EUROPEAN RESEARCH ON THE IMPROVEMENT OF
THE FATIGUE RESISTANCE AND DESIGN OF
STEEL ORTHOTROPIC BRIDGE DECKS

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Abstract

The orthotropic steel deck has been used as a lightweight deck in steel bridges since
many years. This type of structure has suffered several developments since 1950 (e.g.
in view of the shape of longitudinal stiffeners, the distance of cross beams, the
connection of the longitudinal stiffeners to the web of crossbeams etc.) and there are
also differences between railway bridges and road bridges. The fabrication of the
joint details represents a considerable amount of the costs of the bridge.

In the last decades, the traffic intensity and the wheel loads on bridges have
increased considerably, resulting in fatigue cracks in modern steel bridges within
their service lives. Also, detailing of the welded connections and execution of
welding was not always carried out to a sufficiently high standard. Although cracks
are found at many locations, they are usually not immediately threatening the
performance of the bridge. A long time before they can develop to such dimensions,
they have been detected. Repair however, causes large expenses; as bridge decks are
large areas thus include many spots to be repaired. Detail studies of the fatigue design
and the behaviour of steel bridges were required.

As the deck is directly subjected to the severe impact of heavy wheel loads the
research concentrates on the components of orthotropic deck structures. This paper
reviews the European Coal and Steel Community funded research on welded details
of such a deck in relation to observed fatigue failures. It highlights the practical
experience and recommendations on how to reduce the risk of fatigue failures.

Introduction

Early bridges with orthotropic decks have been designed using codes mainly relating
to the behaviour under static loading. Since the early eighties for the assessment of
fatigue various codes exists. For example one of the leading codes in this respect, the
British Standard on fatigue (BS 5400, 1980) defines (i) the fatigue design codes to be
used, (ii) the allowable stress ranges for a service life of 120 years and (iii) the
procedure for the fatigue assessment. However, the codes do not specify the stress analysis and classification of welded details in modern orthotropic steel bridge decks. Due to the higher wheel loads and the increased traffic intensity bridges are more heavily subjected to fatigue loading than in the past. Furthermore the detailing of the welded connections and the execution of welding was not always carried out in the most optimal way. This has resulted in fatigue cracks during service life in modern steel bridges.

To improve the knowledge about fatigue in bridges and to avoid unnecessary pessimism if anyone is aware of possible fatigue damage, several joint research projects have been carried out within the framework of research sponsored by the European Coal and Steel Community (ECSC). Topics included the traffic loading, the resulting stresses in the bridge structure and the fatigue strength of welded details. Initially the investigations were mainly experimental, but nowadays the experiments are combined with numerical and analytical studies. These studies were carried out jointly by the following research institutes: Transport Research Laboratory (United Kingdom), Laboratorium für Betriebsfestigkeit (Germany), Laboratoire Central des Ponts et Chausées (France), Institut de la Soudure (France), Universita di Pisa (Italy), Université de Liège (Belgium) and Delft University of Technology (The Netherlands). Several ECSC reports (Hoffman, 1982; Haibach, 1988; Bruls, 1991, 1995; Kolstein, 1999) summarize parts of these studies. Some relevant conference publications are included in the section “Additional Information” of this paper.

The obtained results during the 1st part of the ECSC research have been contributed to the scientifically basis for the Eurocode on Actions – Traffic Loads on Road Bridges (ENV 1991-3, 1995) which prescribes load models for static design and fatigue verification of bridges. The 2nd part concerns mainly the study on fatigue aspects of orthotropic steel bridges decks. For several details, the stress fields were studied by calculations and/or measurements. Constant amplitude as well as variable amplitude fatigue tests simulating traffic effects have been carried out. After failure some specimens have been repaired to extend the lifespan. The results are almost fully included in two parts of Eurocode 3 on the design of steel structures (prEN 1993-1-9, 2002 and prEN 1993-2, 2003).

This code gives guidance for the design of a durable orthotropic deck. Recommendations for the structural detailing and for the determination of the relevant stress ranges for the substructures, e.g. the stiffness and strength criteria of the deck plate and the longitudinal stiffeners, the detailing for the deck plate splices, for the longitudinal stiffener to cross beam connection, for stiffeners fitted between crossbeams, for connections without cope holes, etc. is given.

