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# WIND AND PEDESTRIAN VIBRATION ASSESSMENT ON THE NEW SWAN RIVER PEDESTRIAN BRIDGE

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**Summary**

The paper presents the problems related to wind and pedestrian induced vibration that have been faced during the design of the Swan River Pedestrian Bridge (SRPB) in Perth (Australia).

It is a three-span arch structure (approximately 80-140-80 m length, respectively). Arches are made by lattice steel structures and the deck is suspended to the arches by sub-vertical strands. Each span has two crossed arches with triangular cross-sections that vary dimensions along the arch. Two cantilever extensions are connected to the crown of the central arch. Arches are supported by quite flexible piles.

For this kind of unique structures, with complex shape and bluff sections, wind effects are challenging and their effects should be studied by wind tunnel tests, as required by most design codes. Static and dynamic wind loads were studied in wind tunnel using scale models of: (i) the full bridge; (ii) portions of the arches; (iii) the deck. Experimental results and a procedure to assess wind effects on the whole structure are outlined.

The pedestrian excitation has been modelled in order to predict the induced vibration on the bridge and to assess the comfort performances. Different models have been implemented among those available in the most recent literature. Due to the uncertain parameters, a "design assisted by testing" procedure becomes mandatory in a such case. The sensitivity of the bridge to be laterally excited (with potential lock-in phenomena) induced to install a TMD on the central span.

**Keywords:** long span footbridge; arch bridge; multi span footbridge; pedestrian induced vibration; wind induced vibration; numerical modelling; vibration mitigation; design assisted by testing

## 1. Introduction

The new Swan River Pedestrian Bridge is formed by three steel arches and three cable stayed steel decks. The geometry of the steel arches follow the free form shape designed by the architects. The total length of the bridge is about 400 m with a central span of 144 m and the two lateral of 84 m. Each arch is formed by four legs, supported by concrete piers. The first and last steel arches have approximately a 84 m free span and 36 m height above the water level. The central arch has a 144 m free span and 75 m height above the water level. Arch cladding is made up of prestressed membrane fabric made with fiberglass coated both sides by PTFE layers and supported by steel purlins.

Stays hang the deck along longitudinal edges of deck structure. Typical longitudinal distance between hanging joints is 12m. The structural deck is composed by longitudinal I beams supporting transversal beams connected with the cast in situ concrete slab. The restrain system allow slow longitudinal movements under the thermal loading and ensure a reaction and a damping when the deck is subjected to a dynamic load as the wind or pedestrian walk. A perspective view of the bridge is reported in Fig. 1.

*Fig. 1. Architectural render of the bridge*

A unique feature of this bridge is that each span has two crossed arches with triangular cross-sections that vary dimensions along the arch. In addition, two cantilever extensions are connected to the crown of the central arch for aesthetic purposes. In Fig. 2 some typical cross sections along the arches are shown. Due to the uniqueness of the structure, design codes require the support of wind tunnel tests on a scale model of the bridge, to assess more accurately static and dynamic wind actions [1] [2]. Considering the complex shape and the lightness of the structure several aerodynamic issues should be assessed: global and distributed static loads on the whole structure, wake effect on the arches, vortex-induced vibrations (VIV) and aerodynamic stability of the deck, of the arches, of the cantilever extensions. To this end, several scales models were tested to investigate specific issues and design countermeasures, if necessary [3].

In addition, due to its flexibility and lightness, the footbridge is potentially prone to vibration induced by human activities and can suffer severe vibration serviceability problems. The AS 5100.2-2004 [2] specifies the comfort frequencies for pedestrian footbridges but those specifications are not particularly significant for large mass bridges (as the SRPB is) because they consider the deterministic traversing of a single pedestrian. The AS 5100.2-2004 itself, recommends to refer to specialist literature for “sensitive” (to pedestrian excitation) bridges. Hence, the analysis and the design process have to account for evaluating the (potential) pedestrian induced vibration and have to provide solutions to confine their amplitude (if necessary) within acceptable values.

## 2. Wind induced vibration assessment

### 2.1 Wind tunnel tests

As a first step, a sectional model of the deck in 1:10 scale was tested to assess steady aerodynamic force coefficients (drag, lift and moment) using a static setup with force balances, and vortex induced vibrations elastically suspending the model. Two different cross sections were tested: the original cross-section and a modified one (with a lower parapet height - see Fig. 2).

