VIBRATION OF THE LONDONM MILLENNIUM FOOTBRIDGE: PART 2 - CURE

David E Newland

Department of Engineering
University of Cambridge
Trumpington Street
CAMBRIDGE
CB2 1PZ, UK
den@eng.cam.ac.uk

Abstract

Part 2 of this paper considers how much damping is needed to ensure that pedestrian footbridges do not experience excessive lateral vibration and describes how the necessary damping was provided for the London Millennium footbridge.

INTRODUCTION

People-excited lateral bridge vibration is likely to occur for pedestrian bridges which have low natural frequencies of swaying movement (less than 3 Hz) and for which the lateral modes have light damping. In the case of London's Millennium Bridge, both these conditions applied.

One solution would have been to stiffen the bridge to increase its natural frequencies and take these outside the excitation frequency range. However the artistic design of the bridge would have been compromised by stiffening and this was regarded as most undesirable. The alternative was to find a way of increasing the bridge's inherently low damping so that self-excitation did not occur. It has been found that, below a threshold damping level, motion would build up, but that above the threshold damping level, self-excitation would not occur. Determining what this threshold level was and then providing a means of introducing the required amount of added damping proved a challenging task. It has involved adding 37 linear viscous dampers and over 50 tuned mass vibration absorbers to the initial structure. As a result, this bridge is now probably the most complex passively-damped structure in the world.

Figure 3 Centre span of the Millennium Bridge

Installing dampers in a way that was consistent with the aesthetic design of the bridge was difficult. A great deal of effort had been put into choosing a preferred design and the concept of “a blade of light” had been adopted and received widespread approval (see Deyan Sudjic (ed), *Blade of Light: The Story of London’s Millennium Bridge*, 2001). So far as possible dampers had to be mounted underneath the bridge deck so that they would be out-of-sight of everyone using the bridge.

RECONCILIATION WITH ARUP’S DAMPING CALCULATION

At page 20 of Fitzpatrick et al (2001), Arup give their formula corresponding to (16) as

$$c_{eff} = c + c_e = c - \frac{Nk}{8\pi fM} \qquad (17)$$

Although this has been arrived at by a completely different approach, it is identical with (16). This can be verified by making the substitutions $\eta_{net} = 2c_{eff}$, $\eta = 2c$, $\gamma = k\omega_n$, $\omega_n = 2\pi f$, and $m = 2M$. Apart from different normalisation of the modal shape function, the main difference is that Arup defined feedback force as proportional to velocity whereas the above analysis begins by assuming that the feedback force is proportional to displacement (at a fixed frequency). Arup use their symbol k not for stiffness but to relate pedestrian feedback force to deck lateral velocity, whereas γ as defined above relates feedback force to deck lateral displacement.

Arup’s computation of their proportionality factor k was done by measuring the acceleration time history under conditions of steady-state crowd loading with a constant number of people walking steadily over each span (in turn) at the correct speed to resonate with the relevant mode. From this time-history, they calculated modal velocity (see Fitzpatrick et al, 2001, p. 14). If F is the amplitude of the modal feedback force (which is assumed proportional to velocity), D is the amplitude of the modal damping force (also proportional to velocity and known from previous measurements), and V is the amplitude of the modal velocity, then for conservation of energy,

$$FV/2 = DV/2 + \frac{d}{dt}\left(mV^2/2\right) \qquad (18)$$

so that

$$F = D + \left(2m/\omega_n\right)\frac{dA}{dt} \qquad (19)$$

where m is modal mass, and A is the amplitude of the modal acceleration, since $A = V\omega_n$. By plotting F/N calculated from (19) (N is the number of people on the span) against modal velocity V, Arup arrived at an average value for k. Some typical values are shown in figure 4 in which physical rather than modal results are plotted. The approximately linear relationship in figure 4 appears to derive from the combined action of two factors: the force per person increases (slowly) with amplitude and more people synchronise with deck movement at larger amplitudes (see the additional material in Fitzpatrick et al, 2001). Linearity is of course a starting assumption for the feedback model in part 1.

