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Dynamic Identification of a Pedestrian Bridge using Operational Modal Analysis

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Abstract

The Postiguet footbridge in Alicante is a very sensitive structure to vertical vibrations and it has been restored with fibre glass composite materials to improve the response to dynamic loads. This paper describes the studies carried out on the walkway in the conditions prior to and after the intervention of structural restoration. This allowed the identification of the characteristic parameters of the dynamic behaviour of the footbridge. The values of the natural frequencies of vibration and the damping coefficients of the structure have been determined for the different modes of vibration in the two structural conditions. Operational modal analysis (OMA) techniques for the modal identification have been utilised and a numerical model has been built to clearly identify the vibration modes and to compare the results.

Keywords: dynamic identification, footbridge, structural reinforcement, resonance, operational modal analysis.

1 Introduction

In this paper a dynamic study has been applied to Postiguet foorbridge in the town of Alicante, Spain. It is placed in front of the Playa Postiguet (Figure 1).

Postiguet footbridge was designed by Prof. R. Irles Más, of the University of Alicante [1] with the principal aim to redevelop 1 Km of the waterfront of the of the coastal landscape of Postiguet, including breakwaters to protect the Alicante coast, and the creation of cycling routes along the "Paseo del Gomiz" both from the sea to the foot of Mount Benacantil.

The pedestrian way is almost parallel to the urban part of N-340 highway also known as "Carretera del Mediterraneo". Following the geometric remodeling the bus lanes were also reorganized. Through a flight of stairs it was possible to connect the

"Calle de la Virgen del Socorro" (14 m s.l.m.) to the pedestrian shopping street (at 2 m above the sea level).

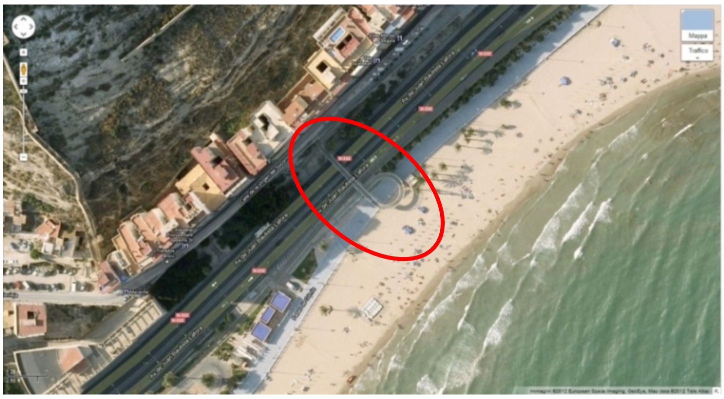

Figure 1: Postiguet footbridge. Postiguet beach, Alicante, Spain.

Subsequently, a restructuring of the walkway was proceeded with the aim to improve its look, which after a few years after its construction (1993) began to show a widespread degradation (Figure 2) due to the continuous exposure to chlorides of the marine environment [2].

Figure 2: Degradation phenomena due to corrosion from chloride.

A previous study of the walkway determined the frequencies of the structure under an excitation produced by the walking of the pedestrians [2] where above all at low frequencies of excitation the structural frequencies were higher than those recommended by the Spanish code.

A similar study was conducted on a historical building in Bari, Italy [3]. By mean of the operational modal analysis (OMA) the values of the natural frequencies of vibration and the vibration modes of the structure have been determined; in this case it was put in evidence the difficulties to modeling a building in cyclopic concrete.

2 Description of the geometry

Postiguet footbridge is characterized by having a deck consisting of a hollow metallic section slabs at its ends, for a total width of the platform equal to 2.20 m. The latter is equipped with longitudinal and transverse stiffeners of the wings of A42b steel and a thickness respectively equal to 10 mm and 8 mm. In the supporting area there are 20 mm thick plates (Figure 3).

