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Human induced vibration vs. cable-stay footbridge deterioration

S. Casciati*

Department of Civil Engineering and Architecture, University of Catania, Piazza Federico di Svevia, 96100 Siracusa, Italy

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Abstract. In this paper, the possibility of using human induced loading (HIL) to detect a decrease of tension in the cable-stays of an existing footbridge is investigated. First, a reliable finite elements model of an existing footbridge is developed by calibration with experimental data. Next, estimates of the tension in the cables are derived and their dependency on the modal features of the deck is investigated. The modelling of the HIL is briefly discussed and used to perform the nonlinear, large strain, dynamic finite elements analyses. The results of these analyses are assessed with focus on characterizing the time histories of the tension in the cables under pedestrian crossing and their effects on the deck response for different initial conditions. Finally, the control perspective is introduced in view of further research.

Keywords: cable tension; footbridge; geometric nonlinearity; human induced load,; stays

1. Introduction

An often adopted solution for the supporting structural skeleton of footbridges is the cable-stayed scheme (Bursi *et al*. 2014, Cantieni 2013). Their response depends on the actual tension in the cables (Liao *et al.* 2012) and this tension should be preserved by an efficient monitoring/maintenance action which is lacking in some of the practical situations. With respect to the large scale cable-stayed bridges open to traffic and supported by hundreds of cables (Virlogeux 2001, Li and Ni 2016, Casciati *et al.* 2016), footbridges offer a simple testbed for structural control solutions aiming to counteract the effects of the cables' tension deterioration. On the other hand, the cost of such solutions might be disproportional with respect to the initial cost of the structure. Therefore, a feasible approach consists of pursuing offline solutions, rather than a continuous monitoring (Cho *et al*. 2010) for online control. However, it must also be considered that the cable tension forces vary in real time because of the loads from the pedestrians crossing and other environmental effects (Li *et al*. 2014, Yang *et al*. 2015).

The implementation of a virtual laboratory offers the possibility to verify the evolution of the structural response to the HIL (Human Induced Loading) vibrations (Zivanovic *et al.* 2005), as the cable tension deteriorates. For this purpose, a reliable numerical model is first created on the basis of a limited experimental dataset. The model is then used to carry out dynamic transient analyses accounting for geometric nonlinearities and targeted to assess the cable performance in operational conditions (away from the resonant regions). Different damage scenarios are considered, where

*Corresponding author, Ph.D., E-mail: saracasciati@msn.com

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damage is herein intended as a tension decrease in one or more of the central stays of the footbridge. A strategy to discern the considered damage scenarios from the responses obtained at the deck mid-span is proposed, and its findings could be useful toward the development of damage detection and localization techniques from acceleration data. Finally, the possibility of counteracting the cables tension deterioration by means of simple and offline tensioning devices is investigated as summarized in the following.

This study is carried out with reference to an existing slender footbridge (Fig. 1) where the main external cables are anchored to the ground. Hence, a low cost control system would consist of increasing the tension only in these main external cables during maintenance. Of course, acting only on the external stays causes a redistribution of forces also on the other stays whose effects on the overall deck response need to be assessed. In view of a proper design of such a structural control solution, this work aims to investigate the effects of increasing the tension on the deck response to the HIL. For this purpose, one of the above mentioned damaged configurations is considered and its initial conditions are varied . The assessment of the results of the nonlinear dynamic transient analyses permits to establish if the deck response obtained by increasing the tension in the cables is consistent with the one of an undamaged situation.

2. Building the virtual laboratory

For studying the evolution of the axial forces in the stay-cables of the footbridge in Figure 1 under serviceability conditions, the following steps were taken:

1) Acquisition of the design data;
2) Experimental campaign and preliminary elaboration of some recorded signals;
3) Implementation of a numerical model for both the structural system and the main excitation components.

Fig. 1 General view of the Farra d’Alpago footbridge

2.1 Geometry and materials data

The timber footbridge of Fig. 1 is located in North-Eastern Italy. It consists of three consecutive spans in a simple cable-stayed configuration. The deck is realized based on a new technology for producing high strength GLT (glued laminated timber). The elements in lamellar larch are combined with the steel elements (transversal beams and horizontal braces) forming the deck skeleton. The footbridge main features are (Casciati *et al*. 2015):

a) Deck, of length 110 m (with a central span of 66 m) and lateral beams in lamellar larch forming a U-shaped cross-section of width 3.08 m and a crossing free width of 2.50 m.
b) Pylons (masts) in steel of height 16 m.
c) Steel cables of diameter 44 mm (the longest ones) and 32 mm (the shortest ones) anchored to both the internal and external sides of the tower.
d) The long external cables are also anchored to the ground. The tensioning devices are preserved, allowing one to re-set the cables tension as scheduled in the maintenance plan.
e) The cable tensioning process consists of two contributions: the actual tensioning of the external cables (by a displacement in the longitudinal direction) and the tensioning achieved when mounting the central element of the main longitudinal beam. The following tension forces in each of the four clusters of four cables were theorized in the design process.

