FATIGUE PROBLEMS IN HIGHWAY BRIDGES

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Abstract

During the past few years there has been an increasing incidence of fatigue cracking in the welded joints of modern steel bridges. Many examples have occurred in welds on main girders of short or medium span highway bridges having concrete decks, and in welded joints in orthotropic steel decks of temporary bridges and long span bridges. Design of British bridges is checked for lives of 120 years involving up to 7.10^8 cycles of stress. Calculations to assess fatigue require a realistic loading spectrum, reliable methods to obtain local stresses, and relevant S/N relationships between stresses and numbers of cycles to failure. This paper briefly describes some of the recent research to improve the background to these three steps for design of bridges, particularly those having orthotropic steel decks. The research includes investigation of the influence of surfacing, type of stress cycle, residual stress, plate thickness, cyclic frequency, rest-periods, weathering and consideration of programmed loading.

Highway bridges in Britain are checked for a fatigue life of 120 years during which time up to 7.10^8 potentially damaging major stress cycles may be produced by the passage of commercial vehicles. Until recent years, fatigue was not considered to be an important feature of design because there were comparatively few cases of reported damage. The picture has changed with the build up of information about the performance of welded joints. Such joints involve stress concentrations due to their geometry and contain incipient flaws so that the crack initiation phase of fatigue is almost entirely eliminated. This causes a drastic reduction in fatigue life as compared with plain unwelded components.

For highway bridges, fatigue problems tend to fall into three classes; those concerned with attachments welded to steel girders of structures having intermediate length spans, those concerned with the welded joints in orthotropic steel decks, and components subjected to resonant vibration. The problems can, of course, be exacerbated by poor design detailing and workmanship. For main girders, the stress ranges are due to gross vehicle weight rather than axle weight. There have been a number of reported cases of fatigue cracks of this type in North American bridges (1 to 4) and it has been shown that the lives can be correlated with calculated figures derived from traffic data and measured laboratory performance. Much of this success is due to the work of Fisher and his colleagues at Lehigh University. There have also been several cases in Europe of fatigue cracking in composite bridges having steel girders and concrete decks. There have been instances of fatigue cracking in the welded joints of orthotropic steel decks. This form of construction involves weld details that have relatively poor fatigue resistance and are subject to high stresses due to the local effects of wheel loads at trough-stiffener to cross-beam welds and at trough-stiffener to deck-plate welds. The latter stresses are influenced by the presence of the asphalt wearing surface (5). The earliest recorded cracks occurred in one of two experimental deck panels installed in a busy road at Denham, England (6). One panel was surfaced with 38mm thick mastic asphalt, the other with 9mm thick resin based material. After 4 years, cracks developed in three of the trough-stiffener to cross-beam welds on the panel having the thinner surfacing. After 9 years, additional cracks had developed but there were none in the panel having thicker surfacing. Cracks have developed in orthotropic decks of temporary bridges. Such bridges, although designed for full traffic loading usually have relatively thin surfacing so that high stresses are developed close to the deck plate. In a Swedish temporary bridge, several cracks were found in the trough-stiffener to cross-beam joints. In a German temporary bridge, cracks and failures were found in a number of the trough-stiffener to deck plate joints.

There have also been several instances of cracks in steel orthotropic decks of permanent bridges having normal thickness of surfacings. In all cases the faults have been repaired to a good standard. An interesting situation arose in the Severn Bridge, England, where temporary bulkheads were welded to the ends of the box sections so that they could be floated down river to the construction site. The bulk heads were welded to the underside of the trapezoidal trough-stiffener giving a connection with a low fatigue resistance. In addition, the bulkhead formed a "hard spot" under trafficking so that relatively high cyclic stresses developed

cracks in the welded joints between the bulkhead plate and the lower flanges of the stiffeners. Effective repairs were made by cutting away the bulkhead plate beneath the stiffener and bolting cover plates on each side of the bottom flange of the trough-stiffener. Later, cracks developed in intermittent fillet welds in a splice joint between "stub" plates and "main" plates, in about thirty diaphragms (7). In this case it was not necessary to remove any material and the overlapping plates were bolted together. About 24 cracks were found in the trough-stiffener to deck plate joints in 1977 (8). None exceeded 1m and the average length was 500mm. The cracked material was removed and the joints rewelded to give sound and effective repairs.

In order to improve the trough-stiffener to cross-beam detail in orthotropic decks, many designs have taken the troughs through a cut-out in the cross beam. This scheme improves the detail at the cross-beam but transfers the problem to the trough connections. Special attention was given to design of joints in trapezoidal stiffeners in the bridge at Ewijk across the River Waal in Holland. Laboratory fatigue tests were made on three types of joint and it was found that the best performance was given by having a 300mm long intermediate stiffener of the same thickness and butt welded on to backing strips (9). A different detail was designed for the Rio Niteroi bridge, the joints having splice plates which were welded to the sides and bottom of the trapezoidal stiffeners. Soon after construction, measurements were made under normal traffic which showed that cyclic stresses occurred whose ranges exceed the fatigue cut-off. It was calculated that visible fatigue cracking could develop after 30 to 40 years service (10).

