of the lateral members. However, the attachments to the main members must obviously be considered.

Out-of-Plane Movement at Lateral Gusset Plates

Even when a lateral bracing system is provided that minimizes interaction with the main load-carrying members under traffic, small movements will occur at the gusset connection. These result from the deflected shape of the structure and can be relatively large when differential vertical deformation occurs. In addition, most lateral bracing systems are built with members that are not symmetrical sections. For example, angles and structural tees are commonly used. Hence, when axial forces are developed in the lateral bracing system, a rotation occurs at the gusset because the neutral axis of the member and gusset do not coincide. (The full eccentricity is not effective because of the flexibility of the members.) Relative deformation of the longitudinal members may also cause out-of-plane movement of the gusset. This is particularly true at gussets near supports where one end of the lateral bracing member is prevented from moving vertically, whereas its other end displaces with the girder. Relatively short stiff lateral bracing members will cause the effects of this movement to concentrate in the gusset, at the gap regions g_2 and g_3 in Fig. 57. These two factors—eccentricity of load and relative deformations—can cause deformation of the gusset, as is shown schematically in Fig. 57a.

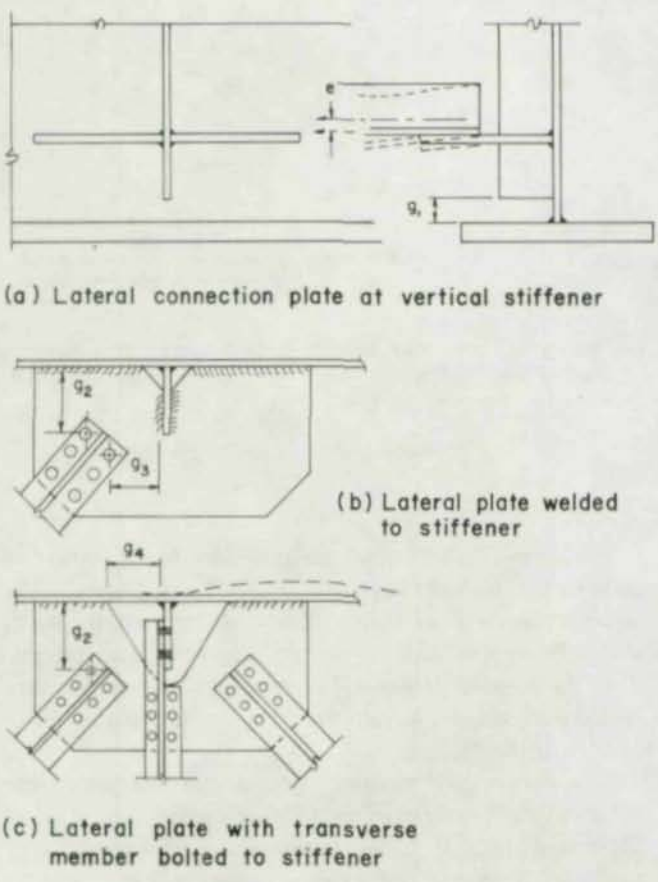


Fig. 57. Schematic showing possible out-of-plane displacement in web and lateral gusset at transverse stiffeners

Another factor may be construction tolerances. The force lines from the lateral system may not exactly intersect at the web center line and this can cause the gusset to twist at the web, as shown in Figs. 57b and 57c. The degree of deformation depends on the cross-frame connection as well.

When the gusset plate is welded to the girder web and transverse stiffener as shown in Fig. 57b, or the transverse member is bolted to the transverse stiffeners, out-of-plane movements can become critical at the end of the transverse stiffener, in the gap g_1. In addition, the significance of vertical movement at the gusset will be dependent on the gap lengths g_2 and g_3. Very short distances are undesirable, as they will result in large bending stresses in the gusset at the weld lines and at the end of the lateral members.

If the gusset is not connected to the transverse stiffener as shown in Fig. 57c, part of the out-of-plane movement will be accommodated by the web in gap g_4. If the gap is very small, very large web bending stresses can be introduced into the gap region. Hence, a reasonable gap length is desirable. Slightly larger out-of-plane movements may develop with larger gaps, because not as much of the transverse-stiffened web is brought into play. When the transverse bracing member frames into the gusset plate and/or stiffener, as shown in Fig. 57c, negligible out-of-plane movement develops in the web space g_4. When only diagonal members frame into the gusset plate, out-of-plane movements are likely to be greater, as the force lines may not intersect. Only gaps g_2 and g_3 are sensitive to vertical movement of the gusset plate.

Figure 58 shows a crack that developed at gussets that were not welded to the transverse stiffener. The lateral gusset plates were welded only to the web on each side of a transverse stiffener. Here it is also clear that the web gap, g_4, between the end of the longitudinal gusset plates and the transverse stiffener, is very small. Hence, any out-of-plane deformation at the web as the structure deforms will result in very high web bending strains. In this example, cracks developed after only a few years of service.

The gusset connection shown in Fig. 58 also created a severe restraint problem at the intersection of the longitudinal and transverse stiffener welds. These intersecting welds should, in general, be avoided wherever possible, because high weld shrinkage strains will occur in the web and because the possibility for larger than usual discontinuities exists at the weld intersections.

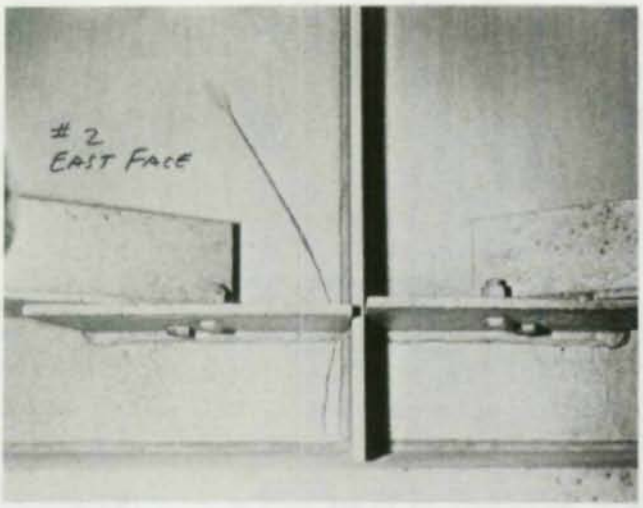


Fig. 58. Cracked web at end of lateral gusset plate (note separate plates are not connected to stiffener)

Fig. 59. Typical lateral bracing gusset without transverse stiffener plate

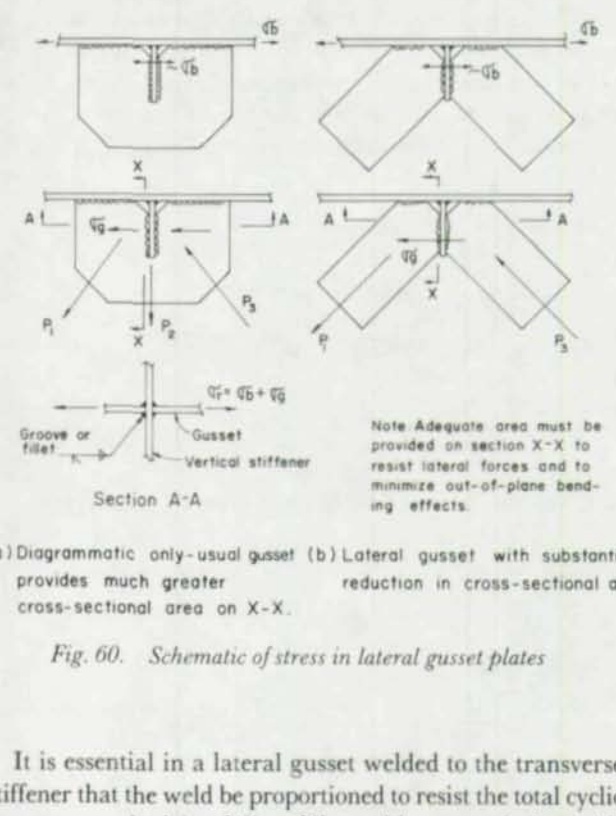
Generally, when gussets are attached to the web in regions without transverse stiffeners, as shown in Fig. 59, there are no difficulties with out-of-plane web movements. The gusset plate is located 6 to 12 in. above the flange. This provides enough flexibility in the web so that large out-of-plane bending strains do not develop in the gap between the gusset and the flange.

The out-of-plane movements discussed in this section will be more severe in curved girder systems, where the diaphragms and lateral bracing system are required to transfer greater forces. Every effort should be made to minimize the out-of-plane movement in gap regions of curved girder bridges.

Stresses in Lateral Gusset Plates

Serious cracking of the web and flange sections of a plate girder, which originated in the connection of a bottom lateral gusset plate,[37] has focused attention on the need to adequately evaluate the fatigue resistance of various components of such connections. When lateral gusset plates several feet long are attached to a girder web or flange, compatibility requires that the gusset plate experience the same level of stress that the girder experiences, even in the absence of forces in the lateral bracing system. When forces develop in the lateral system, stresses due to bracing forces will add algebraically to the stress in the gusset from bending of the girder, as shown schematically in Fig. 60.

Care should be exercised in decreasing the cross-sectional area available to resist the lateral forces. For example, small gusset plates attached to each side of the transverse stiffener, as shown in Fig. 60b, may not provide adequate cross-sectional area at the gusset-to-transverse-stiffener connection. Hence, relatively high stresses can result from the girder bending stresses and the forces P_1 and P_3. This type of connection is also more sensitive to the out-of-plane motion of the lateral gusset plate.



(a) Diagrammatic only - usual gusset provides much greater cross-sectional area on X-X. (b) Lateral gusset with substantial reduction in cross-sectional area.

Fig. 60. Schematic of stress in lateral gusset plates

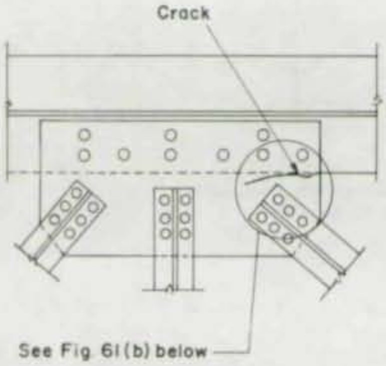
It is essential in a lateral gusset welded to the transverse stiffener that the weld be proportioned to resist the total cyclic stress across the joint. When fillet welds are used, the welds should be sized to limit stresses to Category F requirements. Since the gusset-stiffener weld is perpendicular to the primary bending stress field, any groove weld should be subjected to nondestructive inspection.

It is always good practice to provide an adequate clear length at the web-transverse stiffener-gusset connection as illustrated in Fig. 62. A gap region will provide redundancy in the detail. Intersecting welds permit a crack to propagate into the web from the transverse stiffener-gusset connection. Providing the gap prevents this possibility from developing.