External long cable	External short cable	Internal short cable	Internal long cable
77.4 kN	30.8 kN	25.9 kN	82.3 kN

The high uncertainties, which affect the modelling assumptions, suggest to calibrate the model using empirical data in order to make it realistic and reliable.

2.2 Experimental evidence

An experimental campaign was carried out using a limited number of sensors. In particular, four tri-axial accelerometers were placed along the deck middle-line, and two uni-axial accelerometers were mounted on a pair of long internal stays to monitor the acceleration component in the transversal and vertical directions, respectively. The tests had as excitation either the wind action or the effects of the crossing of a group of six pedestrians. The attention is herein focused on the effects of the HIL. The spectra in Fig. 2 are derived from the acceleration records of the two monitored cable stays.

The presence of several harmonics, which is confirmed by similar plots for the records collected under the wind excitation, suggests that:

1) the low frequencies seem to be more smoothed than the high ones;
2) the presence of a joint tolerance is the source of the harmonics, since the deck and the mast do not show them.

2.3 Numerical modelling

The numerical model in Fig. 3 was created within the commercial software Marc (MCS, 2004), which is particularly suitable to carry out nonlinear analyses in the large strain option by adopting

an updated Lagrangian description. The large strain assumption is necessary to manage the cable elements.
Indeed, in the specific software adopted by the author, the cables are correctly managed as 3D beam elements provided that the analysis is conducted in large strain. The transient dynamic analysis consists of two stages: in the first one, the gravity load is applied as well as the cables are tensioned; in the second stage (from 100 to 200 s), the crossing of the pedestrians occurs as modelled by the time histories that are realizations of the stochastic random field model presented in (Casciati *et al*. 2016).

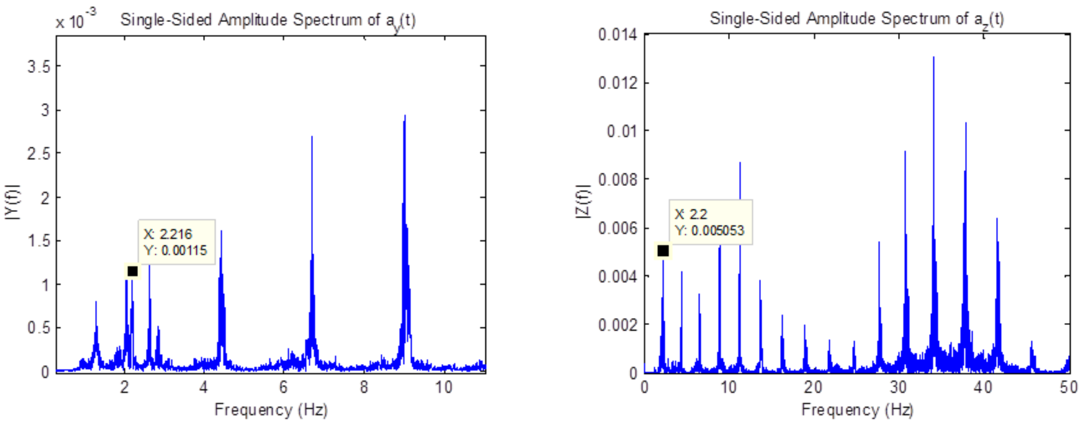


Fig. 2 The spectra computed from the recorded transversal (y) and vertical (z) acceleration components at the bottom of the long internal cable stays

