Dynamic Assessment of a FRP suspension footbridge

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ABSTRACT

In the past decade, the vibration serviceability of slender footbridges has become the subject of serious investigation. Despite the advantages that FRP materials offer in bridge engineering such as higher strength-to-weight ratio and ease of installation, their use in the construction of slender footbridges has raised concerns with regard to their dynamic response, due to the reduced mass and stiffness of these materials compared with their conventional counterparts.

In this paper, the dynamic assessment of a FRP suspension footbridge (the Wilcott footbridge) is described. This is performed using dynamic field testing supported by finite element (FE) modelling: the field testing on the bridge produced values for frequencies, mode shapes and damping which were consequently used to calibrate the FE model. Using the calibrated FE model it was shown that the influence of semi-structural or non-structural elements, such as parapets, on the dynamic properties of the structure can be significant.

The dynamic response of the structure due to human excitation was also measured during the test. The results confirmed that suspension footbridges built from FRP materials are susceptible to vibrations induced by pedestrians. The response levels of the investigated bridge are lower than the threshold levels specified in the relevant code of practice.

1 INTRODUCTION

The interest in using fibre reinforced polymer (FRP) materials for bridge applications increased mainly due to the dramatic effects of corrosion in both steel and reinforced concrete bridges [1]. In addition, increasing labour costs for maintenance work, the indirect costs associated with traffic disruption and the application of expensive non-corrosive de-icing salts, have prompted engineers to seek alternative solutions. Fibre reinforced polymers are gaining momentum in bridge engineering applications, particularly where lower weight and ease of installation are important factors. Due to their reduced weight, special attention needs to be given to dynamic behaviour and the human-structure interaction.

The vibrational behaviour of footbridges has been under consideration in design for some years now, but there is no doubt that the case of the London Millennium Footbridge has focused attention into the problem. The tendency to build slender and elegant footbridges is accompanied by a reduction in stiffness and weight, which leads to increased dynamic response under pedestrian excitation.

Generally speaking, noticeable vibrations can occur to bridges independently of their structural form or their construction material. However, it has been found that "lively" footbridges have similar frequencies. For vertical frequencies the problematic range is between 1.5-2.5Hz whereas for the horizontal direction, vibrational problems occur within the frequency range of 0.5-1.1Hz [2]. It needs to be mentioned here that some footbridges experience problems only after they are loaded with heavy pedestrian traffic as was the case of the London Millennium Footbridge [3]. In addition, because most of the bridges that have experienced vibration problems are made of steel and, hence, have low damping, this factor is also mentioned as a possible source of vibrational problems [4].

Considering pedestrian loading, it has been found that people normally walk with pacing rates of 1.6-2.4Hz [5]. Therefore, vibrations are a result of resonance, i.e. when the frequency of the pedestrian excitation coincides with the frequency of the bridge.

The investigated Wilcott footbridge belongs to the small select group of bridges in Europe having their deck built entirely from FRP material. At present, data about their performance and dynamic properties are limited. Therefore, experimental testing of these types of bridges can provide important information for future structures of similar type. With a dynamic test, the measured data are directly linked to frequencies, mode shapes and damping. The estimation of modal damping is particularly important as it is the only property that cannot be estimated by prior numerical analysis. The measurement of the bridge's actual response to pedestrian crossings and the assessment of its vibrational performance is another benefit from a field test. In addition, the test results can be used to calibrate a finite element (FE) model, which may then be used in further numerical studies.

2 BRIDGE DESCRIPTION

The Wilcott footbridge in Shropshire was completed in February 2003 and was opened in March of the same year. It is a 51.3m single span suspension footbridge with a slightly cambered slender deck providing a footway 2m wide. It spans the A5 dual carriageway trunk road and connects the villages of Wilcott and Nesscliffe [6]. The general arrangement of the bridge can be seen in Figure 1.

