Vibration Assessment of Adelaide's new Riverbank Foot-Bridge

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ABSTRACT

The new Riverbank Bridge in Adelaide is a light-weight steel superstructure for pedestrians, with the main span being approximately 135m long with an internal radius of 182m in the horizontal plane. Due to the relatively light-weight bridge structure, the response of the bridge structure to pedestrian and wind loads was of concern. Pedestrian induced excitation of bridges can occur vertically, horizontally or torsionally. Vertical vibrations are absorbed by legs and joints, with pedestrian movements not synchronised with bridge motion, however pedestrians adjust their walking pattern to synchronise with the lateral motion of a structure, resulting in a potentially significant response. This paper will detail the approach taken by Aurecon to assess and mitigate pedestrian and wind loads.

INTRODUCTION

The new Riverbank Bridge in Adelaide is a light-weight steel superstructure for pedestrians, with the main span being approximately 135m long with an internal radius of 182m in the horizontal plane. There is an internal support to the bridge which is located at 80m from the South end (55m from North end) with V-columns (inclined structural steel columns in an asymmetric shape). The main structural components of the footbridge generally consist of the following:

- 4m wide x 1.6m deep irregular shaped hexagonal structural steel box girder faceted into 5m nominal lengths (to assist with the creation of a visual appearance of a curvature bridge);
- Structural steel outrigger trusses at 2.5m spacing with 200UB22 top and bottom chords on the East and West side of the box girder;
- 150 – 170mm nominally thick slab conventionally reinforced in-situ slab;
- 50 – 70mm pre-stressed precast concrete Deltafloor with a 100 – 120mm nominally thick slab conventionally reinforced topping slab.

This paper describes the approach taken by Aurecon's Building Sciences team to assess vibration excitation from both pedestrians and wind. Additional damping is required in the form of tuned mass dampers (dynamic vibration absorbers), with a relatively simple design developed and integrated within the structural form.

DESIGN BRIEF

The design brief states:

> *"The footbridge structure shall have a resonant vibration due to vibration from pedestrians or wind, greater than 1.3Hz, vertically and 1.3Hz laterally. Where the natural frequency of resonant vibration of the footbridge is below 1.3Hz and the design cannot be adjusted to increase the frequency, low maintenance and easily accessible vibration dampers shall be provided to increase the resonant vibration. Vibration dampers shall be unobtrusive and consider the urban design elements. The design shall address and prevent synchronous lateral excitation by pedestrian movements."*

Figure 1 Artistic Impression of the Proposed Bridge

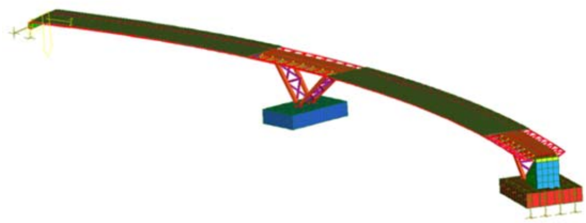

Figure 2 Finite Element Model of the Proposed Bridge

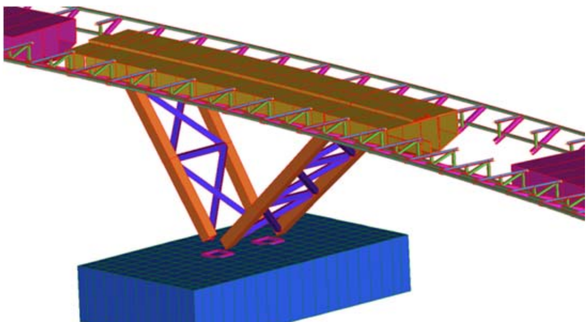

Figure 3 Detailed View of the V-Columns, OutRiggers and Box-Girder

IMPOSED LOADS

Pedestrians

Pedestrian induced excitation of bridges can occur vertically, horizontally or torsionally. Vertical vibrations are absorbed by legs and joints, with pedestrian movements not synchronised with bridge motion. However, pedestrians adjust their walking pattern to synchronise or "lock-in" with the lateral motion of a structure, resulting in a significant response.