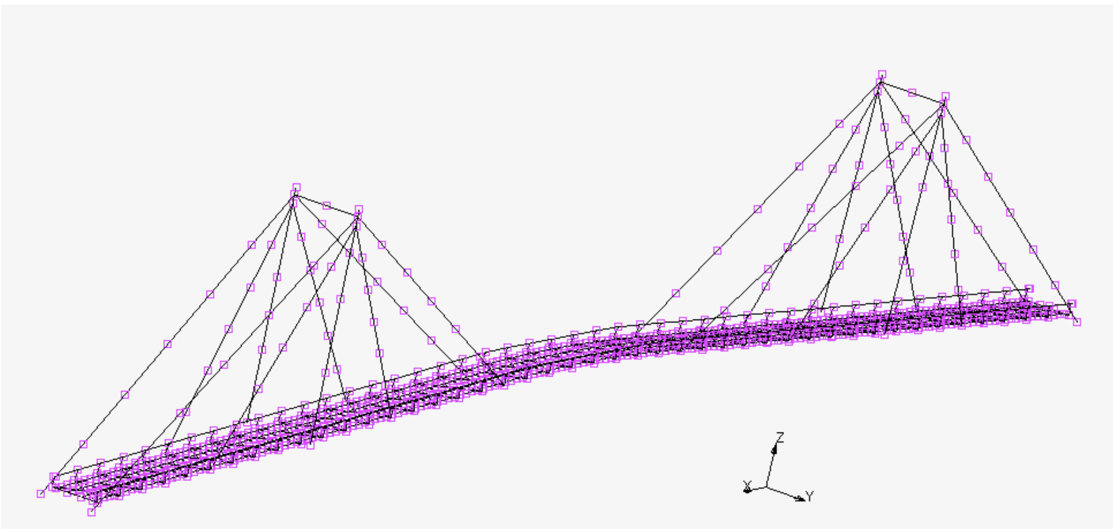


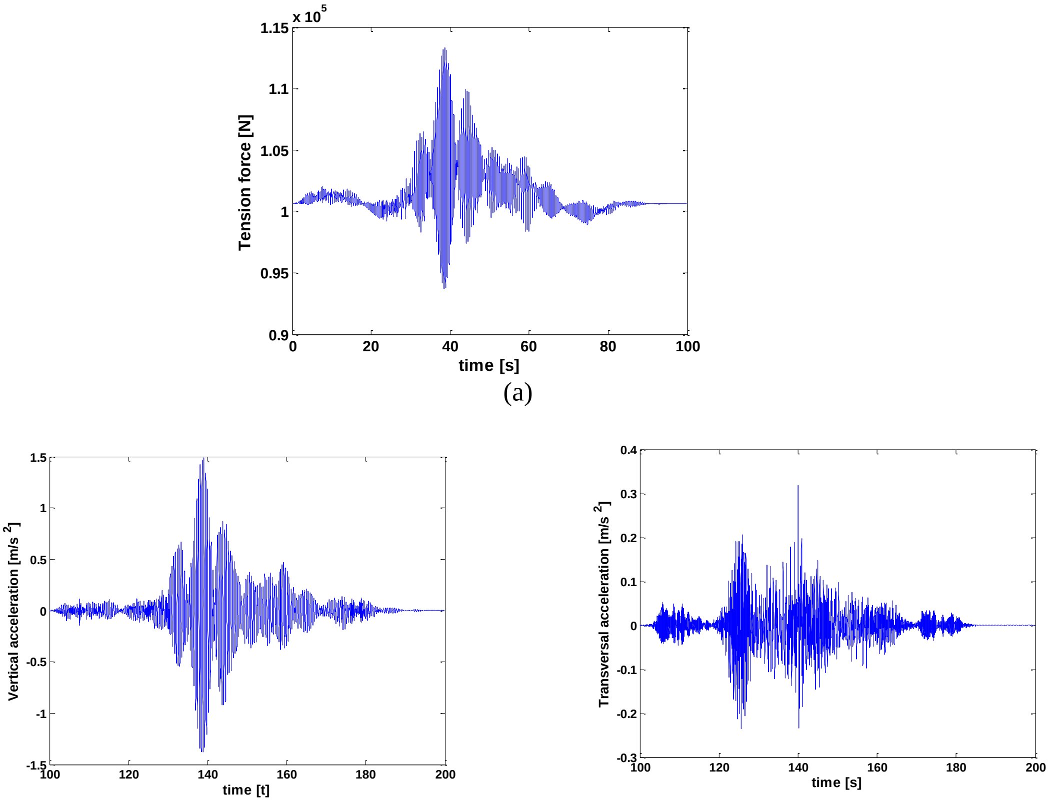
Fig. 3 Numerical model of the cable-stayed footbridge under investigation

As a result of the first stage, the tension force in the long internal cable-stay is estimated to be 103 kN, significantly higher than the assumed design value reported in sub-section 2.1. However, its horizontal component of 87.3 kN is consistent with the design assumption. By adopting as input the value of frequency marked in the two plots of Figure 2, one can verify that the estimates of the tension in the cable calculated by either numerically solving the transcendental equations or by applying the approximate empirical formula found in the literature for similar modelling assumptions (see Table 1) agree well with the one obtained by the proposed finite element approach. Furthermore, by introducing a finite element discretization of the cable, also the spatial variability of the tension along the cable is represented (Ni 2002), together with its sag extensibility and flexural rigidity.

In the existing literature, most studies concern either a single pedestrian motion (Schiehlen 2014) or the crowd load (Venuti and Bruno, 2009) due to the lock-in phenomenon, but the crossing of a group of few pedestrians, as the one used to collect the experimental data, is typically not considered. Several assumptions are introduced to simulate the random field realization. In particular, the considered pedestrian force consists of a stationary part (equivalent to the mean wind velocity, in standard models of the wind action) and a fluctuation (stochastic component) modelled as done for the wind turbulence gust (Solari and Piccardo 2001). The realizations of the pedestrian walking forces in the vertical and transversal directions are derived in (Casciati *et al*. 2016, Casciati *et al.* 2015, Casciati *et al*. 2014). They are herein used to reconstruct the response time histories at each node accounting for the different phases of the pedestrians density spatial distribution along the footbridge model.

