Damage in Railway bridges caused by laden trains[1]

by

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Introduction

A recent opportunity to observe the response of a railway bridge to the passage of fully laden trains has shown that damage can be induced in bridges in a manner previously thought to be impossible.

In the conventional analysis of the response of a bridge to a heavily laden train, it is assumed that vertically acting forces are the design-limiting factor. However, the process of imparting forces to the bridge is not only dynamic, it is also highly non-linear. The imparting of a forcing function, transversally to the direction of the train, is occasioned by a hunting motion of the wheels of trucks on the track. The process is then modified by a variable influence that is a function of the speed of the train and the distance between wheels. The hunting motion (known as 'lacet') is a feature of the design of trains' wheels so as to avoid the appearance of wear preferentially in some parts of the wheels.

The result of this mechanism is that patterns of cracking in bridge supports that were previously not considered worth consideration, may in fact become the principal mechanism for the introduction of significant damage into a bridge system.

This paper describes the mechanism observed and the measurements from a full-scale bridge on which the mechanism was observed.

Forces imparted to a bridge by a train

The basic design philosophy for bridges is that they must support the mass of a fully laden train, that they must be resistant to horizontal forces induced by wind or earthquake, and they must be able to withstand the force created by the train under maximum braking. In each case, the forces generated are opposed by foundations that rely on resistance created by the mass of the Earth.

[1] Damstruct Symposium on Damaged Structures, Brazil, 2005

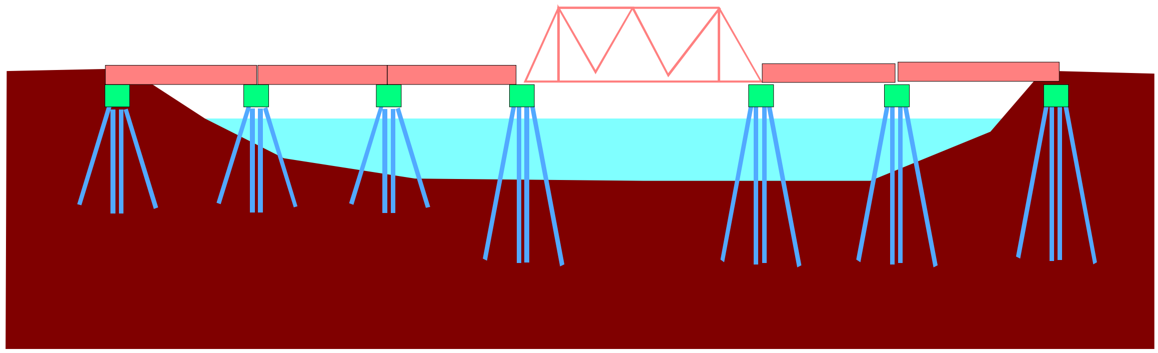

Fig 1: General Layout of the bridge

In the case under consideration the medium for the creation of a reaction to the imposed loads is straked piles driven into bedrock. In the case of the train undergoing full braking then the abutments at each end of the bridge create extra resistance. In the horizontal direction, the forces created by winds blowing along the valley, are normally considered to be the limiting case for design. However, in many locations in Brasil, the wind forces generated are rather light and, as a result, other mechanisms may be of more significance.

The response of a bridge to a train passing

The response of the bridge was monitored using accelerometers that were responsive in the three principal directions. The response of the bridge in the vertical sense, was monitored with a heavily laden train passing. The response shown in Fig. 2 is the type of response that would normally be expected in this situation, and is the condition for which the basic design caters.