Figure 1. View of the completed Wilcott footbridge

The main feature of the bridge is the glass fibre reinforced polymer (GFRP) deck, which was fabricated using pultruded components from the same Advanced Composite Construction System (ACCS) used on the Aberfeldy Footbridge [7]. ACCS is pultruded by Strongwell Corp and is now known by the trading namne Composolite [8]. The cross section of the deck, an assembly of panels, three-way and toggle connectors is depicted in Figure 2. Transverse beams (each a square section constructed using four 3-way connectors) are provided at each parapet post and hanger cable position along the length of the deck. To increase the mass of the deck, the middle panels are filled with ballast. The surfacing is provided by interlocking rubber blocks, manufactured from recycled vehicle tyres.

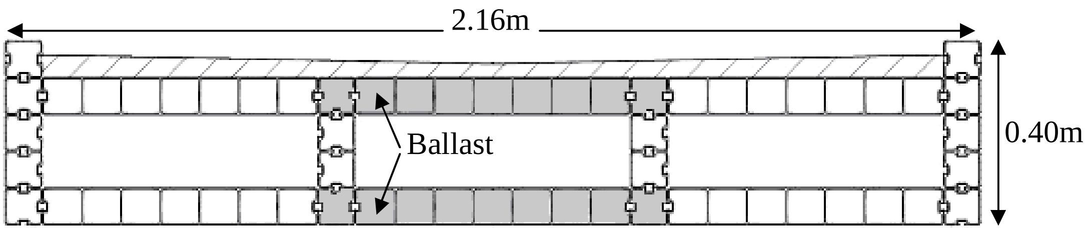

Figure 2. Deck cross section

The GFRP deck was prefabricated in three parts each measuring around 17m in length and connected in-situ. The inclined hangers are steel spiral strands and are connected on the deck via a stainless steel plate backed by four threaded studs bonded into the end of the transverse beam. The hangers are attached on the main cables which are also steel spiral strands with steel clamps; the main cables are in turn anchored at each side to two inclined pylons. The pylons are steel circular hollow sections constructed in tapering form, and they are supported by backstays made of solid steel bars.

The GFRP deck, the pylons, and backstays are supported on a single concrete raft foundation to each side of the bridge. The bridge deck was cast into pockets left in the foundations, and circular plinths are provided for the pylons and the backstays. Finally, a stainless steel parapet system is provided along the length of the bridge. Parapet posts are secured to the deck transverse beams; handrails and footrails are attached to the posts, and a stainless mesh is provided for containment. The footrails and handrails were designed to allow movement along their length through a sliding mechanism.

3 FINITE ELEMENT MODELLING

The first step taken to study the bridge was to use all the available information on the constructed bridge to develop a FE model using ANSYS v.8 commercial finite element program [9]. A three dimensional (3-D) FE model was developed in which the composite deck was modelled in detail using shell elements (Figure 3). Information collected from drawings as well as photos taken during the fabrication of the deck and the construction phases of the bridge provided a valuable insight. In addition, the weight of the deck, measured during the lifting-in-place process, was made available. Thus, considering the deck self-weight and the additional contribution in mass from the ballast, the transverse beams, the polymer inserts etc, the total mass of the bridge's deck was adjusted to comply with the measured value of 27 tonnes.

