DYNAMIC IDENTIFICATION OF A TIMBER FOOTBRIDGE SUBJECT TO VIBRATION TESTS

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ABSTRACT :

Over the last thirty years timber bridges and footbridges have been found considerable use especially by their versatility, good durability and pleasing aesthetic view. First the development of new technology and materials, as the laminated timber, second the use of innovative building, assembling and connection techniques and third the expansion of efficient, certain and inexpensive products for timber protection, have allowed the employ of this type of structure. In this paper the results obtained from two forced vibration tests, carried out in 2000 and 2005, define the dynamic behaviour of a laminated timber footbridge, which has been excited by means of vibrodyne. The first test has been made in the 2000, immediately after the building launch and before its use, whereas the second test has been replicated after five years using the same test set up adopted in the first one. The aim has been to estimate, with hindsight, the dynamic characteristics of the footbridge (natural frequencies, dampings, mode shapes, etc.) and their possible variations. The free vibrations of the system, acquired during only the first test, have been analyzed afterwards to define the modal parameters through the Wavelet Transform. This kind of survey is necessary due both to the material used in the building and the particular using conditions of the footbridge: in fact, it is subjected to particular weather conditions, because it is situated near to the Adriatic Sea.

KEYWORDS: laminated timber, vibration tests, data processing, comparisons experimental results

1. INTRODUCTION

The aim of this paper is on the one hand the definition of the dynamic behaviour of an arc footbridge, constituted by laminated timber, to estimate its response to dynamic action, as wind, pedestrian action order seismic action, which can involve the structure during its use; on the other hand the investigation about an its possible damage through the comparison between its dynamic parameters calculated in two different periods of the life-time of the structure. The choice of the laminated timber as main structural material has been made to satisfy the following requirements: unfavourable weather conditions, high exposure to the sun, limited environmental impact. The durability and the resistance to weather conditions are natural characteristics of the laminated timber, those characterises are improved by several and specific treatments to which the material is subject during the manufacturing process. The laminated timber is completely immune from mildews and mushrooms through specific impregnated products and periodic maintenance and it is able to resist to hard weather conditions. The laminated timber was used with good results in very wet environments, in places near the see, so in marine ambient (as in this case), on the upper mountain and in places characterized by considerable temperature range. Furthermore, it was necessary to build the footbridge with a high span, because the river, that the structure crosses, is very dangerous. The Marecchia river is extremely irregular and is often subjected to flash flood, then the structure is bounded far from its dock. The footbridge reaches a span of 93 meters about in order to satisfy the requirement of safeguard the life of the users and to avoid that possible flood of the river can compromise the structural functionality. In this context the laminated timber was chosen first of all for its lightness and was preferred to the steel because this material resists better in extern aggressive ambient than the steel and is not as deformable as the steel and its resistance to fire is better than the remaining materials. Finally, the footbridge is part of a naturalistic ciclo-pedestrian route and constitutes the entrance on the inside of the Marecchia park and the use of the timber permits a correct integration of the structure in the context where was built and minimizes the impact assessment. The peculiarity of the structure and the uncertainty about the mechanical characteristics of the main material used, entailed the necessity to investigate about its dynamic behaviour through dynamic tests (Ventura, C. et al. (1994).

For this reason, the University of Bologna performed a dynamic test by vibrodyne on the footbridge immediately after the lunch of the structure. In this first instance the assumption about the structural dynamic behaviour, assumed at the planning phase, were compared with the real behaviour of the footbridge. In particular, once the structure modal parameters were found by the interpretation of the experimental results of the two vibration tests, a finite elements model of the structure was realized using the Sap2000 software. In the model the Young module value of the laminated timber has been set to minimize the differences between the experimental natural frequencies and those obtained with Sap2000. The choice of a greater Young module value for the laminated timber (it is usually assumed to be equal to 11 GPa and in this case is considered equal to 18 GPa) can be justified because the vibrodyna brings weak forces to the structure and so causes a low strain in the wood elements. This implies that the strains tend to vibrate around the origin of the material stress-strain law. By this assumption about the resistance of the laminated timber, the results obtained by the finite elements model and the experimental ones are very similar (see Diotallevi P.P. et all (2000)). During the first test the structure was subjected to an shock test, that will analyze in the second part of the paper. The University of Bologna carried out five years later a second dynamic test, very similar to the previous one, to study the possible variation of its dynamic behaviour in the meantime. In general, the interpretation of the response of structural system yields useful indications both for an improvement of the design quality of new structures, both to investigate on the behaviour of existing building. In this paper the dynamic identification was used for both the reasons.

