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Fatigue Crack Locations in Orthotropic Steel Decks

Localisation des fissures de fatigue sur les tabliers de ponts métalliques

Ermüdungsrissbildung in orthotropen Stahlbrückenfahrbahnen

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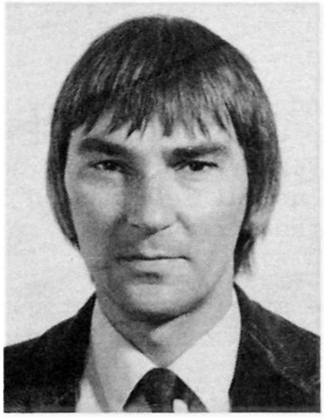

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SUMMARY

Crack locations in orthotropic steel bridge decks are discussed with reference to the main types of welded joint. Instances of cracking in service are given and assessment and strengthening methods described. Joints discussed include two types of web stiffener to deck, web to deck, longitudinal stiffener to crossbeam and to deck, and stiffener splice joints.

RÉSUMÉ

La localisation des fissures sur les tabliers de ponts métalliques orthotropiques est discutée en relation avec les principaux types de soudure. Des exemples de fissures en service sont donnés et les méthodes d'évaluation et de renforcement décrites. Les soudures discutées comprennent deux types de raidisseurs d'âme à la tôle de platelage; de raidisseur longitudinal à la pièce de pont et au tablier, et d'entures de renfort.

ZUSAMMENFASSUNG

Die Orte, an denen in den orthotropen Fahrbahnplatten von Stahlbrücken Risse entstehen können, werden für die häufigsten Arten von Schweissverbindungen diskutiert. Beispiele solcher Rissbildung unter Gebrauchslasten sind zusammen mit den Beurteilungs- und Verstärkungsmethoden beschrieben. Behandelt werden zwei Arten von Verbindungen zwischen Stegsteifen und Fahrbahnblech, zwischen Steg und Fahrbahnblech, Anschlüsse von Längsteifen an Querträger und Fahrbahnblech, sowie Steifenstösse.

1. INTRODUCTION

Orthotropic steel decks are used for their high strength and light weight. Efficient designs use thin plate, for example, some existing bridges have deck plates of 12mm or less and therefore high dynamic stresses occur under traffic wheel loads. Joints attaching stiffeners to the deck plate are frequently made with fillet welds, and so joints close to the wheel tracks of heavy goods vehicles (HGV's) may be prone to fatigue cracking. Cracks have occurred in service in most countries where this type of deck is used, including the UK, France, Germany, Belgium, Holland, Japan and other countries where there has been a large increase in traffic loading in the past two decades [1,2,3,4,5,6].

For new designs the trend is to increase the thickness of the deck plate to 14mm and make greater use of full penetration welds, but there is a need for guidance for design engineers, and existing bridges have to be assessed and may need to be strengthened.

When cracks are found it is frequently necessary to modify the joint to prevent cracks recurring and to modify similar joints to prevent cracking elsewhere. To develop a suitable repair it is first necessary to assess the joint as built. This process yields useful information for designing new decks and for assessing similar joints on other bridges.

With the aid of case studies and fatigue test data, this paper illustrates some of the locations at which cracks can occur.

2. TROUGH TO CROSSBEAM JOINTS

2.1 Short troughs fitted between crossbeams

Decks made in this way are used in a number of bridges, particularly those built before facilities for forming troughs in long lengths were readily available.

In this design longitudinal stiffeners (troughs) were formed in lengths to fit between crossbeams. During fabrication an end load was applied to force the crossbeams into close contact with the trough ends. A fillet weld was then made around the end of the trough.

This joint was first investigated in the UK by Nunn [7]. Cracks occurred in a trial deck panel. Mehue [2] reported cracks in a similar joint in the decks of flyovers and cracks subsequently occurred in a UK bridge [1]. In the course of developing a repair for these cracks, load tests were carried out on a number of trough to crossbeam joints both on the bridge and on trial deck panels in the laboratory. Figure 1 shows typical influence lines of stress under a single wheel load. Measured strains varied widely between individual joints but were not related to weld size. Fatigue cracks occurred at random along the length of the bridge but, as expected, mainly in joints transversely positioned to be under the wheel tracks.

Fatigue tests were carried out on a deck panel by loading alternately at two positions over the trough on either side of the crossbeam, coinciding with the stress peaks on the influence line. Small (1500 x 600mm) specimens were also tested.

Most cracks, both in the fatigue tests and on the bridge, started at the weld root on the web of the trough in a region of tensile residual stress, and grew through the weld throat with the fracture surface parallel to the crossbeam. Fig 2 shows a crack just before it reached the weld surface and became visible.