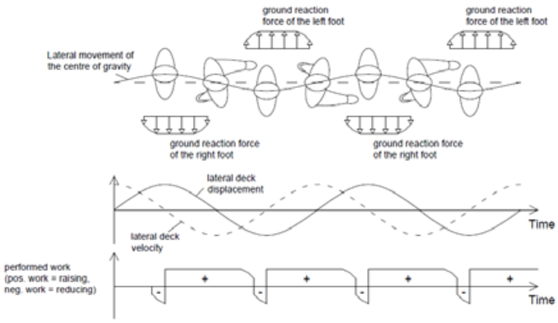

Figure 4 Lateral induced loads by pedestrians

The vibration response to multi-person traffic has been modelled as a multiple of the response to a single person excitation and a factor which is a function of the number of people crossing the bridge at the same time. This approach originated from the work by Matsumoto et al. (1978) and is still popular due to its simplicity. More recently frequency-domain models using ideas from random vibration theory and earthquake and wind engineering have been proposed, but these have not yet been standardised. The forcing function is shown below:

$$f_p(t) = Q + \sum_{n=1}^{k} Q\alpha_n \sin(2\pi nft + \varphi_n)$$

Where Q is the pedestrian's weight, α_n is the load factor of the nth harmonic, f is the frequency of the fundamental or first harmonic, φ_n is the phase shift of the nth harmonic, n is the number of the harmonic and k is the total number of contributing harmonics. The load factor for the nth harmonic is shown below in Figure 5.

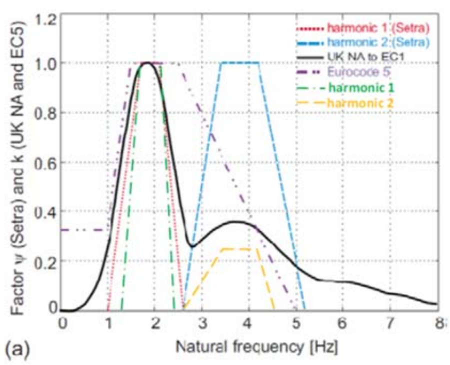

Figure 5 Load factor for harmonic analysis

Loads induced by a group of pedestrians can be modelled by multiplying the load induced by a single pedestrian with a constant derived by Matsumoto et al (1978) as:

$$m = \sqrt{\lambda T}$$

where λ is the mean flow rate of persons over the width of the deck [pers/s] and T is the time in seconds needed to cross a bridge (which can also be expressed as $T = L/v$, where v is the velocity of pedestrians). The product λT is equal to the number of pedestrians on the bridge at any time instant, denoted N.

The multiplication factor $m = \sqrt{N}$ is therefore equivalent to a load due to absolutely unsynchronised pedestrians, and the multiplication factor $m = N$ is equivalent to a load due to absolutely synchronised pedestrians.

Wind Loads

Reference is made to the paper by Gaekwad and Mackenzie (2013) regarding wind engineering analysis of tall or long span structures.

BD 49/01 "Design Rules for aerodynamic effects on bridges" is the only known standard or guideline that specifies design requirements for bridges with respect to aerodynamic effects, including provisions for wind-tunnel testing. BD49/01 classifies the response as:

- Limited Amplitude:
  Vortex-induced oscillations: These are oscillations of limited amplitude excited by the periodic cross-wind forces arising from the shedding of vortices alternatively from the upper and lower surfaces of the bridge deck. They can occur over one or more limited ranges of wind speeds. The frequency of excitation may be close enough to a natural frequency of the structure to cause the resonance and, consequently, cross-wind oscillations at that frequency. These oscillations occur in isolated vertical bending and torsional modes.
  Turbulent Response (Buffeting): Because of its turbulent nature, the forces and moments developed by wind on bridge decks fluctuate over a wide range of frequencies. If sufficient energy is present in frequency bands encompassing one or more natural frequencies of the structure, vibration may occur.
- Divergent Amplitude : Galloping and stall flutter - galloping instabilities arise on certain shapes of deck cross-section because of the characteristics of the variation of the wind drag, lift and pitching moments with angle of incidence or time; and Classical flutter - this involves coupling (i.e. interaction) between the vertical bending and torsional oscillations.
- Non-oscillatory Divergence : Divergence can occur if the aerodynamic torsional stiffness (i.e. the rate of change of pitching moment with rotation) is negative. At a critical wind speed the negative aerodynamic stiffness becomes numerically equal to the structural torsional stiffness resulting in zero total stiffness.

