Fatigue life of Australian railroad bridges

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Résistance à la fatigue de ponts-rails australiens

Ermüdungsfestigkeit australischer Eisenbahnbrücken

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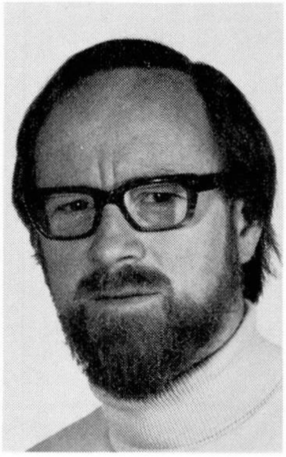

Paul Grundy took his first degrees in Civil Engineering at the University of Melbourne, and his Ph.D. at Cambridge, U.K. After working with consulting Engineers he joined Monash University where he is an Associate Professor of Civil Engineering.

SUMMARY
A series of Australian railroad bridges with identical welded coverplate details are reviewed for expected fatigue life. A method of determining complete sample stress histories from traffic records is developed, with the key factors of influence line and impact being confirmed by field measurements to give a reliable estimate of fatigue loading. Difficulties are encountered in identifying the fatigue category of the detail. Laboratory tests are used to give guidance on crack growth characteristics. Although confident estimates are made of expected life, the risk of premature failure of any one detail in such a large number remains difficult to estimate.

RESUME
Une étude de résistance à la fatigue est entreprise sur une série de ponts-rails australiens présentant les mêmes détails de couvre-joints soudés. Une méthode est proposée pour l'établissement des contraintes en fonction des charges de trafic, prenant en compte la ligne d'influence. La méthode est contrôlée par des mesures in situ et permet ainsi de calculer avec précision les contraintes dues à la fatigue. Des essais en laboratoire ont permis de prévoir les caractéristiques et l'évolution des fissures. Bien que la durée de vie restante peut être prévue avec une bonne précision, il reste le cas de la rupture prématurée de n'importe quel détail constructif, qu'il est difficile de prévoir.

ZUSAMMENFASSUNG
Eine Anzahl australischer Eisenbahnbrücken mit gleichen geschweissten Gurtplatten wird bezüglich ihrer Ermüdungslebensdauererwartung untersucht. Ein Verfahren für die Bestimmung der Spannungsgeschichte aus den Verkehrsaufzeichnungen, unter Einbezug der Einflusslinien und der Stosszuschläge die durch in situ-Messungen bestätigt werden, wird präsentiert. Schwierigkeiten werden in der Bestimmung der Ermüdungskategorie der Details angetroffen. Laborversuche an Schweissdetails werden verwendet, um Auskunft über die Charakteristik des Risswachstums zu erhalten. Obwohl zuverlässige Schätzungen über die erwartete Lebensdauer der Brücken gemacht werden, bleibt das Risiko des vorzeitigen Versagens eines Details, infolge der grossen Anzahl, schwierig abzuschätzen.

1. INTRODUCTION

In 1962 a new rail track was opened between Melbourne and Albury in the State of Victoria, completing the standard gauge link between Sydney and Melbourne shown in Fig. 1. Nearly all the bridges were of three standard spans of approximately 7m, 10m and 13m, consisting of stringers and open decking. The designs use 24" x 7.5" x 95 lb/ft. taper flange girders, which were the largest available, with cover plates. As was fashionable, the coverplates were terminated where they were no longer required to keep stresses within allowable limits. A typical coverplate termination detail is shown in Fig. 2, and a photograph of one ready for fatigue testing is shown in Fig. 3.

Recognising the potential fatigue problem, in the light of more recent research, Victorian Railways began an investigation to establish the risk of fatigue failure, and to establish the remaining life before remedial work should begin. [7-10]

A noteworthy aspect of the problem is the large number of similar structures, presenting over 2000 locations at which a fatigue crack can form, each on a nonredundant load path. Further, each structure is subjected to the same traffic. The rewards for establishing the fatigue reliability of each representative girder with confidence are high.

This paper describes the steps taken to quantify the loads, their fatigue damage effect, the fatigue resistance of the detail, and hence the remaining fatigue life in probabilistic terms.