2. FOOTBRIDGE DESCRIPTION

The footbridge is represented by one-span arch of about 93 m, crossing a river in the city of Rimini; more precisely the main structure consists of a twin arch of section 22x182 cm which bears the lower horizontal deck by means of vertical ties. The trampling level, made in timber, leans on two laminated timber truss of section 22x200 cm. The maximum height of the arc is about 13 meters and its radius of curvature in the vertical plane is 95 meters. The width of the footbridge is about 12 m. To contrast the horizontal loads the two arches are connected by steel cables, constituted by bars of diameter of 50 mm. Furthermore, under the trampling level a steel reticular, of which the upper and lower elements are the laminated timber trusses previously described, is located. The entire bridge is made with laminated timber, with the exception of the connectors (plates and bolts of zinc coated steel) and of the whole frame beneath the walking floor aimed to increase the lateral stiffness. The bridge is conceived for people walking and also for electrical and water transportation. If the electrical cables do not interact with the structural performance of the bridge, water pipes represent a considerable amount of the total load of the structure. Then considerable long lived loads bear down on the footbridge, this aspect was considered in the FEM modellation and in the interpretation of the behaviour of the laminated timber during its use.

Fig. 1 General overview of the footbridge

Each part of the bridge was pre-manufactured (in particular the laminated timber trusses were impregnated before its installation by specific chemicals, which protect it from the attack by atmospheric conditions), then shipped and assembled in the final place. Finally specific varnishes were put on the elements by painting to protect the timber.

3. EXPERIMENTAL SET UP

In both tests, the footbridge was subjected to horizontal and vertical forces, time dependent by a sinusoidal law, through the electro-mechanical shaker, that is available at the Laboratory of the Structural Engineering Department (DISTART) of the University of Bologna. The footbridge response, recorded as acceleration, was acquired seven piezoelectric accelerometers PCB/393B12 whose voltage sensitivity is 10 V/g and three piezoelectric accelerometers PCB/393B12 whose voltage sensitivity is 1 V/g. In the second test the instrumentation was constituted by eight piezoelectric accelerometers PCB/393B12 and two piezoelectric accelerometers PCB/393B12 . The accelerometers were connected, through a signal conditioning unit, to a computer for data processing and recording (the data acquisition and processing were obtained by the software Labview). The mechanical shaker is a device that, firmly attached to the construction, allows the application of sinusoidal time varying forces. It is constituted by masses mounted eccentrically on two disks rotating in opposite directions at the same phase and frequency of rotation fv; each disk has two eccentric masses, whose relative angle α may be changed from 46° to 180° (for each value of the angle α the maximum value of the force applied on the structure is defined) . In this manner, the magnitude of the applied force F(t) can be varied to achieve various force capabilities, up to the maximum value of 20 kN (obtained when α = 46°) . The amplitude and the phase of the sinusoidal generated force depend on some characteristic parameters of the machine; relating to the shaker ISMES of Bergamo the applied force F(t) can be written by the following relation:

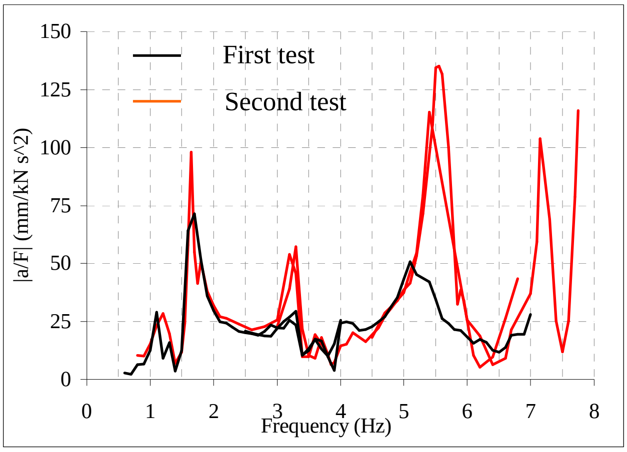
$$\begin{aligned} F(t) &= 1.026\, f_v^2 \cos\alpha/2\, \mathrm{sen}\left(2\pi f_v t + \alpha/2 - 86.9^\circ + \gamma\right) = \\ &= 1.026\, f_v^2 \cos\alpha/2\, \mathrm{sen}\left(2\pi f_v t + \varphi\right) \end{aligned} \tag{3.1}$$