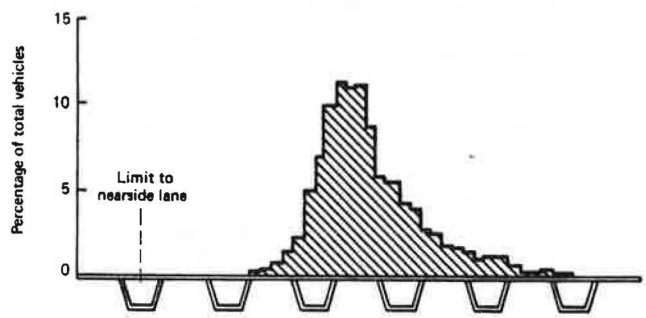
Fatigue can also occur through resonant stresses as opposed to direct traffic induced stresses. In a bridge in England, traffic induced vibrations produced almost continual resonance of lightweight cross-bracing which eventually caused fatigue fractures to occur at welded nodal connections. Fortunately these were readily accessible so that the welds could be repaired cheaply and effectively. The bracing was bolted to a cross-beam to eliminate the vertical vibrations. There have been several instances of failures due to wind induced vibration of bridge components. Examples have been reported of damage to hangers of Japanese bridges (11); there were cracks in the Shitoku-Oshashi bridge which vibrated at 3.45Hz and in the Maroroshi bridge which vibrated at 7.7Hz. Vibrations occurred in the inclined hangers of the Severn Bridge, England, and it was necessary to fit modified Stockbridge dampers to avoid potential problems. Wind and traffic induced vibrations are, however, not normally found to produce significant cyclic stresses in bridge superstructures and reported problems have been confined to lightweight attachments that can vibrate at a resonant frequency.

This paper is mainly concerned with the appraisal of fatigue caused by traffic induced stresses in orthotropic steel decks.

1. Traffic Loading

Considerable attention has been given to collecting axle load and vehicle load data for use in design of pavements as well as bridges. Several types of dynamic weighbridges, for weighing moving traffic without interruption to flow, have been devised (12 to 16). The system developed for British work is composed of three 0.6m square modules made from aluminium castings each having four load cells (12). When used with inductive loop detectors and an axle detector, data can be collected for vehicle speed, length, wheel base, headway and axle weight. It is essential for installations to be in smooth road surfaces free from ruts or undulations which could cause dynamic effects in the vehicle suspension system. Measurements made using different vehicles driven across a specially constructed test track, showed that the range of variations in the measured axle load can exceed 15% of the static value (17). In practice the pavement surfaces adjacent to and across bridges tend to have irregularities associated with features such as expansion joints. Soil compacted behind abutments can settle so that there is a pronounced step in the approach to the bridge. Such irregularities generate extra variations in dynamic loads so that high peak values can occur. In order to assess these effects, a 160kN test vehicle was fitted with instrumentation to its back axle to enable dynamic loads to be measured. The vehicle was driven across thirty motorway and over-motorway bridges (18). It was found that peak values of impact factors (ratio of dynamic to static wheel load) ranged from 1.09 to 1.75. In an extreme case of a bridge which carried an unclassified road over a motorway a peak value of 2.77 was recorded. Measurements were also made on two long span bridges having orthotropic steel decks. Peak values of impact factor were up to 1.7 and the RMS value of the dynamic component of wheel load was 5.8kN for the two bridges (the static wheel load was 50 kN). The impact effects are dominant for the first cycle and rapidly decay. The biggest effects are for unladen vehicles so that the heavy loads which are potentially the most damaging develop smaller dynamic components.

Figure 1. Transverse section of orthotropic deck showing distribution of nearside wheels.

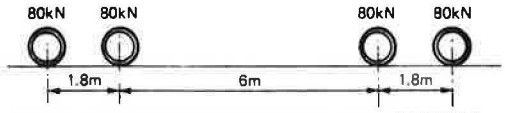


Assessment of the fatigue lives of orthotropic steel decks presents special problems because stresses are very sensitive to the lateral position of the vehicle. Measurements made of transverse positions, using a photographic technique, showed that 71% of all wheels passed with ± 300mm of the centre line of the lane, figure 1. For multi-lane dual carriageways, the heavy vehicles tend to be concentrated in the near-side (right hand) lanes. Measurements of the distributions of vehicles exceeding 15 kN between lanes of dual carriageways, for flows Q ranging from 300 to 850 vehicles per hour, have shown that the proportion of vehicles in the nearside lane is given by the empirical expression $\frac{1200}{1200+Q}$. Thus, for a flow of 500 vehicles per hour, 70% of the heavy vehicles occupied the near side lane (19).