The structure is divided into 6 segments of 8.5+24+13+19+15+12 m and it is fixed on the ends, with hindered rotation around the vertical axis, assuming the static scheme of a continuous beam on support points. The cross section of the walkway has a constant height of 0.5 m, consequently leading to a considerable slenderness ratio of the structure ((height/span = 1/48).

On the other hand, the path has two perpendicular straight alignments, connected by a helix of 270 degrees, with a smooth and continuous variation of the pendant. The straight sections and the helix were connected by two parabolic sections with a uniform pendent of 5.45%. Finally, the ramp on the end next to the beach has a maximum pendant of 10%.

The intermediate supports are composed of five simple supports, in-axis to the deck, with a hollow circular metal profile in A52 steel, joined to the body via metallic supports of confined neoprene. At the union of the deck with the side of the mountain Benacantil (Figure 1) the foundation has a joint made by means of 8 DYWIDAG ∅ 32 bars of 85/105 steel and a length of 12.5 m. Similarly, in the area of the shoulder next to the beach and in correspondence with the first support, the joint is realized through an anchor with GEWI ∅ 32 bars of AEH500N steel (Figures 3 and 4).

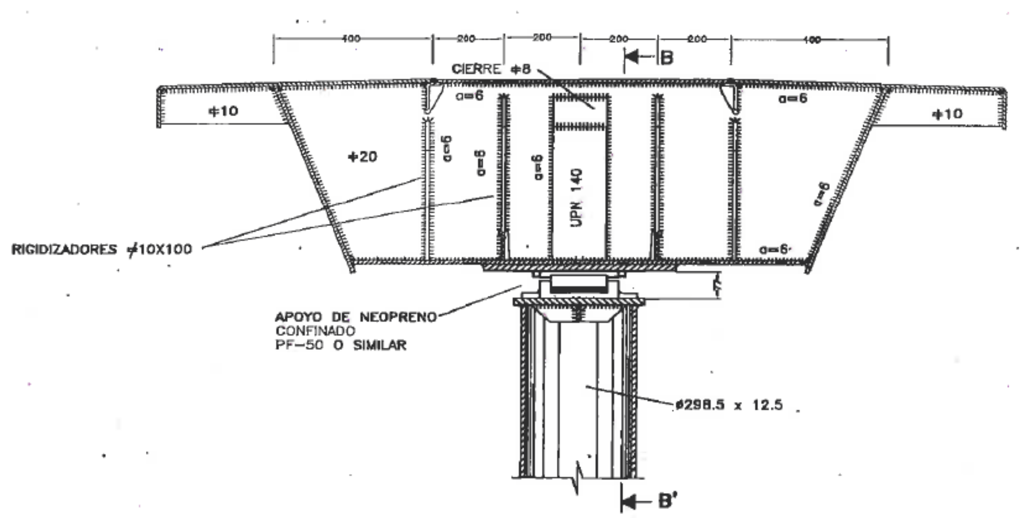

Figure 3: Transversal section of the deck.