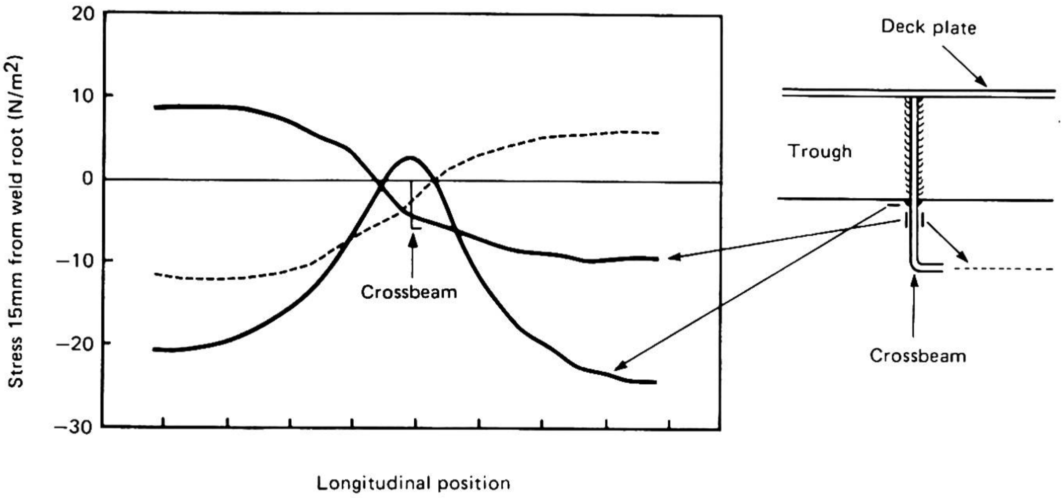

Fig.1 Stress influence lines for trough to crossbeam joint

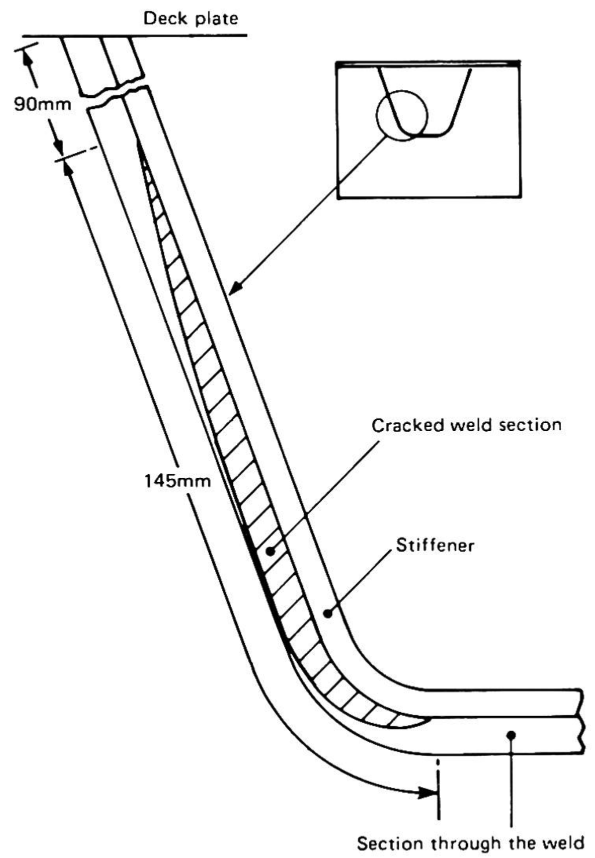

Fig.2 Crack location in trough to crossbeam weld

The stress distribution around the trough to crossbeam joint is complex and the stress in the weld depends on the geometry and fit up of the trough to the crossbeam. Since the applied stress in the trough adjacent to the weld is entirely compressive, the rate of fatigue crack growth depends on the weld stress cycle which, in turn, depends on the residual stress.

2.2 Long troughs passing through crossbeams

Most orthotropic bridge decks are now made with longitudinal stiffeners which are formed in long (up to 16m) lengths, and pass through cut-outs in the crossbeams. This joint is inherently stronger than the old type and fabrication problems such as misalignment of the troughs on either side of the crossbeam are avoided. The new joint has been investigated by Beales [8] in a research programme partly funded by the European Coal and Steel Community (ECSC). It was found that the fatigue strength was higher than the old type, but could be reduced considerably by cracking at an unexpected location.

Three designs of the new type of joint were investigated. Stress influence lines were obtained from static tests on a deck panel under a single wheel load. Fatigue lives were then calculated using the methods of the UK bridge design standard, BS 5400 part 10 [9]. The constant amplitude fatigue strength was determined by fatigue tests on specimens comprising a section of deck 1.5m long by 600mm wide containing a full size trough to crossbeam joint. Specimens were loaded in three point bending to reproduce the stresses in the trough and crossbeam obtained in the deck panel tests.

In two of the designs, the trough to crossbeam weld stops short of the deck plate and a cope hole is left in the crossbeam, to bridge the trough to deck plate weld. This avoids three welds meeting at a point. However, when these joints were fatigue tested cracks occurred in the trough and crossbeam at the upper end of the trough to crossbeam weld, ie at the edge of the cope hole.

