Remaining life of a suite of railway bridges

Remaining Life of a Suite of Railway Bridges

Durée de vie restante d'une série de ponts-rail

Restlebensdauer einer Reihe von Eisenbahnbrücken

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SUMMARY

A suite of short span railway bridges in Victoria, Australia, has been investigated for remaining fatigue life. The study revealed that site variability was significant, that a serviceability impact factor less than maximum is needed for estimating fatigue damage, that test data on fatigue strength is scant for the very large number of stress cycles occurring, and that redundancy of load path increases the uncertainty associated with structural response.

RÉSUMÉ

Une série de ponts-rail de petite portée dans l'état de Victoria (Australie) a été examinée du point de vue de leur durée de vie restante. Cette étude a montré que la variation des caractéristiques d'un ouvrage à un autre était déterminante, que le recours à un coefficient d'amplification inférieur aux valeurs maximales était nécessaire pour évaluer le dommage en fatigue, que les données expérimentales sur la résistance à la fatigue étaient insuffisantes pour les très grands nombres de cycles de contraintes rencontrés, et que l'hyperstaticité augmentait l'incertitude quant à la réponse de la structure.

ZUSAMMENFASSUNG

In Viktoria, Australien, wurde eine Reihe von Eisenbahnbrücken kurzer Spannweite hinsichtlich der Restlebensdauer untersucht. Eigene Untersuchungen zeigten, dass die Konstruktionsart für die Ermüdung weitgehend massgebend ist. Zudem stellte sich bei dynamischen Messungen heraus, dass für die Abschätzung der Ermüdungsschäden nicht die maximalen Stosszuschläge angenommen werden müssen und dass mit dem Grad der statischen Unbestimmtheit die Unsicherheit über den Kraftfluss im Bauwerk zunimmt. Die Resultate aus Untersuchungen an einem bestimmten Bauwerk können auf andere Bauwerke ähnlicher Art übertragen werden. Im Verlauf der Arbeiten wurde ein Mangel an Resultaten von Ermüdungsfestigkeitsversuchen im Bereich von kleinen Spannungsdifferenzen festgestellt.

1. INTRODUCTION

The assessment of the remaining fatigue life of a suite of railway bridges requires a variety of data, all of which is subject to some error of estimation and to various levels of confidence. The broad areas of data may be classified as loads, load effects, and fatigue strength. This data is used in a model of fatigue life or fatigue damage which itself is subject to error inherent in simplified engineering models of systems.

In the case of railway bridges of short span, individual axle or wheel loads, their spacing and sequence are important, since a large number of stress cycles can be generated by the passage of a single train. The loading history has to be estimated, as does future loading, where the assumption has been that axle loads and speed will tend to increase.

The load effects of relevance to fatigue life are primarily stress histories at locations where fatigue cracks are likely to develop. The peak stresses depend on two major factors, the influence line (or surface) for the bridge element, and dynamic amplification. The number of stress cycles is also affected by dynamic behaviour.

The fatigue strength, which is subject to inherent uncertainty of estimation due to variability of defects, geometry and materials of fabrication, is also subject to error of detail classification if the detail does not precisely match the detail used in the test programs.

The abovementioned three areas require equal attention if a forecast of remaining life is to lie within acceptable confidence limits. It has been shown [8] that great effort and precision in two of the three areas while neglecting the third will not achieve a confident result. These observations arise out of experience over the past decade in establishing the remaining life for suites of nominally identical railway bridges of approximately 7, 10 and 13 metre spans in Victoria, Australia. All bridges consisted of simply-supported stringers supporting an open unballasted deck or a ballasted non-composite concrete deck. The stringers were made up of taper flange beams with welded coverplates over part of the length. Since construction in 1961 the coverplate terminations have been identified as critical locations for fatigue.

The investigation has been reported [1, 3, 4], and reasonably definitive conclusions reached [6]. However, the work was based upon detailed field measurements at only one site with only one weigh-in-motion station. Recent work has been completed on site dependent factors, which particularly affect the basic influence line and the relationship of impact to speed. Some findings are presented herein.

The initial work was carried out on the shortest span, where calculations revealed the greatest likelihood of fatigue damage. The study was extended to another site with three spans, and then to sites with longer spans. The additional study has provided the information on site dependent parameters.

2. DETERMINATION OF LOADS

2.1 Data Sources

In the context of short span bridges loads are fully defined in terms of axle or wheel load magnitudes *and* spacing. It is quite feasible to reconstruct past or forecast future loads from train consists and timetables. When this method is used to determine loads it is usual to accept manufacturer's specifications of unladen mass, and to assume that the mass is equally loaded on all wheels. For an initial assessment of fatigue damage this data is sufficiently reliable - perhaps more reliable than the data on load effects and on fatigue strength.