where γ is the phase angle of the phasemeter signal and 86.9° is the phase displacement between the fixed mass and the position of the phasemeter. The shaker has been placed on the footbridge deck to furnish both horizontal forces, having transversal direction with respect to the footbridge axis and vertical forces. In particular, in 2005 any accelerometer was placed on the arcs, because during the test there was an high wind. In the first test the frequencies in the 0.7 – 7 Hz range were considered, whereas in the second test the 0.6 – 8 Hz frequency range was analyzed, with different values of the angle a between the two masses. The frequency step is 0.1 Hz and becomes 0.05 Hz only by the resonance in the second test. A difference between the two trials is the number of sample points (N); in the first test were sample for each acquired signal a number of point N = 256 , equal to four cycles of the force, in the 2005 test N is 1024 , corresponding to sixteen cycles. In both tests N is a power of two, then the Fast Fourier Transform can be applied in the processing of the signal. The theorem of Shannon or Nyquist is respected in the choice of the samples number for either cases.

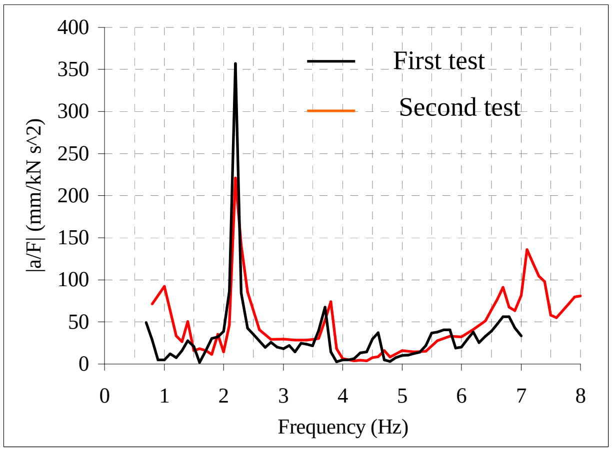
4. COMPUTATION OF MODAL PARAMETERS

The modal parameters of the studied structure (natural frequencies, modal damping and modes shape) are defined through a method in the frequencies domain. In detail, an FRF is computed in module and phase (the FRF is the inertance, because only accelerations are considered): it represents how the ratio between the structure behaviour (registered by some accelerometers) and the related force applied by the vibrodyna changes with the frequency variations (Maia, N. M. M. and Silva, J. M. M. (2007), Ewins, D. J. (1984), Inman, D.J. (2006)). If there are many different vibration modes and if the damping factor is low, then the natural frequencies of the structure are found in correspondence to the inertance peaks, where the FRF read an amplification of the signal input. Because, as already said before, the structure was exited by sinusoidal forces, for each exciting frequency fv, one point of the FRF is, more in detail, the ratio between the amplitude of the acceleration harmonic part a(t) and the amplitude of the force F(t) described in the Eqn. 3.1. The computation of the sinusoidal force amplitude and phase is easy using the Eqn. 3.1, while to define the a(t) amplitude and phase its Fourier transform is computed. Because in both the dynamic tests the number of sample points is raised to the power of two, for computing the discrete Fourier transform the Fast Fourier Transform algorithm (FFT) is applied, that allows a great time-saving because it significantly reduces the number of the computations (for N= 1024 a computation that is 102 times faster than the standard one can be