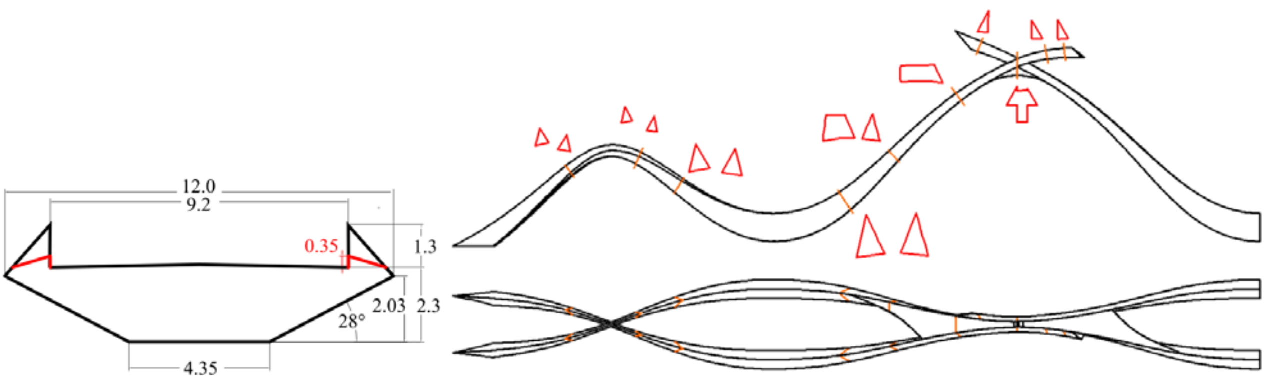



*Fig. 2. On the left, Original and Tailored deck cross section (in red); on the right, detail of the arches with some cross sections.*


*Fig. 3. On the left, 1:50 scale model in wind tunnel: smooth and turbulent flow conditions; on the right, elastic cantilever extension.*

As it will be presented in the results, the tailoring of the original section helped to improve the aerodynamic performances of the deck both for stability and for VIV mitigation. Secondly, a rigid model of the full bridge in 1:50 scale was tested in the boundary layer wind tunnel of Politecnico di Milano (see Fig. 3). The model was instrumented with several pressure taps around several cross sections distributed along the arches (see cross sections in Fig. 2) and with multi-component force balances at the four foundations.

Since the analysis in the frequency domain of the time histories of pressures highlighted the presence of vortex shedding along the whole structure for a large range of wind directions and frequencies, to investigate vortex-induced vibration levels, two large-scale (1:15) sectional aeroelastic models of the "legs", representing the most critical vibration modes, were tested in wind tunnel.

Finally, to assess the vortex-induced vibration of the cantilever extension, the 1:50 model was modified introducing a spring at the connection between the crown and the extension, so that the extension could vibrate, while considering the three-dimensional aerodynamics of the whole structure (see Fig. 3)

The original deck shape, with a very bluff section did not exhibit good aerodynamic performances. Indeed, the slope of its lift coefficient was nearly zero, which is not a good index for the aerodynamic stability [1][4], and the recorded vortex-induced vibrations were large with a wide lock-in range. The subsequent tailoring (see Fig. 2) highly improved the aerodynamic behaviour of the deck.

The comparison of the vortex induced vibration as a function of the reduced wind speed is reported in Fig. 4, using different mass-damping parameters (Scruton number $Sc = 2 \pi m_L h / \rho B^2$, where m L is the deck mass per unit length, B the deck chord, 12 m, ρ is the air density and h is the structural damping ratio). It is possible to notice that the Tailored deck at a Sc equal to 1 do not suffer of any VIV, while to suppress VIV for the original deck it is necessary a Sc = 3.5 (i.e. more than three times of structural damping).

As regard the full-bridge model, the measured global forces did not highlight any particular issue, but, together with the distributed pressure measurements, they allowed the designers to have an accurate estimate of the wind loads both in smooth and turbulent flow. Comparison with the integral of the distributed pressure measured along the arches was also performed; a good matching with the global forces was obtained, this is index that the distributed loads are measured with a sufficient spatial discretization.

Vortex shedding was measured along all the arches for several wind directions and with different shedding frequencies; the corresponding critical wind speeds ranges from 14 m/s (later arches and cantilever) to 50 m/s (central arch). VIV were recorded for both the sectional models, and several Sc values were investigated until vibration suppression. The model of the central arch shows a classical lock-in region, and a Sc value of about 10 is necessary to mitigate VIV.