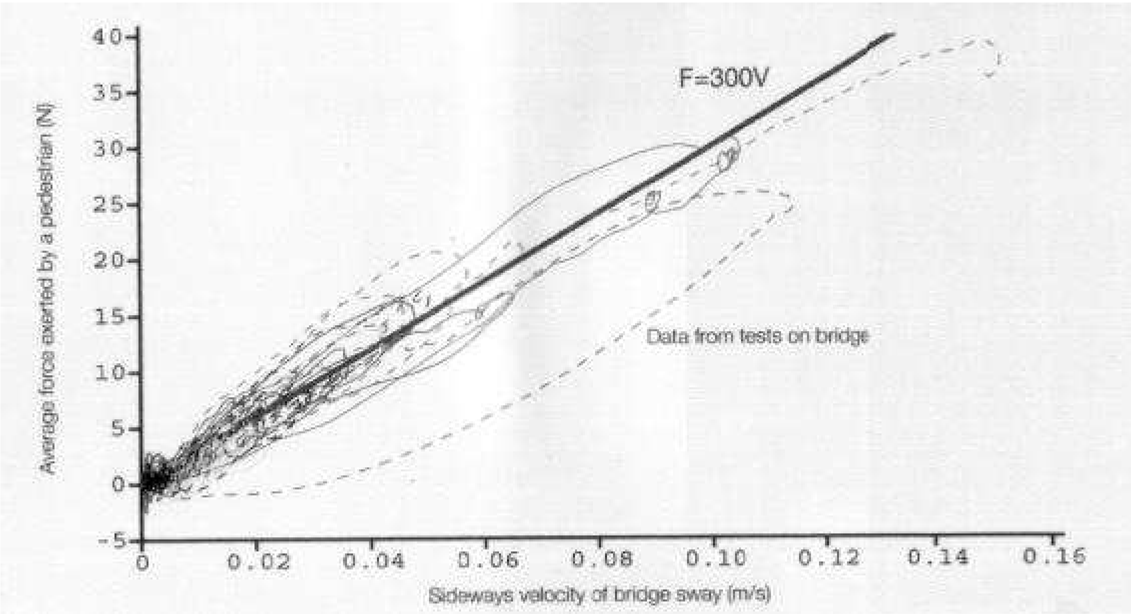

Figure 4. Correlation of pedestrian feedback force and deck velocity. The straight line shows average force exerted per pedestrian (Newtons) plotted against lateral deck velocity (m/s). (Arup figure, reproduced from Deyan Sudjic (ed), 2001, p. 93).

DEPENDENCE OF NET DAMPING ON THE NUMBER OF PEDESTRIANS

It follows from (16) and (17) that the net loss factor will decrease in proportion to the number of walkers on the span. If N_0 is the number of pedestrians on the span when the damping decreases to zero so that $\eta_{net} = 2c_{eff} = 0$, the net loss factor for this mode when there are $N \le N_0$ pedestrians on the span will be

$$\eta_{net} = \eta(1 - \frac{N}{N_0}) \qquad (18)$$

PRACTICAL DAMPING MEASURES

Based on these considerations, Arup decided to aim for 15% to 20% of critical damping for all lateral and lateral/torsional modes below 1.5 Hz and 5% to 10% of critical damping for vertical and vertical/torsional modes below 3 Hz. This is a huge increase in the original damping ratios of these modes which were typically 1% or less. To understand how this was achieved, it is necessary to understand the construction of the bridge. This can be seen from the photograph, figure 3.

The bridge deck is carried on lateral supports spaced periodically. These reach out to clamp onto the four parallel steel cables at each side of the deck. To a first approximation, the bridge vibrates like a taut string passing over supports at the two bridge piers and anchored to fixed supports at the river banks. Therefore lateral vibration involves shearing of the deck structure with no appreciable bending. All the low frequency modes have nodes at the attachment of the cables to the bridge piers and to the river bank anchorages. Although linear viscous dampers can be connected between the bridge deck and these fixed anchorages, the relative motion here is small and dampers fixed here cannot be made to work efficiently. Maximum relative shear displacement occurs between the lateral supports near anti-nodes, and therefore away from the fixed anchorages.

The solution adopted was to fit A-shaped frames to alternate lateral supports, with the points of two A's meeting at the intermediate supports (as shown in figure 5).