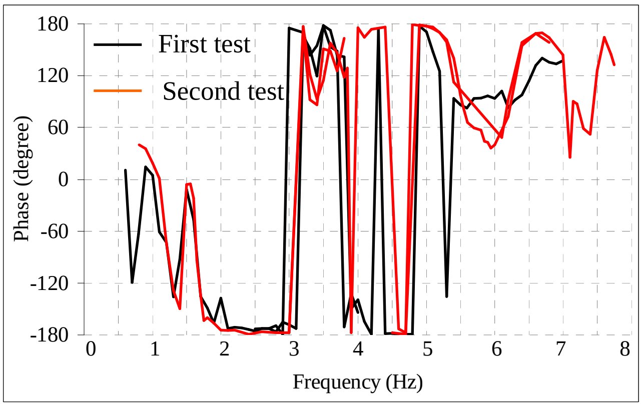
obtained). Therefore, the inertance is computed in module as the ratio between the fast Fourier transform of the accelerometer registration a(t) and the force amplitude, both considered at the same exciting frequency fv.
Because the FFT is a complex function (defined by both a real and a imaginary part or, instead, by a module and a phase), the FRF are represented both in module and in phase; the FRF phases diagram represents how the angle phase displacement between the structural behaviour and the applied force varies with the frequency variation, or, in other words, it represents the difference between the phase of the FFT of the acceleration and the force phase. Below the inertance trend is shown; it is represented in module by the ratio a/F and the phase φ[a/F], computed for the accelerometer 4 with the vibrodyna set in horizontal position, both for the 2000 test (in black) and for the 2005 (in red) one, and the accelerometer 3 with the shaker in vertical position.



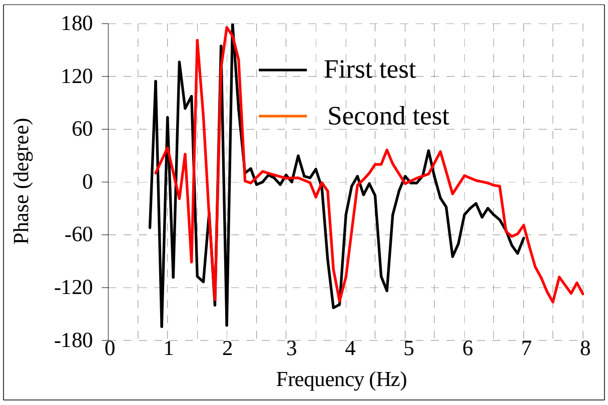
(a) Module FRF



(b) Module FRF



(a) Phase FRF



(b) Phase FRF

Fig. 2 Comparisons between inertance diagraphs for the two tests when the shaker is horizontal (a) and vertical (b)

Therefore, the inertance has been computed in module and in phase for each accelerometer and for both the vibrodyna positions and in correspondence to the related peaks the natural frequencies of the structure have been identified. The results of the two dynamic tests are summarized in table 4.1.

Table 4.1 Natural frequency for the two tests

Vibration mode	Frequency test 2000 (Hz)	Frequency test 2005 (Hz)
1	1.3	1.2
2	1.4	1.4
3	1.7	1.7
4	2.2	2.2
5	3.7	3.8

The computation of the structural damping with sufficient precision is certainly the most difficult aspect of the identification process. For the definition of the modal damping the half-power method has been used at the beginning; this method yields good results only if the vibration modes are distinct, that happens if the structural system is little damped. The half-power method is applied in the neighbourhood of the peak of the FRF, where the frequencies correspond to a value of the FRF module equal to $|a/F|/\sqrt{2}$ (Diotallevi et al. (1999, 2008)). The modal damping is defined by the Eqn. 4.1, where ω_a and ω_b are the extremis of the considered interval (i.e. the frequencies immediately after and before the resonance).

$$\xi = \frac{(\omega_a - \omega_b)}{2\omega_r} \tag{4.1}$$

The extracted results are reported in table 4.2. The assessment of the modal damping depends strongly on the precision by which the maximum value of the module of the FRF is computed. The precision of the peak of the inertance depends on the quality of the recording and on the frequency step used for the acquisitions. The method can be applied successfully only if the damping factor is not too much low. If the structure has a very low damping factor, the peak of the FRF will be very pointed and the percentage error in the definition of maximum value of the inertance increases. Another limit of this method is that its application in real case for a generic MDOF system entails strong errors of approximation, which are often not negligible. So the results obtained through this methodology are probably characterized by the presence of an error; however, neglecting the precision of numerical value of the modal damping, the comparison between the results obtained in the first and in the second test shows a constant increase of the modal parameter, that is an indicator of a possible damage of the footbridge. To obtain a more reliable value of the damping, the Wavelet transforms are applied afterwards for the free vibration of the system. This method is applied only in the first test, because in the second one only the forced vibration test was carried out on the system.