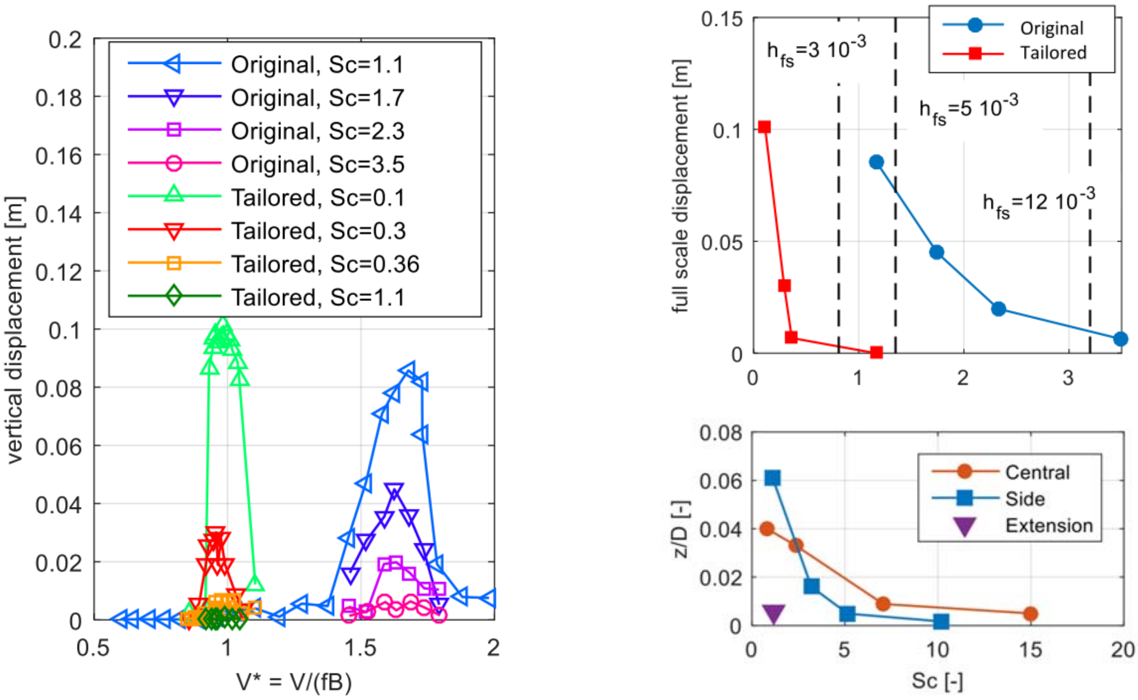



*Fig. 4. On the left, full scale vertical displacements of the deck due to vortex shedding as a function of V* for different Sc numbers; on the right-top, VIV crosswind amplitude as a function Sc for the two deck sections; on the right-bottom, VIV crosswind amplitude as a function Sc for the different parts of the arches.*

On the contrary, the model of the lateral arch, which has two triangular sections, has a complex double lock-in region, but a Sc of 5 is sufficient to suppress vibrations. Moreover, instead of increasing damping, an innovative solution to suppress vibrations was successfully tested research purposes: if a porous surface is used (40% of porosity), vortex-induced vibration is completely suppressed already at the lowest Sc levels. The VIV results are summarized in Fig. 4 (right-bottom), where the crosswind vibration amplitudes as a function Sc for the different parts of the arches are reported. The plot reports also the result obtained for the cantilever extension with the aeroelastic 1:50 model.

### 2.2 Structure behaviour assessment

Using the experimental data and the finite element model of the structure, the following assessment of the performances of the bridge can be made.

As regard the deck, considering a structural damping coefficient for vibration modes equal to $h_{fs}$ = 0,003 (typical for steel structures), the Sc number is 0.8 (using a mass m L of the full scale deck of 7430 kg/m) and using the Tailored deck section VIV are negligible, as summarized in Fig. 4 (left and right-top).

As regard the arched and the cantilever extension, from an aerodynamic point of view, the most critical response is the vibration of the central span arch, since a large Sc number is necessary to suppress it, if compared to the other bluff body sections. However, the following considerations should be done during the assessment of the expected vibration levels: (i) the vibration of the arch is coupled to the vibration of the deck; (ii) deck not only contributes as dead mass, but also with its aerodynamic damping. The structural damping is not known and it has been prudentially assumed equal to 0,003. According to the assumption above, a Sc = 5.6 without aerodynamic damping contribution of the deck is computed, while Sc = 11.1 may be reached considering the aerodynamic damping of the deck. For this Sc value, VIV are reduced to an acceleration level of 1.28 m/s$^2$ and a displacement of 0.04 m. A similar approach shows that lateral arches are less critical. Moreover, the cantilever extension has limited VIV at its structural damping (no deck effect present).