However, several factors may reduce the fatigue strength of bridges, such as the difficulty of maintaining tolerances on fit-up and weld quality on a large fabrication, especially for site welds. The permitted defect level for the highest weld classes is very low and it seems unlikely that such a low level of defects could be guaranteed for all welds on a large bridge. The level of inspection commonly used in bridge works is limited by the cost of close inspection of a large number of welds. On the other hand current non-destructive techniques are not applicable to control for example longitudinal stiffener to deck plate welds, which causes a lot of uncertainties with respect to the quality of these welds.
Orthotropic Steel Bridge Decks

General. *In order to achieve economy in the design of steel bridges one of the main aims must inevitably be to reduce to a minimum the dead weight of the superstructure, and this is particularly so in the case of long span and lifting bridges. One way to assist in the achievement of that objective is, of course, to avoid the use of heavy bridge decks. Although there is more than one possible approach to that problem, a fairly obvious one is to make use of a welded steel deck plate, since that eliminates the relatively heavy concrete deck might be expected to weigh about four times as much as an orthotropic steel deck (Gurney, 1992).*

The earlier orthotropic steel decks were stiffened by flats and bulbs (see Figure 1a), thus allowing for spans of approximately 2 meters. The rather small stiffness and strength of these longitudinal stiffeners caused need for many crossbeams. The fabrication of these structures was laborious and subsequently expensive. This caused the need to develop structures with less welded connections. The introduction of the closed V-shaped, U-shaped and trapezoidal longitudinal stiffeners as shown in Figure 1b, was a large improvement, which allowed spans of approximately 4 meters. The secondary crossbeams and main girders were no longer needed. The amount of work involved reduced as well as the costs.

**Stiffener to deck joint.** The total length of welds required in the fabrication of a deck with closed longitudinal stiffeners is about half of that for a deck with open stiffeners. However, the welding of closed stiffeners to the deck plate raises technological problems, as the welds can only be executed from the outside of the stiffener web. Under traffic loading, particular the effect of local wheel loads, the stiffener to deck plate welds are submitted to local transverse bending moments and is therefore susceptible to fatigue cracking (see Figure 2). Therefore for design and fabrication of this joint specifications are required. However too severe requirements increase the fabrication costs and reduce the advantages of closed stiffeners in orthotropic decks.

**Stiffener splice joint.** Longitudinal stiffeners passing through the cross beams are formed in length which have to be joined end to end. Large bridges are typically fabricated in sections of around 20 metres long. Lifting each section into place and joining it to the sections already in place build the bridge. Thus, some stiffener splice joints will be made in the fabrication shop to make the 20-meter sections. The remainder will be made as positional site welds. For ease of site assembly, a gap is usually left between the adjacent stiffeners (see Figure 3). A splice plate trimmed to the correct length fills this. The spice plate is then welded to both stiffeners. Clearly these welds must be made from outside the stiffener in an unfavorable overhead position. Indeed, this type of joint can account for 40% of site welding and in bridges with very long spans up to 5000 such joints may be required. Their adequacy is therefore of importance.

**Stiffener to crossbeam connection.** This is the most complex joint in the orthotropic deck. Two highly stressed members cross each other and it is impossible to provide a
continuous load path for both. Basically this joint can be made in two ways, see Figure 4. Until about thirty years ago for longitudinal stiffeners it was difficult to form trough sections in long lengths, so sections of trough stiffeners were made to fit between continuous crossbeams and attached with a fillet weld around the end of the trough stiffener. In more recent designs continuous trough stiffeners pass through cutouts in the crossbeams. Short stiffeners are now only used for some movable bridges where the overall depth of the deck is small, and e.g. roll-on roll-off ferry ramps. Here butt welds are applied to connect the end of the stiffener to the crossbeam.