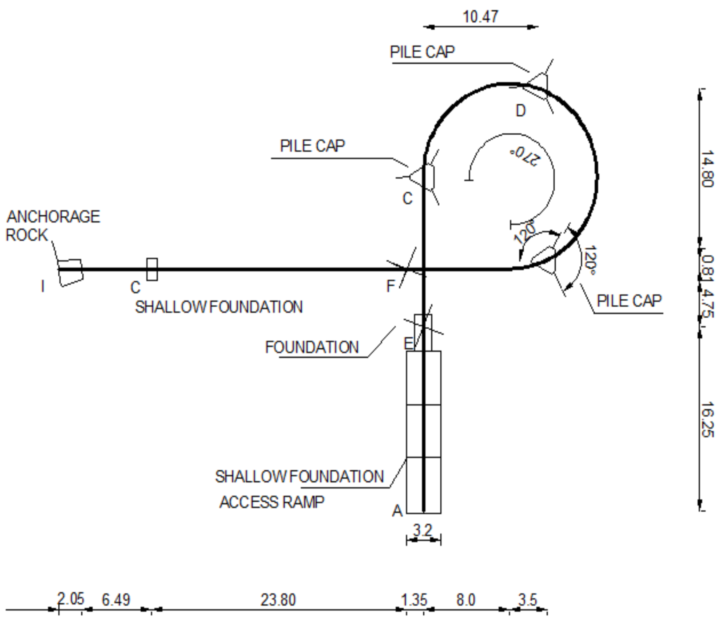

Figure 4: Foundation plan

The other supports have an isolated foundation that consists of a plinth on poles. The latter are, in all cases, constituted by three inclined rods, with a slope 1H/3V, distributed at 120 º in plan, and embedded in the sand up to the rock, with the exception of the foundation closest to the side of Mount Benacantil, which has a superficial foundation in direct contact with the rock.

The restructuring of the walkway is starting back a few years after its construction as a result of the degradation due to a constant exposure to the marine environment. In the restructuring thermosetting (laminated fiberglass with resin) and thermoplastic materials were used. The original steel frame has been kept: the structure was coated with a new material, which has undergone an anti-graffiti treatment and a polishing process able to make the maintenance of the walkway easier. The material has a high mechanical strength, a high corrosion resistance and a low weight (±245 kg/m), which has facilitated the installation of the footbridge [4].

3 Experimental Analysis

3.1 Placement of the sensors

To identify the natural frequencies of the structure the footbridge was closed to pedestrians both in November 2010 for the measurement prior to the restructuring and in July 2011 for the measurement of the natural vibration after the structural restoring.

The first measurements were made in November 2010 with no significant effects due to the wind action that could change the dynamic response of the structure, avoiding the effects of the traffic. For this reason it was chosen to perform the test at night. For the test 11 seismic accelerometers (Table 1) Model 393A03 PCB Piezotronics were used, disposed along the entire structure as shown in Figure 5. PCB signal conditioners and Kynowa Model PCD - 320 data acquisition tools with 8-channel acquisition were also used. Eleven acquisition points (1-11) with a recording time of 30 minutes and a data acquisition frequency of 50 Hz have been utilized.

1	2	3	4	5	6	7	8	9	10	11
V	V	V	V	V	T	L	V	V	V	V

Table 1: Direction of the accelerations recorded in the different acquisition points.

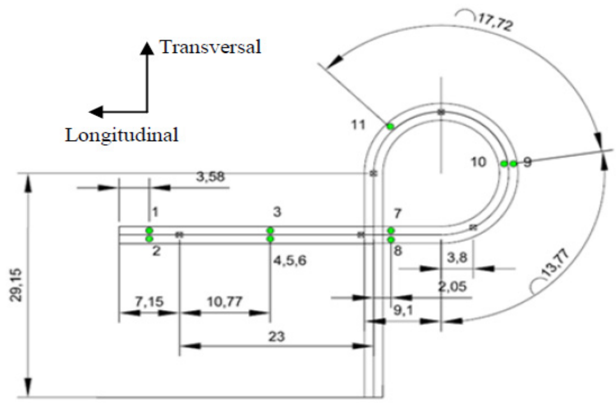

Figure 5: Positioning of the accelerometers.

The reference sensors are those positioned at 4, 5, 6, 7 and 8 as shown in Figure 6a.

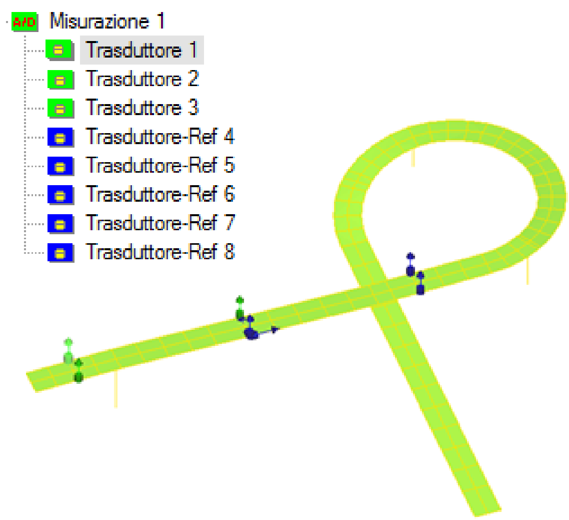

Figure 6a: Location of the reference sensors (in blue) and the variable sensors (in green) before the structural reinforcement.