Vibration of Lateral Bracing

Relatively flexible lateral bracing is often provided. The lateral system may vibrate when the structure is loaded by moving vehicles. These vibrations are near the natural frequency of the system. Generally, such vibrations create large numbers of negligible stress cycles in the bracing.

On occasion the vibration of the lateral bracing members can create difficulties at the member ends. The vertical movement of the member as it vibrates causes out-of-plane movement in the gusset plates, whether attached to the web or flange. At flange connections, a more rigid connection often results, as the torsional stiffness is larger. The consequences of the out-of-plane movement of the gusset can be seen in Fig. 61. It is apparent that the distance between the edge of the flange and the bolted diagonal member is relatively small, i.e., 2 or 3 in. This same type of cracking has been observed in gusset plates welded to the flange or web. At the restraint line along the web or flange, cyclic stresses are created in the gusset by the vertical vibration of the laterals and the resulting rotation at the end of the lateral. A very small gap can lead to high out-of-plane cyclic bending stresses, even with modest amounts of vibration. In addition, large numbers of stress cycles can accumulate in relatively short time intervals.



(a)

(b)

Fig. 61. Cracked riveted-bolted gusset caused by lateral bracing vibration

RECOMMENDED DETAILS AT LATERAL GUSSET PLATES

When significant forces are developed in the transverse diaphragms and lateral bracing during passage of vehicles, it is necessary to minimize out-of-plane movement in the web gap regions. If large forces or deformations are anticipated, the gusset plate should be welded and/or bolted to both web and vertical stiffener, as shown in Fig. 62. Adequate copes need to be provided in the gusset-stiffener corners, so that no welds intersect. A cope distance of four to six times the web thickness or 2 in., whichever is larger, will provide sufficient web gap to prevent adverse effects from weld shrinkage and restraint.

In determining the gusset gap distances g_2 and g_3 (see Fig. 57), consideration should be given to the possible out-of-plane movements that can occur in the connection plates from eccentricity of load, relative vertical displacements between the ends of the lateral member (particularly near supports), and vibration of the lateral system. Near supports, the gaps may need to be larger than the 4 in. shown in Fig. 62.

Very few experimental or theoretical studies are available on the gusset behavior. In one structure, gaps less than 2 in. resulted in relatively large out-of-plane bending stresses in the gusset. Experience suggests a distance of 2 to 4 in. is satisfactory.

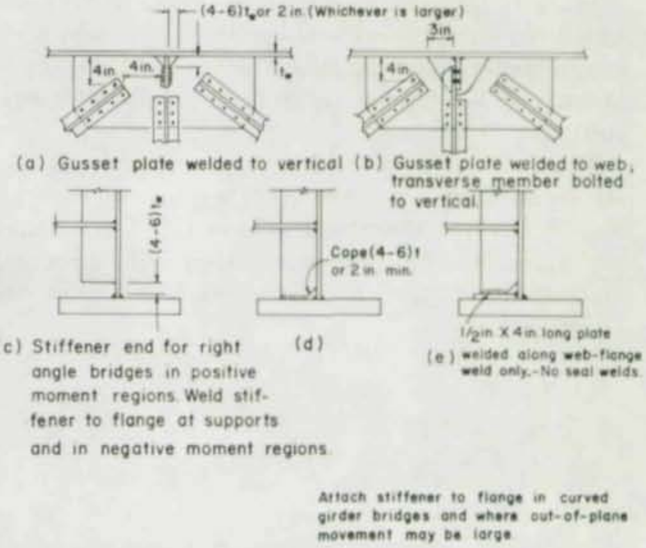


Fig. 62. Suggested lateral gusset connections at transverse stiffeners

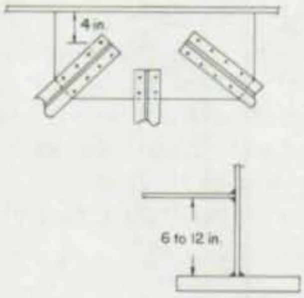


Fig. 63. Suggested lateral gusset plate at points with no transverse stiffener

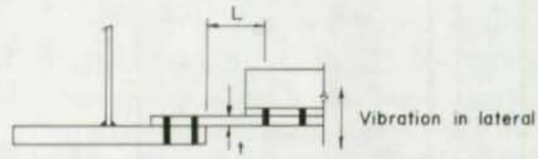
When the gusset is not attached to the transverse stiffener, a gap distance g_4 equal to 3 in. is satisfactory, provided the transverse lateral member is connected to the transverse stiffener and the gusset (see Fig. 62b). This type of detail should not be used when only diagonal members frame into the gusset, unless reasonable estimates of the displacements can be made and the gap length g_4 satisfies Eq. (12).

If the gusset is attached to the transverse stiffener, the end gap g_1, Fig. 57a, must be evaluated as discussed earlier under "Summary and Recommendations for Diaphragms and Cross-Frames" and illustrated in Figs. 56 and 62. When the lateral bracing and cross-frames transmit significant out-of-plane forces into the girder, as is the case in curved girders, the stiffener should be welded or bolted to the flange. This can be accomplished by welding the stiffener directly to the tension flange, as shown in Fig. 62d, or by using a secondary end plate, as illustrated in Fig. 62e. The stiffener can be welded to the plate and the plate in turn only welded along the flange-web weld. This detail has a very favorable orientation when a significant stress gradient exists across the flange, even though its length places it in Category D. Another possibility is to bolt the gusset to the flange. By resisting the lateral forces at the flange, out-of-plane web distortion can be minimized.

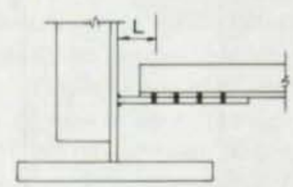
When the lateral plate can be attached to the web at points with no vertical stiffeners, as shown in Fig. 63, many of the gap problems can be minimized. This type of connection can be used if no significant forces are being transmitted into the girder.

The problem of vibration of the lateral bracing system must also receive careful consideration. There are two factors to consider in solving this type of problem. One is to minimize the magnitude of the out-of-plane movement by decreasing the flexibility of the lateral bracing members. This may be accomplished by increasing the member size or providing an intermediate support. The intermediate support (hanger) forces the member to vibrate in a higher mode and decreases the end movement. The other is to minimize the out-of-plane bending stresses at the gusset gap region by making it as flexible as possible. This is illustrated in Fig. 64. The larger the gusset gap, L, and the thinner the plate thickness, t, the smaller will be the stresses produced by the vibration movement.

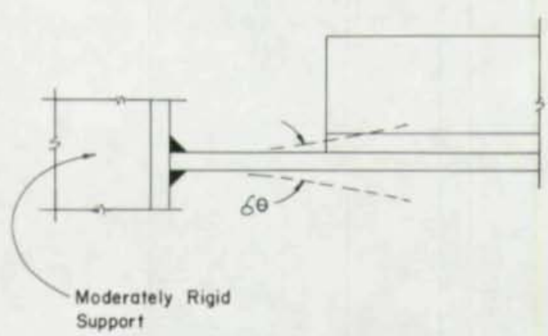
Often the fatigue strength of lateral bracing details is low (i.e., Category E) and may influence the main girder design. Also, the lateral bracing system provides a potential for fatigue damage unless properly designed. Lateral bracing should only be used where necessary. Bracing that is required for erection alone should be removed, so that a major source of secondary stress and potential fatigue problems can be eliminated.



(a) Bolted connection



(b) Welded gusset



(c) Distortion in the lateral gusset gap

Fig. 64. Schematic of vibration displacement at lateral gussets

CHAPTER 7
MISCELLANEOUS FATIGUE CONSIDERATIONS

In the earlier chapters of this booklet, consideration was given to the more commonly used bridge connections. This included their behavior when transmitting primary forces and the secondary effects created because components of a bridge seldom act independently of each other. In this chapter, several miscellaneous fatigue conditions are discussed. Most are indirectly related to several of the cases discussed previously. Some are secondary stress conditions, and others deal with the unusual features of skewed and orthotropic bridges.

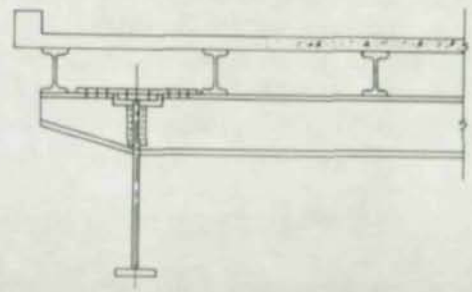
DISPLACEMENT INDUCED SECONDARY STRESSES IN OTHER STRUCTURAL SYSTEMS

Several miscellaneous cases of displacement-induced secondary stresses are reviewed in this section. These include the behavior at cantilever floor beam brackets, truss joints, and hangers in trusses.

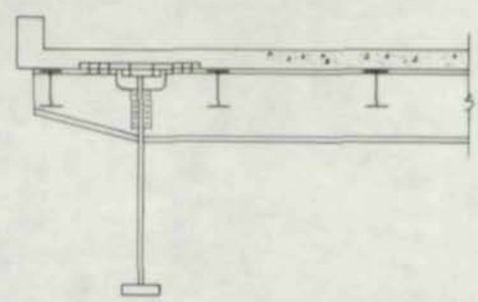
Out-of-Plane Bending in Connection Plates of Cantilever Floor Beam Brackets

Girder bridges are sometimes constructed with floor beams that cantilever beyond the outside of the longitudinal girders, as shown schematically in Fig. 65. The stringer system can either be placed on top of the floor beams, as in Fig. 65a, or framed into them, as in Fig. 65b. The perpendicular intersection of the floor beams and main girders makes each member susceptible to the deformation of the other. Since the members are perpendicular to each other, this results in out-of-plane deformations and higher secondary bending stresses. The floor beam is subjected to this condition when the girder deflects (top flange shortens or lengthens) under live load. The stiff slab will not change length and there will be a relative movement between the slab and the girder. Between expansion joints in the slab, the movement will produce a lateral deflection of the floor beam, as shown schematically in Fig. 66. This is especially critical when the stringers are placed on top of the floor beam, providing a very rigid connection.

A number of bridge structures have experienced fatigue cracking as a result of the type of deformation shown schematically in Fig. 66.[38,39] This has only occurred when the stringers were placed on top of the floor beams. Measurements of the structural response have confirmed that the changes in curvature of the main longitudinal girders under traffic are responsible for very large in-plane bending stresses in the tie plate connecting the cantilever bracket to the floor beam.[38] These displacement-induced stresses are often very high in magnitude and can result in visible fatigue damage in a short period of time.