Experiences on Bridge Decks with Closed Stiffeners

**Stiffener to deck joint.** Failures of the stiffener to deck plate weld have been reported several times. It was concluded that in those cases the relative high stress spectra in combination with the fillet weld or partial penetration weld used caused these failures, see Figure 5 (Gurney, 1992). The stress spectra for this welded detail are strongly influenced by the thickness of the steel plate in combination with the type of surfacing on the deck plate, see Figure 6 (Kolstein, 1997). An excessive gap between the stiffener and the deck plate results in local poor welding and lack of penetration, which lead to a notch effect in the root of these welds. Depending on the location of the crack e.g. stiffener web or deck plate, special repair procedures are required to minimize user delays and avoid repeating within the remaining life of the bridge (Cuninghame, 1987; Mehue, 1981).

**Stiffener splice joint.** Failures of the fillet welded splice connection; have been reported for a bridge after 16 years in service under normal traffic loading (Allan, 1987). An initial assumption was that the pattern of cracking might be related to poor fit-up. However, while fit-up was found to be variable, there appears to be no strong correlation between poor fit-up and incidence of cracking. Also, while the standard quality of welding at the splices is not better than a general commercial quality, no strong correlation appears to exist between weld defects, such as lack of fusion or undersized welds and incidence of cracking. Based on research work on full-scale fatigue tests for the repair procedure butt splice joints with backing strips have been applied.

However, also cracks have been found in various bridges with a splice connection using butt splice joints with backing strips (Kolstein, 1990). X-rayed examination of cores taken from the cracked welds and of cut out splices indicated that the cracking was caused by insufficient root gap between the splice plate and the trough, which resulted in an insufficient penetration of the weld. Besides porosity, undercut and gas cavities were found, see Figure 7. Depending on the length of the crack, the total splice joints have been removed and new joints were installed with a sufficient root gap and quality control of the welding. Small cracks have been removed by grinding up to the backing strip and a new weld was placed.

**Stiffener to crossbeam connection.** Failures of the stiffener to crossbeam connection of the type where the stiffeners are fitted between the crossbeams have been reported
several times (Cuninghame, 1987; Mehue, 1981; Nather, 1991). It was concluded that the relative high stress spectra for the fillet welds used, in combination with the observed root defects, misalignment of the troughs and weld shrinkage effects caused these failures.

If long stiffeners pass through cutouts in the crossbeam a continuous load path along the stiffener is provided, but the cutouts in the crossbeam lead to high stresses, see Figure 8 (Lehrke, 1990). Furthermore the stiffener to crossbeam weld transmits shear from the stiffener into the crossbeam and hence into the supporting structure (Leendertz, 1995). The cutouts in the crossbeam may fit closely around the stiffener, so the weld is continuous, or a cope hole may be provided around the soffit of the stiffener, so that the welds are made on the stiffener webs only. The latter design is easier for fabrication. In several plate girder bridges with relative low crossbeams the stiffener to crossbeam connection with cope holes, some cracks have been observed whereas the fully welded connections showed no cracks. In box girder bridges the stiffener pass trough connecting plates, which have a length of almost a full panel width. The passages are cut outs with cope holes. Some locations were regarded suspect but in inspections they showed (until now) to be no more than cracked corrosion protection.

Fatigue Tests and Numerical Analysis

Stiffener to deck joint. Tests were carried out in a fatigue rig simulating stress distributions in the deck plate and the stiffener web which are compatible with the extreme values observed in full-scale orthotropic bridge decks under actual traffic loading.

Results from Figure 9 (Bignonnet, 1990; Bruls, 1990) show that poor fit-up of the stiffener to deck plate significantly reduce the fatigue strength of this detail. Furthermore it is clear that the fatigue strength increases when using submerged arc welding, which allows larger penetration and larger throat of the weld, see Figure 10).