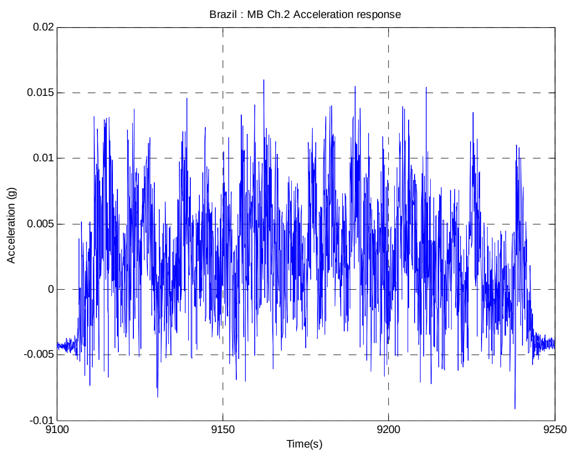

Fig 2: Vertical acceleration response of the bridge

In this case the response can be seen to be caused by the mass of the train. In this respect the mean level of response changes from just below zero to an average of 0.003g in fig 2.

This is accompanied by an increase in the dynamic response as shown by the peaks of the response.

The designer brought up t consider these actions in a conventional sense, will expect that the response will increase if the mass of the train increases. Unfortunately, the situation is much more complex.

In practice, the bridge has a series of modes of vibration that can be excited by the action of the train. If the train happens to produce a forcing function that coincides dynamically with a frequency of resonance of the bridge, then the response of the bridge is increased. In the vertical sense this can be achieved, even with a lightly loaded train, by running it at a speed that induces a frequency of excitation that corresponds with a resonance of the bridge. Fig 3 gives an example in which the response on this bridge in the vertical direction, is larger, for an empty train, than it was for a fully laden train. The detail in Fig 3 shows that a resonance at 1.1 Hz. is excited briefly by this empty train.

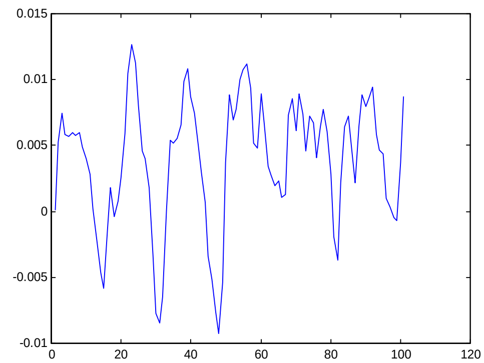

Fig 3: Response caused by an empty train during a 12 second period

The dynamic response of the bridge

The bridge itself was modeled, using the Microstran 8 analysis package. The dynamic response was calculated and validated with the measured response. The validation process was unusual, and particularly complex for this particular structure. Whilst there are several significant resonances in the structure, reference is made just to the first translational mode to make the point about the unusual nature of the damage to this structure.

In figure 4 the model shows the structure to have a fundamental mode of vibration at 0.49 Hz.. However, measurements indicated that the fundamental mode of vibration occurred at 0.60 Hz.. It is unusual that a damaged or aged structure has a higher frequency than that which would occur in the pristine state. This is because it is normal that the stiffness of the structure degrades with time, and since:

$$f_i = \sqrt{\frac{k_i}{m_i}} \quad (1)$$

Where, for mode i, f is the frequency of resonance, k is the modal stiffness, and m is the modal mass[1].

Thus, if the stiffness degrades, it would be expected that the frequency decreases.

In this case, observation of the mode shapes for the pristine structure (fig 4) and the damaged structure (fig 5), it can be seen that if damage occurs at the edge of the main trestle, then less of the structure is involved in the mode of vibration, and the modal mass decreases, with a concomitant increase in the frequency of resonance.