(a) Girder bridge with stringers on top of floor beam



(b) Girder bridge with stringers framing into floor beam and connection plate embedded in slab

Fig. 65. Typical floor beam—girder bridge sections

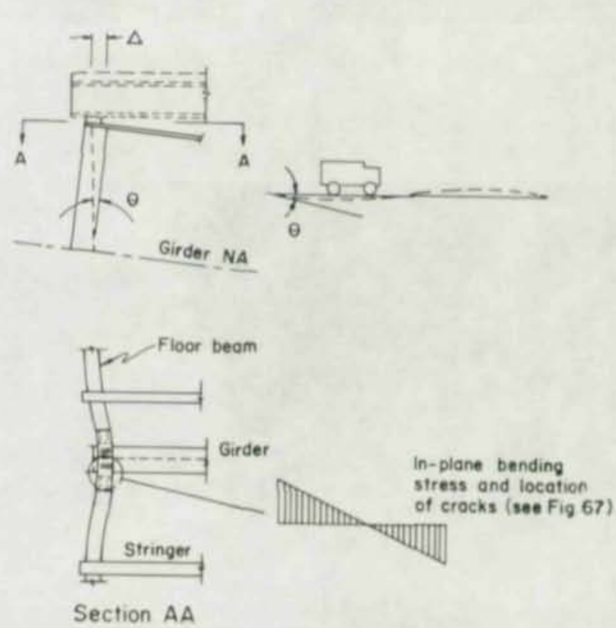


Fig. 66. Schematic of deformation introduced into floor beam-cantilever bracket splice plate (see Fig. 65)

Figure 67 shows a fatigue crack in a floor beam top flange splice plate, at the girder edge. In this case it is also apparent that the crack has grown from a tack weld, which was made to hold the connection plate to the bracket flange while rivets were placed. This, of course, is an undesirable condition; tack welds should always be avoided in regions where high cyclic stress will occur. The AWS specification requires that tack welds be properly removed or incorporated in the permanent weld. Designs should point out areas where no tack welding is permitted. In the example shown, tack welding was not permitted, but did in fact occur. This type of cracking has also developed in plates where no tack welds existed. The measurements reported in Ref. 38 indicated that the stress amplification would result in eventual cracking at bolted and welded connections.

Fig. 67. Fatigue crack in cantilever bracket connection plate

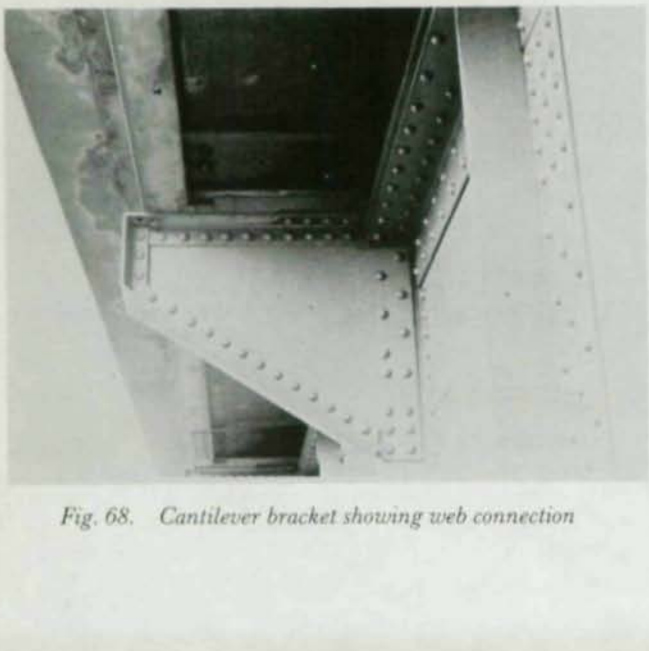


Fig. 68. Cantilever bracket showing web connection

Recommended Details for Cantilever Bracket Connection Plates

One way to avoid the type of cracking that has occurred in the flange splice plates of cantilever brackets is to embed the floor beam and girder flanges in the concrete slab. Measurements on a number of bridges have demonstrated that no in-plane bending stresses develop in the connection plates because the slab is infinitely stiff in its own plane.[39] Hence, the distortion shown schematically in Fig. 66 cannot develop.

When stringers are placed on top of the floor beams as shown in Fig. 65a, the flange splice plates for the floor beam-cantilever should not be attached to the main girders.[39] Releasing this connection will permit the structure to deform without introducing undesirable restraint into the connection plate. Field measurements have demonstrated that releasing the girder connection is the most effective way to minimize the large in-plane bending stresses. Although this provides a more flexible connection, the bracket web connection to the girder must have adequate flexibility to permit the displacement without developing web cracking in the bracket and girder. As Fig. 68 illustrates, the web connection must provide a large enough gap between its connection to the girder stiffener and the flange of the cantilever bracket. Generally, coping the bracket flange to clear the main girder flange will provide adequate "breathing" room.

Secondary Bending Stresses in Truss Joints

Secondary stresses can also be detrimental in truss joints if weld ends are located at points where cyclic stresses occur. This is illustrated schematically in Fig. 69. A gusset at an end panel point had a support shoe welded to the gusset plate. Although the lower chord is a "zero" stressed member at that point, secondary stresses were developed by bending in the gusset plate. Since the support shoe was welded to the edge of the gusset plate, the weld end was located near the stressed region. Crack growth occurred in the gusset under the cyclic secondary bending stresses.

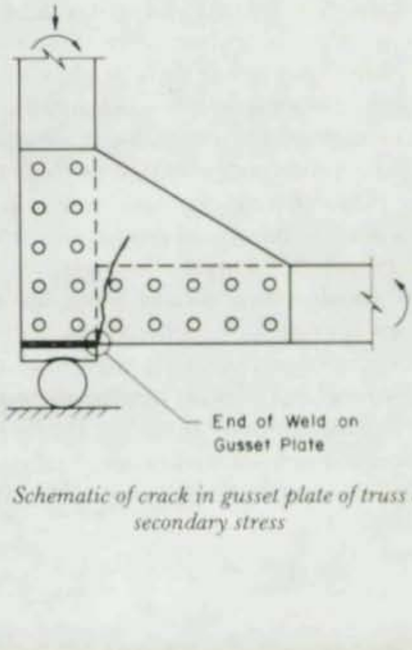


Fig. 69. Schematic of crack in gusset plate of truss caused by secondary stress

In this case, the problem could be avoided by connecting the support shoe to the end post only, rather than partly welding it to the gusset plate edge. Care should be exercised in truss joints to insure that weld ends do not occur at locations of high secondary stresses. This is also true for tack welds which may be used to temporarily hold bolted components in alignment prior to final bolting, as was shown in Fig. 67 for the cantilever bracket.

Floor-beam-to-hanger connections in railroad bridges are particularly sensitive to secondary stress effects.[40] The hanger is both bent and twisted by the floor beam as it deforms. The degree of bending and twisting also depends on the stringer connections to the floor beam. Actual stress measurements have indicated that the stresses in the hanger can be estimated with reasonable accuracy if a three-dimensional model is used and rigid joints assumed even at pinned connections.[41]

The AREA Specifications (Art. 1.3.16) require that for truss web members the calculated stress, when increased by one-third, must meet the fatigue requirements. Some structural systems may result in even higher increases in the hanger stress range as a result of the bending and twisting. Every effort should be made to estimate these effects.

SKEWED BRIDGES

As noted when discussing the out-of-plane deformation at floor-beam-to-girder connections in Chapter 5, the out-of-plane movement can be more severe with skewed bridges. In a skewed bridge, the floor beam not only experiences end rotation as a result of curvature, but also has an increase in end rotation because of the differences in the vertical movement at the ends of the floor beams. Any skewed bridge structure using framing members perpendicular to the main girder will experience some relative movement between the ends of the transverse members.

This means that skewed bridges are much more sensitive to cracking than right-angle bridges, because the out-of-plane movement at the connections are greater. This is illustrated in Figs. 48 and 53b, where examples of web cracking in two skewed bridges are shown.

The design recommendations provided for right-angle bridges will not, in general, be adequate for skewed bridge structures. For example, the vertical connection plates at cross-frames and diaphragms will usually require that the ends of the connection plate be welded to the girder flanges. Furthermore, the magnitude of force in the cross-frame members will be larger, and hence more attention will need to be given to the cross-frame member connections.

Figure 70 shows a suggested diaphragm-girder connection for a deck plate girder railroad bridge. It is preferable to weld the vertical stiffeners and connection plates to the girder tension flange, rather than attempt to provide large gaps between the end of the connection plate and the girder flange. Depending on the bridge configuration, impractical large gaps would be required to accommodate the differential movement of adjacent girders.

Connections between members in skewed bridges, especially those connecting members at other than right angles, need special attention. This is particularly true if "simple" end connections are planned. Figure 71a shows a floor-beam-to-girder connection that experienced cracking in the floor beam web. This probably resulted from restraint as well as out-of-plane distortion. After the flanges were spliced (see Fig. 71b) so that both web bending from the primary forces and out-of-plane distortion stresses were minimized or eliminated, satisfactory behavior was obtained.

At many skewed bridge connections it appears undesirable to use simple shear connections. These connections are sen-

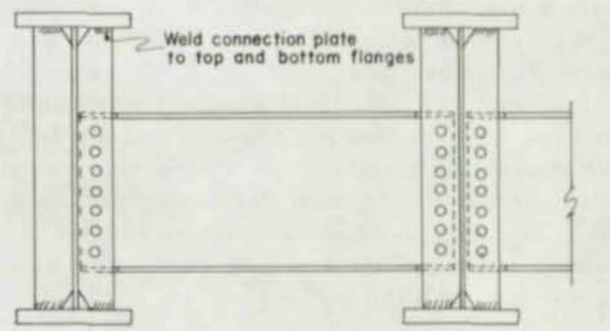
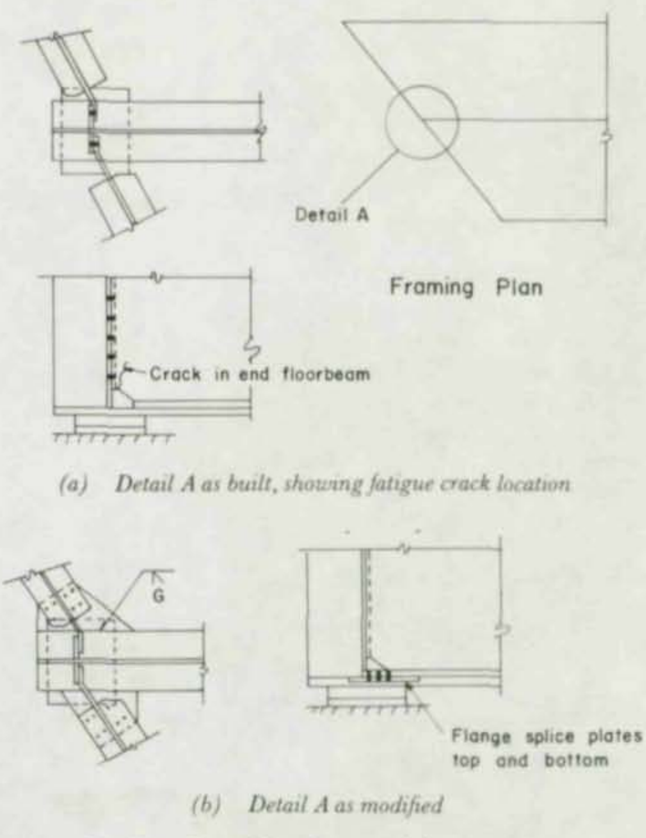


Fig. 70. Diaphragms and connection plates for skewed bridges



(a) Detail A as built, showing fatigue crack location

(b) Detail A as modified

Fig. 71. Skewed floor beam to girder connection

sitive to out-of-plane movement. Many times it is necessary to provide bends in the web or connecting angles that will be even more sensitive to both out-of-plane movement and restraint. Providing a moment resistant connection, as illustrated in Fig. 71b, will minimize the potential problems at such connections. Where cracking has developed at skewed connections, providing restraint to the flange has stopped the cracking and provided a serviceable joint.