Regarding the tests performed on the 1st of July 2011 accelerometers 4, 5, 6 and 7 are reference accelerometers (Figure 6b).

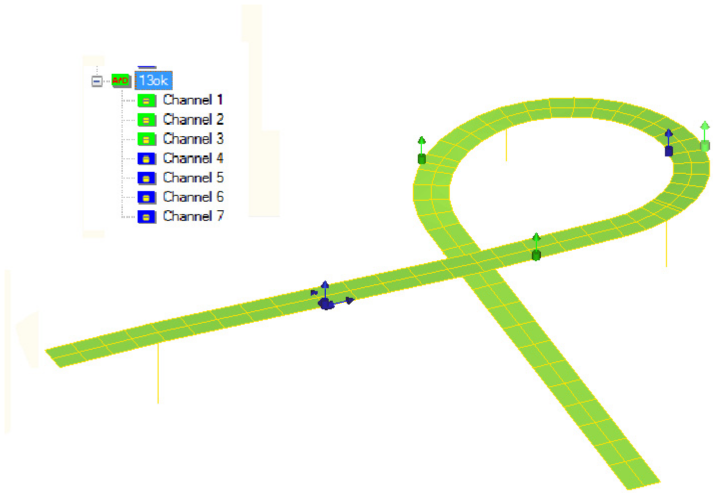

Figure 6b: Location of the reference sensors (in blue) and the variable sensors (in green) after the structural reinforcement.

3.2 Numerical Model

The model was done with Testors ARTEMIS that is a very effective tool to merge the data measured, the geometry information and the data sets acquired by accelerometers recording. The Testor is used to:

- Create the geometric model;
- Organize the measurements and subsequent validation of the same regardless of the hardware used;
- Test planning and allocation of DOF;
- Export the model in ARTEMIS Extractor.

For the actual construction of the model the footbridge was discretized by 61 shell elements while for the columns (pylons) very slender elements were used (Figure 7).

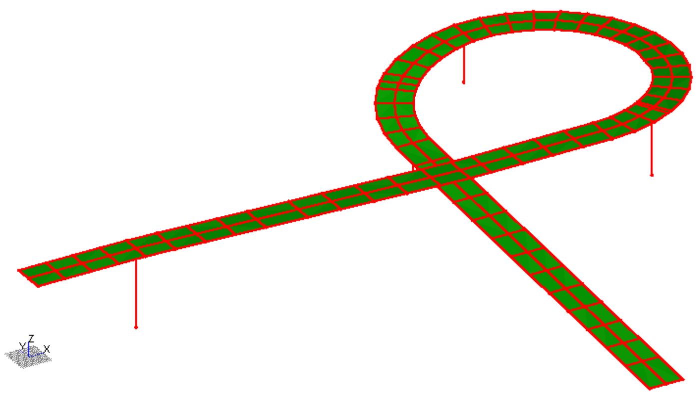

Figure 7: FE model of the walkway in ARTeMIS Testor (2008)

3.3 Operational Modal Analysis (OMA)

3.3.1 Techniques utilized

ARTEMIS software works by identification techniques such as "output-only" [6]. This technique allows to determine the modal parameters by acquiring the experimental data. The modal parameters are typically the mode shapes, natural frequencies and damping ratios (Figure 8).