Table 1 Horizontal component of the tension in the central cables (diameter 44 mm, inclination angle 32°, length 27.3 m, sag 0.07 m, Young modulus (1.65)10^{11} N/m^2, and mass density 5620 kg/m^3) for different modelling assumptions and frequency based computational methods

Cable model		Method	H [N]
3d-beam in large displacement		Finite elements approach	87304
Straight shallow taut string theory		Closed-form solution	88501
Beam with flexural rigidity	simply supported at both ends	Closed-form expression (Humar 1990)	87941
	fixed at both ends	Numerical solution (Humar 1990, Zui *et al*. 1996)	78600
		Empirical formula (Ren *et al*. 2005)	78227
Inclined cable with flexural rigidity and sag		Empirical formula (Zui *et al*. 1996)	91138
Horizontally supported cable with sag extensibility only (no flexural rigidity)		Numerical solution (Irvine and Caughy 1974)	74600
		Empirical formula (Ren et al 2005)	75400
Inclined cable with sag extensibility only (no flexural rigidity)		Numerical solution (Trinitmiantafyllou and Grinfogel 1986, Russel and Lardner 1998)	66611
Horizontally supported cable with flexural rigidity and sag		Simple relationship (Mehrabi and Tabatabai 1998)	57000



(a)

(b) (c)

Fig. 4 Response time histories estimated from the numerical model. (a) tension force at the bottom node of the long internal cable, (b) vertical acceleration component at the deck mid-span and (c) mid-span acceleration component in the transversal direction

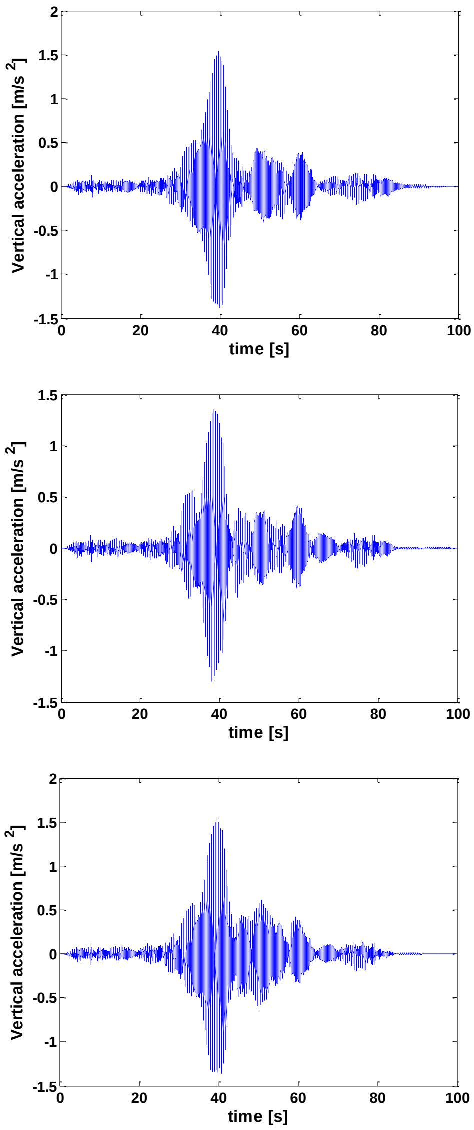
Fig. 4 provides some response time histories as obtained from the model calibrated on the experimental data of the previous sub-section. The evolution of the tension (Fig. 4(a)) in the bottom node of the monitored long internal cable during the pedestrians crossing is assessed. The acceleration time histories at the deck mid-span are also shown in view of a comparison with those obtained by varying the tension initial conditions. The plots of Figs. 4(b) and 4(c) are obtained at the deck mid-span, and they represent the vertical and transversal acceleration components, respectively. The values of the acceleration in the vertical direction (Fig. 4(b)) are much higher than those observed in the transversal direction (Fig. 4(c)) for the considered pedestrians configuration. For this reason, in the following sections, the effects of varying the initial tension are assessed with reference only to the former response quantity.

3. Deteriorating the initial tension in the cable-stays

In this section, the responses of Fig. 4 are herein estimated for different scenarios of cable deterioration in the attempt of establishing a link between the acceleration records and the actual values of the tension force in the cables toward monitoring and control applications.

Three “damaged” cases are considered by varying the initial tension in the cables as follows:

1) The tension in both the internal long cables on the l.h.s of the model in Fig. 2 is decreased to the value of 82.3 kN.
2) The tension decrease is applied to only one of the two cables considered in Case 1.
3) The tension is decreased in both the internal long cable of Case 2 and its correspondent on the r.h.s. of the model in Fig. 2.