If floor beams are framed into a skewed end floor beam and simple end connections are used, the connections must not be sensitive to restraint and out-of-plane distortion. Figure 72 shows one possibility for providing such a connection. The connection plates attached to the end floor beam should be welded to both flanges and the web, so that no out-of-plane movement is introduced into the web. Providing double splice plates on each side of the beam web also increases its resistance to out-of-plane movement.

Experience with skewed steel structures has demonstrated that they are much more prone to significant out-of-plane distortion than right-angle bridges. This must be considered when selecting connections for all components of a skewed bridge, including floor-beam-to-girder and end floor beam connections and secondary members such as cross-frames, diaphragms, and lateral bracing.

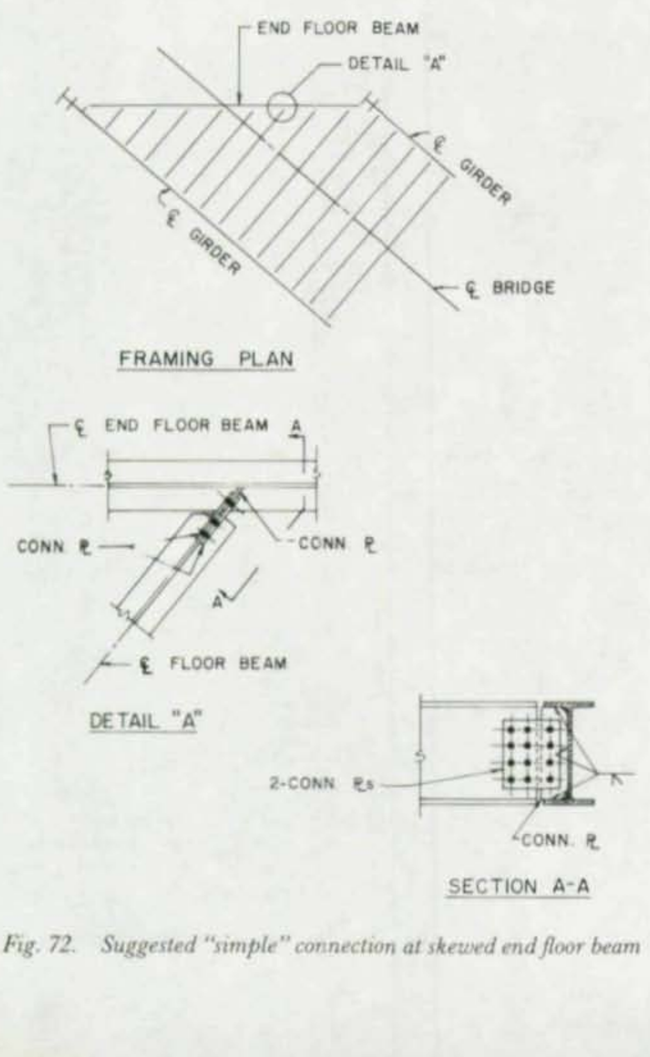


Fig. 72. Suggested "simple" connection at skewed end floor beam

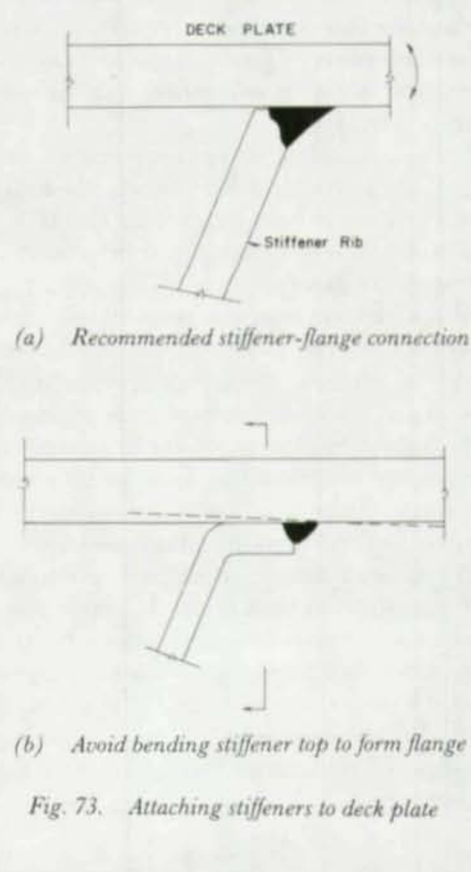
ORTHOTROPIC STEEL DECKS

Care must be exercised in the design of orthotropic steel bridge decks because the frequency of loading and the proportion of stress due to live load is very high. Many details will be subjected to more than one stress cycle per truck, as each axle (or wheel) may cause a stress cycle.

Obviously, low fatigue strength details should be avoided. For example, splice plates which fall into Category E would decrease the fatigue resistance of deck stiffeners substantially. Measurements on an orthotropic bridge deck have indicated that cracking is probable after 20 to 40×10^6 variable stress cycles[42] with Category E details.

Work in England[43] on experimental orthotropic bridge panels and the field studies reported in Ref. 44 have demonstrated that high cyclic stresses occur in the deck transverse to the stiffeners. When stiffeners are attached to the flange plate, the longitudinal welds are often partial-penetration welds, and available studies indicate that Category C is probably applicable, considering the stress range at the weld root.[42,43] In any event, the stiffener should be welded directly to the plate, as illustrated in Fig. 73a. The weld throat should provide at least 80% of the stiffener plate thickness, in order to minimize the notch effect at the weld root.

One should avoid the use of a flanged stiffener as shown in Fig. 73b. This results in a much more severe stress condition at the weld root as the deck plate deforms. The stiffener flange provides a more severe "effective crack length" at the weld root and this enhances the fatigue crack propagation.



(a) Recommended stiffener-flange connection

(b) Avoid bending stiffener top to form flange

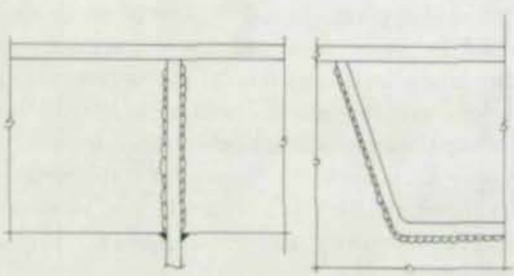
Fig. 73. Attaching stiffeners to deck plate

Another important connection in orthotropic steel decks is between the stiffener and the floor beam. The stiffener should not be interrupted at the web of the floor beam by a welded joint placed at the end of the stiffener. It is preferable for the stiffener to run continuously through an opening in the floor beam without a break. This also permits the stress range to be more effectively controlled, as the critical point is the weld toe termination at the end of the weld attaching the stiffener to the web. By reducing the weld length, a significant reduction in the bending stresses in the trapezoidal stiffener will occur. Figure 74b shows the preferred means of connecting the stiffener to the floor beam. This connection also permits the floor beam web to flex more readily.

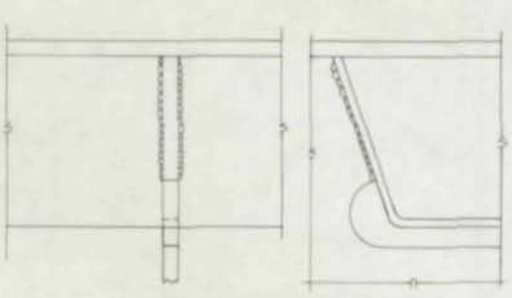
Experience on an experimental orthotropic deck panel, which had trapezoidal stiffening ribs framing into the floor beams without a cutout (Fig. 74a), demonstrated that premature fatigue was possible in the load carrying welds which connected the stiffener to the floor beam web.[43,44]

Splices of the orthotropic deck panels can be made by high-strength bolts and welding. Sometimes a combination of these methods is used, such as welding the deck plate and bolting the ribs and floor beams.[45,46] Under the local wheel load response, the welded joint is near the neutral axis of the deck and is subjected to lower cyclic stress.

Figure 75a shows the field deck splices used for the Port Mann Bridge.[45] High-strength riveted joints provided a good fatigue strength detail. High-strength bolted joints would provide an even better splice. A combination joint is shown in Fig. 75b. As noted above, it places the groove welded splice near the neutral axis for local bending of the orthotropic deck. Hence, a good fatigue strength detail is provided for both deck plate and stiffening ribs.



(a) Welded joint between stiffener and floor beam



(b) Stiffener continuous through cutout in floor beam web is preferable

Fig. 74. Stiffener to floor beam connections

WELD REQUIREMENTS AND SPECIAL CONSIDERATIONS

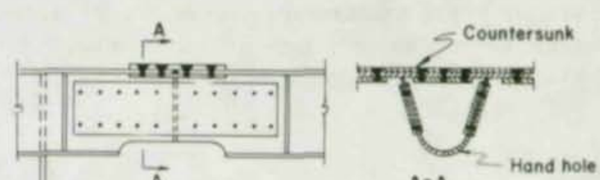
Groove Welded Splices

It has been common practice for many years to require the nondestructive inspection of groove welded flange splices. Both the AASHTO and AREA specifications have required testing of groove welds in main members (see AASHTO *Standard Specifications for Welding of Structural Steel Highway Bridges* (1975) and Art. 3.5.5 of the AREA Specifications).

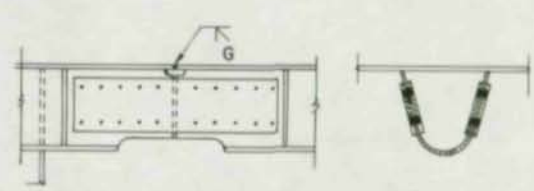
The same degree of quality has not been imposed on many groove welded components that are attached to bridge girders. Several bridge girders have experienced fatigue crack initiation and growth because the groove welds used to splice longitudinal stiffeners had incomplete penetration.[47] Since the longitudinal stiffener was considered an attachment, no weld quality criteria were established and no nondestructive test requirements were imposed on the longitudinal stiffener welds, even though the welded splice was perpendicular to the primary bending stress. Several of these splices existed along the girder length, and in several instances they existed in positive moment regions for architectural purposes. The large flaws permitted crack growth to occur at very low levels of calculated stress range (about 1 ksi and above) making fracture of the girder inevitable. A more detailed discussion of one of the structures is given in Ref. 47.