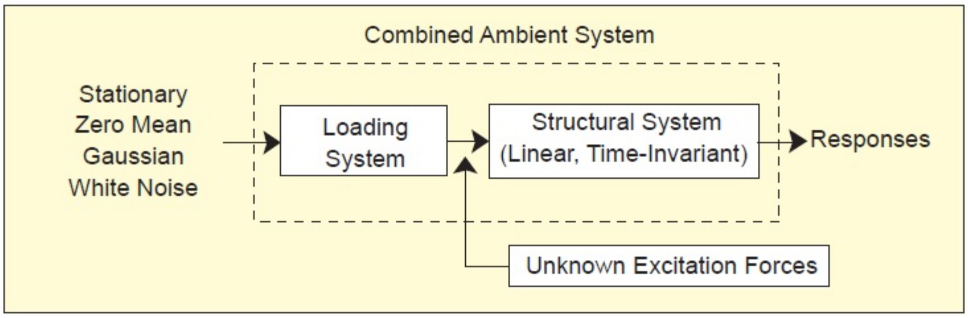

Figure 8: Scheme of virtual load for output-only systems.

The formula behind this approach is an extension of the classic basic frequency domain (BFD) or peak picking frequency (PPF) [7]. The classical theory is based on the use of the discrete Fourier transform (DFT) considering the vibration modes separate, independent and estimated directly from the matrix of spectral density. This classic approach is particular difficult in case that the vibration modes are coupled.

Frequency domain decomposition (FDD) technique removes these disadvantages [8][9]. It makes the singular value decomposition (SCD) of the spectral matrix; this matrix is decoupled in a set of autofunctions of spectral density, each corresponding to a one degree of freedom. It presents no errors when the load is white noise, the structure is lightly damped and the modal shapes of the coupled modes are orthogonal.

The curve-fitting frequency domain decomposition (CFDD) [10] [11] is a new alternative approaching technique that utilizes a curve-fitting of the whole power spectrum S(f) directly in the frequency domain. The principal advantage is a more accurate estimation of the natural frequencies and the damping coefficients both in presence of excitations and of deterministic type. The mode shapes are determined like with the original technique EFDD [8]. With this technique the entire power spectrum S(f) is transformed in a half power spectrum P(f), where P(f) is obtained from S(f) calculating the respective correlation functions obtained by mean of the inverse Fourier transform. The negative values of the correlation functions are assumed equal to zero, while P(f) is obtained from Fourier, transforming again the results in the frequency domain. The modal shapes are obtained as for the original EFDD technique.

The problem of the dynamic identification has been dealt with in absence of a known force by mean of the stochastic subaspace identification (SSI) technique [12] that consists of the modal identification with subspace methods based on the manipulation of matrices using linear algebra operations. In this context, the elements belonging to a row of each matrix can be considered as the coordinates of vectors of a j-dimensional space. The rows of the matrices define, therefore, a base for each linear vector belonging to this vectorial space. Identification with subspaces utilizes in an extensive way the use of matrices that join the matrices of the system.

The principal component (PC) algorithm is a simplification technique of the data utilized in the field of multivariate statistics. An orthogonal transformation is utilized to convert a series of observations connected between them in a set of independent variables that take the name of principal variables. This transformation is made in the way that the first principal component has a higher value of variance.

The canonical variate analysis (CVA) is also a method of linear regression for the analysis of data. This type of analysis quantifies a relation between a variable of expectation and a normalized one for the determination of the correlation coefficient r, the coefficient r^2 and the standard regression coefficient β.

The algorithm unweighted principal components (UPC) is simpler than those proposed in the stochastic field of work and considers both matrices of unit weight and furnishes more approximated data respect to the two techniques mentioned above.

3.3.2 Identification of the natural frequencies of vibration and the damping coefficients

To determine the natural frequencies of Postiguet footbridge before the structural restoring the access to pedestrian has been closed during the night of 25 November 2010. In this way it has been avoided that the measurements would be influenced by the transit of pedestrians, by the vibrations produced by the vehicular flow of the highway below that is greater in the daytime and by particular stresses caused by wind. Dynamic analysis was conducted considering two data sets of measurements that have led to the automatic individuation of 3 vertical modes of vibration with natural frequencies, always under the free vibrations, between 3.65 Hz and 7.68 Hz, as shown in Table 2. The vibration modes have damping coefficients (DR) of about 1-2%. Only the results obtained from the SSI techniques were considered.