(a)

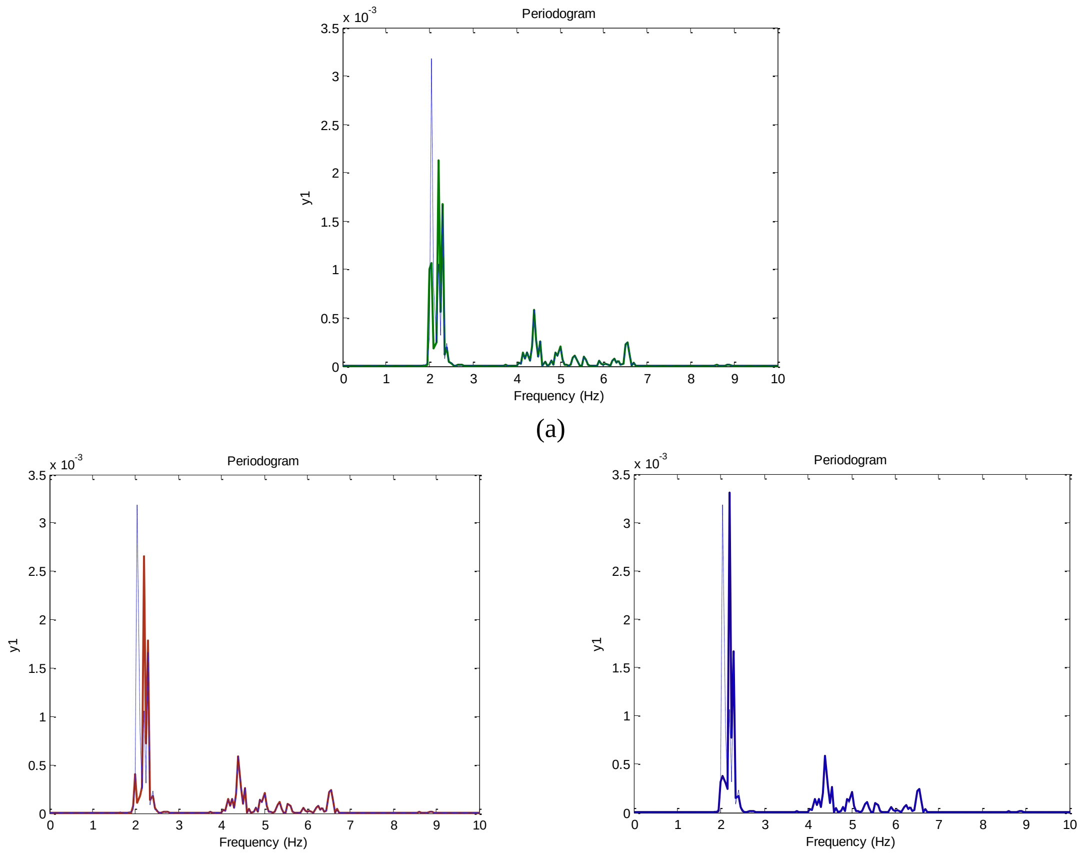
(b)

(c)

Fig. 5Time histories of the mid-span vertical acceleration in the damaged cases listed in the text as 1 (a), 2 (b) and 3 (c), respectively

The effects on the time history of the vertical acceleration component at the mid-span of the deck is shown in Fig. 5 for the three above described “damaged” cases. The plots have to be compared with that of Fig. 4(b) which refers to the initial tensions configuration identified from the “undamaged” monitored condition. A spectral representation of these signals is given in Fig. 6.

The curves in Fig. 6 are actually obtained by considering only the first 20 sec. of the signals in Fig. 5. Indeed, if the entire signals duration of 100 sec. is considered, no significant difference between the spectra can be detected due to the predominant effect of the pedestrians stepping on the measurements point at the deck mid-span. By focusing only on the initial phase of the pedestrians crossing, the effects on the response at mid-point have a uniform order of magnitude, which is similar to the one obtained under ambient vibrations. At this reduced scale, it is possible to detect differences between the spectra as demonstrated by Fig. 6. These differences could be exploited to detect and localize the damage from monitored data.



(a)

(b) (c)

Fig. 6 Periodograms for the three scenarios of reduced tension compared with the original situation (dashed line). The damaged cases are listed in the text as 1 (a), 2 (b) and 3 (c), respectively

In all the damaged cases, an increase of the fundamental frequency is observed, whereas the higher frequencies are unvaried. This could be a consequence of the redistribution of tensions in the cable stays. Indeed, by decreasing the initial tension value in the inner cables, the external ones are subjected to a tension increase which is reflected in the deck response as the pedestrians pass by those cables at the beginning of the crossing.

4. Some off-line control aspects

After a decrease of tension in the cables has been detected and located by the procedure reported in the previous section, one would like to consider the possibility of a countermeasure. Unfortunately, for a footbridge as the one investigated in this paper, online structural control does not apply because:

i) The devices would result too expensive when compared with the initial cost of the structural system;

ii) The structural system is too isolated for preventing the installation from vandalisms.