This incident illustrates that welded butt splices in secondary material subjected to main member stresses must be made in accordance with the requirements for splices in the main material.

A related type of behavior was observed in groove welds that connected lateral gusset plates to transverse stiffeners.[37] In this detail a backup bar was used to make a groove weld perpendicular to the bending stress field. Lack of fusion in this transverse groove weld resulted in fatigue crack growth in those areas.



(a) Deck splice (Port Mann Bridge)



(b) Combination splice with good fatigue resistance

Fig. 75. Deck panel splices

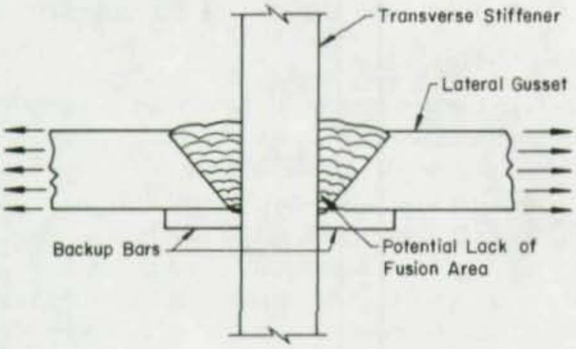


Fig. 76. Schematic of lack-of-fusion area when backup bar is perpendicular to stress field

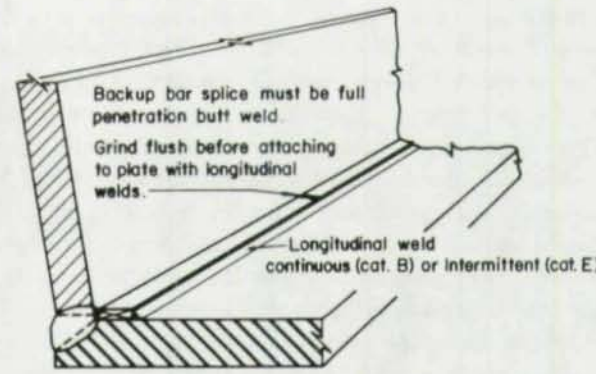


Fig. 77. Schematic showing butt splice in backing bar

The history of crack initiation and growth in the lateral gusset plate, and the propagation to failure of the girder to which it was attached, also illustrate a related problem to which designers should be continuously alert. The fact that the gusset plate transverse groove weld was detailed to come into contact with the girder web provided a path into the girder which eventually destroyed the cross section. This can be prevented by providing interruptions in potential crack paths when attachments are connected to main girders. For example, adequate cope holes between the transverse groove weld and the girder web, as shown in the gusset plate in Fig. 57, would prevent a fatigue crack in the transverse welded gusset plate from growing into the girder web.

This failure also demonstrated that it is undesirable to leave backup bars in place when they are perpendicular to the applied stress field. As illustrated schematically in Fig. 76, this can result in a crack-like defect perpendicular to the stress field. No nondestructive testing process can consistently detect lack of fusion. Any lack of fusion in the weld makes the condition very critical.

Transition Radii

When flanges of different widths have been spliced, it has been the practice to use a transition radius or straight tapered transition. Criteria for the transition radius are given in Art. 1.7.15 of the AASHTO Specifications.

The 1977 AASHTO and AREA specifications permit groove welded and fillet welded attachments with various transition radii. Large radii permit a more favorable stress category. (See Fig. 1.7.2, Example 14, AASHTO Specifications, and Fig. 1.3.13, Example 19, AREA Specifications.)

Care must be exercised when a transition, whether radiused or straight tapered, is used to improve fatigue strength. The specifications require that the weld end be ground smooth. Every effort must be made to insure a smooth transition, with no rejectable weld imperfections. Recent tests have indicated that rejectable weld discontinuities, such as slag inclusion or large gas pockets, must not be apparent in the ground transition radius. Care should also be taken with the root pass of longitudinal groove welds which connect radiused details to a flange. These can also lead to crack initiation sites if a smooth weld is not made.

When permanent backup bars are essentially parallel to the direction of applied stress, they must be continuous. Although backup bars are not usually counted upon to resist the loads, they are in fact subjected to the same stress range as the stressed elements of the section. Therefore, a welded butt splice in the backup bar must be a full-penetration butt weld which is made prior to attaching the backup bar to the member elements as illustrated in Fig. 77. Any lack of fusion that is parallel to the direction of applied stress has no effect on the fatigue resistance.

APPENDIX A
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APPENDIX B
AASHTO AND AREA FATIGUE SPECIFICATIONS

Article 1.7.2 of the 1977 AASHTO Specifications, and Arts. 1.3.13 and 2.3.1 of the 1977 AREA Specifications (with Commentary) are reproduced in full in this Appendix. They contain the following major changes:

(a) Full use of the live load and impact stress range concept, instead of the maximum allowable stress based on stress ratio, R, and tensile strength of steel.

(b) Material subjected to fluctuating compression stresses was exempted from fatigue requirements.

(c) Use of six basic design stress range categories in Table 1.7.2A1 of the AASHTO Specifications and Table 1.3.13B of the AREA Specifications.

(d) Rearrangement of Table 1.7.2B, Stress Cycles, of the AASHTO Specifications to provide for structures subjected to extreme truck traffic volume and for separate transverse member requirements. Development of Table 1.3.13A of the AREA Specifications to account for stress cycles depending on span length, type of member, and frequency of trains.

(e) Inclusion of Tables 1.7.2A2 and Fig. 1.7.2 in the AASHTO Specifications and Table 1.3.13C and Fig. 1.3.13 in the AREA Specifications to illustrate the scope and applicability of each category and detail classification.

AASHTO SPECIFICATIONS—BRIDGES (1977)
ARTICLE 1.7.2 REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

(A) Allowable Fatigue Stress

Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Articles 1.7.1 and 1.7.4.1(B) and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 1.7.2A1 for the appropriate type and location of material shown in Table 1.7.2A2 and illustrated in Fig. 1.7.2.

Main load carrying member components subjected to tensile stresses which may be considered nonredundant load path members—that is, where failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 1.7.2A1 for Nonredundant Load Path Structures.

The 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.

In Table 1.7.2A2, "T" signifies range in tensile stress only; "Rev." signifies a range of stress involving both tension and compression during a stress cycle.

(B) Load Cycles

The number of cycles of maximum stress range to be considered in the design shall be selected from Table 1.7.2B unless traffic and loadometer surveys or other considerations indicate otherwise.

Allowable fatigue stresses shall apply to those Group Loadings that include live load or wind load.

The number of cycles of stress range to be considered for wind loads in combination with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

(C) Charpy V-Notch Impact Requirements

Main load carrying member components subjected to tensile stress require supplemental impact properties as described in the Material Specifications.

These impact requirements vary depending on the type of steel, type of construction, welded or mechanically fastened, and the average minimum service temperature to which the structure may be subjected.[1] Table 1.7.2C contains the temperature zone designations.

Components requiring mandatory impact properties shall be designated on the drawings and the appropriate zone shall be designated in the contract documents.

A514 steel shall be supplied to Zone 2 requirements as a minimum.

Main load carrying member components subjected to tensile stresses which may be considered nonredundant—that is, where failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 1.7.2A1 for nonredundant structures.

[1] *The basis and philosophy used to develop these requirements are given in a paper entitled "The Development of AASHTO Fracture-Toughness Requirements For Bridge Steels" by John M. Barsom, February 1975, available from the American Iron and Steel Institute, Washington, D.C.*

Table 1.7.2A1

Category (See Table 1.7.2A2)	For 100,000 Cycles		For 500,000 Cycles		For 2,000,000 Cycles		For over 2,000,000 Cycles	
	ksi	MPa	ksi	MPa	ksi	MPa	ksi	MPa
REDUNDANT LOAD PATH STRUCTURES[1] — Allowable Range of Stress, F_{sr}								
A	60	413.69	36	248.21	24	165.47	24	165.47
B	45	310.26	27.5	189.60	18	124.10	16	110.31
C	32	220.63	19	131.00	13	89.63	10 12*	68.95 82.74*
D	27	186.16	16	110.31	10	68.95	7	48.26
E	21	144.79	12.5	86.18	8	55.15	5	34.47
F	15	103.42	12	82.74	9	62.05	8	55.15
NONREDUNDANT LOAD PATH STRUCTURES[2]								
A	36	248.21	24	165.47	24	165.47	24	165.47
B	27.5	189.60	18	124.10	16	110.31	16	110.31
C	19	131.00	13	89.63	10 12*	68.95 82.74*	9 11*	62.05 75.84*
D	16	110.31	10	68.95	7	48.26	5	34.47
E	12.5	86.18	8	55.15	5	34.47	2.5	17.24
F	12	82.74	9	62.05	8	55.15	7	48.26

* For transverse stiffener welds on girder webs or flanges.

[1]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 have redundant load paths.

[2]Structure types with a single load path where a single fracture can lead to a catastrophic collapse. For example, flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, caps at single or two column bents have nonredundant load paths.

Table 1.7.2A2

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example No. (See Fig. 1.7.2)
Plain material	Base metal with rolled or cleaned surfaces. Flame cut edges with ANSI smoothness of 1,000 or less	T or Rev.	A	1, 2
Built-up members	Base metal and weld metal in members without attachments, built-up of 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	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

(cont'd next page)

Table 1.7.2A2 (cont'd)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example No. (See Fig. 1.7.2)
Groove welds (cont'd)	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 steeper 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. (50.8 mm) and 12 times the plate thickness, but less than 4 in. (101.6 mm)	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 in. (101.6 mm)	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. (0.610 m) and weld end ground smooth	T or Rev.	B	14
	(*b*) When provided with transition radius less than 24 in (0.610 m) but not less than 6 in (0.152 m) and weld end ground smooth	T or Rev.	C	14
	(*c*) When provided with transition radius less than 6 in. (0.152 m) but not less than 2 in. (0.051 m) and weld end ground smooth	T or Rev.	D	14
	(*d*) When provided with transition radius between 0 in. and 2 in. (0 and 0.051 m)	T or Rev.	E	14
Fillet 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. (50.8 mm) 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. (50.8 mm) and 12 times the plate thickness but less than 4 in. (101.6 mm)	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. (101.6 mm)	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 24 in. (0.610 m) and weld end ground smooth	T or Rev.	B	14

(*cont'd next page*)

Table 1.7.2A2 (cont'd)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example No. (See Fig. 1.7.2)
Fillet welded connections (cont'd)	(*b*) When provided with tansition radius less than 24 in. (0.610 m) but not less than 6 in. (0.152 m) and weld end ground smooth	T or Rev.	C	14
	(*c*) When provided with transition radius less than 6 in. (0.152 m) but not less than 2 in. (0.051 m) and weld end ground smooth	T or Rev.	D	14
	(*d*) When provided with transition radius between 0 in. and 2 in. (0 and 0.051 m)	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

Table 1.7.2B—Stress Cycles

Main (Longitudinal) Load Carrying Members				
Type of Road	Case	(*ADTT*)*	Truck Loading	Lane Loading†
Freeways, expressways, major highways and streets	I	2,500 or more	2,000,000**	500,000
	II	Less than 2500	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*)*	Truck Loading	Lane Loading
Freeways, expressways, major highways and streets	I	2,500 or more	Over 2,000,000	–
	II	Less than 2,500	2,000,000	–
Other highways and streets	III	–	500,000	–

*Average daily truck traffic (one direction).