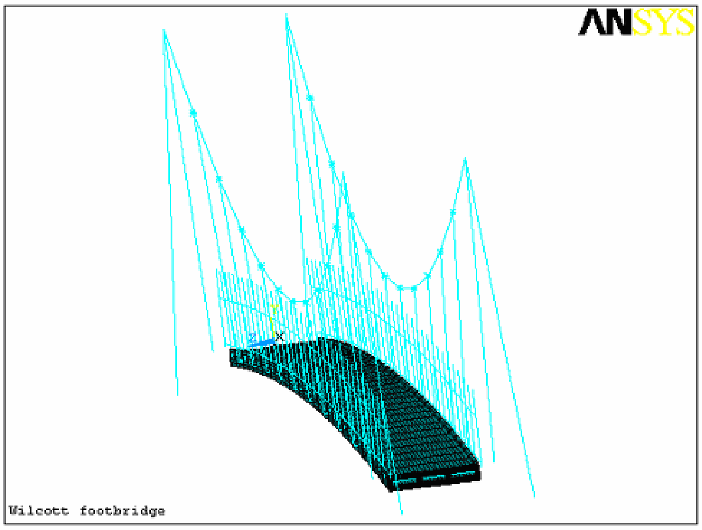

Figure 3. The FE model of the bridge

Having obtained precise information regarding the mass, the remaining parameter which is essential for accurate estimation of the dynamic properties of the structure is the stiffness. This property depends on the material's characteristics such as the modulus of the GFRP deck and also on construction details. For this type of bridge, the latter include the boundary conditions, the amount of tension in the main cables, backstays and to a lesser degree in the hangers. Although the ballast in the cells may have some effect on stiffness, its contribution was not considered and was only included as an additional distributed mass.

The GFRP deck is modelled with 8-node shell elements (SHELL93) with six degrees of freedom at each node. The weight of the ballast and the deck surfacing was allocated as additional distributed mass. The transverse beams were modelled using beam elements (BEAM4) within the cellular box of the deck. The members of the cable system, that is the main cables, the hangers and the backstays, were all modelled as tension only (truss) elements (LINK10) which have stress stiffening capability. The pylons were modelled as solid uniform beam elements (BEAM4).

The boundary conditions at the supports were modelled according to as-built conditions, as verified during site visits, and in accordance with the relevant design drawings; at each end, fully fixed conditions were assumed to prevail over a finite length. The pylons and backstays were also treated as being fixed in all directions at their bases.

It was decided to model the parapets as structural parts rather than as a distributed mass along the edges. This is because it is considered that their contribution to the stiffness can be significant, depending on the degree of continuity achieved between the segments [2]. Hence, an effort was made to capture in the modelling of the parapets, as close as possible, their actual function. All the modelled parts were treated as beams (BEAM4), whereas the connections were modelled so that longitudinal movement between the panels is allowed, but movement is restrained in all other directions. The adopted modelling could also incorporate the modelling of springs at the joints, once field data became available for the calibrating procedure.

The FE model developed following these principles and idealisations was used for a preliminary numerical modal analysis to provide an insight into the possible dynamic behaviour of the footbridge. The modal analysis of suspension bridges is a two-step procedure, and is termed prestressed-modal analysis, because it is performed on the deformed structure in which the structural members are prestressed after a static analysis [10]. This analysis is performed so that the input prestressing in the cables is adjusted to reasonable values. The preliminary modal analysis showed that the footbridge had more than seven modes below 10Hz. Based on these results, and the extracted mode shapes, an effective field test was planned.

4 VIBRATION TESTING

4.1 Testing procedure

A variety of techniques exist for field modal testing, depending on the availability of equipment, the type of structure and the operational conditions. If the input force is not measured, the analysis is done using response data, and is better known as output-only analysis [11]. The latter is associated almost exclusively with the Ambient Vibration Survey (AVS) method, although in some cases human activities can be used to excite the bridge. Ambient vibrations are the vibrations caused by excitation experienced by a structure under its normal operating conditions, therefore allowing the bridge to remain open. According to equipment availability and information from previous tests on FRP footbridges described in [2] it was decided to adopt the AVS approach using as excitation the wind and the under passing traffic.

For the field test a four channel dynamic signal analyser (LDS Dactron Phaser), two high sensitivity accelerometers suitable for low frequency measurements and battery units/ amplifiers to raise the signal level when necessary were used. The acquisition parameters were defined based on the preliminary FE modal analysis.