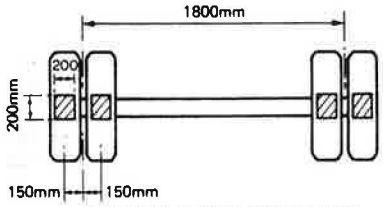
During the trials of orthotropic steel deck panels at Denham continuous measurements were made of axle and vehicle loads, 24 hours a day for three weeks, ie for a total time of 504 hours (20). Axle loads between 1.14 and 272 kN were recorded and classified in 9.1 kN increments. A total of 71,299

axles and 33,619 vehicles was recorded. Of these, 612 axles exceeded the legal limit plus a 10% allowance for impact and 57 vehicles exceeded the gross weight limit. Assuming that fatigue damage is proportional to σ^3 and neglecting cut-off, the overloaded axles have a damage potential of 24%, (σ is stress range). Using these and more recent data, consultants on behalf of Department of Transport have recommended a loading spectrum for the new British Standard for design of bridges. A 25 band spectrum of commercial vehicles (those vehicles having gross weights exceeding 15 kN) has been derived. Numbers of occurrences per million commercial vehicles were given and the damaging effect calculated. From these figures a standard fatigue vehicle was derived, figure 2.

Figure 2. Dimensions and spectrum of standard fatigue vehicle (not factored).



A) AXLE ARRANGEMENT OF STANDARD FATIGUE VEHICLE



B) PLAN VIEW OF STANDARD AXLE

Axle weight kN	Vehicle weight kN	Proportion of vehicles
720	2880	0.00001
402	1608	0.00002
327	1308	0.00003
168	672	0.00044
84.8	339.2	0.1045
65.6	262.4	0.105
57.6	230.4	0.09
29.6	118.4	0.7

C) DESIGN LOAD SPECTRUM FOR MAIN MEMBERS

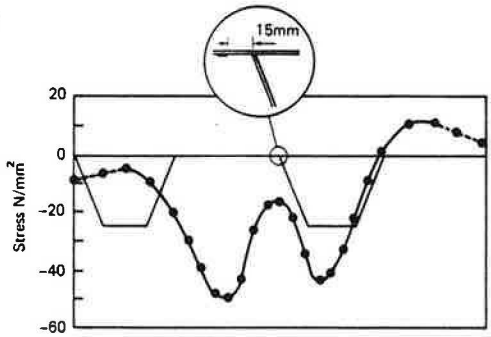
This has an axle arrangement similar to the type of vehicle responsible for half the damage in the 25 band spectrum. It has other advantages including the fact that it has the same axle arrangement as the notional HB design vehicle, so that results of static elastic analysis can be scaled and used for some of the fatigue checks. An equivalent spectrum of these standard fatigue vehicles is given in figure 2.

2. Stress Spectra

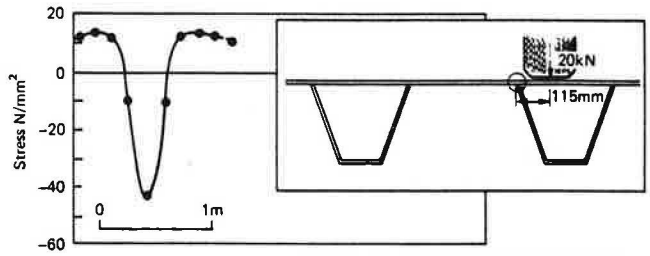
The determination of local stresses may be required for assessing the strength of an existing bridge or for determining a new design. Calculation of stresses in orthotropic decks presents problems due to the sharp stress gradients that develop at the joints and the difficulty in assessing effects of the asphalt wearing surface. The asphalt influences stresses due to its contribution to bending stiffness. Measurements of this composite action have shown that stresses close to the deck plate can be reduced to about 10 per cent of the values without asphalt, depending on features such as the deck temperature, speed of the vehicles, and grade of asphalt (5). A second series of tests on a different grade of asphalt showed that behaviour is influenced by time dependent mechanisms; when the vehicle is stationary the material directly beneath the wheels creeps so that stresses close to the deck plate reduce with time (21). After removal of the vehicle, periods of up to 20 minutes elapsed before the strains returned to the no-load condition. It was found that stresses in the lower surface of the deck plate were reduced by about 30 per cent when vehicle speeds were increased from crawl to 30 km/h, most of the effect occurring between crawl and 5 km/h. The stresses were increased by up to 100% for an increase in asphalt temperature from 20°C to 37°C.