The crack locations and test results are shown in figure 3. Cracks also occurred at the lower end of the weld but at much longer endurances. The fatigue strength of the cope hole detail is consistent with BS 5400 class G or Eurocode class 50 [10]. The estimated life under design traffic loading is less than 5 years. Without the cope hole detail, the fatigue strength (for cracking at the weld toe at the lower end of the weld) is much higher than expected and is consistent with BS 5400 class C, or Eurocode class 125. The design life of the joint is then greater than the 120 years required by the UK design standard.

3. VERTICAL STIFFENER JOINTS

Cases of fatigue cracking at the top of vertical web stiffeners on plate girders have been reported [11,12]. Box girder bridges with orthotropic decks also require vertical stiffeners, on both longitudinal webs and on transverse diaphragms (crossbeams). Whether the stiffener is attached to the deck plate or not there is a risk of fatigue at the top of the stiffener due to local wheel loading. Two such joints have been assessed by means of static load tests and calculations of service life.

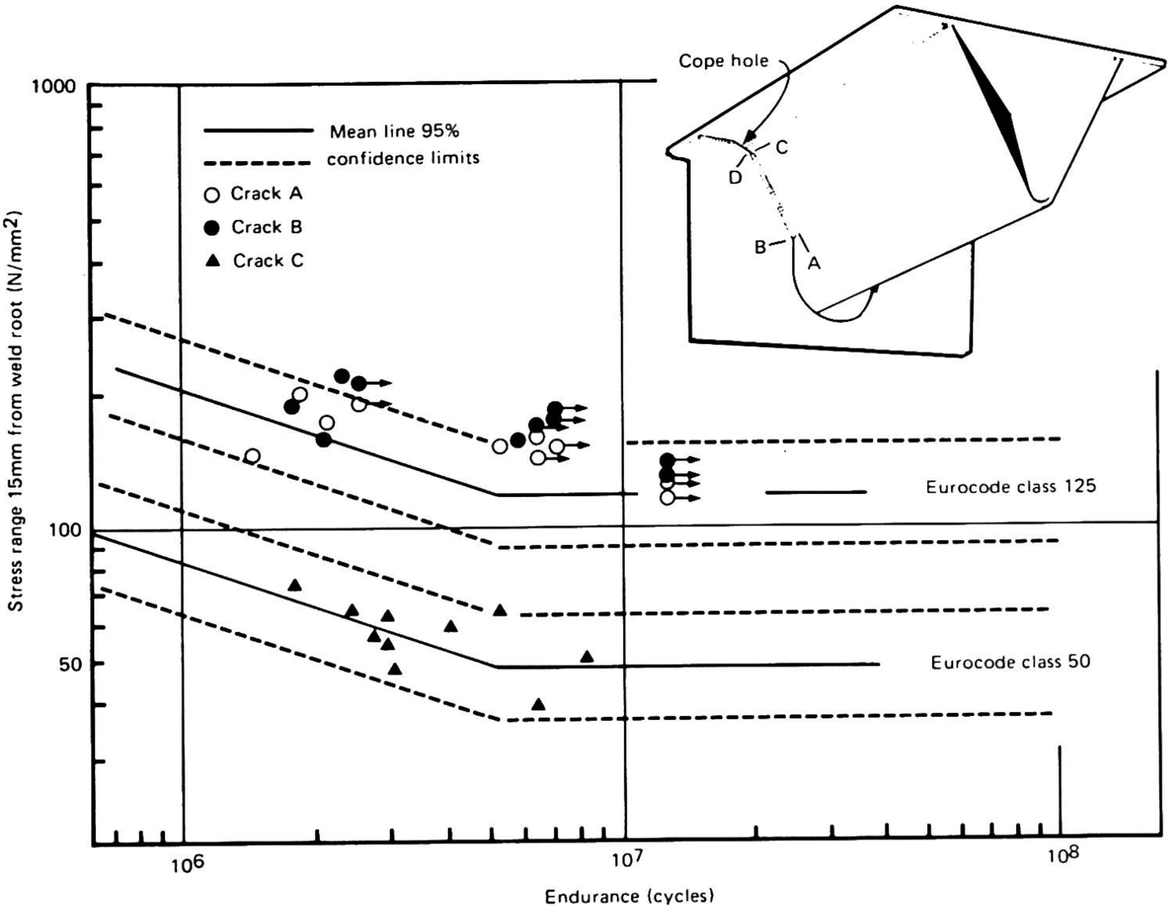

Fig.3 Crack locations and test results for trough to crossbeam joint

3.1 Web stiffener to deck joint

Cracks occurred at the joint shown in figure 4 under the nearside lane of a box girder bridge after 9 years in service.

In order to assess the joint and develop a repair, load tests were carried out on an uncracked joint. An area of the asphalt surfacing 8m long by 2m wide was removed and loads were applied by a two axle test vehicle. As the influence lines are short, strains due to a single front wheel could be measured directly. Single gauges were attached close to the weld toe at each potential crack location, and stress influence lines were calculated from the measured strains. The weld was then removed so that the stiffener was no longer attached to the deck plate and the load test repeated.