Stiffener splice joints. Fatigue tests on the stiffener splice joint with trapezoidal as well as triangular shaped stiffeners have been carried out (Caramelli, 1990; Kolstein, 1990). Testing was carried out on full-scale one-rib specimens with a complete splice. The specimens were all welded in the overhead position. Figure 11a shows that the asymmetric butt-welded connection without a splice plate resulted in a lower fatigue strength than the connection with a splice plate and a butt weld on a permanent backing strip. The difference is negligible if the data is considered in the statistical analysis. As shown in Figure 11b the fatigue strength of the butt-welded connection using backing strips is significantly reduced by lack of penetration defects in the welds resulting from an insufficient root gap before welding. The highest fatigue strength has been obtained by using a root gap of at least 6 mm (equal to thickness of the stiffener), which provides a good access for the root run of the weld. The backing strips are usually held in place by tack welds. Only two test results were found for cracking at an external tack weld and both gave similar the endurance to the butt weld. Using Miner’s calculation there seemed to be a good agreement between the results of the constant and variable amplitude test.
**Stiffener to crossbeam connection.** The crossbeam in an orthotropic steel deck is loaded by bending, shear and the local introduction of external forces (in plane behaviour) as well as by ‘out of plane bending’ caused by rotation of the longitudinal stiffener under traffic loading (Leendertz, 1995).

The ‘in plane behaviour’ of the crossbeam is strongly affected by the cut outs and additional cope holes in the crossbeam which results in Vierendeel effects with respect to the transmission of shear forces. Numerical as well as experimental work of the ‘in plane behaviour’ of the stiffener to crossbeam connection to be applied in a crossbeam in road bridges showed a reduction of the stress concentration at the edges of the cope holes (see e.g. Figure 8) by increasing the notch radius of the cope hole. Several types of cut outs and types of stiffeners have been studied by Finite Element analysis (Lehrke, 1990; Bruls, 1995).

It was concluded that an optimal shape of the cut out (including a cope hole) is such that the cutted area is minimum, and the radius of the free edge is maximum. Therefore, the optimal shape of the cope hole (if it is applied) seems, at present to be circular, see Figure 12.

The welding procedure and fatigue strength of the point where the cut out meets the stiffener web in combination with a traditional cope hole (Figure 8) and with a circular cope hole (Figure 12) has been tested. It was found that the fatigue strength of that point for both types is nearly the same.

The ‘out of plane behaviour’ was studied for triangular stiffeners as well as for trapezoidal stiffeners see Figure 13. The connection with continuous triangular stiffeners provided with a cope hole appeared to be inherently stronger for fatigue than those without a cope hole. For the trapezoidal stiffeners just the opposite was found (Cuninghame, 1990; Kolstein 1995).

A full-scale crossbeam in a test rig, which simulate passing of axles that do deflect the orthotropic deck (in-plane behaviour) and causes rotations in the supports of the deck (out of plane behaviour), i.e. the connections of the stiffeners to the crossbeam, see Figure 14 (Kolstein, 1995; Leendertz, 1995-2). In the crossbeam two types of connections have been tested. For both types the stiffener passes the crossbeam through a cut out in the cross beam. Stiffeners with cope holes and stiffeners being fully welded around, see Figure 15. The latter type results in a longer fatigue life.

**Statistical Analyses of Fatigue Data and Joint Classification**

**Procedure.** The statistical analysis as described in Eurocode 3 (prEN 1993-1-9, 2002) gave a design S-N line based on the lower 95% confidence limit (5% probability of failure). This implies that 5% of a population of welds would be expected to fail before the end of the design life. This procedure is only intended to be applied if at least 12 test results are available. In some cases the analysis of data for orthotropic steel decks is based on a limited sample of test results for each type and category of joint. In this case the data is assumed to follow a Students t-distribution instead of a normal distribution.
**Definition of stress.** All the fatigue data analyzed in this paper were based on strain measurements from gauges close to and at 90° to the weld. Ideally, a design stress is required, which can be obtained consistently from experimental data to establish a S-N curve, and which designers can easily be calculated. However it is difficult to calculate a stress at a welded joint in an orthotropic deck accurately. Designers usually calculate global stresses, but the fatigue strength of a connection depends on the local stress at the weld. For all details the fatigue strength classification closest to the S-N line in terms of measured stress is given in Table 1 to 5, but it should be remembered that this applies to the research data. For the moment it was concluded that the classification in terms of nominal stress couldn’t yet be given for all studied constructional details in a proper way.

