

FATIGUE STRENGTH IN BENDING OF STEEL I-BEAMS WITH WELDED ATTACHMENTS

1. NOTATION

N	endurance		
S_a	maximum nominal alternating principal stress (half range) in beam in critical regions at ends of attachments or weld runs (see Section 2 for different cases)	N/m ²	lbf/in ²

Both SI and British units are quoted.

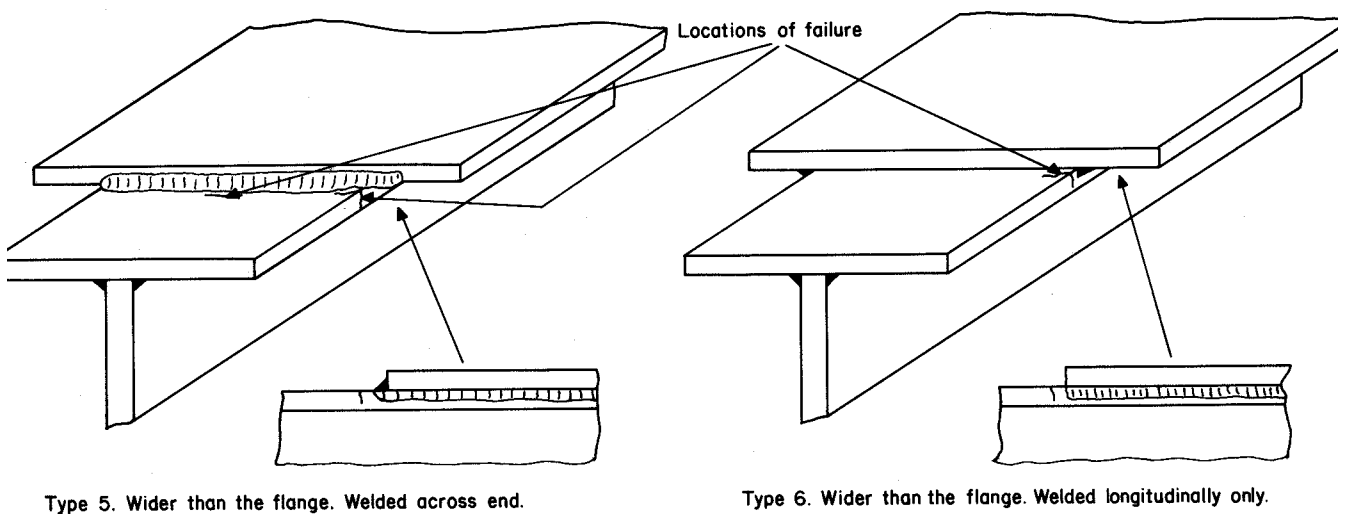
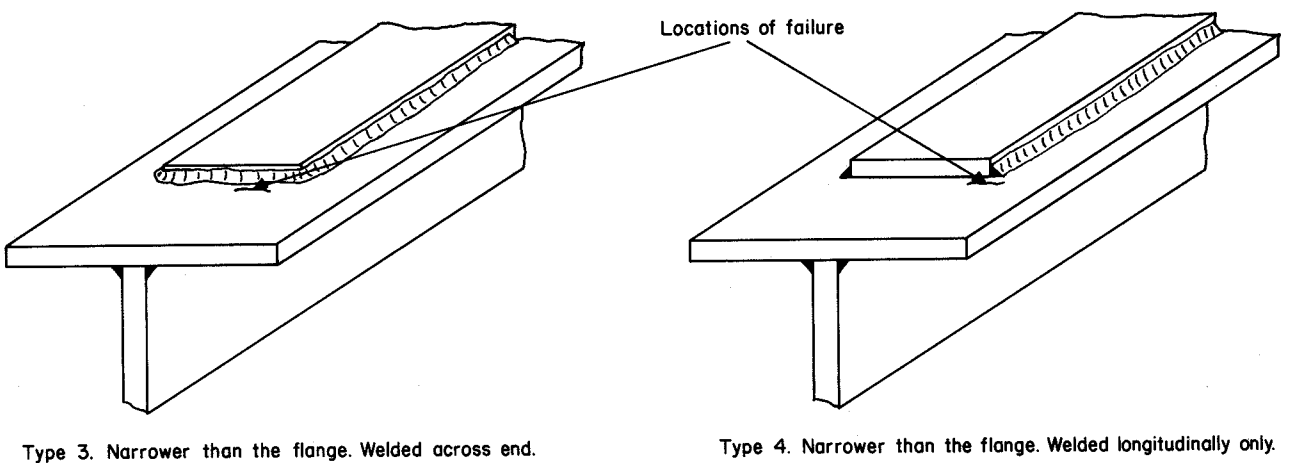
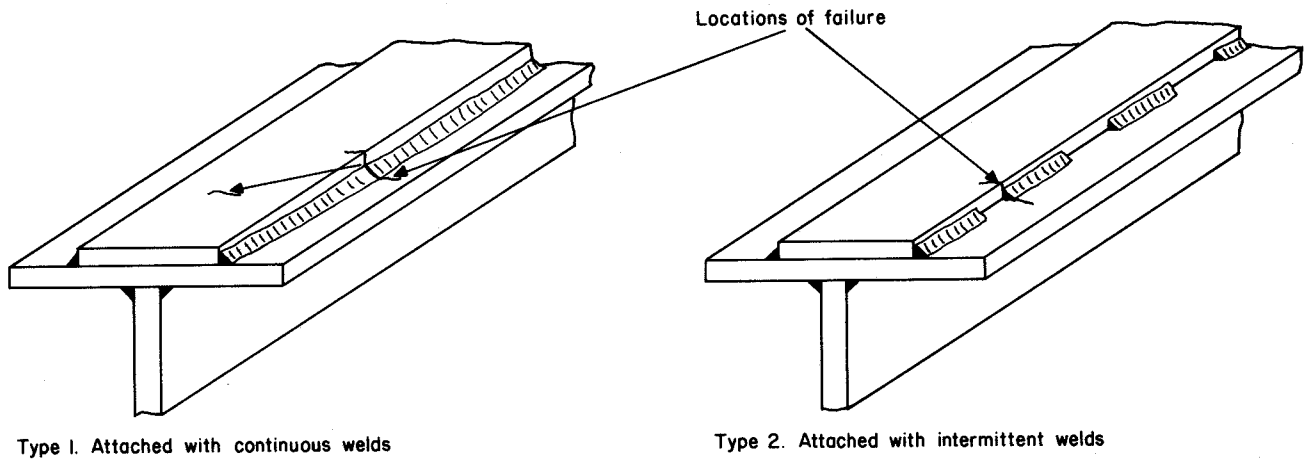
2. INTRODUCTORY NOTES