Table 4.2 Modal damping relating to the two tests

Vibration mode	Damping (test 2000)	Damping (test 2005)
1	-	-
2	0.0416	0.0484
3	0.0315	0.0381
4	0.0175	0.0243
5	0.0177	0.0187

5. EXTRATION OF THE MODAL PARAMETERS THROUGH THE FREE VIBRATION OF THE STRUCTURE

To verify the validity of the modal parameters obtained by the analysis of the acquired signals, the free vibrations of the system were considered afterward. The footbridge was subjected to a shock test only in first instance (2000 test) and the response of the structure was recorded through the same accelerometers previously described. An example of the recorded signal is reported in the figure 3, where accelerometer 1 is considered. In particular, the acquired signal of the free vibrations is described through 16384 sampled points and with a sampling step equal to 1000 points for second, this entails that the frequency step of the Fourier Transform of the same signal is equal to 1000/16384 = 0.06 Hz. At the beginning the Fourier Transform is computed for each signal acquired and the natural frequencies of the system are defined. The interpretation of the first natural frequency of the footbridge is very difficult because two very proximal peaks of the FRF (they are equal to 1.1 Hz and 1.3 Hz) are obtained. The Fourier Transform of the free vibrations clarifies the doubt about this circumstance and says that the first natural frequency of the system is 1.3 Hz.

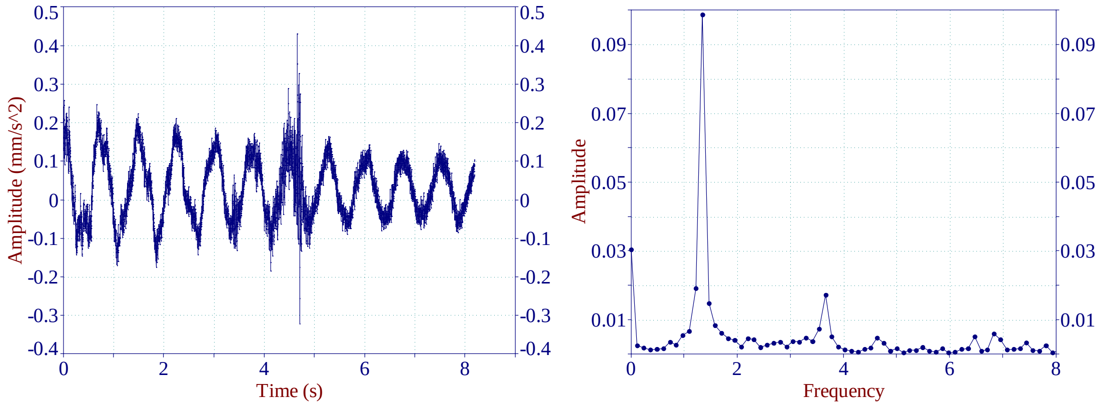


Figure 3: Signal acquired during the shock test and its Fourier Transform

The computation of the FFT gives good information about the frequencies of the analyzed signal but is not able to give any information about its behavior in the time-domain. Then the use of the FFT, when the signal is not stationary, does not describe completely the behavior of the system. For this reason, note the natural frequencies of the system through the classical FFT, for each recorded signal the Wavelet transform, that is able to furnish information both in frequency and time domain, is computed (Chui, C. K. (1992), Ruzzene, M et al. (1997)). In general, for a generic signal x(t), the Wavelet transform is defined by the following integral:

$$CWT_x^{\psi}(\tau,s)=\frac{1}{\sqrt{|s|}}\int_{-\infty}^{+\infty} x(t)*\overline{\psi}\left(\frac{t-\tau}{s}\right)dt \tag{5.1}$$

where the function $\Psi_{\tau,s}=\frac{1}{\sqrt{|s|}}\overline{\psi}\left(\frac{t-\tau}{s}\right)$ is a scaled (s) and translate (τ) version of the mother function Morlet, defined by the following relation $\psi(t)=e^{i\omega_0 t}e^{-t^2/2}$. The computation of the CWT is done at the beginning in an ideal case, where the signal is defined by the expression

$$\mathrm{x}(\mathrm{t})=\mathrm{A}e^{-\xi 2\pi f_v t}\,\mathrm{sen}\left(2\pi\sqrt{1-\xi^2}\,f_v t\right) \tag{5.2}$$

to extract the natural frequency of the signal and its modal damping using only one Wavelet transform. The trend of the CWT is described in the figure 4. If the diagram of the CWT is sectioned at a value of the time constant, the natural frequencies of the signal could be obtained in correspondence of the peaks of the CWT.