For existing decks, it is more satisfactory to measure stresses produced by a vehicle, preferably under dynamic conditions. The fatigue life can either be calculated by relating measured influence lines to assumed vehicle spectra, or by measuring stress spectra at the detail in question. Measurement of influence lines requires both longitudinal and transverse directions so that it is really influence surfaces that are required. Because stresses are so sensitive to lateral wheel position, the measurements are most conveniently made with the vehicle stationary. For transverse influence lines, measurements should be at intervals not exceeding 75mm and for longitudinal influence lines they should be 38 to 100mm. Measurements have been made at TRRL with two types of deck having V-stiffeners and trapezoidal stiffeners (22,23). Loading was applied through a 900 x 20 wheel which could be accurately positioned on the panels. Transverse and longitudinal influence lines were produced, see for example figure 3.

Figure 3. Transverse stresses adjacent to the stiffener to deck plate weld.



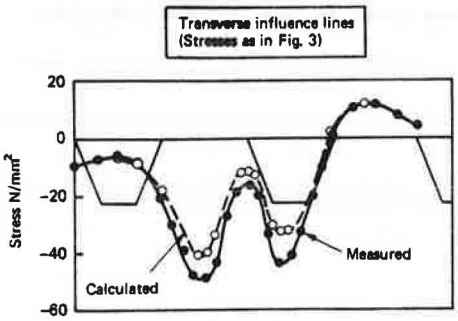
(a) TRANSVERSE INFLUENCE LINE, MID-WAY BETWEEN CROSSBEAMS



(b) LONGITUDINAL INFLUENCE LINE, WHEEL ABOVE STIFFENER

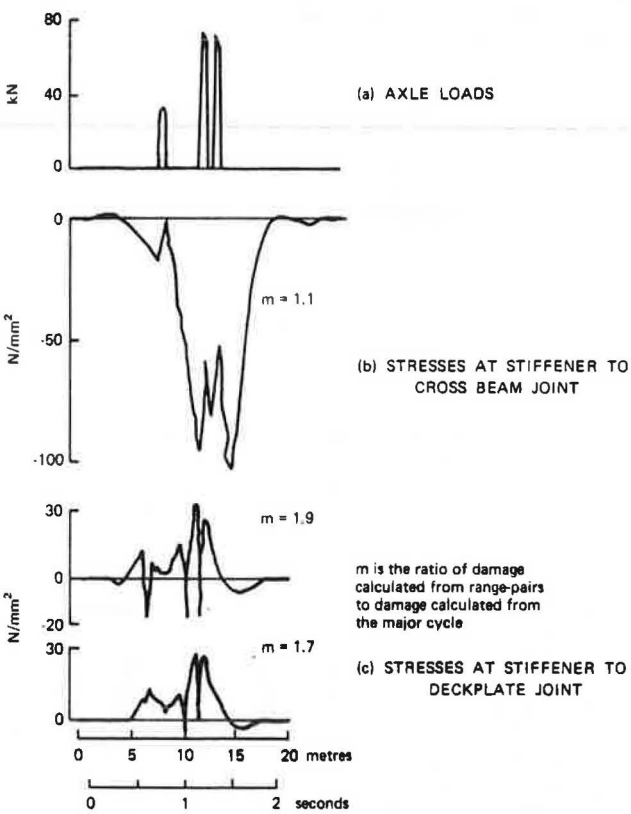
The data for unsurfaced panels tested in the laboratory are suitable for comparison with calculated stresses, partly because the questionable effect of the asphalt is eliminated and partly because the stresses are produced by more accurately controlled positioning of the wheel. Values of the stresses in the stiffener to deck plate joint have been calculated by a method using finite strip analysis. Comparisons between calculated and measured stresses, in figure 4, show good agreement.

Figure 4. Comparison of measured and calculated stresses adjacent to stiffener to deckplate weld.



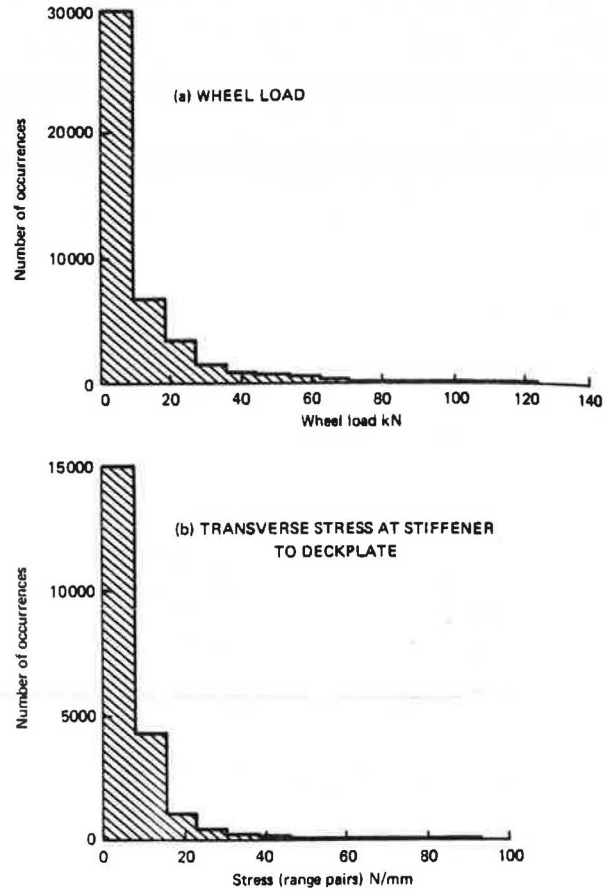
Measurements have been made of the traffic induced stresses in the panels installed at Denham (24). Strain gauges were fixed at different positions on the underside of the panels. The outputs were recorded on analogue magnetic tape and analysed in the laboratory. The shapes of stress cycles were irregular as shown by the examples in figure 5.