When an unfavourable fatigue life prediction is found using paper records to derive loads it often becomes necessary to improve confidence in load estimation by calibration measurements in the field. Unladen weights of nominally identical locomotives and wagons are found to vary significantly, and often to exceed manufacturer's specifications. Moreover, the distribution of mass over the wheels, particularly for locomotives with six axles, is found to be far from uniform. A previous study [6] found measured masses averaging 11% higher than the estimates provided by the railway authority. (The field measurement is itself subject to error.)

Error in vehicle mass estimation is important for all spans, and unequal distribution of mass to the wheels is important for bridges of short span.

2.2 Measurement Program

A test site was selected at Wallan, 25 km north of Melbourne, where an open deck bridge consisting of three nominally identical simply-supported spans of 6.92 m is located on tangent track. Three complete dynamic weighing stations were established nearby.

The weighing stations followed a well established procedure. They consisted of eight strain gauges fixed to the web of the rail in a segment spanning between sleepers. A flat-topped signal is received while a wheel is within the gauged length. The amplitude gives the wheel load. The weighing station must first be calibrated with a load cell. At speed the flat-topped load signal tends to be headed by a dynamic "spike", and followed by a negative spike. An algorithm which ignores this spike in determining load gives more consistent results.

A test train consisting of a diesel locomotive and two wagons was run many times at various speeds in each direction through the system. Individual wheel loads, midspan flexural stresses, and some stresses at coverplate terminations were recorded. The individual wheel loads, which exhibited considerable scatter, were aggregated into axle, bogie (truck) and vehicle loads. The scatter was significantly reduced at each step.

2.3 Load Measurement Results

The results of vehicle mass measurements are given in Table 1. The ideal weighing station is one which gives the same vehicle mass with negligible scatter, independent of speed. The results are far from ideal. At low speeds the weighing stations give results acceptably close to values established by weighbridge (also subject to error of measurement). With increasing speed apparent mass increased significantly at the first weighing station and decreases significantly at the second. Little change was observed at the remaining weighing station. At any individual weighing station scatter (standard deviation) was limited and did not increase significantly with speed. The variations were more marked for the locomotive than for the wagons.

The results show that dynamic weighing measurements are *speed dependent* and also *site dependent.* An effective calibration would need to be done for speed as well as static load. This calibration would be imprecise because the dynamic effects are different for different locomotives and wagons.

The three weighing sites appeared to be identical, but they were at different distances from the bridge. Since the bridge interrupts the continuity of track modulus properties it can be expected to introduce dynamic effects, but these are impossible to quantify.

3. ESTIMATION OF LOAD EFFECTS

3.1 Analytical Models

All the structures in this study are nominally statically determinate. In design the secondary beneficial effects are neglected, but in review for remaining life they must be considered if an accurate prediction is to be made. Figure 1 depicts a typical influence line for flexural stress in a simply-supported beam. Field measurements reveal differences which are particularly significant for short spans. The primary reason for the differences in railway bridges is the spanning effect of the track, consisting of rails, sleepers, sometimes ballast, and sometimes timber or reinforced concrete deck [9].

The spanning capability of these elements rounds off the peak of the influence line, and extends the base length. The peak reduction is typically 10% for open deck short spans, and more than 25% for short spans with nominally non-composite reinforced concrete decks. Careful modeling of the secondary elements has achieved compatibility between analysis and measurement [9].

Other sources of stress reduction, which are difficult to model analytically, are dispersion of concentrated loads through the depth of the web, and arching under live load due to friction or seizure of the bearings.

In other field work on cross beams which are assumed to be simply supported on main girders, some degree of end restraint associated with the torsional rigidity of the main girders is always found, leading to a reduction in peak stress.

The extension of base length of the positive component of the influence line, and the extension of a small negative component both contribute to a reduction in fatigue damage. The range of the stress cycle associated with the passage of each wagon is reduced. There is a dramatic reduction in the number of significant stress cycles per train once the base length of the influence line exceeds approximately 1.5 times the clear distance between axles of a typical wagon.

3.2 Site Measurements

Site measurements have been carried out at two levels of sophistication. At the simpler level the measured static peak stress is compared with the analytical prediction. A better understanding of the system is obtained if the influence line is obtained by test. In each case a test train with measured wheel loads is used, and the "static" test is in effect a slow moving test at 5 kph or less, where the position of the train is closely monitored to obtain the influence line.