The stiffener was then cut down, first to 50mm from the deck plate, then by a further 50mm to 100mm from the deck plate, see figure 4. The potential crack locations were then (i) the weld end at the top of the stiffener and (ii) the web to deck weld. The adjacent deck plate butt weld was also checked. Gauges were attached at all these locations and the load tests repeated.

Fatigue lives were estimated for each gauge location for the as-built and modified joint. 'Design' calculation methods were used as given in the design standard [9]. It was not possible to carry out fatigue tests so the joints were assigned to BS 5400 weld classes based on experience and the guidance in the standard. The following conditions were included.

- The traffic loading was taken from BS 5400 part 10 table 11.
- The transverse distribution of traffic (from BS 5400 part 10 figure 17) was centred on the middle of the traffic lane.
- The lives are for a 2.3% probability of failure.
- The lives are for a traffic flow of one million HGV's per year.
- No allowance was made for the composite action of the surfacing.

The estimated fatigue lives are given in table 1. The life of the as-built joint is 3 years. If the life is recalculated for the actual traffic flow (half a million HGV's per year) and the 30% of welds known to have cracked, the resulting 11 years is in good agreement with the actual life of 9 years.

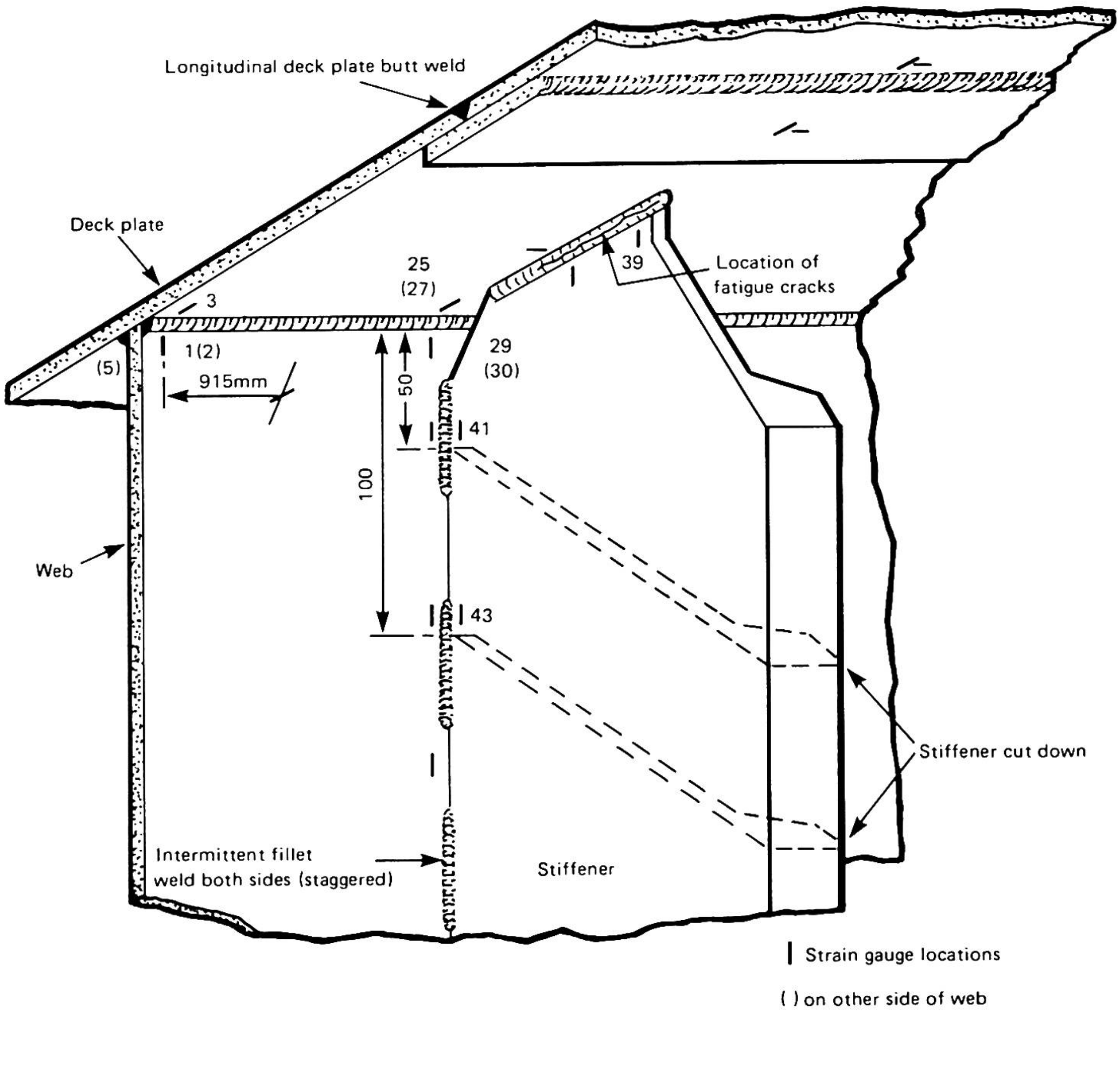

Fig.4 Web stiffener to deck joint

Gauge No	Location	Weld class	As built	Fatigue life (years) for joint: Stiffener to deck weld removed	Stiffener cut down to 50mm from deck	Stiffener cut down to 100mm from deck
39	Stiffener to deck plate joint	G	3			
25	Web to deck joint adjacent to stiffener	F	1112	859	24	21
27			13	13	26	24
29			866	651	78	206
30			55	47	95	159
41	Web plate at top of cut down stiffener	E			41	
43						989
1	Web to deck joint remote from stiffener	F	541			
2			363			
3			30			
5			23			
14	Longitudinal deck plate butt weld	F			137	