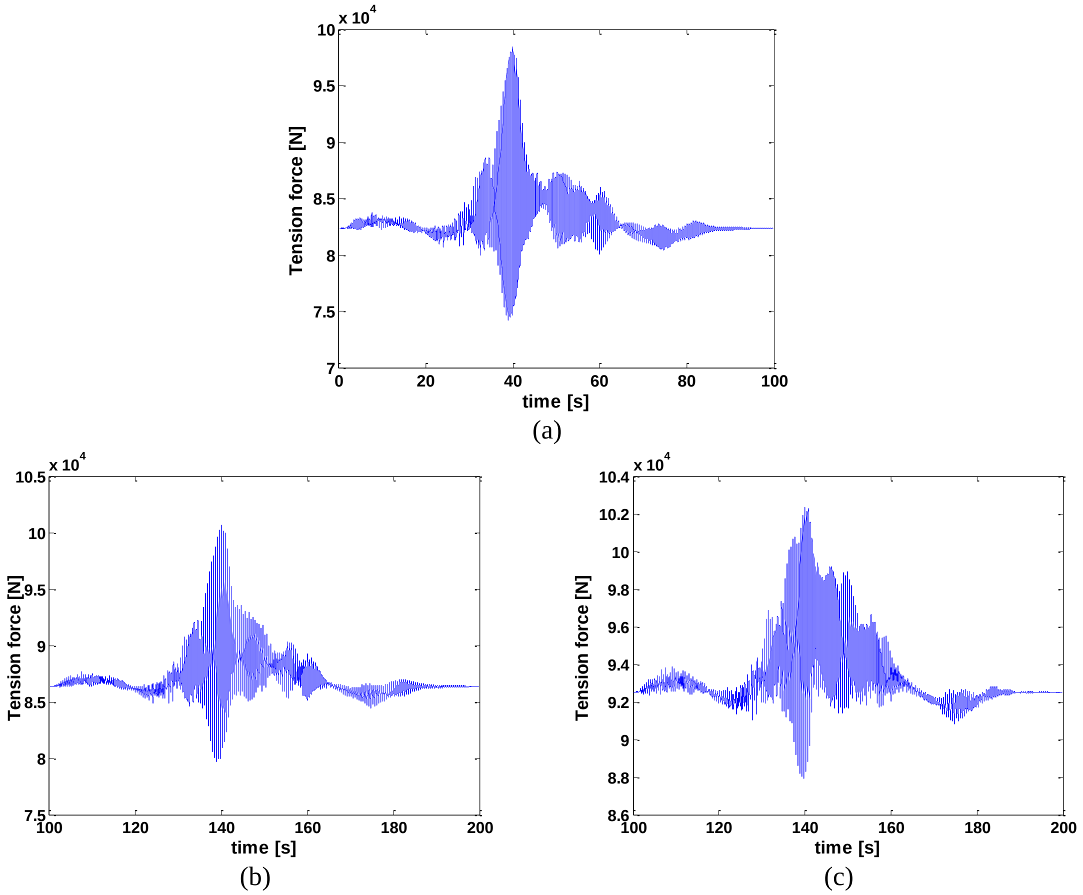
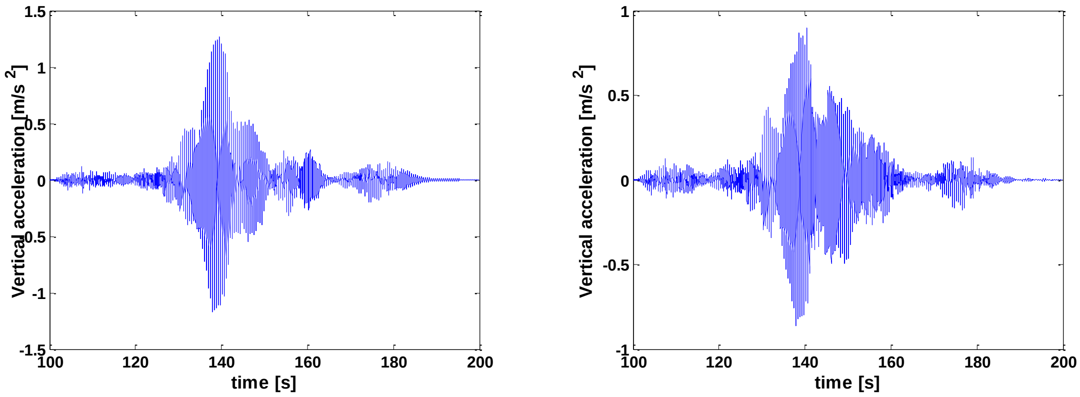