Figure 5. Typical traffic induced stress cycles.



For the cycles considered, it was insufficient to record the peak to peak range and a program was written to count range-pairs and to classify occurrences in increments of 7.72 N/mm^2. In this method, counts are made of constituent cycles within a major cycle caused by one vehicle. Damage estimated from range-pairs for the cycles shown in figure 5, is up to 1.9 times more than that derived from the major cycles alone (the ratio is expressed as m). Histograms of results for 12 hours a day for 6 days are shown in figure 6.

Figure 6. Load and stress spectra measured at Denham.



During the time that has elapsed since the stress data were collected at Denham, there have been changes to the vehicle regulations in Britain as well as differing trends in the distribution of types of vehicle. In addition a new generation of equipment has been developed so that data can be processed more quickly and a wider range of information obtained. It is therefore timely to update the state of the art and work is being done as part of a collaborative project sponsored by the European Coal and Steel Community. The first phase of the programme, which is currently under way, involves collection of 120 hours of data during two visits to each of three steel bridges, ie there will be a total of 360 hours data collected during six sessions at the bridges. Stresses are recorded on eleven channels. Axle loads, axle spacings, vehicle headways and speeds are also recorded (25).

3. Fatigue Relationships

The relevant S/N relationship between stress range and number of cycles to failure, for the component in question, can be selected by reference to a standard classification which gives curves based on conventional laboratory data for common types of welded joint. There is, however, the question of how realistically laboratory testing can simulate service conditions. For traffic loading of highway bridges it is necessary to have S/N data in the range 10^6 to 10^8 cycles, longer lives being too time consuming to produce. Many investigations have stopped at 10^7 cycles and missed the

more relevant longer endurances. For some cases this may not matter because there is a 'knee' in the S/N curve of many joints, at about 2.10^7 cycles, beyond which constant amplitude tests remain unbroken. Under variable amplitude loading however, failures can occur at lower mean stresses and longer endurances.

For investigations related to full scale behaviour it is necessary to use test pieces big enough to hold residual welding stresses which are typically up to yield stress. This is important because residual stresses play a significant role in the fatigue mechanisms of welded joints in bridges, particularly for pulsating - compression.

3.1 Residual Stresses

Most of the current understanding of effects of residual stress is due to the work of Gurney (26) who tested axial specimens having longitudinal non-load-carrying connections. In a co-ordinated series of tests at Welding Institute and TRRL, the main effects due to residual stresses were shown to be as follows:

1. For loading varying from pulsating-tension to fully reversed push-pull, the fatigue life is dependent upon stress range and is almost independent of mean stress and stress ratio, R. For pulsating-compression, lives are a little longer than those for pulsating-tension at the same range of stress, see figure 7.
2. The slope of the S/N relationship is steep-ended and the fatigue strength (the stress to produce a given life) is reduced significantly at longer lives, see figure 8.

Figure 7. Effect of tensile and compressive stress ratio on fatigue.