## 3. Pedestrian induced vibration assessment

In theory, vibrations can cause discomfort to pedestrians and the deterioration of the footbridge's structural integrity. Unfortunately, present bridge design codes (as for instance [1] and [2]) do not provide exhaustive guidelines and information to address such vibration problems and to investigate the dynamic characteristics of slender footbridges under human induced loads. International standards, usually provides comfort criteria in terms of maximum acceleration or deflection (in some cases as a function of the frequencies), even quite different among themselves. The AS 5100.2-2004 itself, recommends to refer to specialist literature for "sensitive" (to pedestrian excitation) bridges. Since the standard codes are quite "poor" on this task, the analysis that have been performed are referred to the most recent literature. In particular, the reference [5] is assumed as the main reference. Other useful references have been [6] and [7].

Practically, different variables introduce uncertainties in the task:
- the dynamic input is quite unknown in both shape and amplitude; researches are still ongoing on this subject;
- the perception and assessment of motion and vibration are not only subjective and, therefore, different for each pedestrian, but also related to some "environment" factors. For instance, some researches demonstrated that users of pedestrian bridges that are located near hospitals and nursing homes, may be more sensitive to vibrations than hikers crossing a pedestrian bridge along a hiking trail; it was also observed that the percentage of individuals feeling disturbed while crossing a sturdier-looking footbridge, is four times higher than for a lighter-looking footbridge;
- horizontal excitation is a typical lock-in phenomenon and usually occurs only significantly crowd conditions; hence, in most cases it can be inopportune to base the design on this criterion;
- the inherent structural damping (which is essential in determining the vibration amplitude and the auto-excitation condition) is quite variable and it depends not only on the basic material and joints typology but also on non-structural elements and on the oscillation amplitude (a reliable estimation of this parameter can be performed only once the structure is completed);
- in the case of the SRPB, even the modal shapes and frequencies are affected by significant uncertainties, due to the interaction with the relatively flexible foundation system and to the quite complex geometry.

Hence, the design criterion based on the natural structural frequencies (which is often adopted and suggested, consisting to avoid that structural modal frequencies "fall" in the critical ranges) can strongly and unnecessarily penalize both economical and aesthetical aspects.

The analysis performed are oriented to check if:
- vibrations due to pedestrian traffic are acceptable for the users,
- the lock-in phenomenon does not arise.

### 3.1 Definition of design situation

The first step is to define the design situations that is the combinations of Traffic Class (TC) and Comfort Level (CL). The TC and CL are defined according to [5].

The following design situations have been defined for the Swan River footbridge, in agreement with the Client:
- DS1 = TC4 + CL3 (inauguration / once)
- DS2 = TC3 + CL2 (stadium / weekly)
- DS3 = TC2 + CL1 (commuter traffic / daily)

For completeness of the analysis, although not required, TC5 has been also simulated in the calculations and the corresponding CL is shown for reference.

### 3.2 Response evaluation

The expected maximum accelerations are evaluated for all the significant vibration modes that involve the deck lateral and/or vertical deflection and are within the following frequency ranges:
- 1,3 Hz $\leq f_i \leq$ 2,3 for vertical vibrations
- 0,5 Hz $\leq f_i \leq$ 1,2 for lateral vibrations

The inherent modal damping ratios are referred to the Table 10.4 of EUR 23318 EN [5].

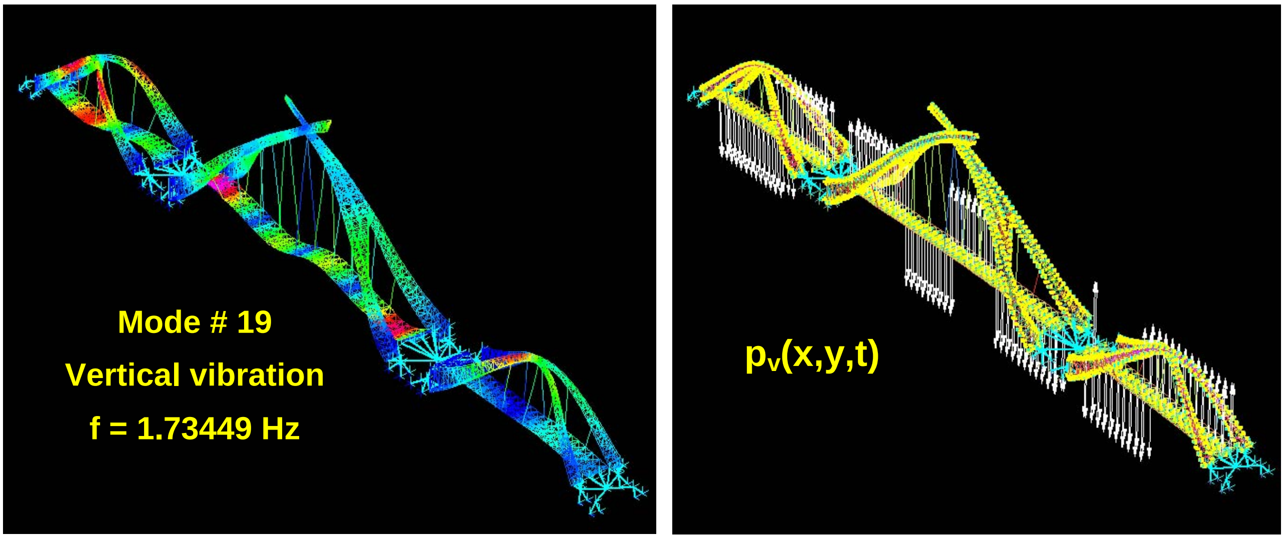
In particular, the following values of □ will be adopted in the analyses:

- $\xi$ = 1.3 % for horizontal modes (concrete slab)
- $\xi$ = 0.6 % for vertical modes (composite members)
- $\xi$ = 5 % for modes that involves foundation motions

Three methods will be used to estimate the acceleration corresponding to the different traffic classes, according to [5] and [6]: the Response Spectra Method, the Single Degree of Freedom (SDOF) method and the Finite Element (FEM) method. The procedure for the latter one is resumed in Fig. 5. As a matter of fact, the parameters that are used in the SDOF method are derived from a FEM modal analysis, by approximating the actual modal shapes with equivalent sinusoidal shapes (equivalent length, mass, load).

Following the procedures above summarized, the maximum (95% fractile) expected acceleration, corresponding to the different traffic classes, have been estimated for the vibration modes. The following aspects are relevant:

- the “modal/generalized mass” that is used for the RS and for the SDoF methods is referred to the maximum-amplitude-point on the deck (in the exciting direction);
- the reference length (L*) for the RS and for the SDoF methods is determined with reference to an equivalent “half-sine” length of the modal shape (in order to be “consistent” with the method hypotheses, that are based on sinusoidal shapes);
- the reference width (B) is safety assumed to be 10 m;
- the harmonic response analysis (H.R.A.) has been performed to evaluate the response with the FEM method.




| Direction | Load per m² |
|---|---|
| Vertical (v) | $d \times (280\text{N}) \times \cos(2\pi f_v t) \times 10.8 \times (\xi/n)^{1/2} \times \psi$ |
| Longitudinal (l) | $d \times (140\text{N}) \times \cos(2\pi f_l t) \times 10.8 \times (\xi/n)^{1/2} \times \psi$ |
| Transversal (t) | $d \times (35\text{N}) \times \cos(2\pi f_v t) \times 10.8 \times (\xi/n)^{1/2} \times \psi$ |

*Load Model for TC 1 to TC 3*

*(density < 1,0 P/m²)*

| Direction | Load per m² |
|---|---|
| Vertical (v) | $1.0 \times (280\text{N}) \times \cos(2\pi f_v t) \times 1.85\ (1/n)^{1/2} \times \psi$ |
| Longitudinal (l) | $1.0 \times (140\text{N}) \times \cos(2\pi f_v t) \times 1.85\ (1/n)^{1/2} \times \psi$ |
| Transversal (t) | $1.0 \times\ (35\text{N}) \times \cos(2\pi f_v t) \times 1.85\ (1/n)^{1/2} \times \psi$ |

*Load Model for TC 4 and TC 5*

*(density ≥ 1,0 P/m²)*

*Fig. 5. Procedure for calculating accelerations with the FEM method.*

The calculation (evaluation of acceleration on the deck) have been performed for any traffic class applied to any vibration mode. Nevertheless, many of such combination are physically meaningless; in fact, the step frequency (hence, the actually interested vibration mode) is related to pedestrian density (hence, to the traffic class), as shown in Fig. 6. Subsequently, only the significant case results are reported in Table 1 (where the pedestrian density has been assumed to be coherent with the step frequency, even if at safe side).