**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 1.3.1(B) for one traffic lane loading.

†Longitudinal members should also be checked for truck loading.

Table 1.7.2C

Minimum Service Temperature	Temperature Zone Designation
0°F and above or (−18°C and above)	1
−1°F to −30°F or (−19°C to −34°C)	2
−31°F to −60°F or (−35°C to −51°C)	3

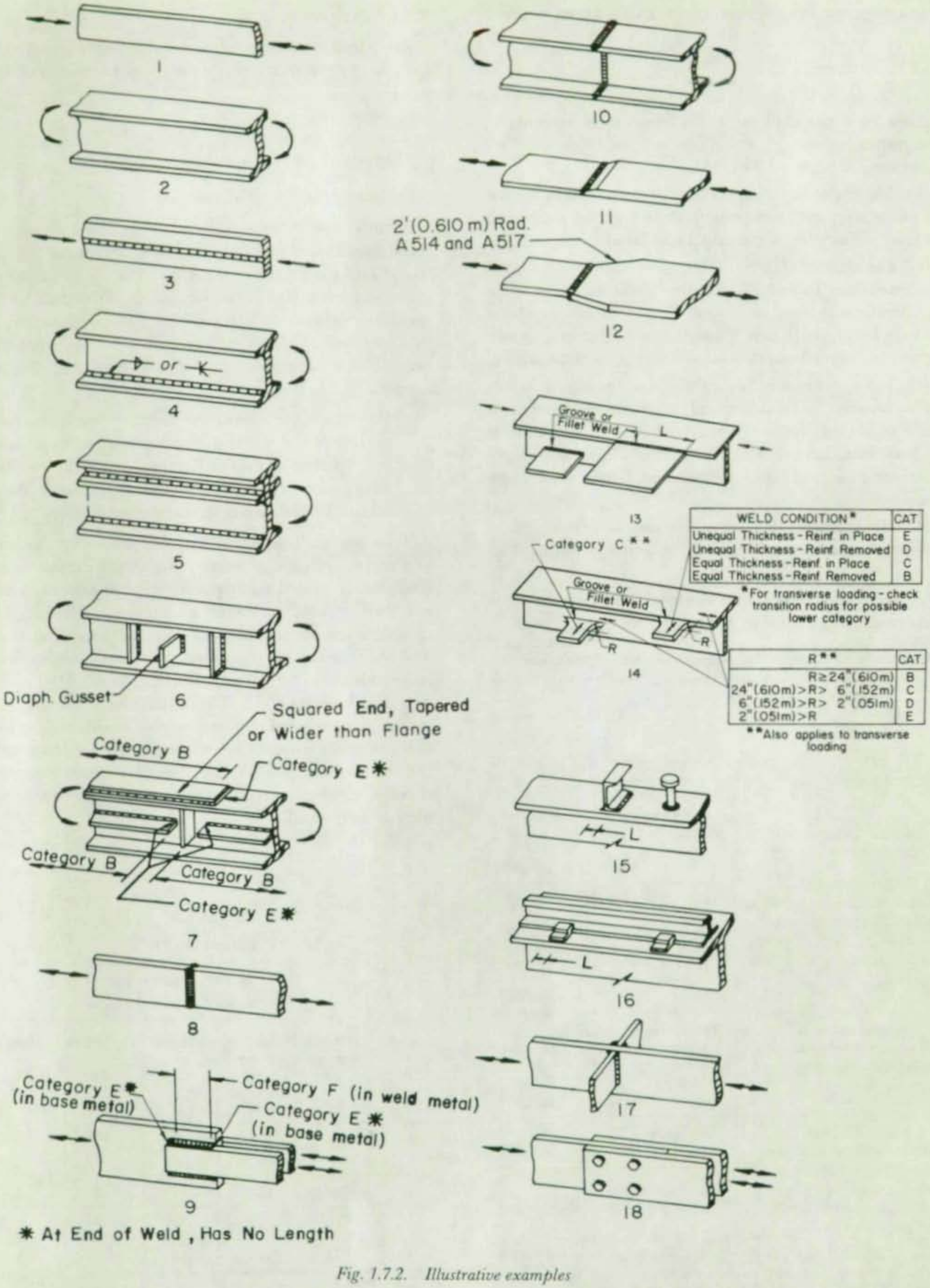

Fig. 1.7.2. Illustrative examples

AREA SPECIFICATIONS FOR STEEL RAILWAY BRIDGES* (1977)

1.3.13 Fatigue

(a) Members and connections subjected to repeated fluctuations of stress shall meet the fatigue requirements of paragraphs (c), (d), (e), and (f), as well as the strength requirements of Section 1.4 or 2.4.

(b) The major factors governing fatigue strength are the number of stress cycles, the magnitude of the stress range, and the type and location of constructional detail.

(c) The number of stress cycles, N, to be considered shall be selected from Table 1.3.13A, unless traffic surveys or other considerations indicate otherwise. The selection depends on the span length in the case of longitudinal members, and on the number of tracks in the case of floorbeams and hangers.

(d) The stress range, S_R, is defined as the algebraic difference between the maximum and minimum calculated stress due to dead load, live load, impact, and centrifugal force. If live load, impact, and centrifugal force result in compressive stresses and the dead load is compression, fatigue need not be considered.

(e) The type and location of the various constructional details are categorized in Table 1.3.13C and illustrated in Fig. 1.3.13.

(f) The stress range shall not exceed the allowable fatigue stress range, S_{Rfat}, listed in Table 1.3.13B.

* *Taken from Chapter 15, Steel Structures, of the American Railway Engineering Association Manual for Railway Engineering (Fixed Properties) of 1976.*

Table 1.3.13A

Member Description	Span Length, L, of Flexural Member or Truss	Constant-Stress Cycles, N
Classification I Longitudinal flexural members and their connections; or truss chord members, including end posts, and their connections	$L > 100'$	150,000
	$100' \geq L > 75'$	200,000
	$75' \geq L > 50'$	500,000
	$50' \geq L > 30'$	2,000,000
	$30' \geq L$	> 2,000,000
Classification II Truss web members and their connections except as listed in Classification III	Two tracks loaded	200,000
	One track loaded	500,000
Classification III Floorbeams and their connections; or truss hangers and sub-diagonals, which carry floorbeam reactions only, and their connections	Two tracks loaded	500,000
	One track loaded	> 2,000,000

Note: Tables 1.3.13A and B are based on bridges designed for E80 loading. For the procedure to be used for a design loading other than E80, see the Commentary, Art. 9.1.3.13, Step 5.

2.3.1 Fatigue

(a) Members and connections subjected to repeated fluctuations of stress shall meet the fatigue requirements of Article 1.3.13.

AREA COMMENTARY

9.1.3.13 and 9.2.3.1 Fatigue

Members subjected to repeated applications of load under certain conditions will fail at a lower unit stress than they would under a single application of load. Such failures are commonly referred to as fatigue failures. All editions of these specifications between 1910 and 1969, inclusive, have required that members subject to reversal of stress (whether axial, bending or shearing) during the passage of the live load be proportioned as follows:

> Determine the maximum stress of one sign and the maximum stress of the opposite sign and increase each by 50 percent of the smaller; proportion the member to satisfy each stress so increased; and proportion the connection for the sum of the maximum stresses.

Tests on small- and medium-size laboratory specimens and tests on full-size structures have shown that under some conditions, repeated loadings will reduce the life of members and their connections even if all stresses are tensile. Thus, reversal of stress is not necessary to cause failures from fatigue. The *Specifications for Welded Highway and Railway Bridges* (now titled the *Structural Welding Code*, D1.1) of the American Welding Society (AWS) has always recognized this fact and has included requirements for modifying the allowable design unit stresses for certain types of welded members and their connections. Tests[6,7] have also shown that riveted or bolted members and connections are similarly affected when there is no reversal.

Table 1.3.13B

Stress Category	Allowable Fatigue Stress Range, S_{Rfat} (ksi) for No. of Constant-Stress Cycles, N				
	150,000	200,000	500,000	2,000,000	> 2,000,000
A	53	48	36	24	24
B	40	36	27	18	16
C	28	26	19	13	10 12[a]
D	24	22	16	10	7
E	19	17	12	8	5
F	14	13	12	9	8

[a] For transverse stiffener welds on webs or flanges.