**Design Recommendations with Respect to Fatigue**

**Stiffener to deck joint.** To avoid cracks at the weld toe in the deck plate, it is strongly recommended that 12 mm (in combination with a deck surfacing ≥ 70 mm) should be regarded as the minimum deck plate thickness. Gaps between the stiffener and the deck plate greater than 0.5 mm should be avoided (a gap of up to 1.0 mm may be acceptable if a suitable weld process is used to give good penetration and no root defects). Weld throat thickness must be ≥ 1.1 times the thickness of the stiffener. Weld procedure trials should be carried out to ensure that the required weld profile is produced consistently. It should be noted that some tolerance must be allowed on the amount of penetration. It is suggested that a nominal value of 80% and a minimum of 50% be specified. In addition automatic welding should be used where possible as this was found to give higher fatigue strength than a manual weld.

The behaviour of the stiffener to deck plate joint can be improved when during automatic welding the stiffener is pressed to the deck plate. After welding the edge of the stiffener remains under prestressing due to shrinkage effects. As a consequence the connection is able to take moments with a relative large inner lever arm. In this way the influence of stress cycles caused by the traffic is reduced and the fatigue life is affected in a positive way.

**Stiffener splice joint.** Fillet welded splice joints are not recommended, except for very lightly loaded structures. So called ‘double lap’ splice joints with fitted splice plates can give good fatigue strength and may be suitable for some special applications. Butt welds must be full penetration and be free from root defects. A weld made from both sides gives good fatigue strength. Where access is from one side only, a backing strip should be used (6 mm thick, width ≥ 30 mm). Tack welds should be of the same quality as the butt weld and contained within it (i.e. no external tack welds). There is clear evidence that a root gap of at least 6 mm is essential and it is recommended that the gap should be equal to or greater than the stiffener plate thickness. This provides access for a good quality root run. Butt welds without backing made from one side only can have a fatigue strength at least as good as a weld on a backing strip. However, a carefully controlled procedure and a team of very skilled welders are required. The length of the splice plate must be ≥ 150 mm. Misalignment between the stiffener and the splice plate and fit-up of the gap with the
backing must be $\leq 1$ mm. MIG/MAG welding is preferred with 50% inspection or MMAW with 100% inspection. Special attention has to be paid at the start and stops of the weld. Special requirement must be made for several straight passes, not using weaving techniques.

At the stiffener splice joint the edge of the total splice plate (at the deck plate side) and the weld length of the connecting stiffener (100 – 200 mm) must be shaped to perform a good weld between the deck plate and the splice plate. Further requirements have been described in previous section.

**Stiffener to cross beam connection.** Stiffeners fitted between crossbeams should only be used where: (i) the overall depth of the deck must be minimized, (ii) traffic loading is very light, i.e. very few heavy vehicles, (iii) for lightly loaded short life structures, e.g. temporary bridges. The connection of the joint must always be made with full penetration welds and 100% inspection is recommended. Also, the stiffeners must be accurately positioned opposite one another on each side of the crossbeam. Any misalignment would cause local bending stresses and reduce the fatigue life of the connection.

Long stiffeners passing through cutouts in the crossbeam may be used with or without a cope hole around the soffit of the stiffener. Cope holes should only be used where an adequate depth of the crossbeam can be provided, e.g. in box girder bridges where the crossbeam is part of a full depth diaphragm. There is some evidence that it is preferable not to have a cope hole even in this case.

An important aspect is the phenomena that the crossbeam is subjected to local and global shear stresses due to wheel loads in the adjacent bays of the deck. Depending on the crossbeam spacing, the load can be substantial. The critical areas for fatigue are adjacent to the points where the crossbeam is connected to the stiffener web, or at the edge of the cope hole. The global shear must be calculated using e.g. a Vierendeel truss analogy, but the calculation of local shear stress in the crossbeam at the edge of the cope hole requires a FE analysis. The crossbeam is also subjected to out of plane bending, which can cause cracks at weld toes in the crossbeam. Therefore, then crossbeam should not be too stiff in the longitudinal direction; any stiffening on the crossbeam should not be placed close to the longitudinal stiffener soffit.