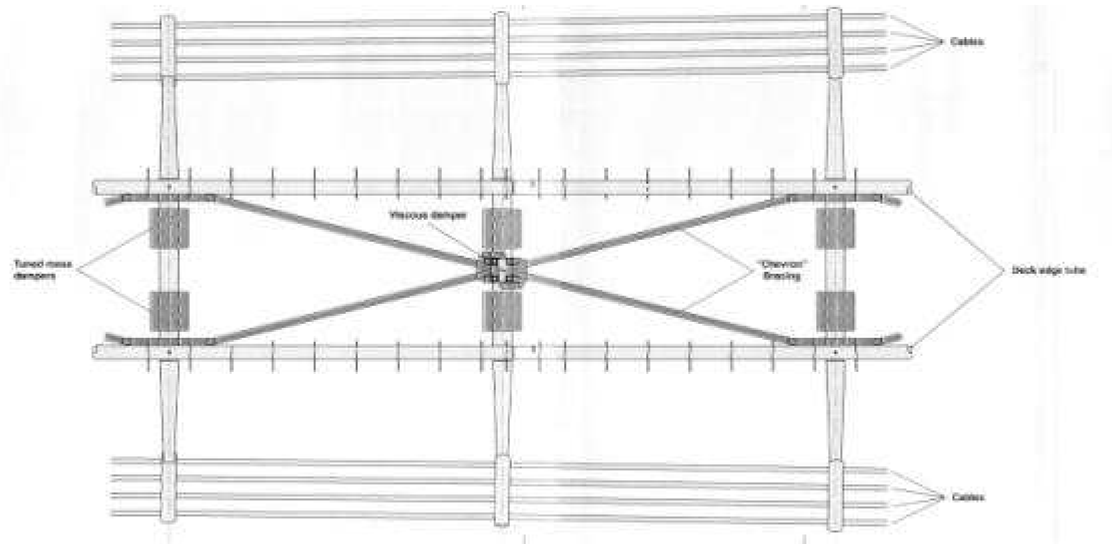

Figure 5. Plan view of underside of deck showing installation of dampers(from Fitzpatrick et al. 2001)

Between the points of each pair of A frames, a linear viscous damper was mounted. It was possible to do this so that the moving parts were supported vertically on the upper-side of the lateral supports. All the viscous dampers were supplied by the US firm Taylor Devices, Inc. and incorporated metal bellows seals so that they are fully-sealed to the environment.

For the centre span, the damping introduced by frame-mounted viscous dampers was supplemented by the action of 4 pairs of laterally-acting tuned-mass vibration absorbers supplied by the German firm Gerb Schwingungsisolierungen GmbH, mounted

on the upper side of the bridge deck's lateral supports, as shown in figure 5.

An additional 26 pairs of vertically-acting tuned-mass vibration absorbers were installed in similar positions on other lateral supports to increase vertical damping. This is to guard against the (unlikely) possibility that synchronous vertical vibration might occur when the lateral problem had been removed. The tuned-mass vibration absorbers have masses between 1 and 3 tonnes and they are located as close as possible to the antinodes of the modes that they are damping.

EXPERIMENTAL VALIDATION

In addition to a range of laboratory tests to study human gait and the interaction of pedestrians and moving platforms, the main experimental tests were carried out on the bridge. These consisted of two essentially different types of tests. Tests with no people, using a mechanical shaker to provide excitation, were carried out to measure modal frequencies and damping. This was done initially for the bare bridge, and then for the bridge with specimen viscous and tuned-mass dampers installed, to verify their action. Tests with walking people consisted mostly of recirculating tests where a metered number of pedestrians walked in one direction across a single span, and then immediately turned round and walked back to their starting point. Results from these tests were used to generate data like that in figure 4 and to confirm the essentially unstable feature of lateral synchronous excitation. A typical result for the north span, without any added damping, is shown in figure 6. A metered number of people were instructed to walk steadily at the speed needed to synchronise with the first lateral mode of the north span. Progressively the number of people walking was increased as shown by the staircase graph. The bridge deck acceleration (plotted below the staircase graph) increased slightly until 166 people were walking, when there was a sudden increase in deck lateral response which was sufficiently violent to stop the test. Since, when fully-laden, the north span can accommodate perhaps 700 people, the reason for the problems on opening day is apparent.

The performance targets for the modified bridge were expressed as rms acceleration levels measured at the quarter and half-span points with a 1 minute averaging time. The lateral target, after filtering with a passband of 0.2 to 2.4 Hz was that the rms should not exceed 25x10^{-3}g laterally; the vertical target in a passband of 0.2 to 4.8 Hz was that the rms should not exceed 50x10^{-3}g vertically. These targets were to be met in the presence of a test in which 2,000 people walked over the bridge three times at 0.6 m/s, 0.9 m/s and 1.2 m/s approximately with the bridge comfortably full of people. A great deal of planning went into the organisation and carrying out of this test which was successfully completed on 30 January 2002. Measured acceleration levels were substantially below the target limits for all the tests, typically less than one sixth of the agreed limits.