This Item presents data on the fatigue strength in bending of structural steel I-beams with welded cover-plates or with stiffening gussets welded to the webs and to one or both flanges. The attachments were of similar composition to the beam materials and the types and configurations of attachments to which the data apply are illustrated in Sketches 2.1 and 2.2. The data were obtained from tests on rolled or welded beams of between 100 mm (4 in) depth by 75 mm (3 in) width and 400 mm (16 in) depth by 200 mm (8 in) width in the as-welded condition. The beams were tested in four point bending in the plane of the web.

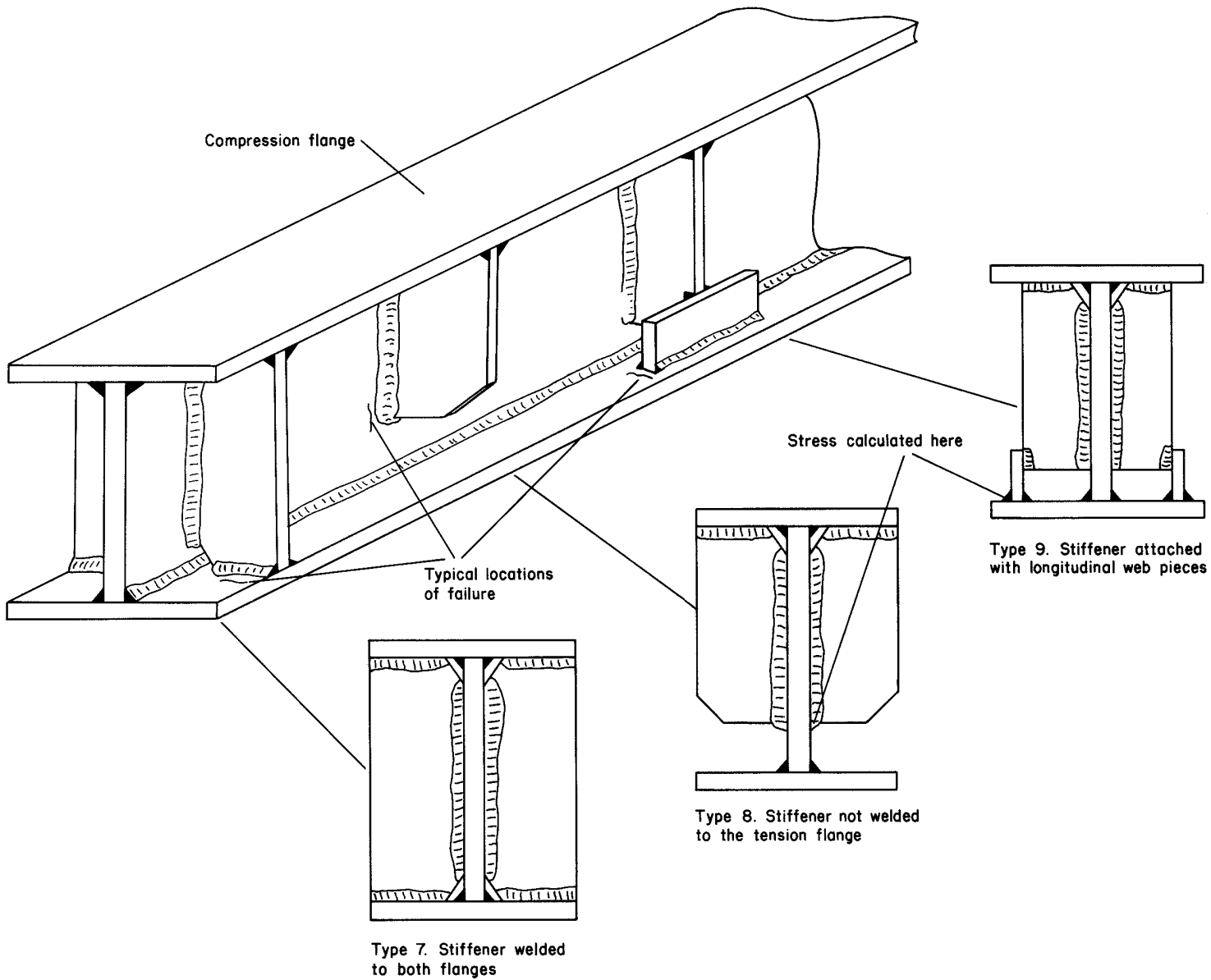
In Figures 1 to 7 data are presented in terms of the nominal principal stress range, $2S_a$, in the beam in the critical regions at the ends of attachments or weld runs. These stresses were determined either by strain gauge analysis or by calculation using simple bending theory. For attachments falling within the constant moment region and for attachments welded to the tension flange outside the constant moment region, the fibre stress calculated using simple bending theory, was taken as the nominal principal stress. Where the critical region was positioned on the web of the beam and outside the constant moment region, shear stresses were taken into account when calculating the nominal principal stress. (When calculating the second moment of area the cover-plate was only taken into account where full length cover-plates existed.)

Provided that the welds are sound and free from major defects, the fatigue strength above 10^4 cycles of structural steel I-beams with attachments having undressed welds is independent of the properties of both plate and weld materials. The fatigue strength is independent of applied tensile mean stress within the range of mean stresses for which data were available (0 to 235 MN/m²). This is because as-welded joints contain a system of localised residual stresses in the direction of stressing which result from the internal constraints developed during cooling of the weld and which are normally of the order of the material yield strength. Thus, the first few load cycles may cause local yielding in critical regions, such as the ends of weld runs in longitudinal welds and along the weld toe in transverse welds. The actual stress experienced in the critical regions will, thereafter, cycle between the material yield strength and the yield strength less the applied stress range irrespective of applied mean stress (except in cases of high compressive mean stress, see Section 3.3). Beams larger than those tested may be expected to exhibit fatigue lives at the lower end of the scatter band due to the full development of these residual stresses.

Mean lines and scatter bands that enclose 95 per cent of the data are provided on Figures 1 to 4. A comparison of the mean lines for the various joints covered by this Item is given in Figure 5. Mean lines and lower bounds of Figures 1 to 4 are compared with those given in BS 5400 for similar joint types, see Section 5 and Figures 6 and 7.



Sketch 2.1 Cover-plate attachments (Types 1 to 6)



Sketch 2.2 Stiffener attachments (Types 7 to 9)
(see Section 4 for notes)

3. BEAMS WITH COVER-PLATES

Data are presented in Figures 1 and 2 for beams with cover-plates welded symmetrically to their flanges. Figure 1 presents data for beams with full length cover-plates attached with either intermittent or continuous welds and compares these with the scatter bands of data for plain welded beams from Data Item No. 76032.