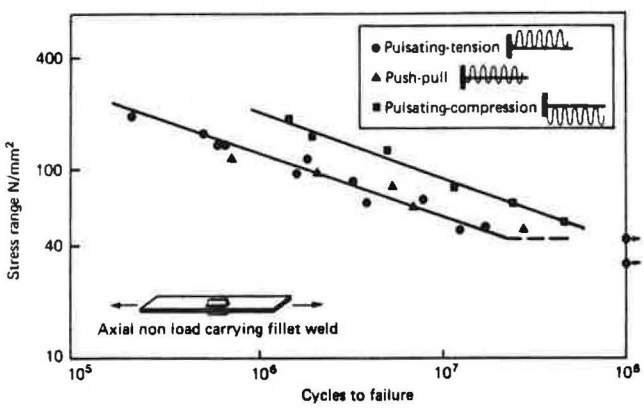
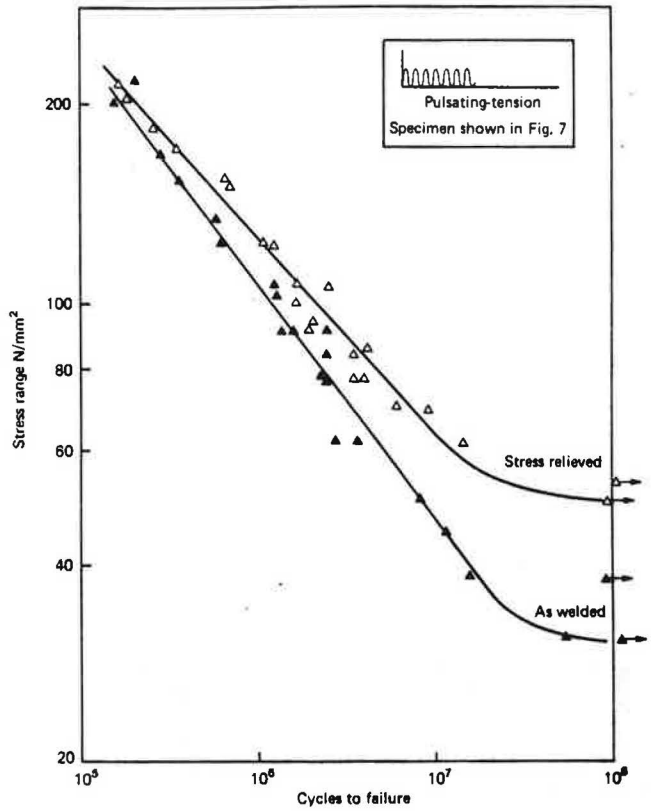


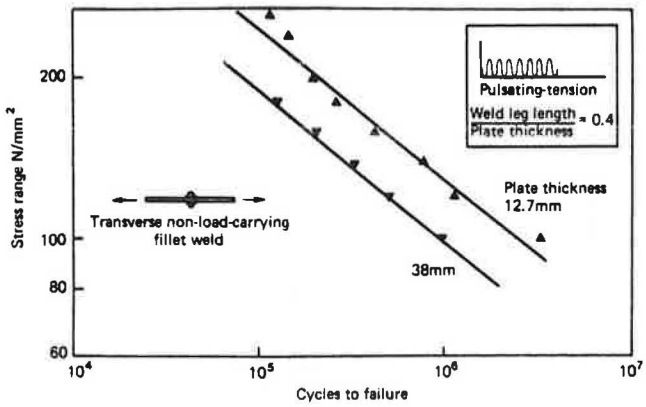
Figure 8. Effect of residual stresses on fatigue.



These effects can be explained from consideration of the stress/strain relationship for structural steel. This has a prolonged flat portion beyond yield stress and prior to the onset of work hardening. For applied stress ranges of yield stress magnitude, the surrounding elastic material is plastically deformed so that regions of high residual stress are relieved and fatigue life is unaffected. For lower stress ranges, the residual stresses are unaffected and lives are reduced due to the higher value of true (as opposed to nominal) stress ratio. For stress relieved specimens the true and nominal stress ranges are the same.

Pulsating-compression can be shown to be effectively the same as pulsating-tension because tension merely produces a small plastic excursion along the stress/strain curve which work hardens to the same stress range. However, it has been shown that pulsating-compression continues to propagate cracks beyond the zones of residual tensile stress and through the zone of compressive stress. In fact it is possible to fracture a welded test piece under the action of pulsating-compression. Such behaviour is almost certainly due to the fact that a crack has a plastic enclave at its tip, which is subject to strain-cycling due to the presence of the surrounding elastic material. Once initiated, the material ahead of the crack continues to experience the same value of strain range irrespective of whether it is in the zone originally containing residual tension.

Figure 9. Effect of plate thickness on fatigue of longitudinal non load carrying fillet weld.