Table 1 Estimated fatigue lives for web stiffener joints

With the weld removed, the weakest point is the web to deck joint above the stiffener (13 years). Cutting the stiffener down to 50mm from the deck plate increased this to 24 years, which is similar to the life of the web to deck joint remote from the vertical stiffener. Cutting the stiffener down to 100mm from the deck plate did not increase the life further.

BS 5400 part 3 (clause 9.13.1) gives guidance on web stiffener to flange connections. A gap of up to five times the web plate thickness is permitted. In this case the web plate is 10mm thick; hence a gap of 50mm is permissible.

The estimated life of the weld end at the top of the stiffener with a 50mm gap was 41 years. This is less than the required life (120 years), but on this particular bridge the traffic is approximately half of the design traffic loading. It was therefore considered preferable to remain within the permitted gap length (ie 50mm), and the joints on the bridge have been modified accordingly.

Regarding the effect on other joints, the life of the web to deck joint is improved from 13 to 26 years, but this is still less than the required life. However, recent fatigue data [13] suggests that the classification should be D rather than F and, as noted above, the traffic on the bridge is less than assumed in the calculations. It was therefore felt that cracking of this joint in service is unlikely.

The deck plate butt weld is not under the vehicle wheel tracks, and the estimated life was satisfactory (137 years). The stress in the apex of the two adjacent troughs was also checked and was unaffected by the modifications to the stiffener.

3.2 Crossbeam stiffener

A similar detail on another bridge was investigated as part of an overall fatigue assessment of the deck. This bridge is also a box girder with the top flange of the box consisting of an orthotropic deck carrying the roadway. The deck is wider than the box so that part of the nearside lane, as well as a cycleway and footpath, is supported by transverse beams cantilevered out from the box at 4.2m intervals along the length of the bridge.

The webs of these beams are stiffened by two vertical stiffeners, figure 5 shows the arrangement. In this case a nominal 25mm gap was left between the top of the stiffener and the deck plate. A survey showed that the actual gap varied between 15mm and 40mm.

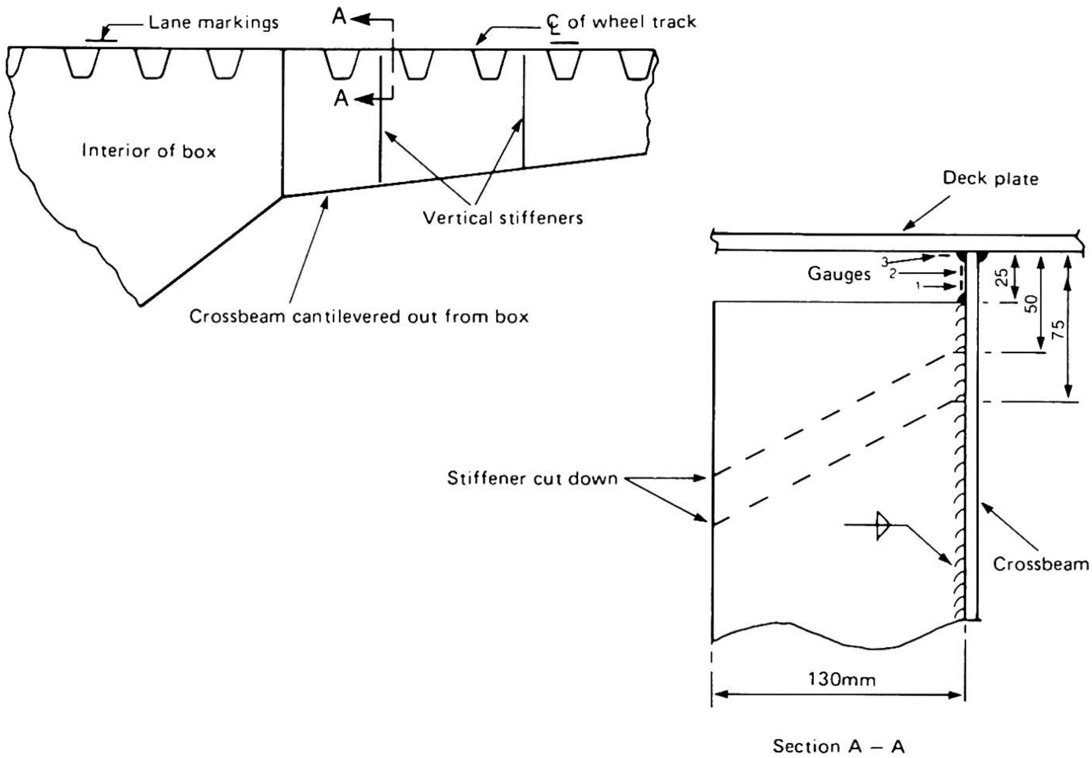

Fig.5 Crossbeam stiffener

Loading tests were carried out with the surfacing removed using the same, two axle, test vehicle. Strains were measured on the crossbeam web at the top of a stiffener and at the crossbeam to deck plate weld directly above, see figure 5.