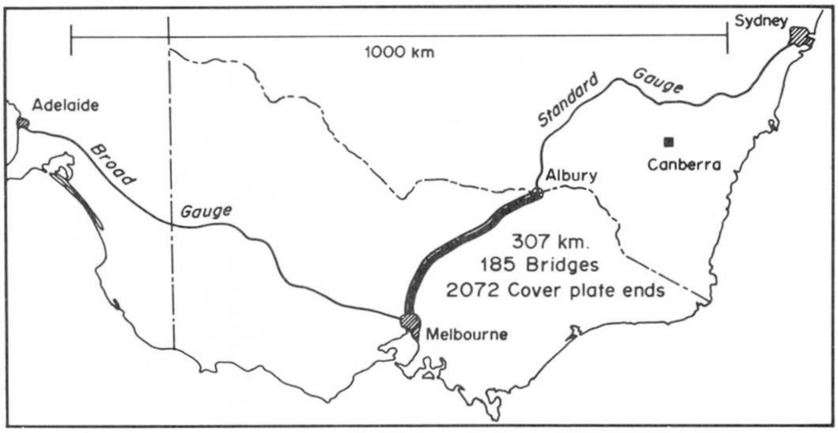

Fig. 1 Location of Bridges under Review.

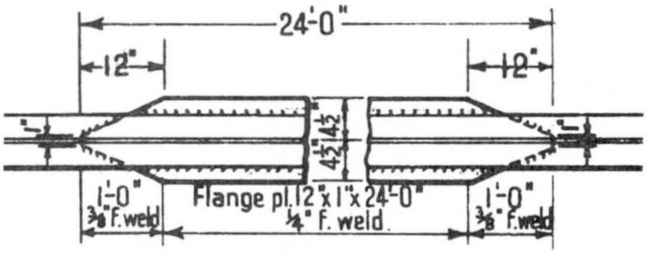

Fig. 2 Detail of Coverplate (from the original drawing).

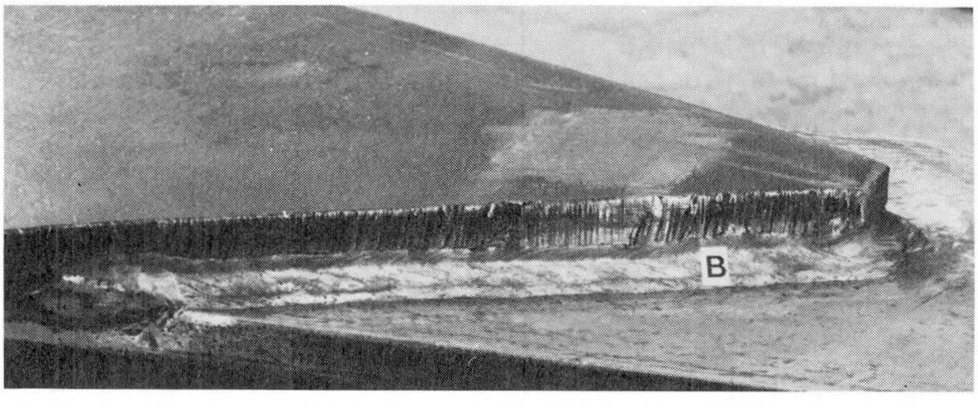

Fig. 3 View of Coverplate terminations

2. REQUIREMENTS FOR PREDICTING FATIGUE LIFE

The two primary requirements for predicting fatigue life are correct estimates of the damage index or fatigue loading, and of the damage sensitivity or fatigue resistance. The relationship between these two parameters determines the reliability of the system in terms of remaining life with an appropriate probability of exceedance [1-4,12].

The fatigue damage index is here defined as $\Sigma D(\Delta\sigma)$ at a specified location on a structure over a specified period, where D is the function relating $\Delta\sigma$, stress amplitude, to damage per cycle, and the sum applies to all stress amplitudes occurring in the specified period. Typically $D(\Delta\sigma) = K(\Delta\sigma)^3$. The index can be determined by field measurements or from traffic data subjected to calculated transfer functions to transform it into a fatigue damage index. There are limitations to either approach, most of which can be eliminated by using both.

In the context of bridges, fatigue damage sensitivity is dealt with by classification of structural details into categories [5], found in all codes of practice. Stress concentration, the use of welding, and other factors influencing fatigue resistance are all encompassed by the category, so that only the ambient stresses need be established by conventional structural analysis.