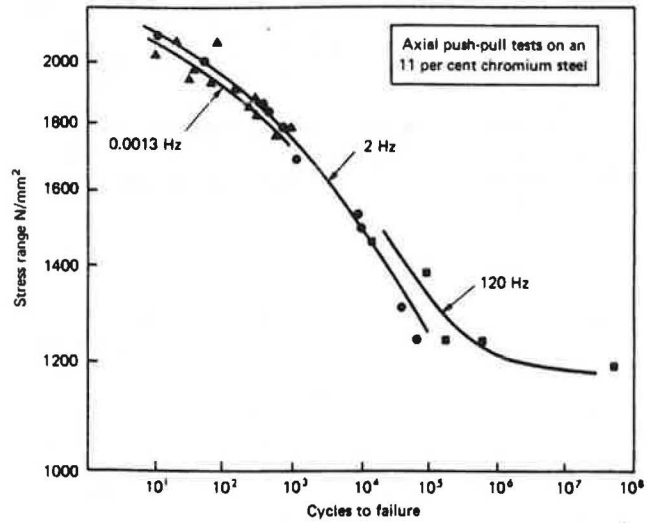
3.2 Plate Thickness

When relating laboratory data to bridge performance, it is necessary to consider size effect in relation to plate thickness. Analysis shows that the size of an initial defect at the weld toe which just allows crack propagation, decreases with increase in plate thickness. Tests have been done at the Welding Institute to confirm the extent of these effects for an axial specimen having a transverse non-load-carrying fillet weld. These show that there is indeed a comparatively big effect for a ratio of weld-leg-length to plate thickness of 0.4, figure 9. The difference in strength at 2.10^6 cycles for plate thicknesses of 12.7 and 38mm, can be seen to be approximately 24 per cent.

3.3 Test Time

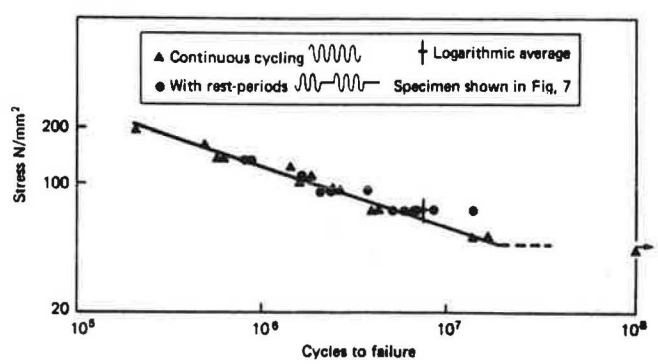
The production of fatigue data for endurances of around 10^8 cycles is essentially the simulation of 120 years design life by laboratory tests lasting for up to about 12 weeks. Such acceleration involves assumption that there are no significant effects due to increased frequency, reduction of exposure time and elimination of rest-periods between heavy vehicles and during off-peak hours. In the past there have been numerous investigations of the effect of frequency. At ambient temperatures and a high ratio of frequencies, there is commonly a small effect whereby number of cycles to failure increases with frequency, eg tests on butt welded specimens exhibited a 20 per cent increase in fatigue strength for frequencies of 8 to 116 Hz (27). In tests on stainless steel at 0.0013 Hz, 2 Hz and 120 Hz, and endurances of up to 10^8 cycles, it was found that in the regions where the respective curves overlapped a sixty fold increase in frequency raised the strength at the longer endurance by a similar factor, figure 10. In practice, commercial vehicles travelling in convoy are likely to cross a given detail at a rate corresponding to a frequency of about 1 Hz. For the types of influence line shown in figure 3, constituent cycles are developed at frequencies of about 30 Hz. The highest frequency that laboratory tests can be run, about 120 Hz depending on the stiffness of the test piece, is unlikely therefore to increase endurance by a significant factor.

Figure 10. Effect of frequency on fatigue life.



There have been a number of investigations into effects of rest-periods. The most notable work was by Whitman (28) who tested mild steel cantilever specimens for durations of up to 23,000 hours. Holes were drilled to lower the fatigue strength and simulate a welded joint. When compared with continuous cycling, it was found that the rest-periods caused increases in endurance of about 18 per cent and raised the cut-off stress by 17 per cent. More recently it was considered necessary to re-examine the effect of rest-periods to check whether behaviour is similar for a welded joint. Tests were conducted at TRRL on a non-load-carrying fillet welded specimen. Different combinations of rest-periods to cycling phases were used but the main part of the programme was for equal times cycling and resting, and with static stress during the rest-period, figure 11. It was found that at the lower stress and longer rest-periods, there may be a very small increase in life. The presence of differing levels of stress during the rest-period had no effect. It seems clear that although rest-periods can cause an increase in cyclic life, the effect is too small to be a significant factor in design. Moreover the effect is opposite to that of frequency so that high frequency continuous cycling should not introduce significant net errors in fatigue assessment.

Figure 11. Effects of rest-periods on fatigue life.



In considering other errors due to accelerating the cycling, the most worrying effect is due to the reduction of real time exposure by a factor of 500, possibly more. Fatigue tests in normal ambient conditions involve a mild form of corrosion considered by some to be the mechanism responsible for frequency effects. This process involves real time and it is not possible to simulate it satisfactorily with the present level of understanding of corrosion over long periods of time. There have been numerous investigations of corrosion and the consensus show that fatigue strength can be reduced by about 50 per cent. Endurances under corrosion fatigue are strongly dependent upon applied frequency, low frequencies giving the shortest lives. The literature on corrosion fatigue has been surveyed by Knight (29) who noted that there is an interaction between corrosion and fatigue so that when the two are simultaneous the strength is lower than for the sum of the individual effects. Laboratory corrosion tests are of necessity accelerated and there is insufficient knowledge about longer term behaviour, for periods of several years and more. The most common type of laboratory test involves cycling a specimen which is immersed in a salt water bath. Whilst such tests give conservative data it is arguably more realistic to consider fatigue of materials exposed in a natural environment for longer times. Tests specifically related to bridges have been conducted on specimens exposed to marine environments for periods of up to two years. It was found that fatigue strengths at 2.10^6 cycles were reduced by between 20 and 40 per cent (30). Tests on reinforcement bars embedded in concrete beams which were loaded in flexure till cracking developed, involved exposures of up to one year. It was found that the fatigue strength at 2.10^6 cycles was reduced by about 37 per cent (31). In the current research programme of the TRRL, specimens are being exposed in a natural marine environment for periods of up to ten years. The first assessment will be in 1980.