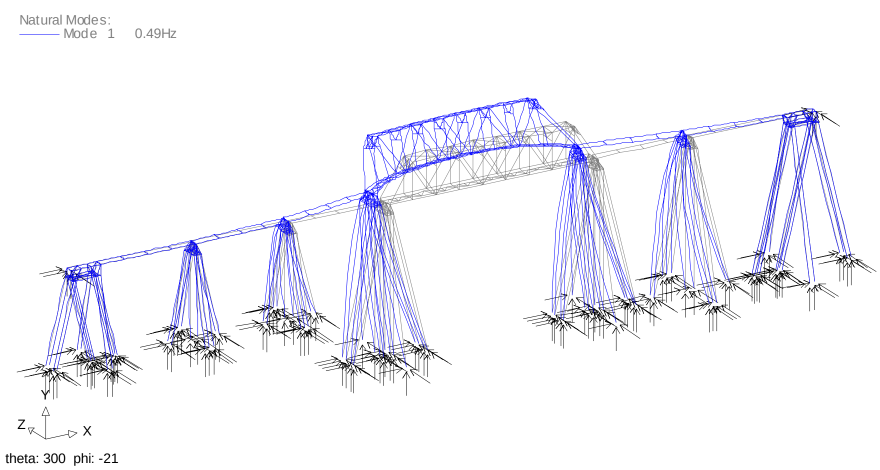

Fig 4: First mode of the undamaged model (0.49 Hz.)

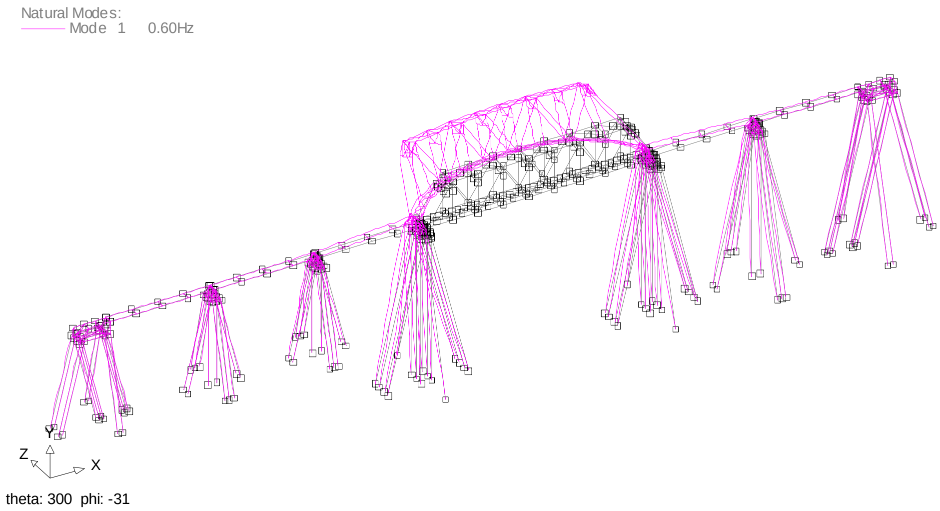

Fig 5: First mode of the model with damage introduced (0.60Hz.)

Unfortunately this observation implied that damage had been caused by the action of a force acting in the transverse horizontal sense. Since the bridge was located in a region where the design wind speed is a light 30 metres/second, this implied that a different action was responsible for the observed damage.

Measurement of horizontal forces induced by passing trains

The horizontal response was monitored with several fully laden trains passing. Fig 7 is typical of the responses obtained for such events.

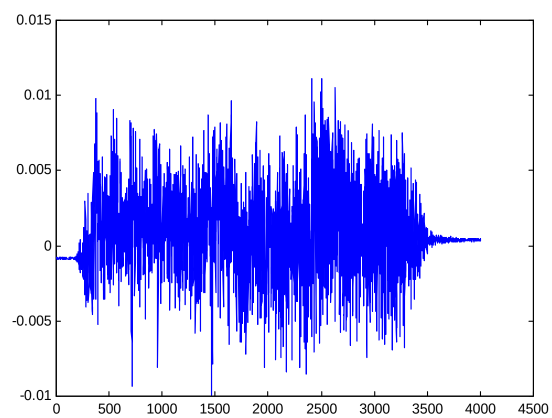

Fig 6: Horizontal response measured on the trelice

In this diagram it is possible to see the effect of the weight of the train on the bridge, and that the dynamic part of the response is not steady. In practice this non-steadiness of the dynamic response was investigated in more detail. Figure 7 is a detail from the event contained in Fig. 6. What it shows is that the maximum response (and as a result the maximum stresses) occur when there is a combination of responses in two modes of vibration. In this case the responses occur at 4.5 Hz and 1.1 Hz. The response at 4.5 Hz. can be seen in figure 7 to be reasonably constant throughout the twelve second period presented. The maximum response occurred at 6.5 seconds into this part of the record, when a single, large-amplitude, cycle of response occurred at 1.1 Hz., and this was superimposed on top of the rather more constant response at 4.5 Hz..

