Fatigue cracks induced by traffic loading on steel bridges’ slender orthotropic decks

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Abstract

Both static and dynamic problems of contact pressures and interaction between vehicle tyres, flexible asphalt pavement and the steel structure of slender orthotropic decks are briefly brought into focus. The main causes of occurrence of observed cracks in an actual large steel bridge which has been, under random traffic loading, frequently damaged by fatigue of welded joints and details are then unveiled. This is done with the aid of numerical modeling and experimental strain measurements performed both in situ and in the laboratory on a prototype scale model of the actual bridge’s steel orthotropic deck, having longitudinal stiffeners of trapezoidal cross-section.

Refined numerical results from the experimentally calibrated finite element model are then used to address the main problems and to envisage solutions that would enhance the ultimate fatigue life of this and other similar steel decks, together with their pavements.

1 Introduction

Slender steel orthotropic decks, having thin-walled longitudinal stiffeners of trapezoidal cross-section, are very common structural components often found in steel box-girders of highway bridges, as illustrated in Fig. 1. Although common in the sense of their use, they are very complex as far as their local and overall static and dynamic behaviour under traffic loading is concerned.

Slenderness of the deck plate and ribs (stringers) brings some important advantages to these structures, such as low height and weight combined with great longitudinal bending stiffness. Alternatively, slenderness also brings higher sensitivity of local stresses to a series of relevant factors, which appear in both static and dynamic problems of contact pressures induced by vehicles’ pneumatic tyres on the pavement and the interaction between these tyres, the flexible asphalt pavement and the slender steel orthotropic deck. Local stresses in the
Figures 1 - Steel orthotropic deck. (a) typical bridge steel box girder with orthotropic deck. (b) detail of slenderness trapezoidal ribs and deck plate, also showing typical instrumentation with strain-gages. (c) longitudinal section of the deck, transversal welded joint and splice.
A steel structure, particularly those related to transverse bending moments, are most sensitive to:

- transverse location of the tyre's contact area with respect to the stiffeners' webs;
- the tyre's contact area, which depends on its size and on the wheel's radius, thickness and hardness of its rubber band and, of course, on its pneumatic pressures and varying loading on the wheel axle;
- geometric imperfections of the thin-walled steel plates that compound the orthotropic deck, leading to strong localized interaction between membrane and bending stiffnesses in the region under contact pressures;
- deviations from the nominal thickness of the thin laminated steel plates;
- effects of welding residual stresses in the thin plate connections leading to geometrical distortions;
- roughness, flaws, deterioration and overall geometric irregularities of the pavement surface, which cause dynamic variations of contact area and applied pressure, and may induce amplification of dynamic loading and therefore of local stresses;
- Multi-mode vibration behaviour, with associated clustered natural frequencies, typical of these easy-exciting slender orthotropic decks.

Hence, because all these sensitivity factors have an inherent and strong random character, one can promptly grasp the high degree of complexity of these common and slender structural steel components. Besides, these factors are interdependent, and therefore random stress variations that may cause premature fatigue cracks in welded details depend simultaneously on them all, in a complex and evolutive cycle of cause and effect.

To unveil the main causes of occurrence of observed cracks in actual structures, a series of in situ experimental strain (or stress) measurement campaigns were carried out by a team of researchers and engineers with the COPPE’s laboratory of structures. These experiments were also carried out on a prototype scale model (i.e. 1:1 geometric scale) of a steel deck shown in Photo 1[1,2]. This refers to the slenderer orthotropic deck (see Figs.1) of the Rio-Niteroi bridge which has been, under stochastic traffic loading, frequently damaged by fatigue cracks in the welded joints and details.

Refined numerical results from parametric studies were obtained with experimentally calibrated finite element models of the orthotropic deck (see Fig. 2), with two and three deck panels spanning on transverse floor beams. In turn these were used to further understand the behaviour and sensitivity of this structural component subjected to traffic loading. It can be noted in the detail shown in Fig. 2 the very localized effect of the wheel’s contact pressure on the transversal deformed shape. With a better insight into the highlighted problems, the calibrated computational model could be used to envisage solutions to attenuate the degree of sensitivity to so many random factors, as well as to enhance the ultimate fatigue life of this and other similar steel decks together with their pavements.

With regard to the service life of the flexible pavement itself, the multi-mode vibration characteristic of these structures (see Fig.3) has a deleterious effect on
Figure 2 - FEM model of the slender orthotropic steel deck and local transversal deformation under wheel loading.

Photo 1 - Prototype scale (i.e. 1:1 geometric scale) model of the Rio-Niteroi Bridge's orthotropic steel deck for static and dynamic test on the strong-slab of the COPPE's Laboratory of Structures.
the performance of the asphaltic concrete and its adhesive layer to the steel plate, leading to premature fatigue damage and disruption of the pavement. The service life of asphalt pavement is often reduced to less than 6 months.