The aerodynamic susceptibility parameter, P_b, shall be derived in order to categorise the structure using the equation:

$$P_b = \left(\frac{\rho b^2}{m}\right)\left(\frac{16V_r^2}{bLf_B^2}\right)$$

Where ρ is the density of air, b is the overall width of the bridge deck, m is the mass per unit length of the bridge, V_r is the hourly mean design wind speed, L is the length of the relevant maximum span of the bridge, and f_B is the natural frequency in bending.

The bridge shall then be categorized as follows:

- Bridges built of normal construction, are considered to be subject to insignificant effects in respect of all forms of aerodynamic excitation when $P_b < 0.04$.
- Bridges having $0.04 \leq P_b \leq 1.00$ shall be considered to be within the scope of the rules set by BD49/01, and they shall be considered adequate with regard to each potential type of excitation if they satisfy the relevant criteria given in BD49/01.
- Bridges with Pb > 1.00 shall be considered to be potentially very susceptible to aerodynamic excitation and shall be verified by means of further studies or through wind tunnel tests on scale models.

The aerodynamic susceptibility parameter was estimated to be less than 0.04.

ASSESSMENT CRITERIA

The response to vibrations is subjective, as acceptable levels vary for each individual depending on their sensitivity to accelerations and deflections. Structural vibration limits are commonly provided in terms of acceleration (RMS m/s^2) as the acceleration of the floor can be more readily compared to a suitable level of human perception.

In all regulations, a limit of lateral acceleration to 0.001g (0.1m/s^2) is generally used to avoid "lock-in". Lock-in is a phenomenon in which pedestrians unconsciously match their footstep to small lateral movements of the structure (below 1Hz), which exacerbates the small lateral vibrations and causes resonance. Pedestrian correlation becomes abnormally high during lock-in, and displacements and accelerations can more easily develop to the point of being so high as to alarm pedestrians.

AS 5100.2 (2010) provides dynamic vibration limits in terms of displacements. These can be readily transformed into accelerations, which vary from about 0.3m/s^2 at 1Hz to 0.7m/s^2 at 5Hz.

The UK National Annex to the Eurocode (2004), recommends a range of comfort between 0.5 m/s^2 and 2.0 m/s^2 subject to the number of people, height and the route redundancy.

ISO 10137 (2005) states that the designer shall decide on the serviceability criterion and its variability. Further, ISO 10137 states that pedestrian bridges shall be designed so that vibration amplitudes from applicable vibration sources do not alarm potential users. In Annex C, there are given some examples of vibration criteria for pedestrian bridges. There it is suggested to use the base curves for vibrations in both vertical and horizontal directions given in ISO 2631-2, multiplied by a factor dependent on use.

SETRA (2006) defines criteria according to a range of acceptable acceleration for a given comfort level, as defined and shown below in Figure 6 and Figure 7:

- Maximum Comfort (Range 1): Accelerations undergone by the structure are practically imperceptible to the users.
- Average Comfort (Range 2): Accelerations undergone by the structure are merely perceptible to the users.
- Minimum Comfort (Range 3): under loading configurations that seldom occur, accelerations undergone by the structure are perceived by the users, but do not become intolerable.
- Unacceptable (Range 4)

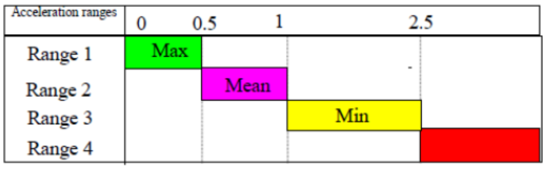

Figure 6 Vertical acceleration limits

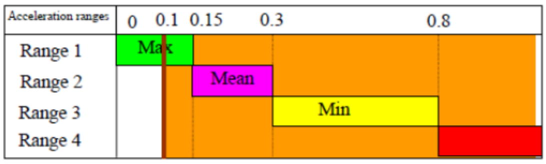

Figure 7 Lateral acceleration limits

MODAL ANALYSIS

A modal analysis was carried out using "Strand7" finite element analysis software, with the structure modelled using the following principles:

- 4m wide x 1.6m deep irregular shaped hexagonal structural steel box girder faceted into 5m nominal lengths (to assist with the creation of a visual appearance of a curvature bridge) were modelled explicitly with the section properties extruded along the centroid of the section following the curve of the bridge;
- Structural steel outrigger trusses at 2.5m spacing with 200UB22 top and bottom chords on the East and West side of the box girder were modelled with the truss elements having individual properties.
- 50 – 70mm pre-stressed precast concrete Deltafloor with a 100 – 120mm nominally thick slab conventionally reinforced topping slab was modelled as a plate element. The composite floor structure is tied into the extruded box girder and outrigger trusses.
- Live load was included as a uniformly distributed load using actual loads of people, rather than code values. The amount of live load was varied to understand the impact on the response.
- Connections between columns, beams and plates were modelled as fixed connections.
- The sub-structure was modelled as an infinite stiff connection, and as an elastic connection with these values varied to understand the impact on the response.

The damping factors used in the model were 0.5% for steel and 1.0% for concrete. Variation of damping will have an impact on the response, and is considered in more detail in the next section. Measured damping ratios under serviceability loads are shown in Figure 8.

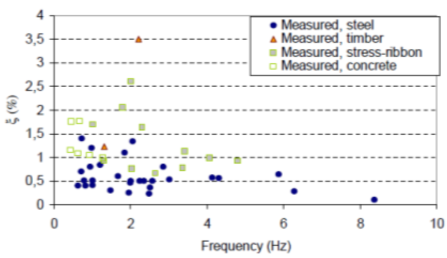

Figure 8 Measured damping ratios of bridge structures

The results of a modal analysis of the bridge structure with dead and live load and infinitely stiff foundations, is shown in Figure 9 via mode shapes, with the frequency of each mode and corresponding modal mass, stiffness and damping noted in Table 1.

Modes 1 and 2 are dominated by vertical motion, though with some longitudinal motion. Mode 3 has a strong longitudinal motion (demonstrated by participation in X and Y directions). Mode 3 has very strong lateral motion, but is well outside the range of lateral excitation (though the second harmonic of the forcing function needs to be considered). Mode 4 is similar to mode 3 but is effectively the second harmonic of lateral motion, generating minimal participation in the Y axis (given anti-phase response either side of the support). Mode 5 is effectively the second harmonic of mode 1 etc.

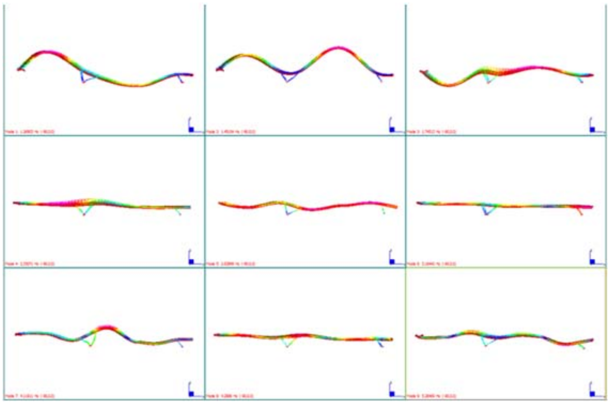

Figure 9 Mode shapes

Table 1 Modal analysis results (dead plus live load)

Mode	Frequency	Modal Mass	Modal Stiffness	Modal Damping
No.	Hz	Tonnes	10^6Kg.s^{-2}	(%)
1	1.23	538	32.2	0.50%
2	1.46	406	33.9	0.54%
3	1.72	874	102	0.38%
4	2.29	912	188	0.44%
5	2.46	1,859	444	0.42%
6	3.16	1,127	445	0.25%
7	3.63	615	319	0.27%
8	4.20	413	287	0.49%
9	4.32	628	463	0.49%
10	4.83	183	169	0.41%

The stiffness of foundations cannot be considered as infinitely stiff, and the effect of foundation stiffness needed to be assessed. Geotechnical engineering confirmed that the equivalent spring stiffnesses (force per unit deflection) of the pile group as a whole (i.e. equivalent spring stiffness at top of pile cap, which will therefore encompass the combination of pile stiffness and soil stiffness, and reflect the total number of piles in the group), are as follows. Both vertical and horizontal spring stiffnesses vary with load level (as expected), and the ranges given below show this variability:

- vertical spring stiffness for SLS (dead+live) loading = 500-1300 kN/mm
- horizontal spring stiffness in tangential direction for SLS (dead+live) loading = 750-900 kN/mm
- horizontal spring stiffness in radial direction for SLS (dead+live) loading = 270-360 kN/mm
- rotational spring stiffness for overturning about radial axis for SLS (dead+live) loading = 30-60 x E6 kNm/rad
- rotational spring stiffness for overturning about tangential axis for SLS (dead+live) loading = 15-30 x E6 kNm/rad

The results of the modal analysis using values for the stiffness of the sub-structure are given in Table 2, with the modal frequencies increasing by about 10% (compared with dead load only above), however the modal mass (and therefore stiffness) change significantly.

Table 2 Modal analysis results (foundation stiffness)

Mode	Frequency	Modal Mass	Modal Stiffness	Modal Damping
No.	Hz	Tonnes	10^6Kg.s^{-2}	(%)
1	1.40	310	23.9	0.61%
2	1.53	287	26.6	0.60%
3	2.20	658	126	0.63%
4	2.84	466	149	0.68%
5	3.21	587	239	0.73%
6	4.39	319	243	0.60%
7	4.54	577	469	0.64%
8	5.10	116	119	0.59%
9	5.34	152	171	0.59%
10	5.46	96.5	113	0.59%

HARMONIC RESPONSE ANALYSIS

Any sustained cyclic load will produce a sustained cyclic response (a harmonic response) in a structural system. Harmonic response analysis provides the ability to predict the sustained dynamic behavior of a structure. Harmonic response analysis is a technique used to determine the steady-state response of a linear structure to loads that vary sinusoidally (harmonically) with time. The structure's response is calculated at each frequency interval, with a frequency response. This analysis technique calculates only the steady-state, forced vibrations of a structure. The transient vibrations, which occur at the beginning of the excitation (or following each impact), are not accounted for in a harmonic response analysis.

Transient dynamic analysis (or time-history analysis) is used to determine the dynamic response of a structure under the action of any general time-dependent loads. This type of analysis is used to determine the time-varying displacements, strains, stresses, and forces in a structure as it responds to any combination of static, transient, and harmonic loads.

Willford (2006) recommends a harmonic analysis is used below 10Hz, and a transient analysis above 10Hz. This is because the structure responds harmonically to forced excitation from footfalls below 10Hz, while the energy in fourth harmonics and above is not sufficient to excite higher order modes, and these modes respond as free vibrations (in a transient manner).

Vertical and lateral loads have been applied assuming very dense crowd movement (of 2 persons/m^2). Figure 10 shows vertical accelerations exceed the comfort criteria without mitigation. Likewise Figure 11 shows a similar exceedance of lateral vibration criteria without mitigation. Lateral accelerations were well within the criteria for “lock-in”, and it was anticipated that reduction of vertical accelerations would also reduce lateral accelerations given joint participation in the vertical and lateral directions of the offending mode.

Vibration levels were also predicted for dense and sparse crowds, for small groups jogging and for vandalism loads. These levels were well within the design criteria.