The use of a train rather than a single axle load makes the determination of the influence line a difficult numerical process, which is sensitive to measurement errors. A single axle loading is not practical, although it has been attempted. With a single axle lifting of the rail away from the point of load application introduces non-linear behaviour which is partially suppressed in practice by neighbouring axles.

Some scatter is always observed on repeated measurement of peak stress or influence line. Apart from errors of measurement the system is in a different state of initial stress after each passage of a train due to the non-linear effects of friction and slip of the bearings and the track system on the beams.

After the static calibration test runs are made at various speeds in each direction to provide statistical data on the effect of speed. It is regrettable that early studies of impact between 1940 and 1960 discarded all the test data except peak values considered significant, thereby losing the statistical properties of impact.

Some of the results at the Wallan site are given in Table 2 and Figure 2. At this site there are three consecutive spans of 6.92 m consisting of a pair of stringers which are taper flange steel joists with welded coverplates. The bridge is open deck (no ballast), and the rail continuously welded on timber sleepers. Effectively the three spans are three sites for the purpose of identifying site dependent parameters.

The results given are the peak stresses recorded at midspan of each girder for each test run, using the test train documented in Table 1. For fatigue damage all stress cycles are significant, but the volume of data and its interpretation is too extensive to report here.

Table 1 - Mass Measurements at Wallan

			All Locations		For Each Loocation					
	Speed kph	No.Runs	Mean A Tonne	Std A Tonne	Mean 1 Tonne	Std 1 Tonne	Mean 2 Tonne	Std 2 Tonne	Mean 4 Tonne	Std 4 Tonne
G523	5	6	127.63	1.64	127.19	1.04	126.66	1.53	129.32	0.94
126.91	10	8	128.56	1.14	128.03	0.51	127.74	0.52	130.14	0.34
	15	5	128.75	0.86	128.32	0.26	128.23	0.41	129.93	0.59
	30	4	127.85	2.96	130.19	1.84	124.92	2.09	128.63	1.55
	60	3	129.76	8.32	141.00	0.90	121.37	1.10	126.90	0.96
	80	2	128.01	10.92	142.04	1.94	115.60	0.83	126.40	0.15
	95	4	126.79	11.53	140.75	2.45	112.92	1.51	126.70	1.86
NOBX	5	6	72.55	0.76	72.64	0.56	72.13	0.89	72.94	0.49
76	10	8	73.09	0.70	73.76	0.61	72.38	0.21	73.15	0.13
	15	5	73.28	0.45	73.00	0.23	73.75	0.13	73.03	0.43
	30	4	72.75	0.94	73.51	0.05	71.75	0.84	73.06	0.32
	60	3	72.22	1.39	74.11	0.14	71.65	0.32	70.91	0.21
	80	2	72.83	0.97	73.76	0.10	71.51	0.12	73.22	0.20
	95	4	74.21	5.40	81.57	1.84	70.24	0.80	70.83	1.37
AOCX	5	6	74.88	0.89	74.62	0.53	74.48	1.01	75.66	0.47
78	10	8	75.69	0.65	75.73	0.46	75.07	0.39	76.38	0.27
	15	5	75.25	0.94	74.97	0.57	74.77	0.83	76.52	0.18
	30	4	74.87	1.14	75.16	0.57	73.88	1.13	75.81	0.56
	60	3	74.62	1.06	76.06	0.34	73.74	0.31	74.06	0.15
	80	2	74.93	0.46	74.77	0.07	74.73	0.51	75.30	0.41
	95	4	74.74	3.06	78.52	1.68	71.88	0.98	73.82	0.99

Table 2 - Peak Stresses (MPa) at Wallan, 6.9 m Span

Speed kph	No.Runs		R1	L1	R2	L2	R3	L3	CPT
0 - 20	19	Mean	41.01	42.75	38.36	40.70	39.56	39.59	36.70
		S.Dev	1.08	0.72	0.87	0.44	0.63	0.36	0.42
30 - 60	6	Mean	42.52	41.44	40.51	41.60	41.53	42.56	36.97
		S.Dev	1.66	0.87	1.17	0.52	0.35	0.67	0.81
80 - 100	7	Mean	47.63	45.42	43.81	43.61	45.68	43.03	41.26
		S.Dev	0.89	1.71	0.62	0.70	1.06	0.72	2.66
All Runs	32	Max	48.80	46.92	44.82	44.58	47.00	43.92	45.66
		Min	38.62	40.34	36.16	39.34	38.62	38.62	36.04
		Mean	42.74	43.09	39.96	41.51	41.33	40.94	37.67
		S.Dev	2.90	1.69	2.37	1.28	2.57	1.68	2.21