2 In situ experimental measurements

Vibration modes and related frequencies were experimentally measured on the actual steel deck of Rio-Niteroi bridge, by installing micro-accelerometers underneath it. Fig. 3 shows a typical frequency spectrum of vertical accelerations at the center of a deck panel in between box-girders webs and two consecutive floor-beams.

Strain measurements were also carried out on this steel deck, which was subjected to both normal traffic of heavy vehicles and a controlled loading given by a known weighed three-axle truck. Over one hundred of strain-gages were installed underneath the slender part of the steel orthotropic deck of this bridge. Figs 1b and 1c illustrate typical transversal and longitudinal sections of the deck with some of the installed strain-gauges. Traffic on this bridge, brought into service in 1974, has risen to figures well beyond the initial estimates reaching now over one hundred thousand vehicles per day; almost 20% of these being heavy trucks.

Typical stress responses under either normal traffic loading or controlled truck have similar features, with variations in peak amplitudes dependent on differences in truck weights and on some dynamic amplification due to local irregularities of the pavement surface. Typical stress responses obtained from longitudinal and transverse measured strains are shown in Figs. 4a and 4b, where the peak amplitudes associated with the passage of each of the three wheel axles of the truck under controlled speed (60 km/h) and weight (20 tons) can be seen. What can be readily observed in these figures is the sharp effect of wheels on the resultant stresses, particularly on the transverse ones.

On the other hand the randomness of these stresses is only captured for long term measurement campaigns on the actual steel deck under normal daily traffic loading. This is shown by histograms of stress illustrated in Figs. 5a and 5b, respectively for the transverse and longitudinal stresses generated at welded details by the corresponding bending actions and vibration characteristics. A direct comparison between current measurements and measurements taken around 20 years ago, which were taken just after the bridge was open to traffic, are also shown in these figures. What is noticeable in these figures is the current higher percentage of lower stresses. This feature may be explained (or interpreted) by the following attenuating effects occurring along the years:

- decreasing of individual wheels loads with the increase of number of axles of the heavy vehicles;
- reduction of contact pressure with the increase in size of tyres and consequently their contact area with pavement;
- new and more effective suspensions of the modern heavy vehicles;
- reduction of the excitation induced by the wheels rolling on rough and uneven surface with the increase in the speed of heavy vehicles.
Figure 3 - Frequency response function of vertical acceleration at the center of the deck panel under impact excitation.

Figures 4 - Typical stress responses at mid-span of the deck panel due to a three axles truck passing at controlled speed (60 km/h). (a) longitudinal stress at bottom flange of a rib. (b) transversal stress at the rib web to deck plate welded connection (see Fig. 1).
3 Fatigue life estimation

The implications of changes in the distribution and range of stress variation on fatigue life estimates is remarkable. The large differences found in terms of years to a crack initiate at each one of the most relevant welded joint details (fillet welds with partial penetration) are summarized in Table 1. The results were obtained by applying the Miner-Palgren rule for calculating cumulative damage, appropriate S-N curves [3] and taking either stress histograms as if they were obtained from measurements taken around the same date (in 1974) just after the bridge was brought into service. The intention here with this straightforward comparison is twofold: (i) to show that traffic loading is less adverse today than it was in the past, as far as fatigue problems are concerned – the evidence for this is given in the last paragraph of previous section; (ii) to demonstrate the great sensitivity of the transverse bending stresses (at the stringer webs to deck plate fillet welded joint) to the various random factors mentioned previously along with the introductory remarks to this paper.

Table 1 – Fatigue life estimates for welded details

<table>
<thead>
<tr>
<th>Welded joints details (fillet weld with partial penetration)</th>
<th>Welded detail A*</th>
<th>Welded detail B**</th>
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</thead>
<tbody>
<tr>
<td>Estimates† for LEHIGH (1976) measurements‡‡</td>
<td>6.3</td>
<td>8.7</td>
</tr>
<tr>
<td>Estimates‡ for COPPETEC (1997) measurements‡‡</td>
<td>10.6</td>
<td>43.7</td>
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Notes: (+) Estimation of cumulative damage made from 1975 on.
(‡‡) Measurements are related to a situation of a relatively new pavement patch. A not too conservative multiplier coefficient (=1.4) was considered in the calculations of cumulative damage to take into account stress concentration and the increasingly deteriorated conditions of a pavement lacking adequate maintenance.
(*) SN curve G and SN curve W [3] applied respectively to welded details A and B corresponding respectively to approximately 15.9% and 2.3% probability of failure.