When the damage index and damage sensitivity have been established, it is possible to estimate the remaining design life of the structure at the detail under review [4]. Design life is determined from fatigue resistance curves, which are lower confidence limit curves, not mean life curves. If the estimate is correct, it implies a probability of prior failure of 0.05 in American and Australian Codes, and of 0.023 in European Codes. The owner of a railway system is reluctant to accept such a risk unless a failsafe program of inspection and maintenance can be implemented. These considerations are compounded by the large population of similar details at risk, where the cost of a failsafe program can become significant. It is possible to estimate the expected time to the first fatigue failure, but in the context of no failure being permitted the owner would like to know the expected time to the probability of first failure exceeding a low threshold, say 0.023. Such a prediction, based upon assumed functions describing the lower tail of probability distribution, would lie outside the database of laboratory tests, and would therefore carry little credibility.

3. ESTIMATION OF FATIGUE DAMAGE INDEX

For the bridges under review there exists remarkably complete documentation of all trains to the extent that the entire history of axle loads, spacing and sequence can be reconstructed for the whole life of the bridge. In practice sample days are chosen to represent typical traffic to give a daily damage rate. The annual gross tonnage is compared with the sample day to confirm the representativeness of the sample. The procedure, detailed elsewhere [7-11], is

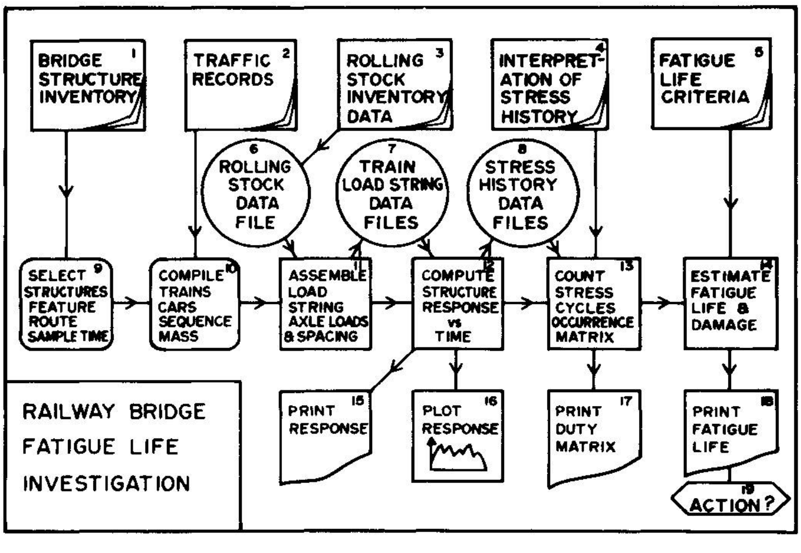

Fig. 4 Bridge Fatigue Life Flow Chart.

summarised in Fig. 4, where items 1-5 are documentary sources, items 6-8 are computer data files, items 9-10 are user inputs, items 11-14 are computer programs processing the data, and items 15-18 are outputs. Items 14 and 18 require an input on fatigue resistance. Apart from these, the entire program develops a general damage index for the structures on the chosen route.

For the given load history, item 7, the corresponding stress history is computed for a given span (influence length) and shape of influence line. The influence line is readily determined for these statically determinate structures. However, the effects of continuity of track and the dispersion of load from the top of the rail to the bottom of the girder where the critical details were located needed to be taken into account. The influence line was deduced from strain measurements with a locomotive stationed on the bridge (Fig. 5).

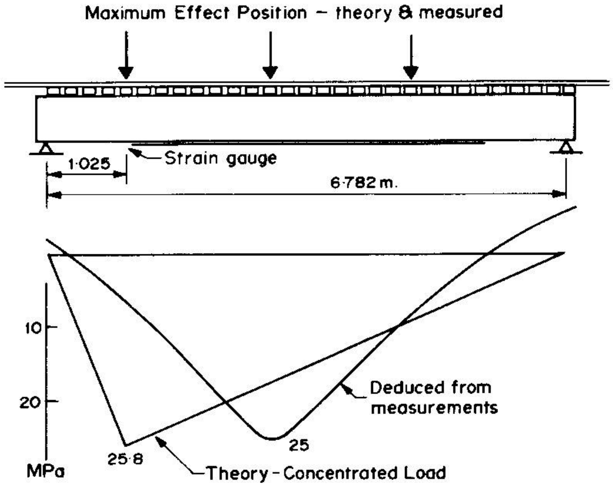

Fig. 5 Influence Lines for Stress 1.025m from support for 115.7kN wheel load.