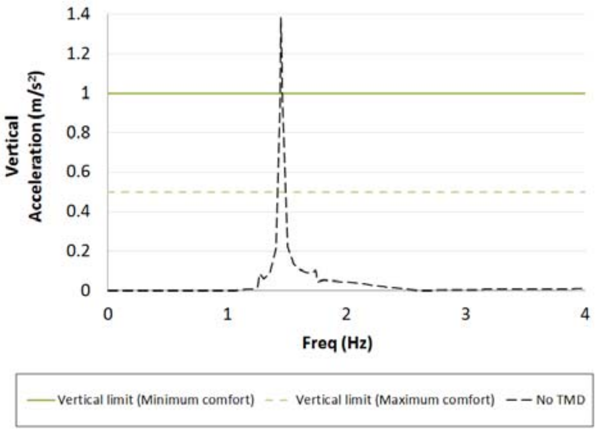

Figure 10 Predicted vertical acceleration with very dense crowd

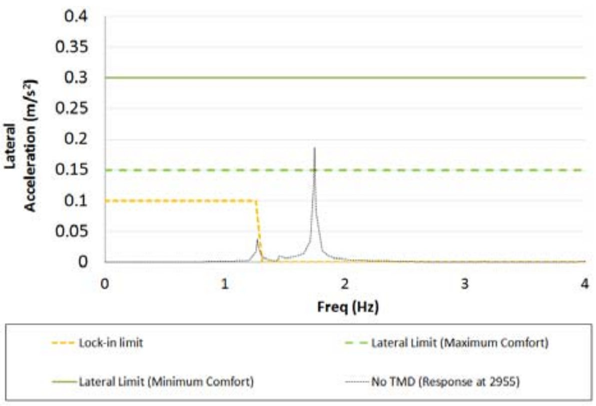

Figure 11 Predicted lateral acceleration with very dense traffic

CONTROL STRATEGIES

Vibration of the structure can be reduced by shifting the modes of structural vibration outside the frequency range of excitation, increasing the modal mass, or increasing structural damping using passive dampers. Given cost and aesthetic design imperatives, modifications of stiffness and mass are not possible to increase the modal frequencies beyond the range of excitation frequencies.

Tuned mass dampers or viscous dampers are often used to increase damping of a structure. General principles for the design of the damping system include:

- Accessibility
- Low maintenance
- Corrosion prevention
- Buffers to prevent damage to the structure in the event of vandalism loads
- Allow for tuning and adjustment

The efficiency of a viscous damper depends on the possibility of installation of the damper between a fixed point and a point on the structure with significant relative velocity. The most significant advantage of viscous dampers is their ability to control various modes of vibration. The maximum velocity for the riverbank bridge typically occurs mid-span as shown by the modal analysis results. It is not practical in this instance to accommodate viscous dampers at appropriate locations.

Figure 12 Viscous Damper

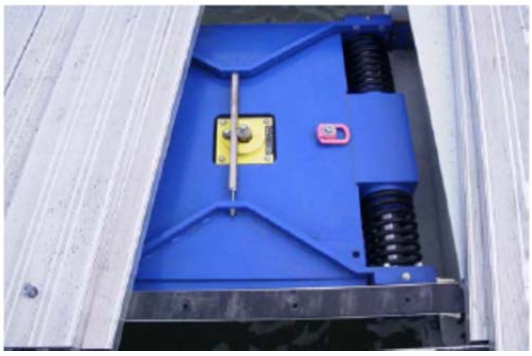

Figure 13 Tuned Mass Damper

TUNED MASS DAMPERS

A tuned mass damper or dynamic absorber is shown conceptually in Figure 14 below.

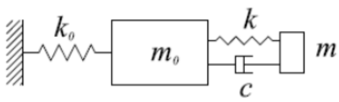

Figure 14 Concept of a Tuned Mass Damper

It can be shown that the Dynamic Magnification Factor (DMF) of the mass, with the TMD included becomes (there is a similar relationship for the TMD mass with the numerator being $\omega_0^2\omega^2$):

$$DMF = \frac{\omega_0^2[\omega_d^2 - \omega^2 + 2i\varsigma_d\omega_d\omega]}{\omega^4 - [\omega_0^2 + (1+\mu)\omega_d^2]\omega^2 + \omega_0^2\omega_d^2 + 2i\varsigma_d\omega_d\omega[\omega_0^2 - (1+\mu)\omega^2]}$$

Where

$\omega_0^2 = \frac{k_0}{m_0}$, $\omega_d^2 = \frac{k}{m}$, $\varsigma_d = \frac{c}{2\sqrt{km}}$ and

$\mu = \frac{m}{m_0}$ as the important mass ratio.

Figure 15 shows the DMF for various damping ratio, ς_d:

- "curve 2" with infinite damping, and the TMD simply adding additional mass to the mass-spring system.
- "curve 1" represents zero damping, with two distinct frequencies either side of ω_0, and
- "curve 3" being optimal damping with the peaks in the response at the two frequencies either side being equal.