Figure 2 gives data for beams, either rolled or welded (built-up), with partial length cover-plates welded to each flange. The cover-plates were attached with either intermittent or continuous manual or automatic longitudinal welds and either with or without transverse end welds. No effects of welding method could be discerned from these data. These data are compared with those for plates in tension with longitudinal cover-plates, see Item No. 75016*. The partial length cover-plates were either wider or narrower than the flanges to which they were attached and had cover plate thickness to flange thickness ratios of between 0.75 and 2.0.

The available data, given in Figures 1 and 2, show no positive knee in the high cycle region and are represented by a straight line on a log endurance/log stress basis. The stress used to present the data is the stress range in the flange between the loading points and in Figure 2 is the stress at the toe of the weld where transverse end welds were used and at the end of the weld run where only longitudinal welds were used. In the case of beams with full length cover-plates the cover-plate was taken into account when calculating the second moment of area. Most tests relate to beams with cover-plates attached symmetrically to both flanges. The effects of cover-plate geometry on fatigue strength are described in Section 3.1. The effect of using cover-plates of different lengths on each flange is discussed in Section 3.3.

Failures in beams with full length cover-plates welded continuously start at a weld ripple, or at defects on the cover-plate surface, or at the ends of weld runs where intermittent welds are used. Failures occurred by the propagation of cracks across and through both cover-plates and flanges. Failures in beams with partial length cover-plates occurred from cracks initiated in the beam flange at, or close to, the ends of transverse welds or at the ends of longitudinal weld runs, which propagated across and through the flange of the beam and into the web. Failure locations are indicated in Sketch 2.1. In general, the crack propagates in a direction perpendicular to the direction of stressing.

3.1 Effect of Cover-plate Geometry on Fatigue Strength

No effect of cover-plate width on fatigue strength of beams with partial length cover-plates could be discerned from the collected data. None of the beams tested with cover-plates narrower than the flange had weld runs that interfered with the edges of the flange.

The data in Figure 2 are for beams with cover-plates of thickness between 0.75 and 2.0 times the flange thickness. No effect of cover-plate thickness could be discerned from these data. The majority of standards and codes of practice recommend that no cover-plate should be more than 1.5 times the flange thickness, although this restriction may be for reasons other than fatigue strength.

All the beams tested had cover-plates longer than 1.5 times the cover-plate width if transverse end welds were used and longer than three times the width if only longitudinal welds were used.

Data are included in Figure 2 for cover-plates having round, tapered, or re-entrant ends. These data do not disagree with those for square-ended cover-plates.

3.2 Effect of Welds at the Ends of Cover-plates on Fatigue Strength

End welds appear to have no beneficial effect on the fatigue strength where the cover-plate is narrower than

* Data Item No. 75016 "Fatigue strength of transverse fillet and cruciform butt welds in steels".

the flange. However, data from coherent sets of tests suggest that the fatigue strength of beams with cover-plates wider than the flange and attached with only longitudinal welds is about 25 per cent less than that of those with end welds. The post-weld treatment of end welds on cover-plates, for example dressing, can significantly improve the fatigue strength, see Data Item No. 78016.*

It should be noted, however, that while the above conclusions hold for simple structural details the load path in more complex structures may be altered by the addition of end welds and may lead to premature failure.

3.3 Failures in the Compression Flange

Failure may occur in the flange in which the mean applied stress is compressive. This arises if the applied cyclic stress in the flange is greater than that in the other flange for reasons of asymmetry of cover-plate geometry or loading. Some data for asymmetric cover-plates are given in Figure 2. The reason for this behaviour is that the applied compressive mean stress is more than eliminated by the tensile residual stress. It is, therefore, unsafe to assume that no failure will occur in flanges where the mean stress is compressive. The few data available for compression flange failure are for beams with asymmetric cover-plates, the compression flange cover-plate (that is a flange in which the nominal mean stress is compressive) being shorter than on the flange with a mean tensile stress and therefore having a higher nominal principal stress range. It would appear from Figure 2 that these failures are likely to occur at a much higher endurance than those in the tension flange for a given stress range.