3.4 Programmed Loading

The term programmed loading refers to cycles of varying amplitude, whether random or non-random. Although fatigue testing is usually conducted at a constant amplitude of stress, bridges experience a spectrum of stresses which must be interpreted in relation to the available data. There has been a very considerable volume of research into crack propagation and rupture life under programmed loading and it has been found that summation of linear cumulative damage* can be used successfully in a variety of situations. (*The Palmgren-Miner hypothesis (32,33) first proposed in 1924 in a study of the durability of ball bearings.) It has subsequently been shown that a fracture mechanics analysis of crack propagation gives the same results as the linear summation of cumulative damage. The hypothesis is currently very widely used and is embodied in the new British Standard for bridge design.

For analysis of crack propagation under programmed loading, Barsom (34) showed that the RMS value of the spectrum can be correlated with constant amplitude loading. This gave very good results for different spectra. Overbeeke (35) conducted an unusually comprehensive programme of tests on spot welded lap joints. This work is notable because it involved endurances of up to 10^8 cycles and the tests included narrow-band Rayleigh and broad-band Gaussian spectra. The rupture data correlated using RMS values of stress. This work is very significant because there is a dearth of data for broad-band spectra relevant to fatigue of joints in orthotropic bridge decks. Yamada and Albrecht (36) tested I-beams with welded cover plates and butt jointed plates, using a stress spectrum recorded on a relevant bridge detail. In analysis of the data the programmed loading was expressed in terms of RMC (Root Mean Cube) rather than RMS because summation of crack propagation using fracture mechanics involves an exponent of 3. It was confirmed that RMC stresses correlate with constant amplitude fatigue. Schilling and Klippstein (37) tested similar specimens using a Rayleigh distribution which was shown to represent a measured spectrum of stresses. It was found that the equivalent stresses, whether calculated from cumulative damage or RMS values, correlated with constant amplitude data.

Clearly a large proportion of traffic induced stresses are below the fatigue limit (cut-off stress). The fatigue limit is the lowest stress that will propagate cracks from a representative size of defect and is therefore influenced by the initial state of the specimens. Once a crack has propagated a finite distance, lower stresses become damaging. Gurney (38) calculated the extent of this effect for several conditions and in a typical case found that a joint having a cut-off of 56.6 N/mm^2 for constant amplitude fatigue has an effective cut-off of 42 N/mm^2 under programmed loading. This work is currently being extended to the spectrum given in the proposed British Standard.

4. Conclusions

There have been a number of cases of fatigue cracks developing in steel bridges under normal traffic. These may be loosely classified as being associated with welded attachments to main steel girders, welded connections in orthotropic steel decks, and components subjected to resonant vibration. In design of welded connections it is necessary to assess fatigue performance using the best current procedures. In this paper special attention is given to orthotropic steel decks and the following points emerged:-

1. Vehicle induced stresses in unsurfaced orthotropic deck details may be calculated with adequate accuracy using methods such as finite strip analysis. The composite action of asphalt surfacing reduces stresses close to the deck plate by factors of up to 10 to 1. Current practice is to ignore this composite action in design calculation and regard it as a hidden factor of safety.
2. For the types of traffic induced stress cycle in orthotropic decks, it is necessary to count constituent cycles within the major cycle produced by a vehicle. Damage assessed from range-pairs can be about twice that assessed from major cycles alone.
3. In laboratory tests to simulate fatigue of highway bridges it is necessary to use large specimens that can retain residual welding stresses and have realistic plate thicknesses. Applied stresses should be selected so that cyclic endurances exceed 10^7 cycles and should preferably run to 10^8 cycles using relevant programmed loading.
4. Residual stresses play a significant role in fatigue processes. The slope of the S/N curve is steepened and the stress range is the dominant factor for pulsating-tension and push-pull loading. Pulsating-compression has the same logarithmic S/N slope as pulsating-tension but exhibits longer endurances.
5. Accelerated testing at high frequency and

continuous cycling is unlikely to introduce serious errors in estimation of lives up to about 10^7 cycles. Effects may be more significant at lower stresses associated with cut-off.

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