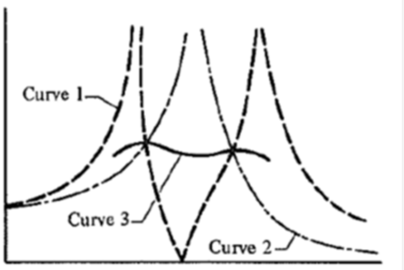

Figure 15 Effect of damping ratio on the system response

It can be shown that the optimal damper frequency is:

$$\frac{\omega_d}{\omega_0} = \frac{1}{1+\mu}$$

With the DMF at the peaks in "curve 3" (corresponding to frequencies $\omega_{A,}\omega_B$) becoming:

$$DMF_{A,B} = \sqrt{\frac{2+\mu}{\mu}}$$

Finally, it can be shown that the optimal damping ratio for the TMD is:

$$\varsigma_{OPT}^2 = \frac{1}{2}\frac{\mu}{1+\mu}$$

It can be seen that the mass ratio has a significant impact in the response, with the results shown in Figure 16 and Figure 17, below for damping ratios of 1% and 10%. Low values of the mass ratio can result in significant amplitudes of the TMD mass, exceeding those of the structure. Typically a mass ratio of about 2.5% results in similar amplitudes of vibration of the TMD mass as that of the structure. This can be confirmed by considering the equation for the DMF at the peak in the response for the TMD, which is given by:

$$DMF = \frac{1+\mu}{\mu}$$

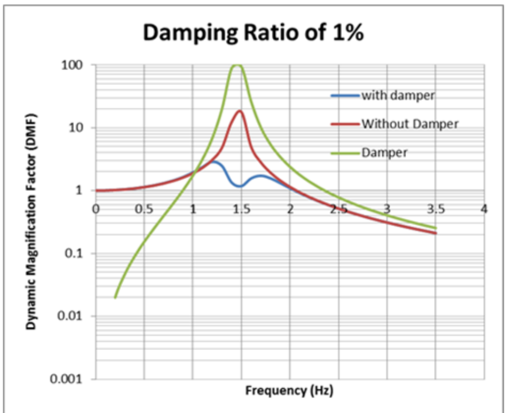

Figure 16 DMFs for a damping ratio of 1%

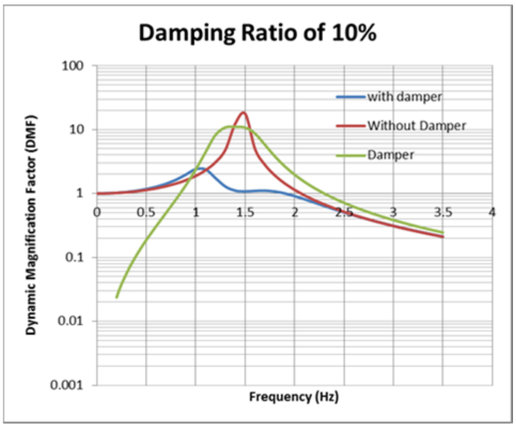

Figure 17 DMFs for a damping ratio of 10%

Proprietary TMDs are available from Maurer Söhne (among others), with their literature noting the following important parameters for the design of TMDs:

- Mass Ratio – As noted above, if too low large vibrations of the TMD mass result. They suggest a range of between 4% and 8% to allow for variations in the frequency to be controlled.
- Modal Frequency – As shown above under Modal Analysis, the variation in the modal frequency can be as much as 10% given uncertainties in geotechnical conditions, and modelling parameters.
- Damping – The response is not as sensitive to the damping ratio for the TMD. Deviations of +/- 25% result in minor changes to the TMD efficiency.

DESIGN OF TMD

Given the time constraints, it was decided to design bespoke TMDs for the riverbank bridge. Calculations for the design parameters for multiple TMDs applied at mid-spans along the bridge are shown below in Table 3. Corresponding DMF curves for the structure with and without the TMDs, and that for the TMDs themselves, are shown in Figure 18 below. It is important to consider the motion extents of the TMD to ensure it is not constrained by space and does not reach the limits of its restraints.