Table 3 - Measured Peak Stresses (MPa) at Mangalore, 9.9 m Span

Speed (kph)	No.Runs		Mid 1	Mid 2	Mid 3	Mid 4	CPT 1	CPT 2	CPT 3	CPT 4
0 - 20	17	Mean	29.08	36.06	44.68	25.84	33.68	31.79	45.74	22.71
		S.Dev.	0.59	0.34	0.68	1.30	1.08	2.38	2.03	0.88
30 - 60	25	Mean	30.48	37.57	46.65	28.03	34.87	33.58	47.17	24.95
		S.Dev.	0.55	0.46	1.04	1.44	2.72	4.23	1.53	0.98
80 - 100	9	Mean	33.95	41.03	47.15	34.94	37.89	36.69	50.16	28.30
		S.Dev.	1.47	1.72	6.75	8.61	2.57	3.19	2.51	1.52
All runs	51	Mean	30.63	37.68	46.07	28.53	35.01	33.53	47.22	24.79
		S.Dev.	1.86	1.90	3.15	4.99	2.69	3.89	2.44	2.20
		Max	37.20	44.48	51.90	58.90	42.03	40.82	54.16	30.76
		Min	28.22	35.65	28.49	24.06	30.02	28.49	42.50	21.35

Table 4 - Properties of Fatigue Life Variables

Variable	Mean	Coefficient of variation	Remarks
κ	1.00	0.24	Fatigue strength
α	0.87	0.08	Influence line peak (measured/theory)
ψ	1.11	0.11	Axle loads (measured/estimated)
ϕ	1.19	0.04	Measured impact factor
λ_T	1.16	0.28	Damage per train factor

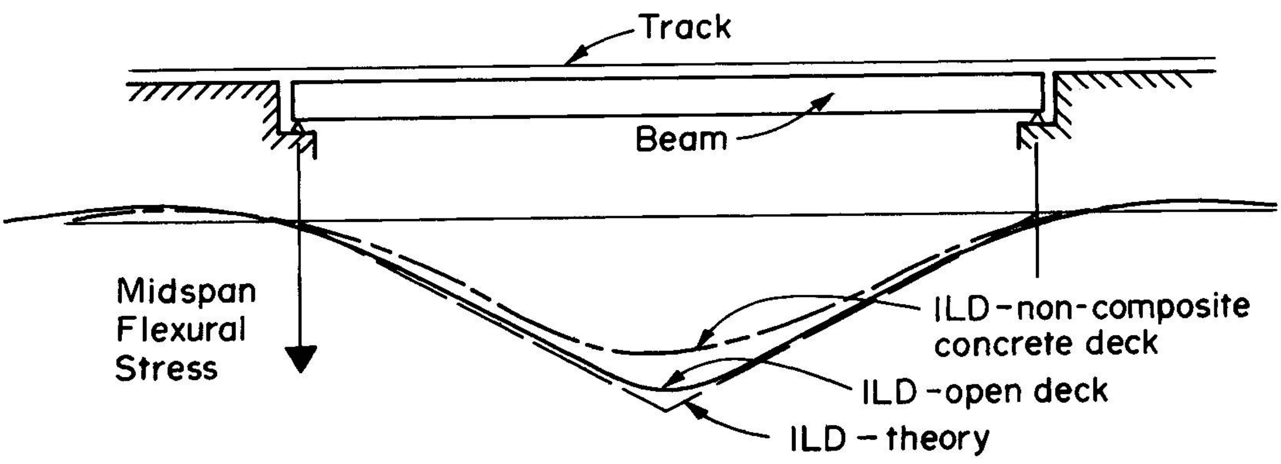

Figure 1 - Influence lines for a simply-supported stringer supporting railway

Further results are given in Table 3 and Figure 3 for a bridge at Mangalore, 110 km north of Melbourne. This is an open deck bridge of three spans of 9.9 m. Each span is supported by four stringer beams in parallel with welded coverplates over part of the span. Table 3 gives the statistical results of 50 test runs with the train consisting of locomotive G523 and wagon NOBX. Half the runs were made with the train headed "up", and half "down". Table 3 gives the stresses at midspan and at coverplate terminations at one end of all four girders in one span. Only the stresses at coverplate terminations are documented in Figure 3.