The modal superposition has been considered by performing time history analyses, with an appropriate frequency band. With this analysis, each pedestrian is simulated with an sinusoidal force, that is characterized by a Gaussian distributed frequency (around the frequency of the considered mode) and a uniformly distributed phase. The parameters for the stochastic simulation are defined according to [5]. The "central frequency" of the band is set (safely) equal the most excited mode the falls in the range (resulting from simplified analysis - Spectral, SDoF, FEM). The time history is repeated for different "central frequencies", corresponding to the each of the most excited modes, and each of them is characterized by an appropriate frequency band (defined by $\sigma_f$).

### 3.3 Vibration mitigation tools

Although the lateral modes seem to be not particularly prone to pedestrian excitation (due to their important mass), the uncertainties in the numerical modes and the irksomeness of lateral vibrations (in case of lock-in), induce to adopt additional precautions.

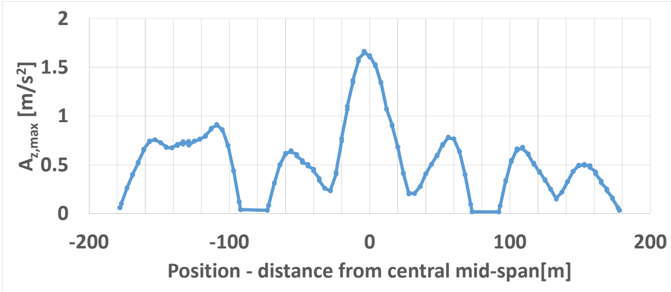
Lateral span horizontal vibrations involve even the motion of the piles; hence, the involved modal mass and the inherent damping are really very large. Excitation of these modes seem to be averted, even if the final judgment can be given only after the dynamic characterization on the real structure.

Central span, instead, seem to be close to the lock-in trigger acceleration, in case of crowded conditions. In fact, for the TC4 and TC5 traffic classes, the estimated accelerations are close to the lock-in limit (as defined in the referred literature). Hence, due to the particularly annoying effect of lateral vibration induced by the lock-in, a transverse-direction TMD has been implemented. By this way, the lock-in is sufficiently far from the estimated accelerations, because the damping added by the TMD ($\Delta\xi \cong 4.5\%$) strongly reduces the lateral vibrations.

| Modal data | | | | | | Traffic | | Deck max acceleration [m/s$^2$] | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Mode # | Dir. | f [Hz] | $\xi$ [%] | M* [kg x 10$^6$] | L* [m] | d [p/m$^2$] | TC | Spectral | SDOF | FEM | T.H. |
| 2 | h | 0.565 | 1.3 | 0.631 | 144 | 1.5 | 5 | 0.086 | 0.117 | 0.122 | \ |
| 3 | h | 0.792 | 1.3 | 6.382 | 240 | 1.5 | 5 | 0.012 | 0.015 | 0.014 | \ |
| 6 | h | 0.925 | 5 | 4.273 | 200 | 1 | 4 | 0.006 | 0.004 | 0.004 | \ |
| 7 | h | 0.949 | 5 | 3.393 | 200 | 0.5 | 3 | 0.005 | 0.005 | 0.005 | \ |
| 9 | h | 1.045 | 5 | 7.420 | 240 | 0.2 | 2 | 0.002 | 0.002 | 0.002 | \ |
| 12 | v | 1.457 | 0.6 | 0.758 | 120 | 1.5 | 5 | 0.036 | 0.159 | 0.099 | \ |
| 13 | v | 1.481 | 0.6 | 7.339 | 70 | 1.5 | 5 | 0.268 | 1.175 | 1.131 | 0.578 |
| 14 | v | 1.561 | 0.6 | 0.974 | 120 | 1.5 | 5 | 0.280 | 1.198 | 0.963 | \ |
| 15 | v | 1.566 | 0.6 | 0.516 | 160 | 1.5 | 5 | 0.611 | 2.607 | 1.483 | 1.101 |
| 16 | v | 1.620 | 0.6 | 0.518 | 100 | 1.5 | 5 | 0.489 | 2.055 | 1.523 | 1.170 |
| 17 | v | 1.631 | 0.6 | 0.384 | 80 | 1.5 | 5 | 0.593 | 2.480 | 2.088 | 1.660 |
| 19 | v | 1.734 | 0.6 | 0.648 | 220 | 1 | 4 | 0.621 | 1.988 | 1.507 | 0.980 |
| 20 | v | 1.761 | 0.6 | 2.485 | 70 | 1 | 4 | 0.092 | 0.293 | 0.520 | \ |
| 21 | v | 1.777 | 0.6 | 0.395 | 140 | 1 | 4 | 0.823 | 2.604 | 2.272 | 1.230 |
| 22 | v | 1.853 | 0.6 | 0.451 | 140 | 1 | 4 | 0.735 | 2.282 | 2.140 | 1.080 |
| 23 | v | 1.883 | 0.6 | 0.632 | 140 | 0.5 | 3 | 0.460 | 0.521 | 0.534 | 0.720 |
| 25 | v | 1.972 | 0.6 | 0.413 | 80 | 0.5 | 3 | 0.544 | 0.602 | 0.488 | 0.670 |
| 26 | v | 2.025 | 0.6 | 0.401 | 80 | 0.2 | 2 | 0.359 | 0.392 | 0.288 | \ |
| 27 | v | 2.045 | 0.6 | 0.349 | 70 | 0.2 | 2 | 0.388 | 0.422 | 0.477 | 0.380 |
| 31 | v | 2.252 | 0.6 | 0.560 | 140 | 0.2 | 2 | 0.357 | 0.371 | 0.221 | 0.390 |
| 34 | v | 2.312 | 0.6 | 2.267 | 60 | 0.2 | 2 | 0.058 | 0.060 | 0.043 | \ |