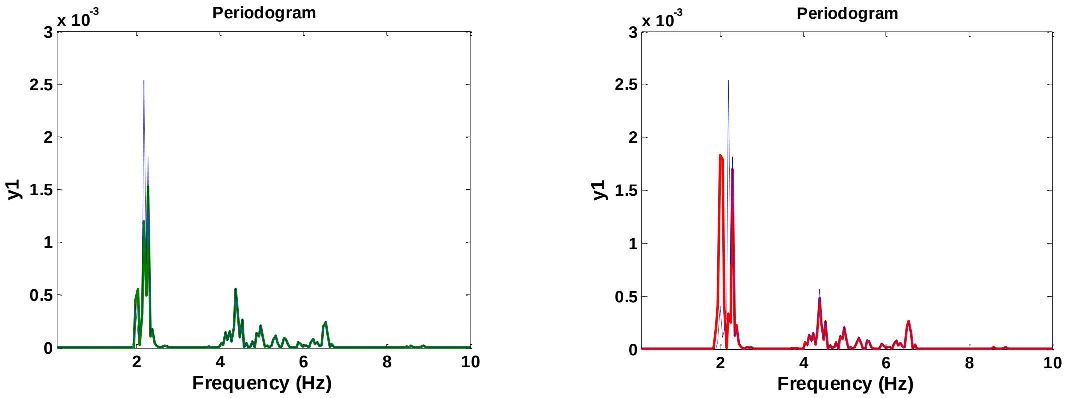
Fig. 7 Time histories of the tension at the bottom node of the long internal cable for: (a) the “damaged” case 1, (b) after an extra-elongation of 2 cm and (c) after an extra-elongation of 5 cm

The idea therefore is to act offline by properly assigning an extra-elongation of the external cable(s). Indeed, the anchorages installed when mounting the cables are still available at the site and can be easily used for this purpose. However, the easiness of practical implementation comes with the challenge of handling a highly nonlinear problem. Indeed, the initial condition monitored before decreasing the tension in the inner cables cannot be exactly restored by acting only externally. The effects of such an intervention need to be preliminarily evaluated so that the amelioration of the deck response does not result in an excessive increase of the tension in the external cable stays.



(b) (c)

Fig. 8 Time histories of the vertical acceleration component at the deck mid-span: (a) after an extra-elongation of 2 cm, and (b) after an extra-elongation of 5 cm



(b) (c)

Fig. 9 Periodograms: (a) after an extra-elongation of 2 cm, and (b) after an extra-elongation of 5 cm. The original situation is reported in dashed line for comparison

As an exemplification, the developments of this section assume that the damage scenario listed as 1 in the previous section is encountered. Starting from such a damaged initial configuration, different values of extra-elongation are assigned to the long external cable-stays, and the resulting configurations are used as initial conditions to carry out the nonlinear transient dynamic analyses on the model of section 2 subjected to pedestrians crossing.
The time histories of the tension force at the bottom node of the long internal cable in the “damaged” situation of case 1, and after an extra-elongation horizontal component of 2 cm and 5 cm are shown in Figs. 7(a)-7(c), respectively. The effects of the extra-elongations of the external cable-stays on the response at the deck mid-span are assessed by focusing on two aspects: the reduction of the peak vertical acceleration component (Fig. 8), and the spectral content of the acceleration signal at the beginning of the pedestrians crossing (Fig. 9). From these plots, it can be observed that the highest extra-elongation value of 5 cm results in a further reduction of the vertical acceleration peak at the deck mid-span, but it also leads to a spectral representation which does not match well the original one. Thus, the intervention using the lowest extra-elongation value of 2 cm seem to be preferable.

6. Conclusions

Cable-stayed footbridges are interesting in that they themselves may need monitoring and control and they can provide insights into the design, maintenance or control of larger structures. Because full-scale online monitoring systems are most likely cost prohibitive for footbridges, a virtual approach supported by periodic monitoring and model calibration is appropriate. In this paper, such an approach has been presented with reference to a wooden cable stayed footbridge in Northern Italy and the issues related to the tension in the cables, i.e., their monitoring and, if it is the case, their control are addressed. The studied structure has 16 cables and it is shown that human induced loading (HIL) can be conveniently used to detect a decrease of tension in the central cables. For this purpose, some specific time windows of the recorded response are proposed and used.

Since real time structural control does not apply to such a kind of structures for reasons of cost and exposure to vandalism, the potential of an off-line control by a simple extra-elongation of the external cables is discussed. Using such an approach may be effective but also requires studies of the associated nonlinear behaviours and effects on other bridge elements. In this paper, some insights into these aspects are provided by analysing the results from the nonlinear dynamic analyses under HIL and varying initial conditions. Also in this case, it is shown that the HIL response can be conveniently used to establish if the deck response obtained by increasing the tension in the cables is consistent with the one of an undamaged situation.

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References

Bursi, O.S., Kumar, A., Abbiati, G. and Ceravolo, R. (2014), "Identification, model updating, and validation of a steel twin deck curved cable-stayed footbridge", *Comput. - Aided Civil Infrastruct. Eng*., **29**(9), 703-722.