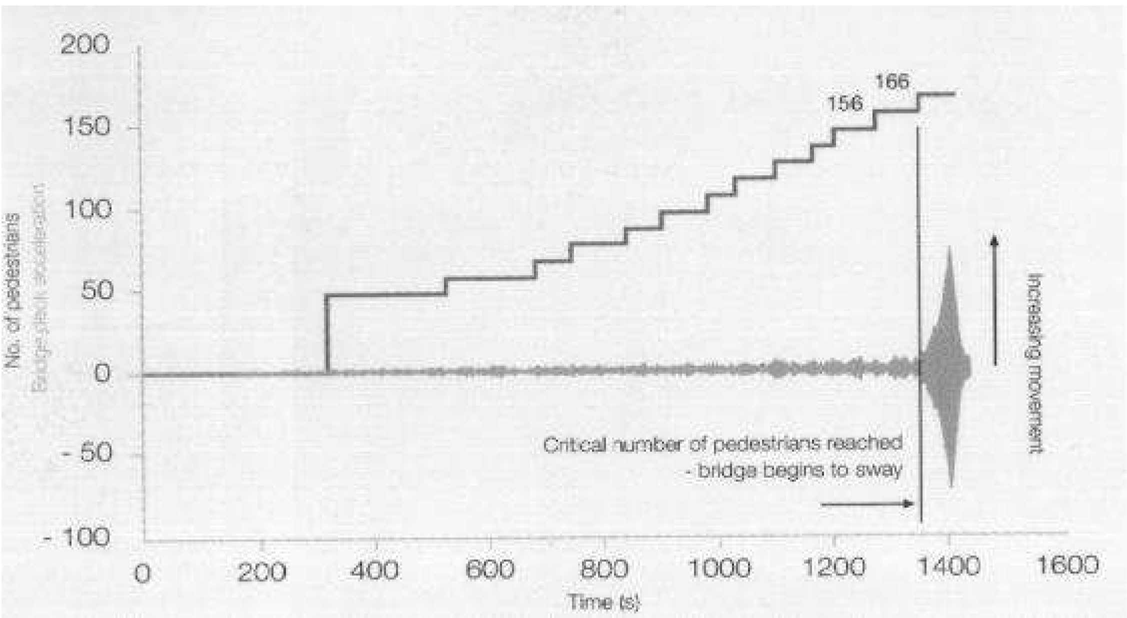

Figure 6. Onset of instability in crowd test on undamped north span, fundamental mode. As the number of people walking on the span (upper graph) increased to 166 progressively, the bridge lateral acceleration (lower graph)increased only slowly until instability was reached. The scale for acceleration is not given, but the peak acceleration reached was about $80x10^{-3}$ g at the right-hand side (Arup figure from Deyan Sudjic (ed), 2001, p. 93)

CONCLUSIONS

The introduction of damping by a combination of frame-mounted viscous dampers and tuned-mass vibration absorbers has cured the London Millennium Bridge's famous wobble. It was caused by synchronous lateral excitation from pedestrians, a phenomenon that was not well-known at the time but for which there is now a good understanding and good data.

ACKNOWLEDGEMENTS

The bridge's engineering designers were the Ove Arup Partnership and they designed, tested and supervised the construction of the vibration control system described in this paper. The author's role was as independent technical advisor to the London Millennium Bridge Trust (the bridge's principal funding body) for the duration of the remedial work described above. During this period, July 2000 to January 2002, he was pleased to work with the Ove Arup Partnership, and with the W. S. Atkins Group who were advising the London Borough of Southwark and the Corporation of London. Two detailed papers by Ove Arup about the bridge are listed below. The interpretation of experimental results in terms of the feedback model described in part 1 of this paper is original to the author.

REFERENCES

- H Bachmann, "Case Studies of Structures with Man-Induced Vibrations", *Journal of Structural Engineering*, Trans ASCE, **118**, 631-647 (1992).
- Hugo Bachmann and Walter Ammann, "Vibrations in Structures Induced by Man and Machines",*Structural Engineering Document 3e*, International Association for Bridge and Structural Engineering (IABSE), ch. 2: Man-Induced Vibrations, and Appendix A: Case Reports (1987).
- Pat Dallard, Tony Fitzpatrick, Anthony Flint, Angus Low, Roger Ridsdill Smith, Michael Willford and Mark Roche, "London Millennium Bridge: Pedestrian-Induced Lateral Vibration", *J. of Bridge Engineering*, Trans. ASCE, **6**, 412-417 (2001).
- Tony Fitzpatrick, Pat Dallard, Sophie Le Bourva, Angus Low, Roger Ridsdill Smith and Michael Willford,"Linking London: The Millennium Bridge", Royal Academy of Engineering, London, Paper ISBN 1 871634 99 7 (2001). An amended version of this paper with additional material was subsequently published as P. Dallard et al. "The London Millennium Footbridge", *The Structural Engineer*, **79**, No. 22, 17-33 (2001).
- Yozo Fujino, Benito M Pacheco, Shun-ichi Nakamura and Pennung Warnitchai, "Synchronization of Human Walking Observed during Lateral Vibration of a Congested Pedestrian Bridge", *Earthquake Engineering and Structural Dynamics*, **22**, 741-758 (1993).
- Deyan Sudjic (ed), *Blade of Light: The Story of London's Millennium Bridge*, Penguin Books Ltd, London (2001), chapter by David E. Newland, "Vibration: problem and solution", 88-93.