It should be noted that it is virtually impossible to see such an effect using conventional analysis techniques[2]. A single cycle that is averaged over a much longer period produces a spectral moment that is vanishingly small, and would be difficult to distinguish from background noise in a spectrum.

In this particular case the train was travelling at approximately 70 km/hr (20 m/s), and a distance of approximately 4 metres between wheels (passing over gaps or imperfections in the track) would excite this frequency. Accordingly it must be assumed that the response at 1.1 Hz. is caused by the lacet effect, and that when a sideways impact of the wheel flange with the track occurs, then a large low cycle excursion occurs.

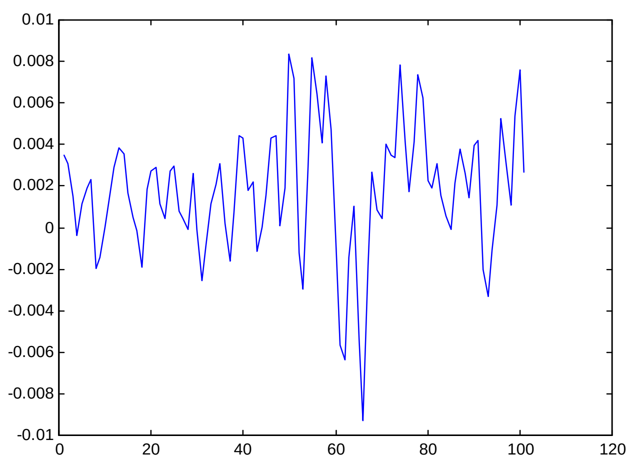

Fig 7: Detail from the response in Fig 7

In practice the stresses induced by such an action, produce stresses that are five times larger than the values that would be assumed from the action of the wheels of the train interacting with track imperfections.

Effect of damage on damping characteristic

In this particular case, damage induced by horizontal actions were apparent in this bridge. The opportunity was taken to make a recording of the response of the bridge to light winds over an eight hour period, so as to be able to assemble a non-linear damping characteristic using the random decrement amplitude-related signature[3-6].