The stress data was used to estimate the fatigue life of the joint by the same methods as described above. The highest stress occurred at the end of the stiffener to crossbeam weld between the top of the stiffener and the deck plate (gauge position 1, see figure 5).

The lowest estimated fatigue life of 3 years is similar to that of the web stiffener to deck plate weld described in section 3.1, but no fatigue cracks have been found in this joint although the bridge is older and more heavily trafficked. Part of the reason for this is the effect of the transverse position of the traffic relative to the joint, see figure 5. With the present position of traffic lanes on the bridge, the joint is 450mm from the centre of the vehicle wheel track. It is calculated that this will increase the life to 12 years.

Some improvement was thought necessary, so tests were carried out on stiffeners modified in a similar way to those described above, ie by increasing the gap between the top of the stiffener and the deck plate, see figure 5. This time it was not possible to remove the surfacing so dynamic tests were carried out on the surfaced deck, with the same vehicle as in the static tests.

Strains were measured at a number of locations, the effects were similar at all of them. The results are summarised in table 2.

Location (refer to fig 5)	Type of surfacing *	Gap at top of stiffener	Vehicle speed (Km/hr)	Asphalt temp. (^{o}C)	Stress (N/mm^2) Max	Min	Range
	none	23	static		+48	-92	140
	RB+MA	25	13	15	+4	-18	22
	RB+MA	25	31	15	+4	-12	16
1	RB+MA	25	45	15	+4	-13	17
	RB+MA	50	11	15	+3	-29	32
	RB+MA	50	31	15	+5	-25	30
	RB+MA	50	45	15	+5	-22	27
	RB+MA	75	11	18	+3	-29	32
	RB+MA	75	33	18	+4	-26	30
	RB+MA	75	46	18	+5	-25	30
	None	23	static		+51	-75	126
2	RB+MA	25	13	15	0	-10	10
	RB+MA	50	11	15	0	-17	17
	RB+MA	75	11	18	0	-20	20
	None	23	static		+1	-59	60
3	RB+MA	25	13	15	-1	-9	8
	RB+MA	50	11	15	-1	-16	15
	RB+MA	75	11	18	-1	-19	18

* RB+MA is rubber bitumen (3mm thick) + mastic asphalt (35mm thick)

Table 2 Peak stresses at crossbeam stiffener joint for 31.4kN wheel load

It will be seen that for this joint there is no benefit from increasing the gap at the top of the stiffener, in fact stress ranges are increased by about 50%. However, all the stresses were considerably reduced by the surfacing.

It was concluded that there was no advantage in modifying this joint by cutting down the stiffener. However, fatigue cracking is unlikely to occur provided the surfacing remains effective (the effect of surfacing is discussed further in section 8).

4. TROUGH TO DECK WELDS

The trough to deck joint is perhaps the most important in the deck because the length of weld is greater than for other joints. A four lane wide deck may contain 70 metres of trough to deck weld per metre length of the bridge. On a long span bridge this may amount to 150Km of weld. Therefore the most economical fabrication method must be used consistent with adequate performance.

The edge of the trough may be cut square, or machined to fit closely to the deck; fatigue tests have been carried out on both types [14,15,16]. Janss compared the results for square edged and machined troughs and found no significant difference provided the gap between trough and deck plate did not exceed 0.5mm. Gurney and Maddox showed that increasing the size of the fillet weld, or making a partial penetration weld increased the fatigue strength. For a typical deck with 12mm thick deck plate and 6mm thick troughs, the fatigue strength is equivalent to BS 5400 part 10 class F for a 6mm fillet weld, and class D or greater for a 9mm fillet or a penetration weld.

Cracks have occurred on a UK bridge with 6mm fillet trough to deck welds [1]. Cracking began at the weld root and propagated to the surface on a broad front. A section of a trough web was removed for examination of the fracture surface. It was found that subsurface cracks extended well beyond the end of the visible crack, and there were other cracks further along the weld which were not visible at all. Ultrasonic test equipment with specially developed twin probes enabled sub surface cracks to be detected when they extended halfway to the surface. Because of the difficulty of determining the extent of cracking, it was assumed when making repairs that the full length of the weld between crossbeams was cracked, once a visible crack was present anywhere in that length.

Cracks occurred mainly under the vehicle wheel tracks in the nearside lanes of both carriageways but were distributed randomly along the length of the bridge. All the joints under the wheel tracks were strengthened by machining an edge preparation on the trough web (removing the fillet weld in the same operation) and rewelding with a three pass semi automatic weld. Laboratory tests showed that the fatigue life of the modified joint will be adequate provided the weld throat thickness is at least 7mm, with 2mm penetration of the trough web.