4. BEAMS WITH STIFFENING GUSSETS

In some standards and codes of practice for beams with welded attachments, stiffeners welded to the tension flange of beams are put into a lower fatigue strength category than stiffeners not welded to the tension flange (attached only to the web and/or the compression flange). For this reason this Item separates attachments into the same two categories.

4.1 Stiffeners Welded to the Tension Flange

Data are presented in Figure 3 for stiffeners welded to the web and to both tension and compression flanges. The data are for beams with stiffeners arranged symmetrically about the web of the beam, Type 7 of Sketch 2.2. The data are presented in terms of stress range at the toe of the transverse weld on the inner surface of the tension flange. Data are also given in Figure 3 for beams with stiffeners attached to the tension flange using additional longitudinal web pieces, Type 9 of Sketch 2.2, presented in terms of stress range at the end of the longitudinal weld runs. These data are compared with data for plain welded beams.

4.2 Stiffeners not Welded to the Tension Flange

Data are presented in Figure 4 for stiffeners welded to the web and compression flange of the beam but not attached to the tension flange, Type 8 of Sketch 2.2. For a given bending moment the stress experienced at the end of a weld run depends upon the distance of the critical region from the neutral axis of the beam. Data are therefore presented in terms of the stress range at the end of the weld run. These data are compared with the scatter band of the data of Figure 5. It should be noted that failures may still occur from stiffeners attached in regions of nominally compressive fibre stress, see Section 3.3. Only some of the stiffeners for which data are presented in Figure 4 were welded across the end of the stiffener. The data in Figure 4 are for failures originating at, or close to, the end of the weld attaching the stiffener to the web, see Sketch 2.2. These data agree well with data for failures from stiffeners attached to the tension flange. Failures usually occurred by the initiation and propagation of fatigue cracks perpendicular to the direction of stressing. Some

* Data Item No. 78016 "The effect of post-weld treatments on fatigue at fillet welded attachments".

failures began from longitudinal cracks formed parallel to the beam flange at the end of stiffeners welded around the end, these cracks turning to run transversely across the web, parallel to the stiffeners.

5. COMPARISON OF DATA OF FIGURES 1 TO 4 WITH BS 5400

In Figure 5 the mean lines of Figures 1 to 4 are compared with the mean line of data for beams with continuous web-to-flange welds.

British Standard 5400, Part 10 (see Derivation 21) provides guidance on the design of beams with welded attachments under fatigue loading. The standard classifies joints in terms of fatigue strength using a letter coding, see Appendix A, and presents the fatigue strength of a variety of joint types, weld and stress configurations by single straight lines on a log stress/log endurance plot.

The mean and lower bound lines of BS 5400 Classes C to G are given in Figures 6 and 7 and are compared with the mean and lower bound of the data of Figures 1 to 4. However, since Figures 1 to 4 relate to specific joint types and the BS 5400 Classes C to G each cover a wide variety of joint types, only limited conclusions can be drawn from such a comparison. These conclusions are as follows.

Full length cover-plates – Types 1 and 2 (Sketch 2.1)

Good agreement between Figure 1 and BS 5400 Class D. It should be noted that Class C welds fall mainly outside the scope of this Item.

Partial length cover-plates – Types 3, 4, 5 and 6 (Sketch 2.1)

Figure 2 gives fatigue strengths slightly below Class G for this type of joint.

Stiffeners welded to the tension flange – Types 7 and 9 (Sketch 2.2)

Joints covered by Figure 3 appear to have welds up to, or close to, the edge of the flange and would therefore fall into Class G. The data fall well above the Class G line but agree well with the Class F line which covers welds that do not interfere with the edges of the flange.

Stiffeners not welded to the tension flange – Type 8 (Sketch 2.2)

Figure 4 is in good agreement with Classes E and F which cover this type of joint.

6. DERIVATION

This section lists selected sources that have assisted in the preparation of this Item.

- | | | |
|----|-----------------------------|---|
| 1. | LEA, F.C.
WHITMAN, J.G. | The failure of girders under repeated stress. <i>Weld. J. Res. Supplmt</i> , Vol. 18, No. 1, pp. 28s-33s, January 1939. |
| 2. | WILSON, W.M.
MUNSE, W.H. | The fatigue strength of steel beams. Bulletin No. 377, Engineering Experiment Station, University of Illinois, USA, 1948. |
| 3. | WILSON, W.M.
MUNSE, W.H. | Fatigue strength of various details used for the repair of bridge members. Bulletin No. 382, Engineering Experiment Station, University of Illinois, USA, 1948. |