Although the measured peak value almost agreed with the computed one, the positive influence length is shorter and there are adjacent negative influence regions as a consequence of the structural continuity of the track. For the span shown in Fig. 5, the measured maximum static stress for the locomotive was only 79% of the computed value. If the computed stress amplitudes on this simple statically determinate structure were used without calibration by field measurement the damage index would have been overestimated by nearly 100%.

4. DYNAMIC EFFECTS

In estimating stress amplitudes it is necessary to apply the correct impact factor for dynamic effects. Since it is the average impact factor which is important for cumulative damage, the maximum values used in design codes are of no use. Since data gathered on impact factors on one type of bridge of a given span cannot be applied to a different type or different span of the same type, it is invariably necessry to measure inpact factors for the bridge in question.

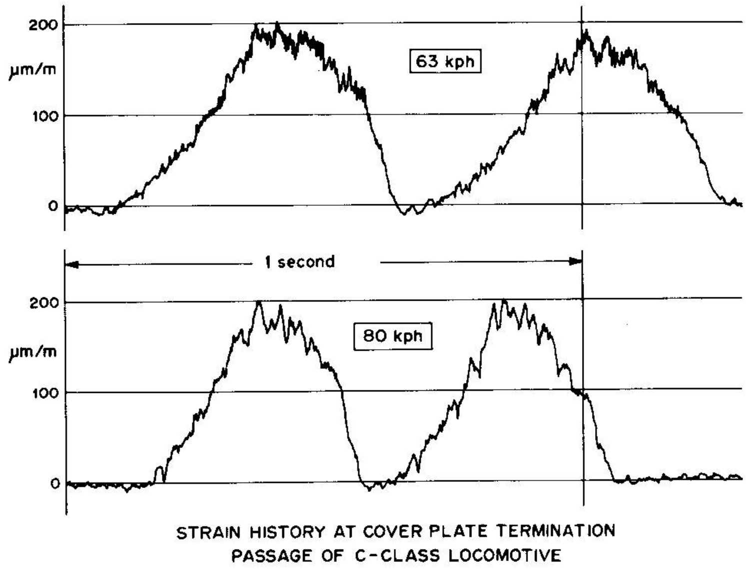

Fig. 6 Measured Response to Locomotive.

Two methods of measurement were employed in this study. In the first, dynamic traces of the of the strain at the ends of the coverplates were made for the locomotive previously used in static calibrations traversing the bridge at various speeds. Two such traces are shown in Fig. 6. Superimposed on the quasistatic response, which is not amplified, is a vibration associated with the suspension of the locomotive. This vibration adds on about

8% to the maximum static stress amplitude, and it creates a significant number of minor stress cycles not previously present. If the latter are disregarded, the dynamic effect can be included by amplifying the static stress amplitudes uniformly 8%. The method is imprecise, but vastly superior to using the impact factor specified in the relevant Code, which is 1.58.

The second method integrates the dynamic effect for a whole train in terms of fatigue damage. The strain history was recorded for two trains and then subjected to analysis by the Rainflow method [3]. For all stress cycles, the sum $\Sigma(\Delta\sigma)^3$ was formed as a measure of damage, D_{meas}. The procedures of Fig. 4 were then followed for the same two trains to generate the corresponding stress history, and the measure of damage, D_{calc} was formed in the same way, without using the 0.93 static calibration factor derived from field measurements. The expression $\sqrt[3]{D_{meas}/D_{calc}}$ is an integrated estimate of impact factor for all stress amplitudes, which includes the calibration factor relating measured static stress to computed stress.

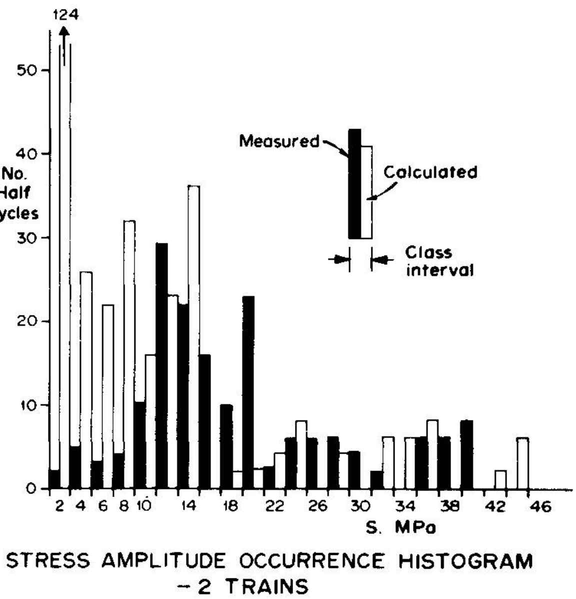

Fig. 7 Stress Amplitudes by Rainflow Method.