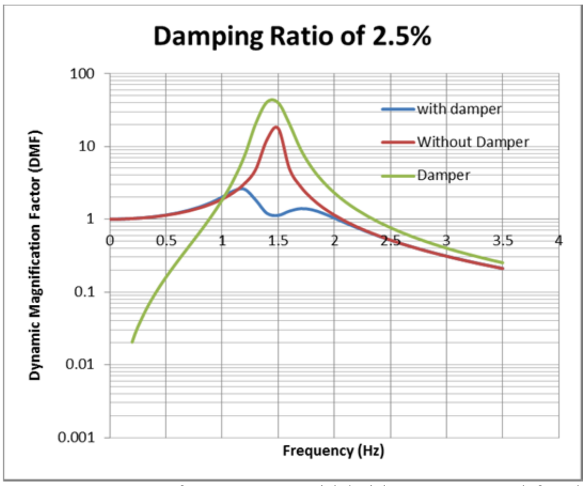

Figure 18 DMF for structure with/without TMD and for the TMD itself

A relatively simple form of TMD has been developed, and is shown in Figure 19, and consists of 4 off springs and dampers supporting a mass of steel plates.

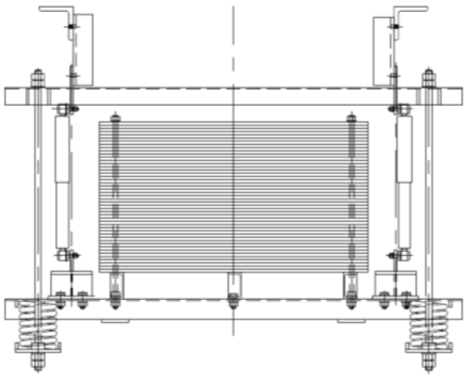

Figure 19 Tuned Mass Damper Design Drawing

Table 3 Multiple TMD Parameters

No. TMD	4		
Mass/TMD (kg)	2,536		
Dimensions (0.5 x 0.5 x L), (m)	1.30		
Stiffness/TMD	20.31	Damping/TMD	5,012
No. Springs	4	No. Dampers	4
Spring Stiffness/TMD (kg/mm)	5.08	C (Ns/m)	1,253
Mass/Spring (kg)	634		

The results of the application of TMDs to the structure are shown in Figure 20 and Figure 21. Significant reductions in vertical acceleration are achieved with the TMDs located mid-span.

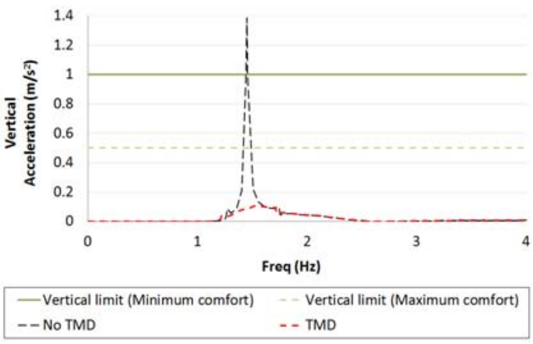

Figure 20 Reduction of vertical accelerations using vertical TMDs

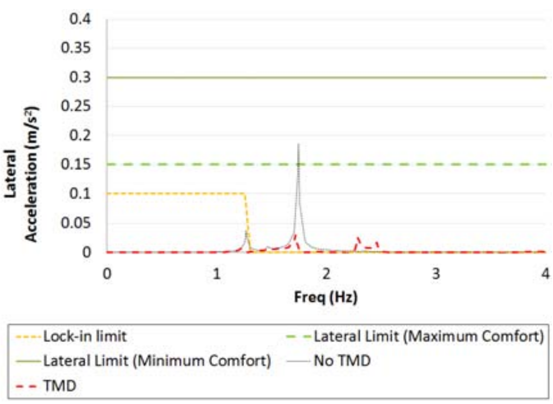

Figure 21 Reduction of lateral accelerations using vertical TMDs

By careful placement of the TMDs (ie. at locations of extreme displacement of the vertical and torsional mode-shapes), the tuned mass dampers managed to significantly reduce the brige's vertical vibration and effectively counteract the bridge's lateral motion at the same time.

CONCLUSIONS

There has been a great deal of research on pedestrian induced excitation of light-weight bridges since the opening of the Millennium Bridge in London, when vibration levels were considered extremely uncomfortable due to lateral excitation. This paper has outlined a detailed approach to consider vibration induced loads from pedestrians and wind. It also provides useful information to enable the design of bespoke tuned mass dampers to reduce vibration as required.

ACKNOWLEDGEMENTS

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