There may be a case for designing decks for new highway bridges with more fatigue resistant welds under the vehicle wheel tracks only, to reduce fabrication costs. The British design code requires fatigue life to be calculated for traffic centred within ±300mm of the centre of the traffic lane. However, any cost saving would have to be judged against the possible future need to move the position of the traffic lanes. Alternatively, it may be possible to obtain satisfactory weld penetration without expensive machining of the trough web.

5. WEB TO DECK WELDS

Steel box girder bridges frequently have longitudinal webs attached to the deck plate under the nearside traffic lane, ie close to the heaviest wheel loading. The transverse influence line for stress at the web to deck weld is fairly short so that fatigue life is highly dependent on the transverse position of the traffic.

Tests on Tee shaped specimens similar to the web to deck joint with a double fillet weld [13] showed that the fatigue strength is equivalent to BS 5400 class D. Cracks generally occurred at the weld toe in the deck where fit-up was good; and at the weld toe in the web in specimens with a gap of 2mm between web and deck.

Poor fit-up is most likely to occur near the vertical joint between two web plates, where the top edges of adjacent web plates may be at slightly different heights.

6. TROUGH SPLICE WELDS

Trough splices are required as a consequence of the improved trough to crossbeam detail, with troughs passing through the crossbeams. Splice joints are made during erection of the bridge, so they are affected by the quality of the site welding. A number of studies have been carried out of the fatigue performance of butt welded splice joints [17,18], and the effect of weld defects [19].

It was shown that for V-shaped troughs, the distribution of residual stresses around the trough leads to cracking in the web rather than the more highly stressed soffit of the trough. The same pattern occurs in trapezoidal troughs.

Fillet welded splice joints have also been tested [13]. They may be easier to fabricate, but fatigue strength is lower. Joints with a single cover plate fitted over a V-shaped trough cracked through the weld throat on the trough web. Joints with plates fitted both inside and outside the trough cracked at the weld toe on the trough, but still at lower fatigue strength than the butt welded splice. Cracks have occurred in service in fillet welded joints in a deck with trapezoidal troughs [20].

7. SOME OTHER CRITICAL LOCATIONS

Fatigue cracks have occurred at other locations, at joints which may not have been assessed in the original design, for example, temporary attachments not properly removed after use. An example of this occurred at the fillet welds attaching temporary diaphragms to the soffit of longitudinal trough stiffeners [1]. The diaphragms prevented deflection of the troughs under traffic, leading to high stresses at the weld toes. A partial solution was to disconnect the diaphragms from the troughs. More permanent repairs were made by cutting out the cracked area and fitting a bolted splice. Similar examples have been reported elsewhere [21]. Anticipating and avoiding problems of this type is largely a matter of education, and effective communication between designer, fabricator and erector.

8. EFFECT OF SURFACING

The reduction of stress at welded joints in the deck due to composite action between the surfacing and the steel deck is well established [22,23,24].

It is generally assumed that the reduction is less for joints further from the deck plate. However, measurements on the soffit of the trough at several trough to crossbeam joints on a bridge indicated a reduction in stress sufficient to give an increase in fatigue life by a factor of at least three.

It is clear from the tests on the crossbeam stiffener (section 3.2) that composite action gives a very large reduction in stress at the joint. The reason that cracking was not prevented at the web stiffener to deck plate joint (section 3.1), is that there is a tendency for the surfacing to crack along the line of the web. It is common practice to 'regulate' this cracking by sawcutting the surfacing over the web and filling the slot with a flexible sealant. The result is a significant loss of composite action for both the stiffener joint and the web to deck joint.

Cracking of the surfacing can occur over the crossbeam, but the deflection of the deck plate under traffic loading is less, so cracking is less likely.

The stress reduction due to the surfacing depends on many factors, eg the grade and thickness of the asphalt, the temperature of the asphalt and the vehicle speed. The benefit can be lost for a number of reasons such as cracking of the asphalt or poor transfer of stress between the asphalt and the steel deck plate. Therefore the effect is highly variable and the UK design code states that 'this effect should only be taken into account on the evidence of specialist tests or specialist advice'. The aim of the TRRL work has been to strengthen welded joints to provide the required service life without taking account of the surfacing.

9. CONCLUSIONS

Welded joints in orthotropic steel bridge decks are prone to fatigue cracking. It is difficult to predict where cracks may occur because a number of factors affect the formation of cracks.

Whether a particular joint suffers fatigue cracking depends on the applied stresses and the fatigue strength of the joint. Experience of cracks occurring in service suggests that the factors which govern crack location are :-

- The location of the joint relative to the vehicle wheel tracks.
- The extent of composite action of the surfacing and the steel deck which depends on the properties, bonding and condition of the surfacing, and the distance of the weld from the deck.
- The joint geometry and load path.
- The quality of the joint, ie fit-up and weld quality especially of site welds.
- The magnitude and distribution of residual stresses around the joint.