Before the initiation of any measurement, the behaviour of the footbridge was observed during passing of vehicles and crossings of pedestrians. The vibrations induced by pedestrians were noticeable whereas passing of vehicles produced merely perceptible vibrations. The exception was large HGVs which triggered definitely noticeable vibrations. Additionally, it was observed that during pedestrian crossings, and as vibrations intensified, the suspended cables oscillated laterally. This implied that a cable mode was being excited due to the lateral component of the pedestrian induced force.

The first acquired data were used to verify the position of the reference station. The measured frequency spectrum was dominated by two clear peaks at 1.5Hz and 2.2 Hz, thought to be the second vertical (V2) and third vertical (V3) mode based on the FE model predictions. This also suggested that it is walking at a pacing rate of 2.2Hz that causes the lateral movement of the cables: based on the FE model, the first local cable modes were around 1Hz, and therefore can be excited by the lateral component of the walking force. It is worth mentioning that for normal walking at 2.2Hz the lateral force is exerted at 1.1Hz [12].

To assess the serviceability of the footbridge and to measure damping, tests were carried out using controlled walking with the aid of a digital metronome. The footbridge resonant frequencies that lie close to the normal walking range (1.6Hz-2.4Hz) were targeted in these tests. Thus, for investigating mode V3, walking at 1.5Hz for V2 and at 2.2Hz was undertaken. In addition, with a view of examining mode V1, for which a prior frequency estimate was around 0.95Hz, jumping tests were conducted as walking at this frequency is outside the normal range.

4.2 Data analysis

The data processing was performed using SPICE (Signal Processing In Civil Engineering) which is a code written in MATLAB and in which system identification for output-only analysis is implemented using the Stochastic Subspace Identification (SSI) technique or the Peak-Picking method [13].

Modal parameters

The frequencies were estimated using data from both the reference station (to enable the construction of a record over one hour long) and the measurements points. The resulting frequency spectrum is shown in Figure 4. The identified peaks represent modes in the vertical direction. In total, eight vertical modes with their associated mode shapes were identified in the range 1-8Hz. The mode shapes and the modal ordering were in agreement with the predicted modes from the FE analysis.

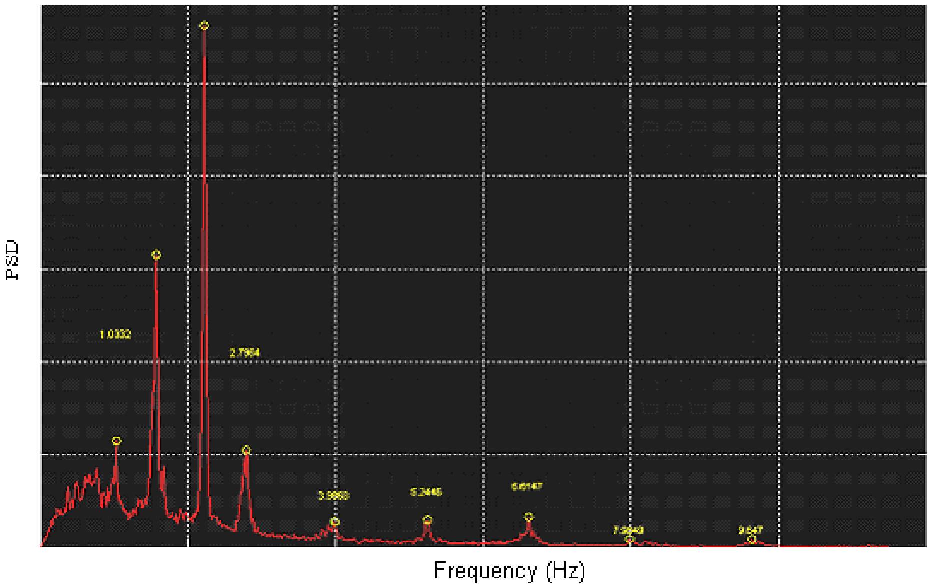

Figure 4. The frequency spectrum obtained from the field test

The damping was measured using pedestrian tests. Two types of controlled activities were used to measure damping: normal walking along the bridge and stationary stamping (s/s) at the antinode of the excited mode. The latter was found easier to apply than jumping. In each case the damping is estimated from the decaying response after the input excitation is stopped. The basic difference between these two tests is the presence of an extra person on the bridge. In most cases the s/s or jumping test gives higher damping values than normal walking, because of the contribution to damping of the standing person. The primary method used to estimate damping was the logarithmic decrement method [14]. A typical response decay after walking at 1.5Hz (V2) is shown in Figure 5.