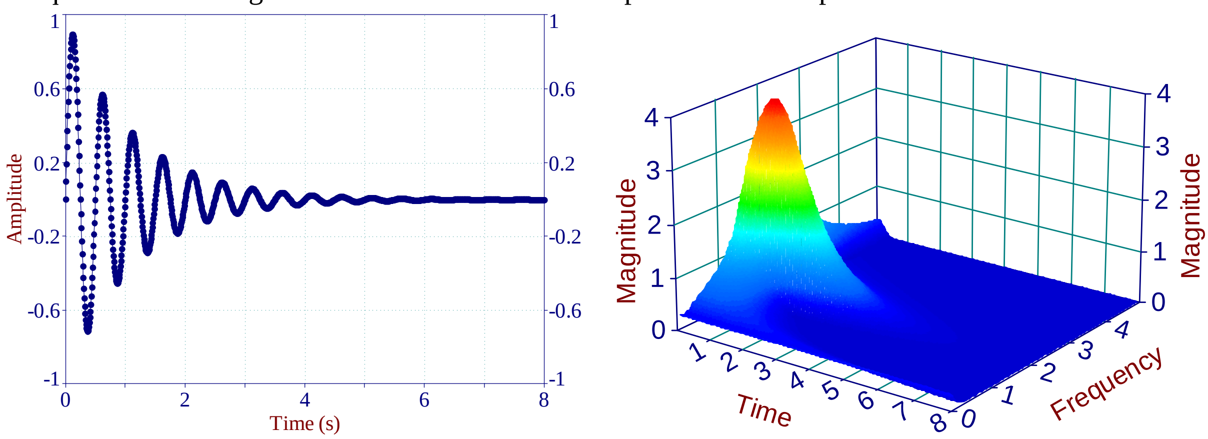
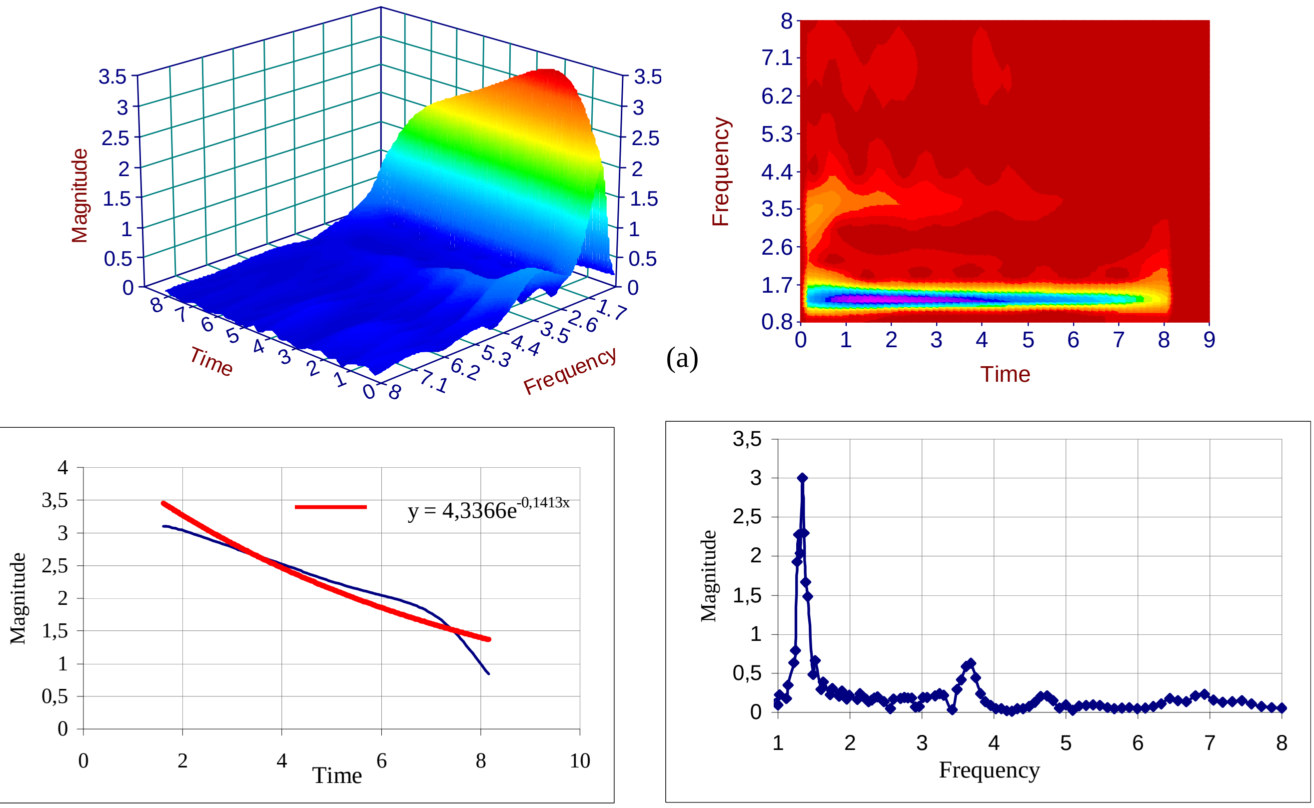