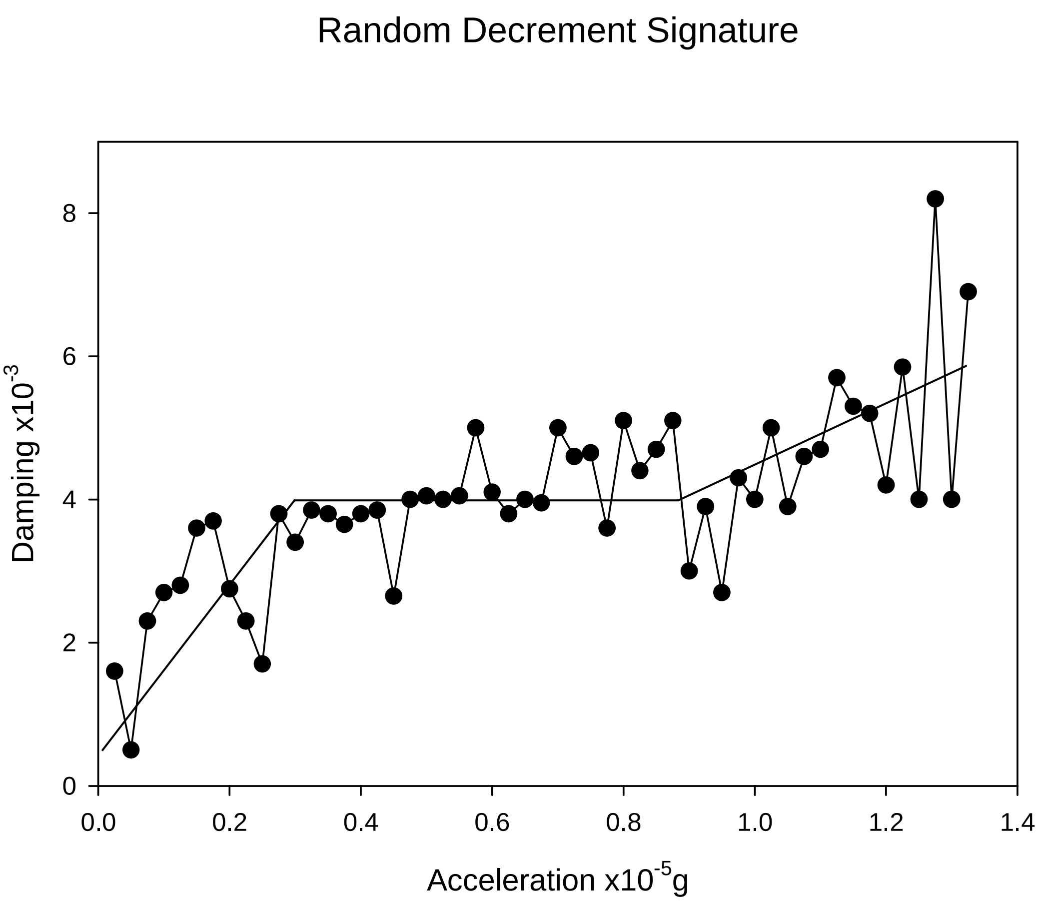

Fig. 8: Horizontal damping characteristic for the bridge

The striking feature of this characteristic is that the damping characteristic rises to a plateau, and then rises again. The plateau region in the centre of the graph would be consistent with the appearance of a large (approximately 30 cm) horizontal crack. Conventional estimates of damping[7-9] tend to predict a steadily rising characteristic, whilst non-linearities in this characteristic have previously been shown to be caused when damage appears in the structure[10-11].

Conclusions

A new mechanism for damage occurring in railway bridges has been identified. Horizontal actions caused by the ‘lacet’ effect (yawing) of trains introduces a horizontal action into the bridge that may in turn impart forces that are larger than would conventionally be anticipated. In the measurements made on the bridge cited in this paper the action produced forces that were five times larger than those generated considering the conventional approach.

References

1. Jeary A.P. (1997) Designer’s Guide to the Dynamic Response of structures, E & F Spoon, 235 p.
2. Bendat J. and Piersol A.(1971): Random Data: Analysis and Measurement Procedures. New York. John Wiley and Sons.
3. Cole Jr., H.A. (1968), "On-the-line analysis of Random Vibrations", AIAA/ASME 9th Structural Dynamics Materials Conference, Palms Springs, USA
4. Cole H.A.jr. 1973. On-line Failure Detection and Damping Measurements by Random Decrement Signatures. NASA-CR-2205. March 1973.
5. A.P.Jeary(1992) Establishing Non-linear Damping Characteristics of Structures from Non-stationary response time-histories. The Structural Engineer. Vol 70 No. 4.pp 61-66. 18 February 1992.
6. Tamura Y., Yamada M. and Yokota H.(1994) Estimation of Structural Damping of Buildings. ASCE Structures Congress and the IASS International Symposium, Atlanta Georgia. April 24th to 28th
7. Brownjohn, J.M.W. (1993), "Damping in Suspension Bridges: Sources, measurements and errors", Third Asia Pacific Symposium on Wind Engineering. PP 217-222. Hong Kong. 13-15 December 1993.
8. Eyre R & Tilley G.P.(1977) Damping Measurements on Steel and Composite Bridges. Symposium on Dynamic Behaviour of Bridges. Transport and Road Research Laboratory Crowthorne Berkshire, UK. 19 May 1977.
9. Jacobsen L.S.(1965) : Damping in Composite Structures. Proc. 2nd World Conference on Earthquake Engineering. P. 1029-1044.
10. Jeary A P (2002) The Identification of Damage in Large Structures, Damstruc 2004.
11. Jeary A P , Chiu G C & Wong J C K (2001) Wholistic Structural Appraisal, Conf. on Structural Monitoring. ICOSSAR. San Diego, June 2001.