The estimation of damping for V3 required a person to walk at 2.2Hz. It was clear that as the person was walking along the bridge, the main cables started to oscillate laterally. As previously mentioned, this is because a local cable mode exists at half the selected walking rate. The effect of this mode coupling was evident in the measured decay response which had two parts: at higher amplitude the rate of decay is exponential but, as the amplitude decreases, the rate of decay becomes constant which is actually a sign of non-linearity. Therefore it is considered that the damping of this mode has two parts: the average of the first part was 0.716% and for the second was 0.5145%.

This hypothesis about the nature of damping of V3, regarding the cable participation was verified when the s/s test was applied; no lateral force was generated for this activity and the cable mode was not excited and as a result the form of the decay signature was exponential.

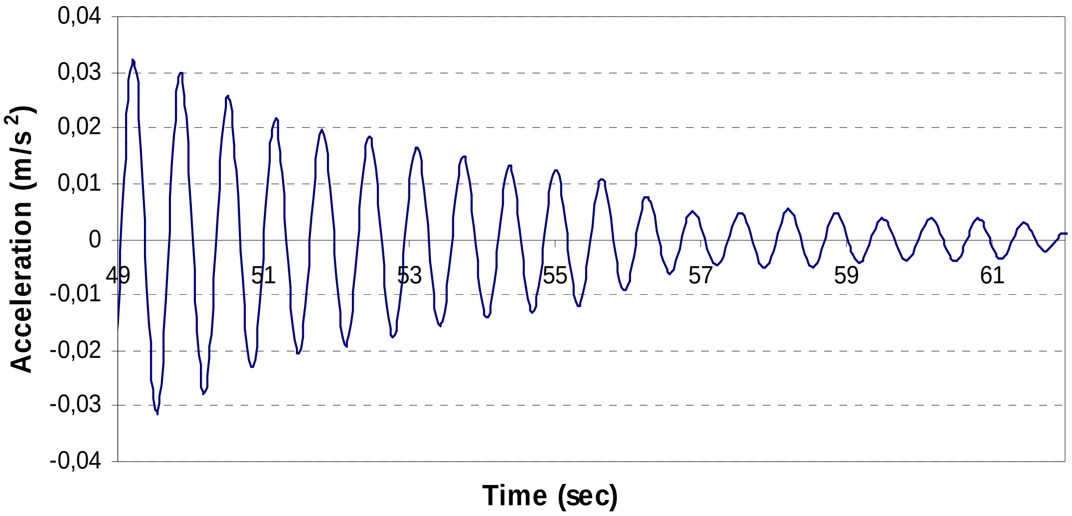

Figure 5. Decay response of mode V2 after walking at 1.5Hz

Table 1 summarises the measured modal parameters for the eight vertical modes investigated. In addition, damping estimates obtained from the system identification procedure are also provided. The damping is shown as damping ratio and is generally higher than values listed in literature for footbridges associated with lively behaviour.

Table 1. Modal parameters summary

No	Frequency (Hz)	Pedestrian tests Damping (ζ%)		System Identification using SSI (ζ%)
		Walking	Stationary Stamping s/s	
V1	1.03	-----	-----	1.46
V2	1.55	1.64	1.84	1.94
V3	2.22	0.5145-0.716	1.5	0.69
V4	2.77	-----	-----	1.58
V5	3.97	-----	-----	0.72
V6	5.26	-----	-----	1.60
V7	6.61	-----	-----	1.41
V8	7.93	-----	-----	0.81

Dynamic response

The "filtered" acceleration time-response of the footbridge, measured at the antinode of the excited node was used to determine the peak acceleration. For mode V2, a pacing rate of 1.5Hz was selected on-site, whereas for mode V3, the walking frequency used was 2.2Hz. The objective was to compare the measured accelerations with the acceptability limits defined by the UK bridge code, BS5400 [15]. The results are listed in Table 2. Based on this assessment, the bridge complies with serviceability criteria. The response for V2 is close but still below the limit, whereas the response of V3 is well below the limit.