Table 1.3.13C

General Condition	Situation	Kind of Stress Range	Stress Category (See Table 1.3.13B)	Illustrative Example No. (See Fig. 1.3.13)
Plain material	Base metal with rolled or cleaned surfaces. Flame cut edges with ANSI smoothness of 1000 or less	T or Rev.	A	1, 2
Built-up members	Base metal and weld metal in members without attachments, built up of 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	T or Rev.	E	7
Groove welds	Base metal and weld metal at transverse full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush, and weld soundness verified by NDI	T or Rev.	B	8, 10
	Base metal and weld metal in or adjacent to transverse full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of applied stress, and weld soundness verified by NDI	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	T or Rev.	C	8, 10, 11, 12
	Base metal at details attached by groove welds subject to transverse and/or 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, 14
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading when the detail length *L* is greater than 12 times the plate thickness or greater than 4 in.	T or Rev.	E	13, 14
	Base metal at ends of details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length:			
	(a) When provided with 24 in. or more transition radius and weld end ground smooth	T or Rev.	B	19
	(b) When provided with 6 in. or more transition radius and weld end ground smooth	T or Rev.	C	19
	(c) When provided with 2 in. or more transition radius and weld end ground smooth	T or Rev.	D	19
Fillet welded connections	Base metal at intermittent fillet welds parallel to direction of stress	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	15, 16, 17
	Base metal adjacent to fillet welded attachments (or details) with length *L* in direction of stress between 2 in. and 12 times the plate thickness but less than 4 in.	T or Rev.	D	14, 15, 16
	Base metal adjacent to fillet welded attachments (or details) with length *L* in direction of stress greater than 12 times the plate thickness or greater than 4 in.	T or Rev.	E	14, 16
	Base metal at details attached by fillet welds regardless of length in direction of stress:			
	(a) When provided with 24 in. or more transition radius and weld end ground smooth	T or Rev.	B	19
	(b) When provided with 6 in. or greater transition radius and weld end ground smooth	T or Rev.	C	19
	(c) When provided with 2 in. or greater transition radius and weld end ground smooth	T or Rev.	D	19

(cont'd next page)

Table 1.3.13C (cont'd)

General Condition	Situation	Kind of Stress Range	Stress Category (See Table 1.3.13B)	Illustrative Example No. (See Fig. 1.3.13)
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 riveted connections or bolted connections not covered above	T or Rev.	D	18
Fillet welds	Shear stress on throat of fillet welds	Shear	F	9

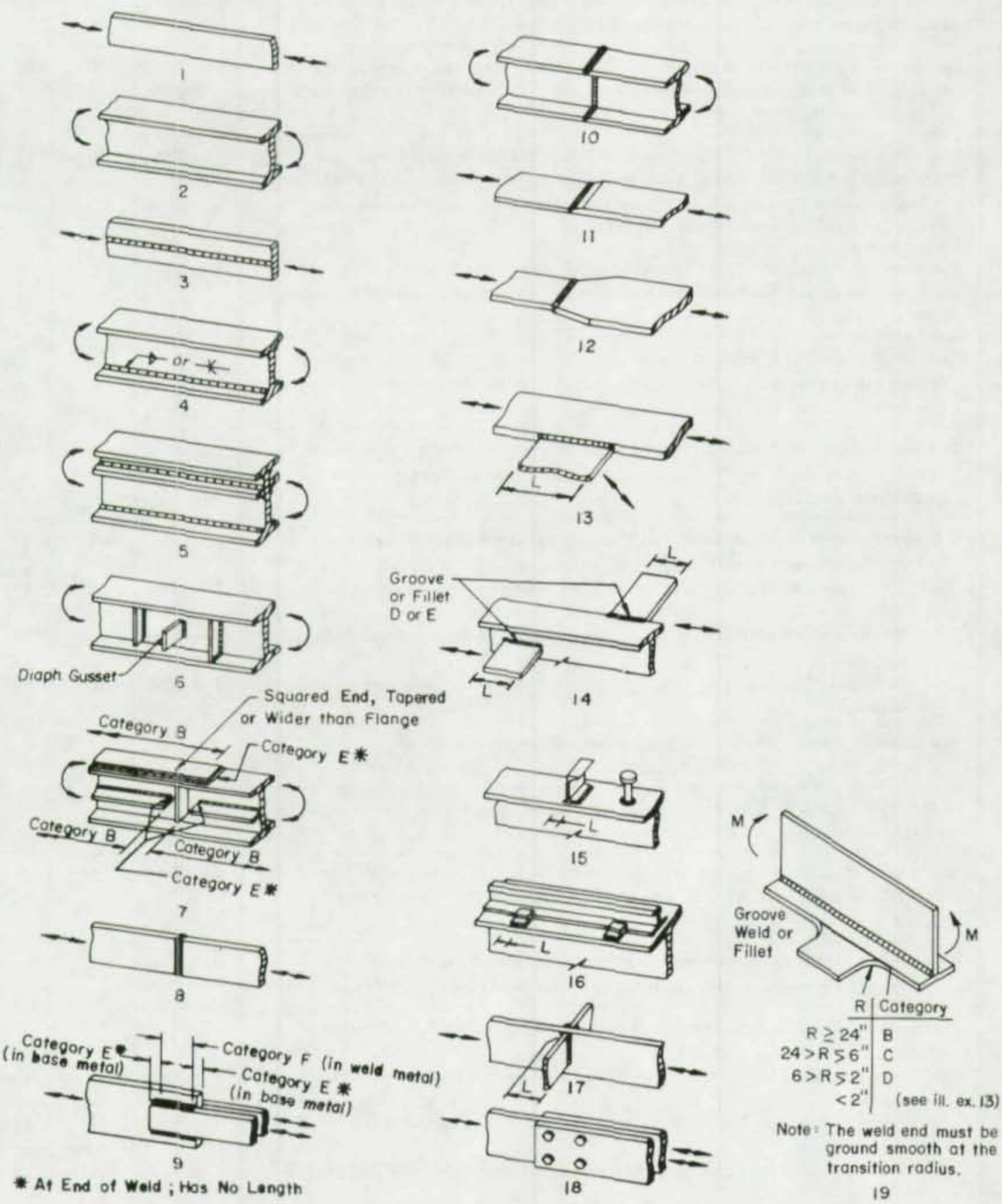

Fig. 1.3.13. Illustrative examples

The fatigue formulas in Parts 1 and 2, 1969 edition of these specifications, were based on the formulas in AASHO (now known as AASHTO) *Interim Specifications, Bridges,* 1966 and 1967,[8] and on additional data published in 1968 and 1969.[29,30] These formulas included consideration of:

(1) Frequency of applications of the critical loadings. Two cases: 500,000 constant-stress cycles or less, and more than 500,000 constant-stress cycles. Wind load plus dead load was *not* included as a case.

(2) R, the ratio of the minimum stress to the maximum stress.

(3) The methods used to fabricate members and fastener materials used to connect members.

Since 1969 additional research[32,33] has demonstrated that:

(1) Stress range (S_R) is the significant factor for fatigue strength, rather than the stress ratio (R).

(2) Cracks that may form in fluctuating compression regions are self-arresting. Therefore, these compression regions are not subject to fatigue failure.

(3) Allowable stress range (S_R) for the various details can be expressed in terms of the number of constant-stress cycles (N).

S_R-N curves, which were developed[33] by using 95% confidence limits for 95% survival applied to test data, are shown in Fig. 9.1.3.13A.[34,35] The categories, A through F, have the same definitions as given in *AASHTO—Interim Specifications, Bridges,* 1974. A discussion of the effect of various welded details on the fatigue life of a typical bridge member is included in *Bridge Fatigue Guide / Design and Details.*[38]

The relationship between the allowable fatigue stress ranges, S_{Rfat}, and the equivalent number of constant-stress cycles, N, was determined as described hereafter.

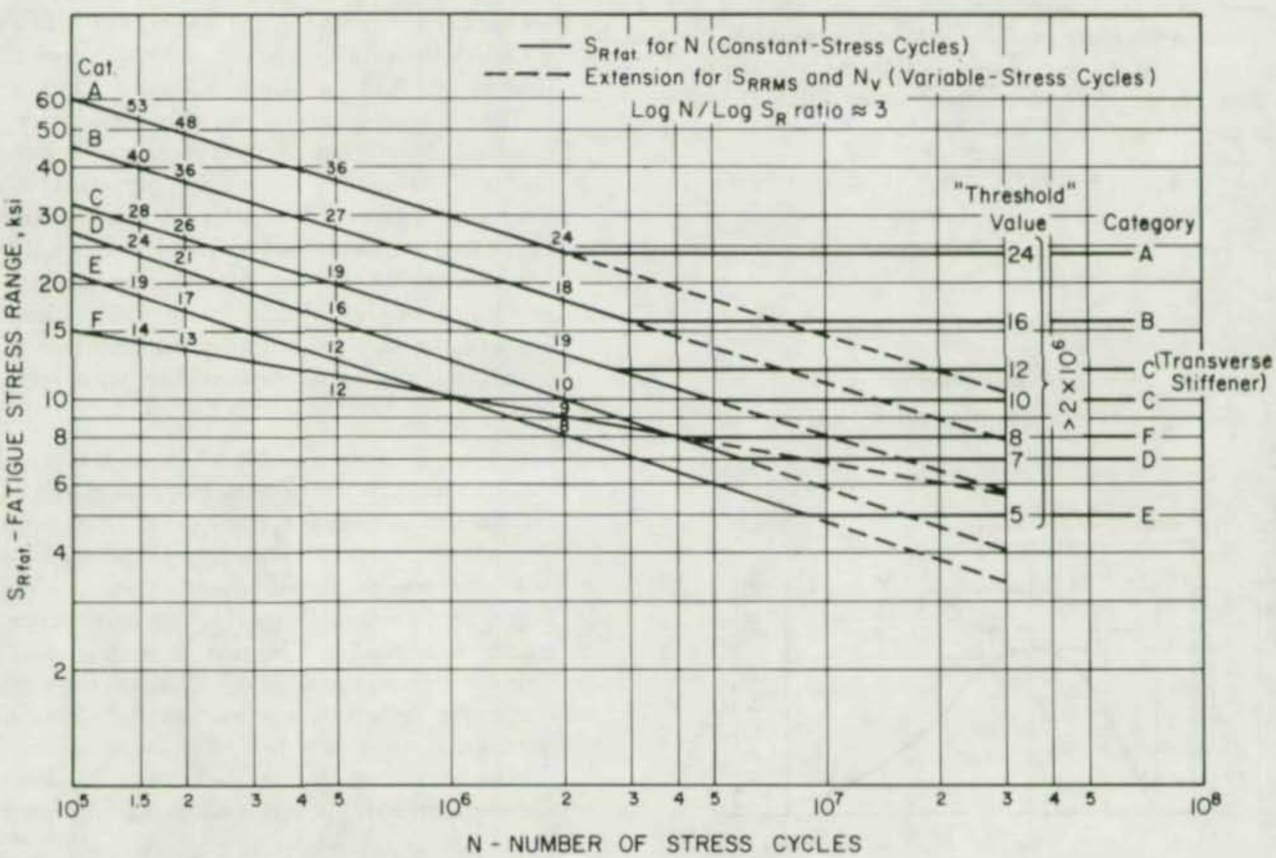

Fig. 9.1.3.13A. S_R-N curves

A Rayleigh probability density function was used to describe the frequency of the stress range experienced by various members and connections. Figure 9.1.3.13B shows schematically the assumed stress range distribution. For derivation of the design stress cycles given in Table 1.3.13A, S_{Rmax} was taken as αS_{Rdes} (usually, for E80 loading), $\phi = 1$, $S_{Rmin} = 0.1S_{Rmax}$, and $S_{Rd} = 0.3S_{Rmax}$. The regular traffic is usually given as E-loading. Since S_R is directly proportional to the applied E-loading, the function is described by S_R.

The following steps were followed when estimating the number of constant stress range cycles:

(1) The loadings to which a bridge will actually be subjected were assumed to have a frequency distribution comparable to the Rayleigh probability density function. This assumed characteristic was based on limited experimental data[30] obtained on older structures which had been designed for E72 and higher loadings including impact. This frequency distribution was used to obtain a relationship between the maximum E-loading and the root-mean-square E-loading.