Mode	Modal identifications										
	FDD	EFDD		CFDD		PC		UPC		CVA	
	f(Hz)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)
1	3.65	3.65	0.35	3.65	0.25	2.87	1.79	3.76	1.86	3.65	1.43
2	4.66	4.65	0.42	4.65	0.35	3.65	1.42	4.65	1.80	4.67	1.95
3	7.68	7.68	0.27	7.68	0.23	7.71	2.43	7.42	1.09	7.35	1.24

Table 2: Characteristics of the vertical vibration modes of the structure. Pre-reinforcing.

The identification of the natural frequencies of the structure after the structural retrofitting were conducted in a regime of free vibrations in July 2011. Four stable vertical modes of vibration have been identified by the software ARTEMIS Extractor [5] with frequencies between 3.064 Hz and 6.458 Hz (see Table 3). The damping coefficients present values between 1 and 1.5%.

Mode	Modal identifications										
	FDD	EFDD		CFDD		UPC		PC		CVA	
	f(Hz)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)	f(Hz)	DR(%)
1	3.06	3.06	0.95	3.06	0.51	3.10	0.91	3.09	1.50	3.09	0.25
2	3.50	3.50	1.07	3.50	0.57	3.58	2.02	3.57	3.13	3.58	2.67
3	4.10	4.11	1.14	4.11	0.63	4.11	1.03	4.11	0.91	4.10	0.83
4	6.46	6.46	0.43	6.45	0.20	6.52	1.58	6.53	1.58	6.50	1.38

Table 3: Characteristics of the vertical vibration modes of the structure. Post-reinforcing

The data obtained from the operational modal analysis (OMA) in the two cases studied, before and after the structural reinforcement, show a substantial change in the values of the modal characteristics. Considering the first three vertical modes of vibration of the structure it is possible to notice the differences described in the following. The main parameters of the vibration modes before restructuring are shown in Table 4.

Mode	Frequency f (Hz)	DR(%)
1	3.66	1.7
2	4.65	1.13
3	7.7	0.75

Table 4: Characteristics of the vibration modes before the structural reinforcement.

The vibration modes listed above show a bending type modal shape regarding to vertical vibration modes (Figure 9a-b-c).

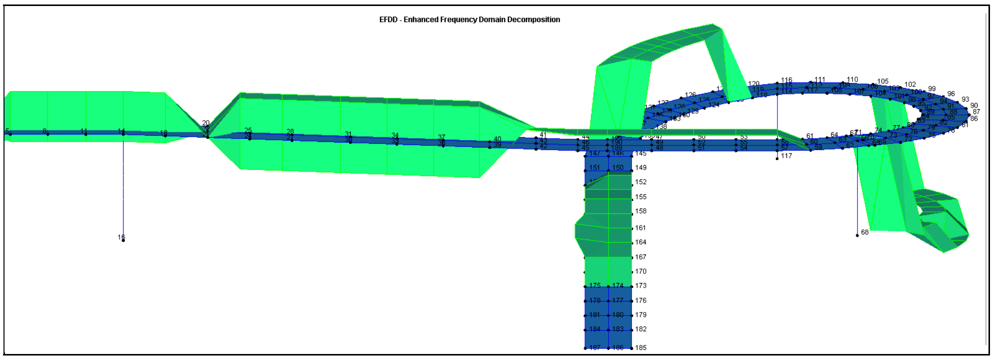

Figure 9a: Shape of the 1st vibration mode before the restoration.