Table 2. Dynamic response due to walking

Mode	Accelerations (m/s^2)	
	BS5400	Measured
V2-1.5 Hz	0.62	0.47
V3-2.2 Hz	0.75	0.21

Two crossings were performed for each case using the same pedestrian. An example of a measured acceleration record obtained from walking at 1.5Hz is shown in Figure 7.

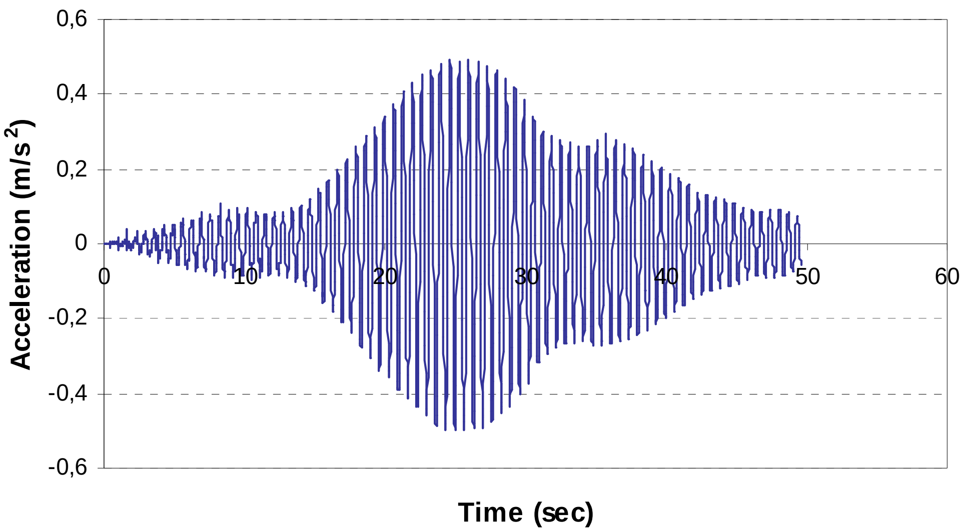

Figure 7. Measured acceleration response for walking at 1.5Hz

5. FE model updating

Model updating is a process in which the numerical model is adjusted so that values of key response parameters match their experimentally recorded counterparts. Usually the discrepancies between the models are due to simplifying assumptions, uncertainties in material and geometric properties and boundary conditions. The updating process aims to minimise the differences between the predicted and experimental values. The updating procedure was implemented using sensitivity analysis (in Excel) and optimisation techniques as provided by the FE program ANSYS.

The most effective parameters employed in the updating process were the following: (1) the orthotropic properties of the GFRP deck, i.e. the longitudinal elastic modulus E_x, the transverse and vertical modulus E_y=E_z, the shear modulus G and the density; (2) the elastic modulus of the main cables and hangers; (3) the amount of initial strain in the cable members; and (4) the stiffness of the handrails where two spring elements have been used, one in the vertical and the other in the longitudinal direction. The contribution of parapets to the structure's stiffness was significant. By assuming that the parapet segments are fully continuous and rigidly connected throughout, the lower frequencies are increased by more than 30%, compared with the case of modelling parapets as simple attachments, made from individual panels with no connection between them. Thus the stiffness of the springs was varied between these two extremes during the calibration to reach an optimum value.

The final values from the updating process are listed in Table 3.