If cope holes are used, special attention must be given to providing a smooth edge to the cope holes. Any notches are to be ground. Welds to be turned around the edges of the cope holes. The weld throat thickness must be $\geq 50\%$ of the diaphragm plate thickness. Special attention must be paid to undercuts. The joint without cope holes can be realized with fillet welds with a throat thickness $\geq 50\%$ of the crossbeam thickness. Starts and stops at the cold-formed parts of the stiffener must be avoided. For both types of connection 50% of the weld must be inspected.

In all types of connection, a cope hole at the deck plate weld decreases fatigue life and must be avoided.

**Deck plate splices.** Deck plate splice joints are subjected to high stresses due to local wheel loading and so good fatigue strength is essential. For this reason full penetration butt welds are always used. Only one set of tests is available for this joint
in orthotropic bridge decks, consisting of two tests each on transverse and longitudinal welds made with a backing strip. They gave fatigue strength far above the standard classifications.

Most joints can be made in the fabrication shop, but site connections are also required during assembly of large bridges. Where possible, welds should be made from both sides. Where a backing strip is used, there should be no external tack welds and careful attention should be paid to the weld quality, particular to prevent crack-like defects, e.g. lack of penetration or lack of fusion. As with any butt-welded joint, misalignment of the plates must be controlled to achieve good fatigue strength. On some types of bridges, e.g. box girders; failure of a transverse deck plate weld could lead to structural failure. Therefore more attention should be paid to the quality of these welds and they should be made from both sides wherever possible.

**Crossbeam and main girder to deck plate connection.** Good fatigue strength can be achieved by good quality double fillet weld of adequate throat thickness. Single fillet welds should not be used. Intermittent welds are not recommended for the same reason. The gap between the connected plates must \( \leq 1 \) mm. If possible the location of the web to be located outside the wheel tracks.

**Concluding remarks**

In some cases, very high fatigue strength was obtained for welded details in the orthotropic steel bridge deck structure with closed longitudinal stiffeners. However, several factors may reduce their fatigue strength, such as the difficulty of maintaining tolerances on fit-up and weld quality on a large fabrication, especially for site welds. The permitted defect level for the highest weld classes is very low and it seems unlikely that such a low level of defects could be guaranteed for all welds on a large bridge. The level of inspection commonly used in bridge works is limited by the cost of close inspection of a large numbers of welds.

The fatigue design of most joints in an orthotropic deck is straightforward, with exception of the longitudinal stiffener to crossbeam joint where the classification is very complex and not fully solved.

The new requirements for fatigue assessment in the Eurocodes will not focus on fatigue cracking of load bearing components but also give guidance on the procedure how possible fatigue damage can be avoided. An important missing part in the fatigue assessment until now is a simple method of stress analysis for all parts of the orthotropic deck. A proper fatigue analysis of an orthotropic steel deck is rather time consuming but based on parametric sensitivity studies which are currently carried out, the procedure can be simplified for design by classifications.

The composite action between the steel deck plate of orthotropic bridge decks and the surfacing is an important aspects of the performance, in particular because the stiffness of the combined unit helps to reduce strains in the welded structure as well as in the surfacing, which results in a longer life time (Kolstein, 1989, 1997). Since the stiffness behaviour of the bituminous surfacing is strongly influenced by temperature, loading frequencies and composition of the total surfacing, the stress
reducing effect in the steel components cannot yet easily be described and included in
design rules. Without this effect the calculated design life of the orthotropic decks
would be considerably underestimated.

Experiences with existing bridges demonstrate that their durability is
restricted, mainly due to shortcomings in design and execution (Kolstein, 1998; Jong
de, 2002). In future there may be more repair work and replacement of old bridges.
Development of a strategy, concerning the evaluation of damages and repair methods,
will be of a high priority.