(2) The maximum and minimum E-loadings for the Rayleigh function were selected using E80 loading as the design reference loading and assuming that the regular traffic would be between E40 to E55 loadings with occasional heavier loadings. Since the S_R at a given location is directly proportional to the E-loading producing it, S_R was used to describe the Rayleigh function (see Step 5 for restrictions on the use of S_{Rmax}).

The stress range mode value for the function, $S_{Rd} = (S_{Rmax} - S_{Rmin})/3$, gave a reasonable fit to available test data. For this distribution, the desired relationship becomes:

$$S_{RRMS} = 0.46S_{Rmax} + 0.54S_{Rmin} \qquad \text{Eq. 1}$$

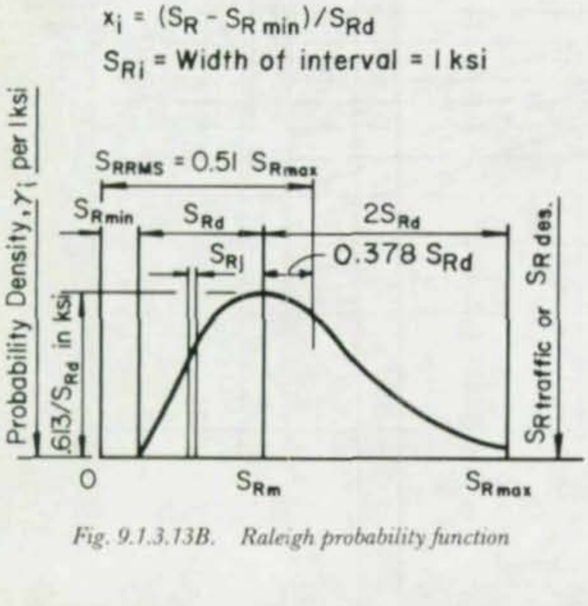

Fig. 9.1.3.13B. Raleigh probability function

For $\Phi = 1$ and $S_{Rmin} = 0.1S_{Rmax}$:

$$S_{RRMS} = 0.51S_{Rmax} \qquad \text{Eq. 1a}$$

Miner's Rule, $\Sigma n/N$, expressed in terms of stress range, $S_{RN} = (\Sigma\gamma_i S_{Ri}{}^B)^{1/B}$, would give a similar relationship if $B = 3$ were used instead of $B = 2$, as was used with the Rayleigh distribution.

(3) αS_{Rfat} represents the S_R creating the fatigue damage and may be substituted for S_{Rmax} in Eq. 1a, which then becomes:

$$S_{RRMS} = 0.51\alpha S_{Rfat} \qquad \text{Eq. 2}$$

Since E_{design} is proportional to S_R:

$$E_{RMS} = 0.51\alpha E_{design} \text{ in terms of E-loading}$$

The assumed values of α for the various span lengths and the respective values of S_{RRMS} are listed in Columns 6 and 7, Table 9.1.3.13A.

(4) The reciprocal of the slope of the S-N log-log-curves represents the N/S_R relationship and has an approximate value of 3 for most details.[36] N, then, varies inversely with $S_R{}^3$. Using Eq. 2, for which $S_{Rmin} = 0.1\alpha S_{Rfat}$, the relationship between N and N_v may be approximated by:

$$N = (0.51\alpha)^3 N_v \text{ or } N/N_v = (0.51\alpha)^3 \qquad \text{Eq. 3}$$

Substituting the N_v values from Col. 5, Table 1.9.3.13A, into Eq. 3 provided the approximate values of N which are listed in Tables 1.3.13A and B and in Col. 9. In Table 9.1.3.13A, values of S_{Rfat} for each value of N were taken from Fig. 9.1.3.13A and listed in Table 1.3.13B.

The projected number of variable stress cycles, N_v, given in Col. 5 corresponds to an average main-line volume of 60 daily trains over an 80 year period. Where more specific data is available for a bridge, those values can be used to estimate the variable stress cycles. If more precise estimates of α are available, they also can be substituted for the values listed in Table 9.1.3.13A. Likewise, the actual frequency distribution, if known, can be utilized in place of the idealized Rayleigh distribution for Col. 7.

(5) Steps 1 through 4 and Tables 1.3.13A and B apply when the maximum predicted loading is approximately equal to the design loading, $\Phi \approx 1$, and the Rayleigh function shown in Fig. 9.1.3.13B is used.

$\Phi > 1$ represents a second condition when the design loading for the structure is less than the desired maximum regular traffic loading to be used on the structure.

$\Phi < 1$ represents a third condition when the design loading for the structure is more than the maximum regular traffic loading to be used on the structure.

With reference to Step 3, the scope of Eq. 2 can be widened to include all three conditions by the insertion of Φ:

$$S_{RRMS} = \Phi\,(0.51\,\alpha)S_{Rfat} \qquad \text{Eq. 2a}$$

Table 9.1.3.13A. Parameters (used to develop Tables 1.3.13A and 1.3.13B)

Classification	1	2	3	4	5	6	7	8	9
	Span Length L (in feet)	Life in Days (80 yr)	Daily Trains	Stress Cycles per Train Crossing	Projected N_v	α	$\frac{S_{RRMS}}{S_{Rfat}}$ (for $\Phi = 1$) Eq. 2	$\frac{N}{N_v}$ Eq. 3	N (Used in Table 1.3.13A)
I & III	$L > 100$	29,200	60	2.0	3.5×10^6	.70	.357	.045	1.5×10^5
	$100 \geq L > 75$	29,200	60	2.0	3.5×10^6	.80	.408	.068	2×10^5
	$75 \geq L > 50$[a]	29,200	60	3.0	5.3×10^6	.85	.433	.081	5×10^5
	$50 \geq L > 30$	29,200	60	12.0	21.0×10^6	.85	.433	.081	2×10^6
	$30 \geq L$[a]	29,200	60	60.0[b]	105.0×10^6	.90	.459	.097	$>2.0 \times 10^6$[c]
II	Web Member 2 Tracks	29,200	60	1.0	1.8×10^6	.95	.484	.113	2×10^5
	Web Member 1 Track	29,200	60	2.0	3.5×10^6	.95	.484	.113	5×10^5

[a] Also includes members in Classification III, Table 1.3.13A.
[b] Based on one cycle per car or engine for trains averaging 60 load units (cars or engines).
[c] See Figure 9.1.3.13A—probably unlimited fatigue life.

Similarly, in Step 4, Φ should be inserted in Eq. 3:

$$N = (\Phi\, 0.51\alpha)^3 N_v \qquad \text{Eq. 3a}$$

In summary, the N values in Table 1.3.13A should be multiplied by the factor (Φ^3) to obtain the proper value of N for the desired condition. For $\Phi > 1$, design loading smaller than traffic loading, N will be increased for the same value of N_v. Appropriate values of S_{Rfat} for the new values of N may be obtained from Table 1.3.13B or scaled from Fig. 9.1.3.13A. These values for R_{Rfat} will be smaller than those for $\Phi = 1$.

For $\Phi < 1$, N will be decreased for the same value of N_v. Again, appropriate values of S_{Rfat} for the new value of N may be obtained from Table 1.3.13B or scaled from Fig. 9.1.3.13A. These values of S_{Rfat} will be increased over those for $\Phi = 1$.

For short spans and transverse members which receive one or more stress cycles per car passage, an additional column, ">2,000,000", has been added to Table 1.3.13B. The S_{Rfat} values listed represent the "threshold," or probable fatigue limit, for each category.

In Table 1.3.13C, base metal at the gross section of high-strength (H.S.) bolted friction-type connections was placed in Category B. Existing test data on H.S. bolted connections were plotted[37] with the S_R-N design curve, Fig. 9.1.3.13A, for Category B. The plot showed that all data were well above the stress ranges permitted by Category B. Similarly, for the base metal at the net section of riveted connections, a plot of existing test data and the S_R-N design curve for Category D gave the same result. Category D was then used for riveted connections.

The existing test data which were used as described in the preceding paragraph also showed that the failures were in the connected material and not in the fasteners. If the fasteners and connected material are proportioned in accordance with Sections 1.3.13 and 1.4, the fasteners will have a greater fatigue life than the connected material. Thus, no categories for bolts or rivets in shear or bearing are required to replace the 1969 formulas.

For the usual design condition, only the bending S_R needs to be considered for details such as transverse stiffeners, which are influenced by shear stresses as well. The classification into design categories has taken the shear influence into account. Therefore, principal stresses need not be considered in the usual design condition. For unusual design conditions, details may be used which require the principal stresses to be considered.

List of Symbols

B	=	Reciprocal of the slope of the log-log S_R-N curves or the N/S_R ratio. See Fig. 9.1.3.13A.
E_{design}	=	Design load based on E-loading
n	=	Number of occurrences of constant stress cycles which would cause fatigue damage equivalent to the fatigue damage caused by a larger number, N_v, of variable stress cycles

n = Number of stress cycles for each of the stress-range values represented in the distribution being considered

N_v or Σn = Total number of variable stress cycles in the distribution or life

S_R = Stress range, the algebraic difference between the maximum stress and the minimum stress for a stress cycle

S_{Ract} = Stress range actually created at a given location in the structure by a moving load

S_{Rd} = Stress range at the peak value based on the starting point of the function. Therefore, $S_{Rd} = S_{Rm} - S_{Rmin}$

$S_{Rdesign}$ = Required S_R for E-design loading

S_{Rfat} = Allowable fatigue stress range as listed in Table 1.3.13B

S_{Ri} = Width of the interval that was used to subdivide the Rayleigh function

S_{Rm} = Stress range at the peak value, or mode, for the Rayleigh function based on the origin or $S_R = 0$ point

S_{Rmax} = Maximum stress range or upper limit value for the function being considered

S_{Rmin} = Minimum stress range or lower limit value for the starting point of the function being considered

S_{RN} = Stress range which corresponds to N constant stress cycles for a given detail

S_{RRMS} = Stress range for the Root Mean Square (RMS); equals the square root of the sum of the squares of each value of $(S_R - S_{Rmin})$ within the function being considered plus S_{Rmin}.

α = S_{Ract} / S_R or $E_{act} / E_{applied}$ ratio when S_R is calculated by using the same load which was applied when S_{Ract} was measured. Field measurements have shown the measured S_R is equal to a factor, α, times the calculated S_R. This reduction reflects the beneficial effects of participation by the bracing, floor system, or other three-dimensional response of the structure and, also, the fact that full impact does not occur for every stress cycle. Since S_R at a given location is directly proportional to the loading used, $E_{act} / E_{applied}$ also equals this ratio

γ_i = Probability density for an interval of 1 ksi, or the ratio of the number of occurrences of S_{Ri} to the total number of occurences, Σn, within the function being considered

Φ = Ratio of the maximum traffic loading to the design loading for the structure. For Tables 1.3.13A and B, $\Phi = 1$.

References (Arts. 9.1.3.13 and 9.2.3.1)

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