When applied to the 7m span under review, the second method yields a nett impact factor of 1.01. If the calibration factor remains at 0.79 for all axle configurations, the real average impact factor becomes 1.27 - much higher than the value from the first method. One reason for the difference is that the vibrations noted create more stress cycles at lower amplitudes, and the amplification factor is greater for stress cycles of lesser amplitude. Histograms of the measured and calculated occurrences of stress half-cycles are shown in Fig. 7. The number of major stress cycles corresponded between theory and measurement, with the measured maximum being 0.91 of calculated maximum. This is consistent with the lower impact factors measured with the locomotive. Low amplitude cycles were filtered out of the field measurements, but in the middle range many more cycles were measured than calculated.

It is noted that 90% of damage is done by stress cycles exceeding 16 MPa. This confirms the observation that the small cycles are not significant, and that fatigue damage estimates are not sensitive to the shape of the S-N curve at low stresses under variable amplitude.

In assessing the fatigue damage index, more confidence was placed in the second method. Taking all documentary evidence with the confirmation of field measurements into account lead to a confident prediction of the fatigue damage index. A similar effort was required to establish the fatigue sensitivity of the coverplate detail.

5. FATIGUE RESISTANCE OF COVERPLATE ENDS

Although coverplate ends have been extensively studied [5,6,13], it is evident from Figs. 2 and 3 that this series does not readily compare with those tested by others. The overlapping thick coverplate leads to a choice of USA fatigue category E' [6], proposed to allow both for overlapping coverplate and for size effect. In this case the manual fillet weld is continuous through the overlap. It is undersize by modern standards. The choice between E and E' represents a factor of 4.1 on estimated life. Two girders removed from service are being tested to give four results - not enough for statistical predictions of life, but useful for monitoring fatigue crack growth.

So far one test has run out and two have failed in the same way, by a crack propagating outwards from the root of the fillet weld (Fig. 8). The crack surfaces almost simultaneously across the full width of the coverplate end and then turns down through the side fillet welds into the top flange (Fig. 9). The two specimens which failed had virtually identical crack growth rate and form, but very different times to the initial visible crack. There appears to be insufficient data to support a change from E' classification to E, but the stable crack growth gives confidence in implementing a program of visual inspection, since the cracks are large and easily seen over a long period of the fatigue life.

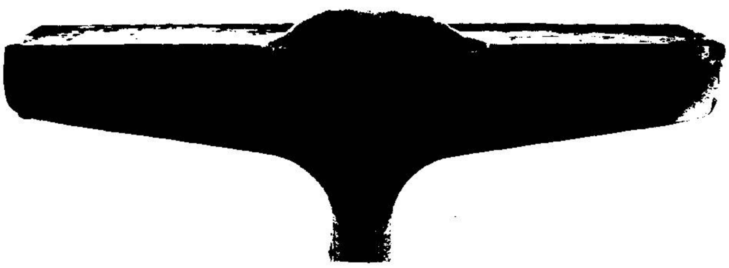

Fig. 8 Fracture Surface.

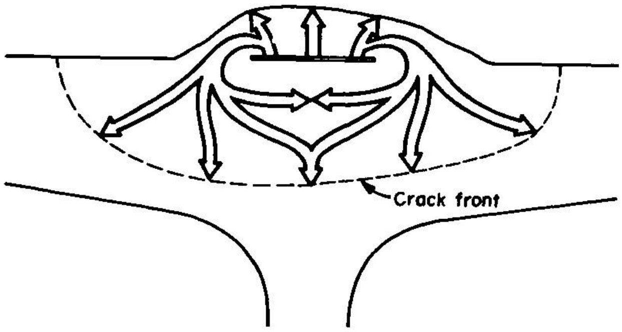

Fig. 9 Key to Crack Growth of Fig. 8.

6. CONCLUSION

This paper describes an investigation in which, unlike most situations, it has been possible to document load effects more accurately than resistance. It has resulted in a predicted life of 40 years for the coverplate end on the short span beams [10-12], based upon E' classification. As this life represents 95% probability of survival, it is practically certain that at least one beam out of the large population will have failed. Attempts to translate this finding into a credible estimate of current risk of failure, after 20 years of service, have been unsuccessful. If the detail is Class E, the estimated fatigue life becomes acceptable.

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