The results are discussed in the following sub-chapters.

3.3 Site Dependency

Of the six girders monitored at Wallan the highest mean peak stress (all runs) at 43.09 MPa was 7.8% higher than the lowest of 39.96 MPa, shown in Table 2. This is not a surprising result. It is probable that data from other bridges of identical specification would increase the scatter, but this data has not been collected. The scatter revealed by these tests is therefore not a conservative estimate of the scatter to be expected for all similar spans within the railway network.

Sufficient data has been gathered to show that, given the rather small standard deviation on peak stress for any one girder, the difference in mean stress between girders is systematic, not stochastic.

3.4 Effect of Speed

A major gap in design data lies in an impact factor (dynamic increment) to be used for the purposes of estimating fatigue damage. Bridge design specifications give impact factors as a function of speed and span, but these are only suitable for determining peak load effects. Fatigue damage accumulates for all stress cycles, so that the *mean* impact factor provides a more creditable estimate of damage to be expected. The magnitude of the standard deviation of impact factor has only a secondary effect, depending upon the value of the exponent, m, used in cumulative damage calculations.

An idea of the effect of speed can be drawn from Figures 2 and 3. There is apparently a slight dynamic effect at 60 kph compared with crawling speeds - about 4% increment. There appears to be a significant step between 60 kph and 80 kph - about 15% increment compared with crawling speeds. To facilitate this assessment the data have been divided into three speed groups for the two bridges - Slow: 0-18 kph, Medium: 27-66 kph, and Fast: 80-99 kph.

The statistical results appear in Tables 2 and 3. The dynamic increment is also shown. The low scatter in each speed group indicates a fairly reliable estimate of impact.

The impact factor from the AREA Specification for the short span and a speed of 80 kph is 1.58, which is vastly exceeds any values measured here or on any other bridge checked by the authors.

3.5 Redundant Load Paths

With four girders in parallel the bridge at Mangalore can be described as a redundant load path structure. The method of construction is simple, with timber cross members sitting on the four stringers. It is inevitable that the cross-member sits on just two of the stringers until deflection under load brings it in contact with the remainder. Table 3 and Figure 3 show very unequal distribution of load between the stringers, exceeding all expectations. The mean peak stress of the most heavily loaded beam is 47.22 MPa compared with an average of 35.14 MPa. The stresses in the most highly stressed girder at the coverplate termination exceed the constant amplitude fatigue limit, whereas the average peak stresses do not.

The paradox of this result is that redundancy has greatly increased the likelihood of fatigue damage occurring within design life, but not of catastrophic collapse. This poses problems for the owner of the system with limited resources to allocate for inspection of fatigue sensitive details in both redundant and non-redundant load path structures.

4. ESTIMATION OF FATIGUE STRENGTH

4.1 Use of Detail Classification

For fatigue strength reliance must be placed on the detail classes found in structural codes of practice. Information on mean fatigue life as well as design (mean minus two standard deviations) fatigue life is useful if a risk assessment is to be made. However, compromises have been made to present design curves which are simple to use, leading to some inaccuracies.

Some aspects of fatigue strength are still being developed. Two areas of significance are thickness effects and fatigue strength at very high cycles.

Thickness effects are specifically mentioned in relation to welded coverplates. The transition in Detail Class occurs at 25 mm thickness, and the change is sufficient to halve the fatigue life (for exponent, m, equal to 3.0). The coverplates in question have thicknesses of 22 mm and 25 mm, making the interpretation of fatigue strength difficult.

In addition to the thickness question the coverplates overlapped the flanges, and were tapered at the ends, with a continuous fillet weld wrapping around over and under the coverplate. The weld had a leg length of 8-10 mm - small for the thickness of plate.

It was evident that there were enough differences between this detail and those used in laboratory tests for doubt concerning the choice of Detail Class. To improve confidence in predicting remaining life this doubt had to be reduced.

4.2 Laboratory Tests

There is a paucity of test data at very high cycles, exceeding 20 million cycles. For short span bridges more than 100 million cycles of varying amplitude can be expected within the design life. The test data statistics with regard to standard deviation really only apply where most of the test data lie, less than 10 million cycles. It has been reported previously [6, 7, 10] that tests on girders taken out of service provided four new data points in excess of 20 million cycles, one of which was a run out without failure. Even to get these data the Miner's mean stress range was much greater than occurred in practice, with block loadings up to 30% higher than the maximum stress range observed in the field.

The tests were too few in number for statistical inference on their own, but the results were compatible with the higher strength Detail Class (no thickness effect).