*Table 1. Comparison of deck accelerations, calculated with different methods.*

| Pers/m² | $v_m$ [m/s] | $f_{s,m}$ [Hz] | $\sigma_f$ [Hz] |
|---|---|---|---|
| 0.2 | 1.45 | 1.93 | 0.089 |
| 0.5 | 1.30 | 1.81 | 0.076 |
| 1.0 | 1.04 | 1.61 | 0.054 |
| 1.5 | 0.79 | 1.41 | 0.033 |



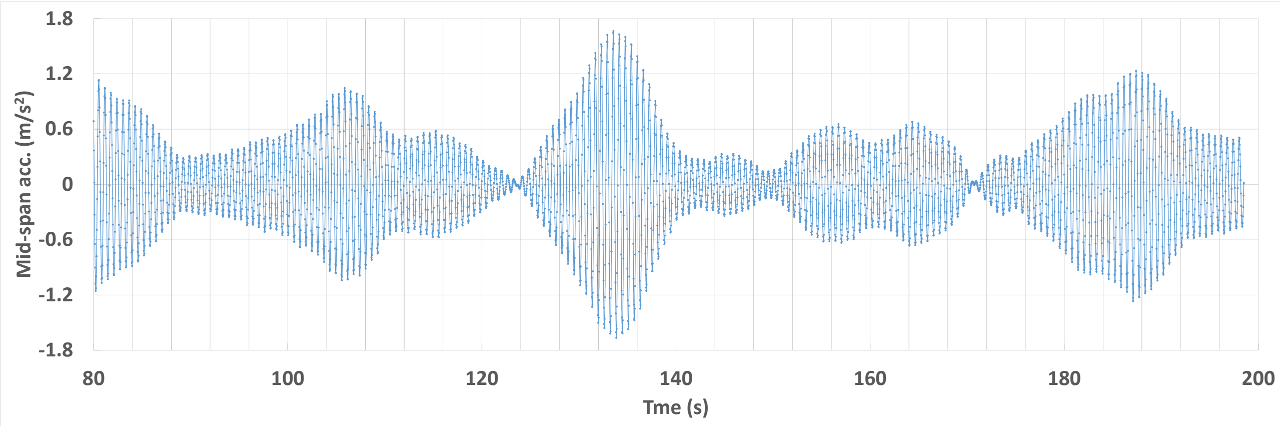




*Fig. 6. Example of time history analysis. Study case parameters: stream mean frequency 1.631 Hz, corresponding to mode # 17; frequency SDT 0.057 Hz; stream velocity 1.07 m/s; pedestrian density 1.5 P/m$^2$; deck maximum acceleration 1.66 m/s$^2$.*

Based on the assumed reference models for pedestrian excitation, response evaluation and comfort criteria, the installation of the TMD will avoid any kind of horizontal synchronization phenomena and unacceptable vibration. Nevertheless, due to (i) the uncertainties that still characterize the available models, (ii) the irksomeness of lateral vibrations (in case of lock-in) and (iii) the importance of the SRPB, additional precautions have been adopted. In particular, provisions for a future installation of additional dampers is made (at the deck ends - see the scheme of Fig. 7).

As regard the vertical vibration, on the other hand, they are less worrying (due to the absence of lock-in phenomena) but numerical evaluations seem to denote a proximity of the reference limits (defined design situation) for different vibration modes.

As contingency plan, the design of the bridge contains provision for vertical damping devices (TMD). The requirement and subsequent installation of these devices will depend on the results of the post-erection dynamic characterization. The provisional vertical TMDs, if needed, will be placed below the concrete slab, between the main beams.