4 Correlations between theoretical and experimental results

Under the local action of the wheel loading the longitudinally stiffened anisotropic deck, in between two consecutive floor-beams, displays a deformed shape as shown by detail in Fig.2b. Table 2 presents a comparison between theoretical and experimental stress values at the indicated locations and directions in the structure (see Figs.1b and 1c) obtained from the numerical model (see Fig.2) and from tests on the actual steel deck under controlled loading of a known truck crawling along a marked alignment on the pavement within a traffic lane. As it can be seen in this table, the theoretical and experimental longitudinal stresses at the bottom flange of the ribs compares very favorably,
while transversal bending stresses show very poor correlations depending on their particular locations. The latter results show once more the sensitivity of the transversal bending stresses to some of the already cited random factors. In regard to the found discrepancies related to the transversal bending stresses, it is worth stating that no significant contribution to either global or local bending stiffness of the deck has been found to come from the flexible asphalt pavement. Enlarging of the effective contact area to account for pressure spreading through pavement thickness is an assumed, but not necessarily correct, effect contribution. More important is the effect of tyres contact hardness.

Table 2 Comparison between experimental and theoretical stress values at strain gage locations indicated in Fig.1b obtained from tests on the actual steel deck.

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<th>analysis</th>
<th>stress values (MPa) at strain-gages locations</th>
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<td>experimental</td>
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<td>theoretical</td>
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<td>soft contact</td>
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<td>hard contact</td>
<td>11</td>
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Then, in order to have reasonable control over some of these factors – at least those associated with the size and location of the contact area between wheels tyres and the deck plate, plus the softness of the tyre’s contact with the road surface – an experimental programme has been carried out at COPPE’s laboratory of structures in which extensive strain measurements have been made on a well instrumented prototype scale model (i.e. a model in 1:1 geometric scale factor; see Photo 1) of the actual steel orthotropic deck. By being able to measure both strains and the initial size and location of the contact areas, besides [as well as?] the variation of these areas with the progressive deformation of the pneumatic tyres under increasing applied load, it became possible to better understand the local structural behaviour and to achieve a better correlation for the theoretical bending stresses. Table 3 presents a comparison between theoretical and experimental stresses at the locations on the structure’s model indicated in the enlarged detail in Fig.6, which also shows the transversal bending moment diagram obtained from the numerical model. What can now be readily seen by comparing results given in Table 2 and in Table 3 is the better agreement between theoretical and experimental results obtained under well controlled loading device and measured strains, together with a more precise measurement data for the location, magnitude of the wheel load and variation in size of the contact pressure area.

Table 3 Comparison between experimental and theoretical stress values at locations indicated in Fig.6 obtained from tests on the prototype scale model.

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<thead>
<tr>
<th>analysis</th>
<th>stress values (MPa) at strain-gages locations</th>
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<td>experimental</td>
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<td>theoretical</td>
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<td>soft contact</td>
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<td>hard contact</td>
<td>-76</td>
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<td></td>
<td>-73</td>
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Figures 5 - Histograms of stress variations under daily traffic loading. (a) transverse stress at toe of fillet weld on rib web. (b) longitudinal stress on rib flange at toe of fillet weld of splice plate.

Figure 6 - Details of transverse bending moment distribution and gage locations on the prototype scale model tested in laboratory.
5 Concluding remarks

It has been shown that the complex behaviour of slender steel orthotropic decks of highway bridges can be unveiled with the aid of careful experimental measurements of strains – both in actual structure and in its prototype scaled model – and of numerical results from a refined finite element model of the 3D structure composed by thin-walled deck plate and folded-plate ribs with trapezoidal cross-section.

From the obtained experimental and computational results one may conclude that the stress variations due to the interaction between vehicles’ tyres, flexible pavements and the steel orthotropic deck has strong local and random characteristics. These are exacerbated by a high sensitivity to inevitable deviations from certain given values of geometrical and physical parameters.

It has also been shown that theoretical fatigue life estimates during the designing stage of the welded steel joints and details of these structures are not an easy task. This is so because the amplitudes of stress variations in the welded connections of this slender and thin-walled plates are strongly sensitive to small variations in the magnitude of contact pressure and the size and location of contact area of wheel tyres on deck surface in relation to the stiffeners webs.

From the obtained results it becomes clear that the very slender orthotropic steel deck lacks transversal bending stiffness and proper damping in all multiple and clustered frequencies vibration modes. Any alternative remedial measure for reducing crack initiation and propagation in welded details of this structure should comply and fulfill these lacking properties.

Long-term experimental ‘in-situ’ measurements for monitoring more slender patchs of the orthotropic steel deck of Rio-Niteroi bridge structure are currently under way. Two alternative solutions have been envisaged and tested on the prototype scale model at COPPE’s laboratory of structures, and are to be tested under normal traffic loading very soon. In both alternatives the flexible asphalt pavement has been substituted for a reinforced concrete slab of equivalent thickness, in order to attenuate the stress amplitudes and enhance the ultimate fatigue life of these structural components together with durability of the new pavement. A full description of these structural alterations and their details are not included here due to lack of space.

References