The tests were most valuable for two observed phenomena. Firstly, the cracks initiated at the root of the fillet weld and propagated for quite some time through the weld before entering the flange and reducing the effective cross section. Secondly, cracks were clearly visible for more than half the life, and growth rates were virtually identical for all tests. Both observations were important for subsequent analytical modeling of behaviour.

Normally laboratory fatigue tests are too expensive or simply not feasible. In this case the large number of identical bridges at risk justified the investigation.

4.3 Analytical Refinement

When the Detail Class is uncertain a fracture mechanics analysis with finite elements incorporating the quarter point crack tip element will provide useful information - at a cost. The uncertainties surrounding the detail on the bridges investigated prompted the finite element analysis [7, 10]. The result has to be calibrated against the standard detail analysed by the same method.

In this case the analysis found the detail close to the normal coverplate termination in behaviour. It also found that the crack was more likely to grow from an initial defect at the root rather than the toe of the fillet weld.

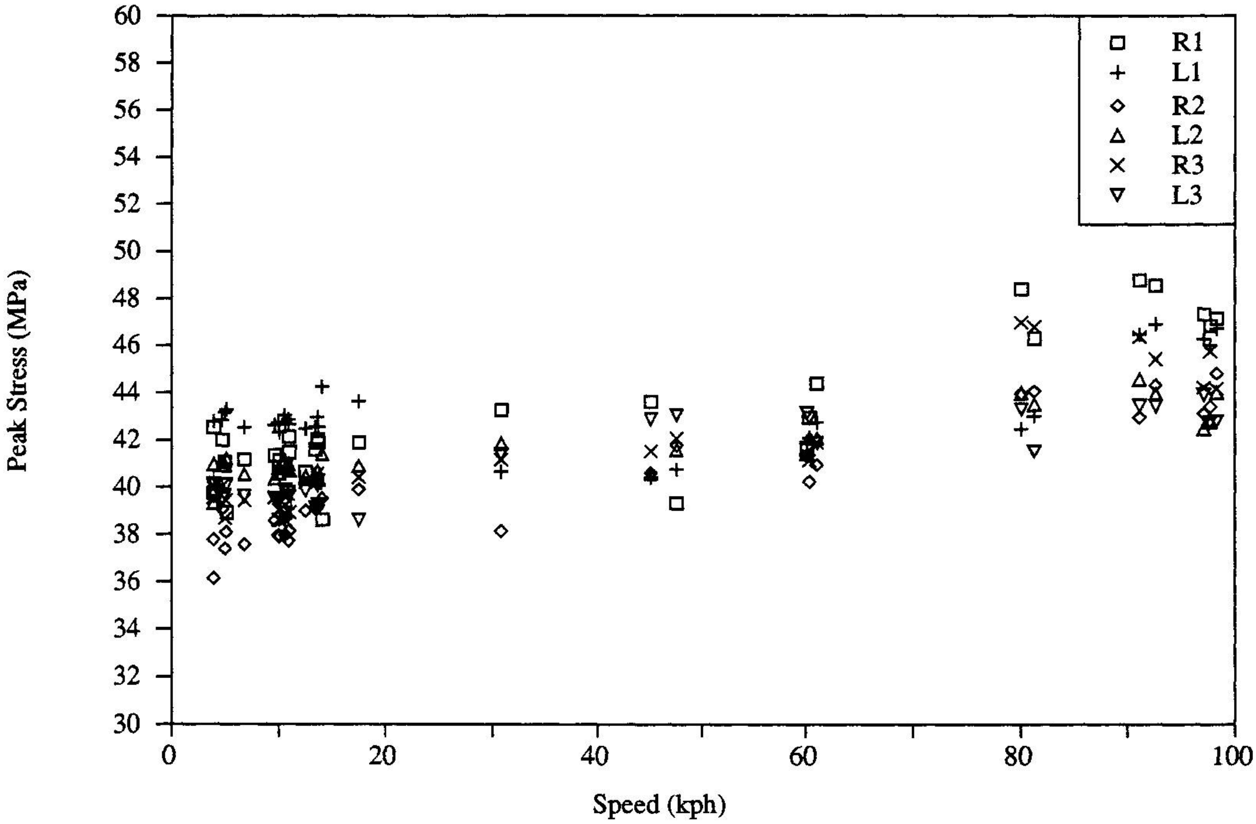

Figure 2 - Midspan stresses on three 6.9 m spans at Wallan

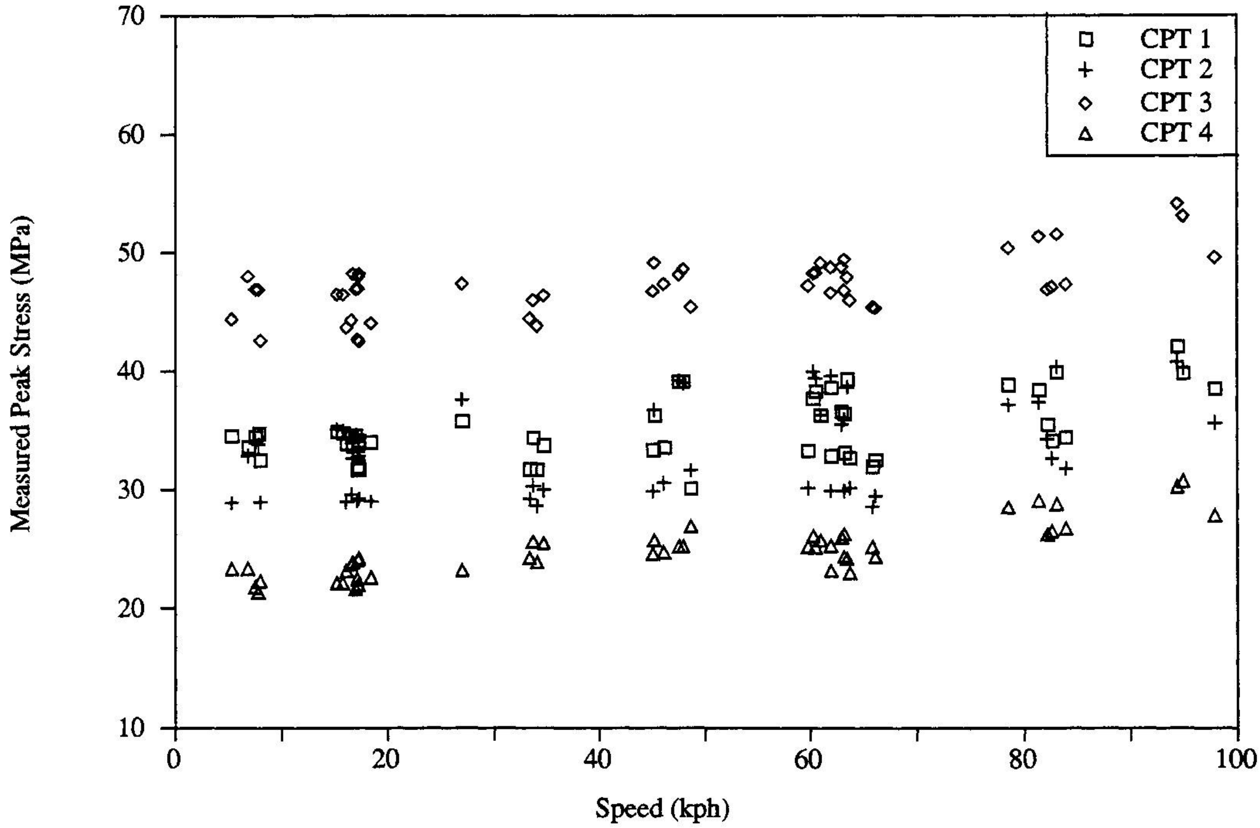

Figure 3 - Stresses at coverplate terminations, 9.9 m span at Mangalore

The fracture mechanics analysis called for an effort comparable to that for the laboratory tests. In normal practice it could only be justified when a detail does not match an existing Detail Class.

4.4 Significance of Detectable Crack Growth Phase

The constant crack growth phase leads to the conjecture that a statistical model assuming normal distribution of life will lead to conservative estimates of a lower confidence limit. It also permits the development of a rational inspection program which will detect damage before it is critical.

5. USE OF SITE MEASUREMENTS

5.1 Verification of Analytical Models

In this study measurements on site were used to establish wheel loads, influence lines and impact factors. Other important phenomena were revealed by site measurements.

Wheel load measurements were useful provided that the effects of speed were recognised and allowed for. Apart from the calibration runs reported herein, the weighing stations were useful for collecting data on regular traffic, which could be related to stress histories on the bridges.

The field measurements of influence lines could be reconciled with improved analytical models. This gave confidence to the determination of influence lines for other short span bridges without the luxury of site measurements.

Although no analytical model of the observed impact factor is available, the value, typically 1.15 for speeds up to 100 kph provides a starting point for assessing fatigue damage of short span bridges similar in character to those which were instrumented. The site measurements revealed that the impact effect was primarily due to the vibration of the bridge in its fundamental mode.