Table 3. Updated parameters values

Parameter	Value
E_x, GPa	23.8
E_y=E_z, GPa	9
G, GPa	11
Density, Kg/m3	1930
Hangers, E_x, GPa	199
Main cables, E_x, GPa	165

The correlation of the numerical modal parameters resulting from the updating process with their experimental counterparts was very good: for the frequencies the difference was 1%. For the mode shapes where the MAC criterion is used to quantify correlation, values of 0.9 were achieved with unity corresponding to perfect correlation. The extracted mode shapes for the first four modes, from the FE model and the field test are plotted in Figure 8.

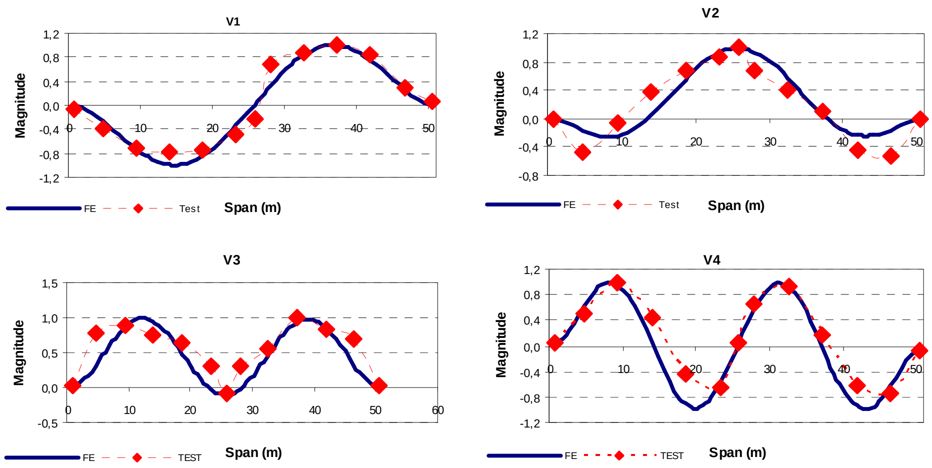

Figure 8. Mode shapes

CONCLUSIONS

$$a_{\max} = 0.5\sqrt{f_0}$$

$$\zeta = \frac{\delta}{\sqrt{(2\pi)^2 + \delta^2}}$$

$$\delta = \frac{1}{n}\ln\left(\frac{peak\,0}{peak\,n}\right)$$

The footbridge was tested using the AVS technique and the modal analysis was completed with the extraction of the modal characteristics for the first eight vertical modes (V1 to V8) using the program SPICE and the stochastic subspace identification (SSI) technique.

The eight measured nodes appeared in the range of approximately 1Hz to 8Hz. The fundamental frequency at 1.03Hz is in line with current trends found in slender footbridges. The obtained mode shapes were of very good quality despite the relatively short acquisition time. The last two modes (V7-V8), although visually excellent, were affected by the limited measured points which resulted in lower MAC values than the first six modes. However, if further structural assessment is to be performed, better quality shapes are required, and, on the evidence of this test, improved quality can indeed be achieved.

Damping values were obtained only for modes V2 and V3 which are the most important for a vibration serviceability assessment. The results showed that the presence of stationary persons on the bridge can increase damping. For V3, cable oscillation had a marked effect on modal damping. The introduction of non-linearities due to coupled motions affected the expected exponential form of the response decay.

For the Wilcott footbridge, modes V2 and V3 can be excited by normal walking. The results of pedestrian tests showed that V2 exhibits the larger response but both modes result in values smaller than the respective acceleration limit set in BS5400. Therefore, although perceptible vibrations are indeed experienced while on the bridge, the level of vibration lies within acceptable boundaries.

The field data were also used to calibrate the developed FE models. The contribution of parapets to the stiffness of slender footbridges is very important and their effect should not be neglected. The updated FE model can be used in further sensitivity studies and can also be used as a benchmark to assess durability influences that might arise as a result of the bridge's exposure to the environment (e.g. moisture uptake, etc).

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