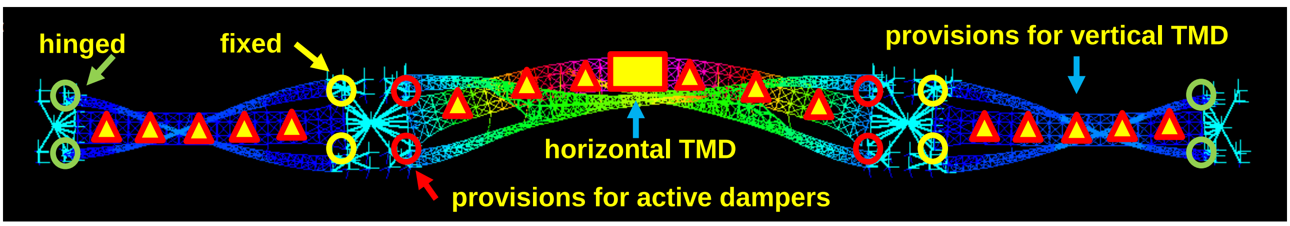



*Fig. 7. Layout of the deck restraints and vibration mitigation devices.*

## 4. Dynamic characterization and monitoring

In addition to the modal characterization (frequency, shapes, damping ratios), the actual pedestrian effects have to be checked during the “dynamic characterization / monitoring” stage (for instance, by testing the effects of pedestrian groups that cross the bridge with different walking conditions, that intentionally try to excite it, etc.).

The EUR 23318 EN [5] refers these type of testing as:

- “level 1” - identification of structural parameters, with purpose of calibrating numerical models and eventually tuning control devices. Natural frequencies, vibration modes and damping coefficients are parameters of interest;
- “level 2” - measurement of the bridge dynamic response under human excitation for assessment of comfort criteria and/or correlation with the simulated response.

Specifically, the characterization and monitoring program should be articulated in the following stages:

1. check of the excitable modal frequencies and shapes. The first stage should aim to define the modal shapes and frequencies that can be potentially excited by pedestrian on the deck and by vortex shedding on arches. The acceleration calculated in section 3.2 will be re-calculated using the actual natural frequencies and damping of the structure measured from these tests to check if they still meet the acceptance criteria described in Section 3.1;
2. estimation of the pedestrian induced vibrations on the deck (based on the reference assumed models). The second stage is aimed to improve the theoretical estimation of the pedestrian and the wind (VIV) induced vibration;
3. mean and long term monitoring of the bridge behaviour during its first lifetime, in order to check the actual response to pedestrian and to wind actions. Given the special nature of the SRPB structure, a further stage (monitoring) during service life of the bridge is suggested to bridge owner even if not strictly required for design purposes. The monitoring duration should be sufficient to “keep” a significant occurrence of pedestrian density and wind speed conditions. This monitoring stage will provide records of actual bridge behavior to be compared to design assumptions.

## 5. Conclusions

In conclusion, using a test case of a recent bridge, this paper highlights the challenges that designers can face when light structures with complex shapes are designed against wind and pedestrian actions.

It shows how wind tunnel results can be used during the design stage, and provides some guidelines and an innovative solution to suppress negative effects like vortex-induced vibrations.

On the other hand, pedestrian induced vibration have been estimated by mean of numerical models derived from the most recent and advanced literature on this theme. Nevertheless, uncertainties are still notable, in both theoretical approaches and design parameters. In addition, often, different evaluation methods gives rise  to different results.

Based on such considerations, reliable tests and information can be achieved only after the completion of the structure. Since the estimated wind and pedestrian induced vibrations do not undermine the structural safety but just the service limit states, the most adopted design procedure is the following:

a) at the design stage is foreseen the possibility to implement some kind of vibration mitigation device; an appropriate combination of restraints and dampers will mitigate the most annoying horizontal vibrations, one horizontal TMD will be installed in the main span deck at the construction stage and provisions are made for possible future needs; for vertical vibration the dynamic analysis results shows a satisfactory bridge behaviour with no need of further mitigation tools. The bridge will be designed with provisions of vertical TMDs which will be only installed if the bridge response resulting from the dynamic characterization test will differ significantly from design assumptions;
b) after the completion of the structure, dynamic characterization tests should be performed and specifically oriented to measure frequencies and inherent damping ratios;
c) finally, if the previous phases outline the opportunity to mitigate the wind/pedestrian induced vibrations even in the vertical direction, TMDs could suitably be installed and tuned.

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