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11. REFERENCES

1. CUNINGHAME J R. Strengthening fatigue prone details in a steel bridge deck. Proceedings of International conference on Fatigue of Welded Structures, The Welding Institute, Abington, 1987.

2. MEHUE P. Cracks in steel orthotropic decks. Proceedings of Bridge management conference (ed. J E Harding, G A R Parke and M J Ryall), Elsevier, London, 1990. ISBN 1-85166-456-4.

3. MEHUE P. Fatigue cracks in dismountable metal viaducts. 13th IABSE Congress, Helsinki, June 1988.

4. YAMADA K. Fatigue assessment of orthotropic decks of box girder bridges. Proceedings of Workshop 'Remaining fatigue life of steel structures', IABSE, Lausanne, 1990.

5. Kolstein H and De Back J. Fatigue behaviour of field welded rib joints in orthotropic steel bridge decks. Proceedings of Workshop 'Remaining fatigue life of steel structures', IABSE, Lausanne, 1990.

6. Wolchuk R. Lessons from weld cracks in orthotropic decks on three European bridges. ASCE Journal of Structural Engineering, 1990.

7. NUNN D E. An investigation into the fatigue of welds in an experimental orthotropic bridge deck panel. Department of the Environment, TRRL Report LR629, Transport and Road Research Laboratory, Crowthorne, 1974.

8. BEALES C. Assessment of trough to crossbeam connections of orthotropic steel bridge decks. Department of Transport, TRRL Research Report RR Transport and Road Research Laboratory, Crowthorne, 1990.

9. BRITISH STANDARDS INSTITUTION. Steel, concrete and composite bridges. Part 10: Code of practice for fatigue. British Standard BS 5400, British Standards Institution, London, 1980.

10. COMMISSION OF THE EUROPEAN COMMUNITIES. Eurocode No. 3: Design of steel structures, part 1. 1989.

11. CHATTERJEE S AND DAWE P H. Fatigue cracking in two steel bridges. Proceedings of International Association of Bridge and Structural Engineers (IABSE) colloquium, Lausanne, 1982.

12. FISHER J W, YEN BT, AND LEE J J. Fatigue cracks at diaphragm connection plates of bridge girder webs. Proceedings of International conference on Fatigue of Welded Structures, The Welding Institute, Abington, 1987.

13. CUNINGHAME J R. Fatigue classification of welded joints in steel orthotropic bridge decks. Department of Transport, TRRL Research Report RR259, Transport and Road Research Laboratory, Crowthorne, 1990.

14. MADDOX S J. The fatigue behaviour of trapezoidal stiffener to deck plate

welds in orthotropic bridge decks. Department of the Environment, TRRL Report SR96UC, Transport and Road Research Laboratory, Crowthorne, 1974.

15. JANSS J. Fatigue of welds in orthotropic bridge deck panels with trapezoidal stiffeners. Journal of Constructional Steel Research, vol 9, pp 147-154, 1988.

16. GURNEY T R AND MADDOX S J. Fatigue tests on joints in orthotropic decks. Proceedings of International conference on Fatigue of Welded Structures, The Welding Institute, Abington, 1987.

17. TROMP W A J. Fatigue of field splices in ribs of orthotropic steel bridge decks. Stevin Laboratory, Department of Civil Engineering Report no 6-74-15, Delft University of Technology, 1974.

18. CUNINGHAME J R. Steel bridge decks: Fatigue performance of joints between longitudinal stiffeners. Department of the Environment, TRRL Report LR1066, Transport and Road Research Laboratory, Crowthorne, 1982.

19. KONDO A, YAMADA K, KIKUCHI Y, MIYAGAWA K AND AOKI H. Fatigue strength of field welded rib joints of orthotropic steel decks. Proceedings of IABSE colloquium, 'Fatigue of steel and concrete structures', IABSE, Lausanne. ISBN 3 85748 030 0, 1982.

20. DARNELL W J, EDMONDS F D, RUTLEDGE J C, YONG P M F. Fatigue problems in the orthotropic deck of the Auckland Harbour Bridge. Pacific structural steel conference, Auckland, New Zealand, 1986.

21. SMITH I F C AND HIRT M A. Fatigue resistant steel bridges. Journal of contructional steel research, vol 12, pages 197-214, 1989.

22. NUNN D E AND MORRIS S A H. Trials of experimental orthotropic bridge deck panels under traffic loading. Department of the Environment, TRRL Report LR627, Transport and Road Research Laboratory, Crowthorne, 1974.

23. MORRIS S A H. Stresses under dynamic wheel loading in a surfaced steel orthotropic deck with V-stiffeners. Department of the Environment, TRRL Report SR237, Transport and Road Research Laboratory, Crowthorne, 1976.

24. CULLIMORE M S G, FLETT I D AND SMITH J W. Flexure of steel bridge deck plate with asphalt surfacing. Proceedings of International Association of Bridge and Structural Engineers IABSE Periodica P-57/83, (1-15), February 1983.