5.2 Direct Prediction of Fatigue Damage

The large scale field measurement program undertaken was justified on account of the large number of structures potentially at risk. It gave an insight into the basic parameters relevant to remaining life, so that rational decisions could be made for the entire system. Such a program would not be economically justified for a unique structure.

A less costly procedure in the case of a unique structure would be to identify the potentially critical locations, to attach strain gauges, and to gather the stress history over a sample period of time. Equipment now exists for cumulative Rainflow analysis of data in real time. This procedure eliminates the need to estimate loads and to calculate load effects. In fact, the uncertainty regarding prediction of fatigue life is reduced at the instrumented site, compared with building a model of response through loads and load effects. This approach, however, does not provide the insights which enable the observations to be applied to other situations without instrumentation.

5.3 Site Dependency

The investigation has clearly demonstrated that at every stage variations in the parameters occur which are associated with the particular site. The parameters are

- axle load distribution
- vehicle mass measurement
- system influence line (surface), i.e., load effect
- dynamic increment versus speed
- notch geometry (weld profile)
- defect type, size and location (for fatigue crack origin).

There are thus two parameters each relating to loads, load effects and resistance, although the last two related to fatigue strength are normally deemed to be allowed for in the statistics of fatigue strength, and as such are site independent.

6. PREDICTION OF REMAINING LIFE

6.1 Previous Predictions

The predicted life has been the subject of successive refinements [6]. The first estimates were alarmingly short, prompting the field and laboratory testing to replace guesses with measured values. With parameters based on data, and using the more accurate two-slope fatigue strength curve (since most stress cycles were below the constant amplitude fatigue limit), the mean predicted life was 77 years, with a standard deviation of 14 years. This prediction assumes no error in identifying the detail as AREA Category E. This result was based upon the statistical data from field measurements given in Table 4 [6]. In using this data it was assumed that constant amplitude fatigue limit was considered deterministic.

6.2 Revised Predictions

In the light of more recent work on loads, and the discovery that dynamic weighing is speed sensitive, the previously measured wheel load load factor of 1.11 is suspect. If this factor is ignored the predicted mean is increased by 37% for m = 3.0, and 68.5% for m = 5. The latter figure is more likely as the reduction drops most of the stresses into the lower range of the two-slope fatigue life curve.

In addition, with peak stresses now typically 45-55 MPa, and treating the constant amplitude fatigue limit as a variable with the code value a lower confidence limit, a significant number of the details could be free of fatigue damage because the constant amplitude fatigue limit has never been reached.

6.3 Reconciliation with Observed Damage

The structures were put into service in 1961. The need to revise the predicted life is prompted by the observation that, for the 1,422 coverplate terminations on 6.92 m spans on the route in question, not one fatigue crack has been detected after 29 years in service. The apparent conflict between prediction and observation needs to be resolved.

Several explanations are possible. Firstly, the heaviest axle weights were 19 tonnes until 1977, when new locomotives with axle weights 22.5 tonnes were introduced. The chance of not exceeding the constant amplitude fatigue limit before 1977 was quite high. The effective period of damage so far would then be reduced to 12 years.

Secondly, even with the heavier axles, the constant amplitude fatigue limit might not have been exceeded in many cases.

Thirdly, for cracks initiating at the root of the weld, current inspection methods will not discover these until they reach the surface, which takes a large number of cycles.

7. CONCLUSIONS

7.1 Site Dependency

Many key factors affecting stress amplitudes in critical locations for fatigue have been shown to be site dependent. Accurate life predictions will always require some site measurement to obtain acceptable confidence limits to the predictions.

Where a critical detail is repeated at many nominally identical sites sufficient data must be gathered at several sites to quantify site dependent variability.

7.2 High Cycle Life Predictions

Life predictions for bridges are being made for numbers of cycles far in excess of those applied in laboratories to establish fatigue strength curves. Coefficients of variation given for laboratory data can only be applied to structures in service with lives similar to laboratory specimens, typically less than five million cycles. Extrapolation to 50 million or more cycles introduces additional uncertainty.

7.3 Dynamic Effects

A serviceability impact factor is badly needed for estimating fatigue life of bridges, to replace the current ultimate or maximum impact factor. Measured values near 1.15 have been found where the maximum specified by codes is 1.58.

7.4 Redundancy

The greater safety provided by redundant load path structures may be offset by the greater variability of distribution of load between the redundant elements.

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