Figure 4: Simulated signal with natural frequency 2 Hz and modal damping 7.2 % and its Wavelet Transform

When the natural frequency of the simulated signal is known, the same diagram is sectioned at constant frequency equal to its resonance frequency. It is possible define how the amplitude of the signal damps as the time changes. The exponential function, that better represented this trend, obtained through the minimum square method, permits to define the damping factor of the system. The same methodology is applied in the real case; in the following figures the trend of the CWT is defined for the signal acquired by the accelerometers 1. The section of the diagram at constant time (see figure 5b) allows the individuation of the natural frequencies of the footbridge. For each value of the resonance frequencies the CWT is sectioned and the variation of the amplitude of the signal in the time is obtained (see figure 5c). By the exponent of the exponential function that better describes the trend of the variation of the magnitude of the CWT, it is possible define the damping factor for each vibration mode of the footbridge.



(b) (c)

Figure 5: Wavelet Transform of the accelerometer 1 (a) trend of the Magnitude with time (b) and frequency (c)

The obtained results are summarized in the following table and compare with the value previously obtained through the forced vibration test. The frequencies are in good agreement between the two methods, instead the modal dampings are very different.

Table 5.1 Comparison between the modal parameters obtained by the two methods

Frequency (FRF)	Frequency (Wavelet)	Damping (Half power)	Damping (Wavelet)
1.3	1.34	-	0.0165
1.4	1.52	0.0416	0.0125
1.7	1.75	0.0315	0.0087
2.2	2.18	0.0175	0.0085
3.7	3.69	0.0177	0.0106

6. CONCLUSION

In situ dynamic tests have been conducted in order to estimate the dynamic behaviour of a timber footbridge. The lowest frequencies have been evaluated to be in the range between 1 and 2Hz including both flessional and torsional mode shapes. It was thus argued that the footbridge is sensitive to low frequency loads, such as human pacing, mainly due to the material choice compared to the bridge shape and dimensions. This can entail low comfort level for the users, because the footbridge can resonate during pedestrian crossing. A light decrease of the first natural frequency of the footbridge was obtained in the first test compared to the second. This entails a light decrease of the rigidity of the structure, which is attributed to a possible damage of the footbridge, as the modal damping variation confirms. A possible interpretation of this variation can be the effect of the duration loads to the resistance. The long lived loads entail the reduction of the laminated timber resistance (see Giordano G. (1993)). Moreover the second test was carried out on the footbridge only five years after, then a little variation of the modal parameters of the structure are presumable. It can be interesting repeat again the test after a longer time. The natural frequencies and the modal dampings of the system are estimated through the free oscillations of the footbridge, extracted by a shock test realized only in 2000, during its launch. The modal parameters are calculated through the Wavelet Transform, applied for each signal acquired by the accelerometers. The frequencies of the system are almost the same, whereas the modal dampings are very different. This because the half-power method can be applied successfully only if the vibration modes are distinct. This hypothesis entail that the each mode cannot be influenced considerable from the another ones. The peaks of the inertance are very proximal between each other as it is possible to see in figure 2. The diagram shows that the contribution of the modes, which are immediately before and after the considered mode, cannot be neglect (see for example the first and the second natural frequencies of the system in figure 2 black line). The obtained results show that the computation of the damping factor of a generic system with sufficient preciseness is very difficult and is the most complicated aspect of the process of identification.

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