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Additional Information


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Figure 1. Basic types of orthotropic steel bridge decks

(a) Deck with open stiffeners

(b) Deck with closed stiffeners

Figure 2. Local wheel loading effects – stiffener to deck joint (Bruls, 1995)
Figure 3. Longitudinal stiffener splice joint

Figure 4. Basis types of stiffener to crossbeam joints

Figure 5. Fatigue failure stiffener to deck joint (Gurney, 1992)
Figure 6. Measured stress spectra stiffener deck plate connection – 60 mm surfacing (Kolstein, 1997)

Figure 7. Butt-welded stiffener splice joint with insufficient root gap (Kolstein, 1990)

Figure 8. Stress distributions along the edge of a cope hole (in plane behaviour) (Lehrke, 1990)
Figure 9. Fatigue tests stiffener to deck joint (Bigonnet, 1990; Bruls, 1990)

Figure 10. Improved stiffener to deck joint (Bigonnet, 1990)
Figure 11. Fatigue test results stiffener splice joint (Caramelli, 1990; Kolstein, 1990)
Figure 12. Improved shape of cope hole (Bruls, 1995)

Figure 13. Out of plane bending test (Kolstein, 1995)
Figure 14. Combination of “out of plane” and “in plane” testing (Kolstein, 1995)

Figure 15. Typical stress distributions of stiffener crossbeam details (Leendertz, 1995)
Table 1. Fatigue Classification Stiffener Splice Joint

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Table 2. Fatigue Classification Stiffener to Deck Joint

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<td>1 0.5mm&lt;gap&lt;2mm</td>
<td>16</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>2 gap&lt;0.5mm manual weld</td>
<td>26</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>3 gap&gt;0.5mm autom. weld</td>
<td>13</td>
<td>90</td>
<td></td>
</tr>
<tr>
<td>throat thickness &gt;6mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress in the deck plate. (at the weld toe in the deck plate)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>125</td>
<td>nominal stress in the deck plate at the weld toe</td>
<td>125</td>
</tr>
</tbody>
</table>

Table 3. Fatigue Classification Stiffener to Crossbeam Joint - Stiffener fitted between crossbeams

<table>
<thead>
<tr>
<th>CONSTRUCTIONAL DETAILS</th>
<th>DESCRIPTION</th>
<th>RESEARCH DATA</th>
<th>DESIGN RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Measured stress</td>
<td>stress to be used</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No of tests</td>
<td>Class</td>
</tr>
<tr>
<td>Fillet Welded Joint</td>
<td>42</td>
<td>45</td>
<td>nominal stress 15mm from the root of the weld</td>
</tr>
<tr>
<td>Butt Welded Joint with good penetration and no root defects</td>
<td>16</td>
<td>56</td>
<td></td>
</tr>
</tbody>
</table>
### Table 4. Fatigue Classification Stiffener to Crossbeam Joint - Continuous stiffener

<table>
<thead>
<tr>
<th>CONSTRUCTIONAL DETAILS</th>
<th>DESCRIPTION</th>
<th>RESEARCH DATA Measured stress</th>
<th>DESIGN RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No of tests</td>
<td>Class</td>
</tr>
<tr>
<td>Welded All Around</td>
<td>V-shaped stiffener</td>
<td>8</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>Trapezoidal stiffener</td>
<td>4</td>
<td>&gt;80</td>
</tr>
<tr>
<td>Cope Hole at Soffit</td>
<td>loc.1 crack along weld toe</td>
<td>10</td>
<td>125</td>
</tr>
<tr>
<td></td>
<td>loc.2 crack in cope hole</td>
<td>5</td>
<td>125</td>
</tr>
<tr>
<td></td>
<td>loc.3 crack in cope hole</td>
<td>7</td>
<td>125</td>
</tr>
<tr>
<td></td>
<td>loc.4 crack at end of weld</td>
<td>11</td>
<td>112</td>
</tr>
<tr>
<td>Cope hole at deck plate</td>
<td>crack at end of weld</td>
<td>10</td>
<td>45</td>
</tr>
</tbody>
</table>

### Table 5. Fatigue Classification Crossbeam or Web to Deck Joint

<table>
<thead>
<tr>
<th>CONSTRUCTIONAL DETAILS</th>
<th>DESCRIPTION</th>
<th>RESEARCH DATA Measured stress</th>
<th>DESIGN RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No of tests</td>
<td>Class</td>
</tr>
<tr>
<td>Double Fillet Welded Joint:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Stress in deck plate</td>
<td></td>
<td>38</td>
<td>112</td>
</tr>
<tr>
<td>2 Stress in crossbeam or web</td>
<td></td>
<td>23</td>
<td>125</td>
</tr>
</tbody>
</table>