Cantieni, R. (2013), Cable-Stayed Footbridge: Investigation into Superstructure and Cable Dynamics. (Ed., A. Cunha), Topics in Dynamics of Bridges, Volume 3: *Proceedings of the 31st IMAC, A Conference on Structural Dynamics, 2013, Conference Proceedings of the Society for Experimental Mechanics Series 38*, DOI 10.1007/978-1-4614-6519-5_2, The Society for Experimental Mechanics Inc.

Casciati, F., Casciati, S. and Faravelli, L. (2016), "A contribution to the modelling of human induced excitation on pedestrian bridges", submitted for publication.

Casciati, F., Casciati, S., Elia, L. and Faravelli, L. (2016), "Optimal reduction from an initial sensor deployment along the deck of a cable-stayed bridge", *Smart Struct. Syst*., **17**(3), 523-539.

Casciati, S., Casciati, F., Faravelli, L. and Bortoluzzi, D. (2014), "Modelling the human induced vibrations in a cable-stayed pedestrian timber bridge", *Proceedings EWSHM2014*, Nantes, 2014.

Casciati, S., Faravelli, L. and Casciati, F. (2015), "HIL Excitation in the Damage Assessment of a Timber Footbridge", (Eds., F.K. Chang and F. Kopsaftopoulos), *Structural Health Monitoring 2015: System Reliability for verification and implementation*, **1-2**, *(IWSHM)*. Book Series: Structural Health Monitoring, 367-374.

Cho, S., Lynch, J.P., Lee, J.J. and Yun, C.B. (2010), "Development of an automated wireless tension force estimation system for cable-stayed bridges", *J. Intell. Mat. Syst. Str.*, **21**(3), 361-376.

Humar, J.L. (000), *Dynamics of structures*, Prentice Hall, Upper Saddle River, NJ.

Irvine, H.M. and Caughey, T.K. (1974), "The linear theory of free vibrations of a suspended cable", *Proc. Roy. Soc. Lon. A*. **341**, 299-315

Li, H., Zhang, F. and Jin, Y. (2014), "Real-time identification of time-varying tension in stay cables by monitoring cable transversal acceleration", *Struct. Control Health Monit.*, **21**, 1100-1111.

Li, M. and Ni, Y.Q. (2016), "Modal identifiability of a cable-stayed bridge using proper orthogonal decomposition", *Smart Struct. Syst.*, **17**(3), 413-429.

Liao, W., Ni, Y. and Zheng, G. (2012), "Tension force and structural parameter identification of bridge cables", *Adv. Struct. Eng.*, **15**(6), 983-996.

Mehrabi, A.B. and Tabatabai, H. (1998), "Unified finite difference formulation for free vibration of cables", *J. Struct. Eng. -ASCE*, **124**, 1313-1322.

MSC (2004), http://www.mscsoftware.com

Ni, Y.Q., Ko, J.M. and Zheng, G. (2002), "Dynamic analysis of large-diameter sagged cables taking in account flexural rigidity", *J. Sound Vib.*, **257**(2), 301-319.

Ren, W.X., Liu, H.L. and Chen, G. (2008), "Determination of cable tensions based on frequency differences", *Eng. Comput*., **25**(2), 172-189.

Russell, J.C. and Lardner, T.J. (1998), "Experimental determination of frequencies and tension for elastic cables", *J. Eng. Mech. - ASCE*,1**24**(10), 1067-1072.

Schiehlen, W. (2014), On the historical development of human walking dynamics, in: *The History of Theoretical, Material and Computational Mechanics – Mathematics Meets Mechanics and Engineering, Part I* – Pages 101-116, Springer, 2014, DOI 10.1007/978-3-642-39905-3_7

Solari, G. and Piccardo, G. (2001), "Probabilistic 3-D turbulence modeling for gust buffeting of structures", *Probab. Eng. Mech.*, **16**(1), 73-86.

Triantafyllou, M.S. and Grinfogel, L. (1986), "Natural frequencies and modes of inclined cables", *J. Struct. Div. - ASCE*, **112**(1), 139-148.

Venuti, F. and Bruno, L. (2009), "Crowd-structure interaction in lively footbridges under synchronous lateral excitation: A literature review", *Phys. Life Rev.*, **6**, 176-206.

Virlogeux, M. (2001), "Bridges with multiple cable-stayed spans", *Struct. Eng. Int.*, **11**, 61-82.

Yang, Y., Li, S., Nagarajaiah, S., Li, H. and Zhou, P. (2016), "Real-time output-only identification of

time-varying cable tension from accelerations via Complexity Pursuit", *J. Struct. Eng. - ASCE*, **142**(1).
Zivanovic, S., Pavic, A. and Reynolds, P. (2005), "Vibration serviceability of footbridges under human-induced excitation: a literature review", *J. Sound Vib.*, **279**, 1-74
Zui, H., Shinke, T. and Namita, Y.H. (1996), "Practical formulas for estimation of cable tension by vibration method", *J. Struct. Eng. - ASCE*, **122**(6), 651-656.