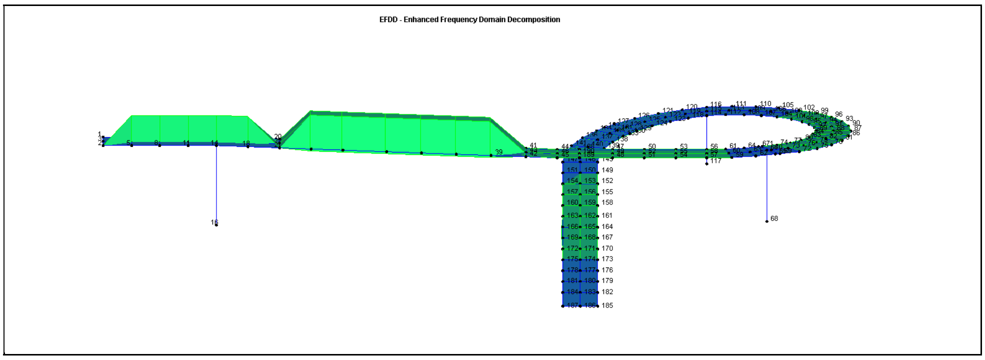

Figure 9b: Shape of the 2nd vibration mode before the restoration.

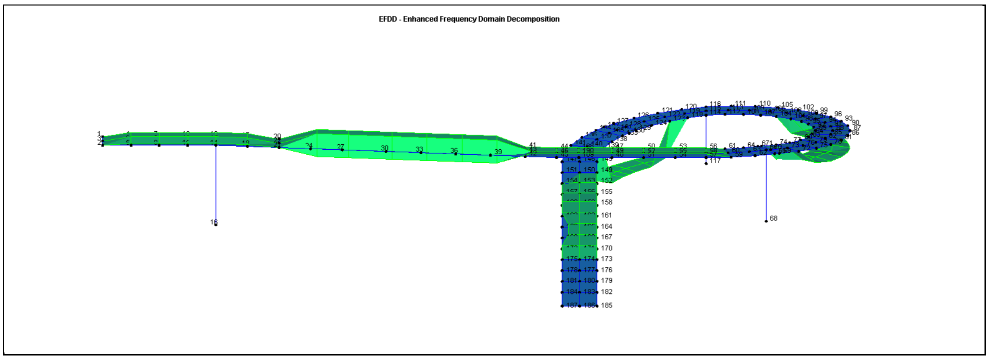

Figure 9c: Shape of the 3rd vibration mode before the restoration.

From the test after the structural restoration it has been found values of frequencies lower than the previous case and higher damping coefficients (Table 5). The shapes of the vibration modes are represented in Figures 10a-b-c.d; in this case the first two vibration modes are flexio-torsional one while the third and fourth modes are flexural ones.

Mode	Frequency f (Hz)	DR(%)
1	3.06	1.92
2	3.58	2.53
3	4.1	1.28
4	7.09	1.02

Table 5: Characteristics of the vibration modes after the structural reinforcement.

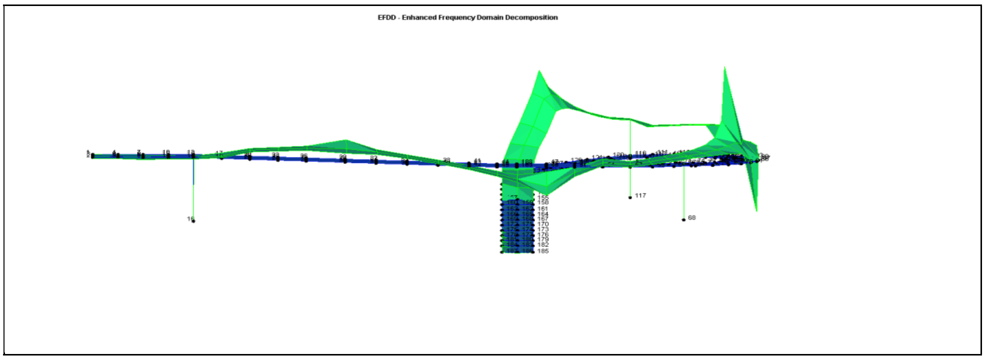

Figure 10a: Shape of the 1st vibration mode after the restoration.

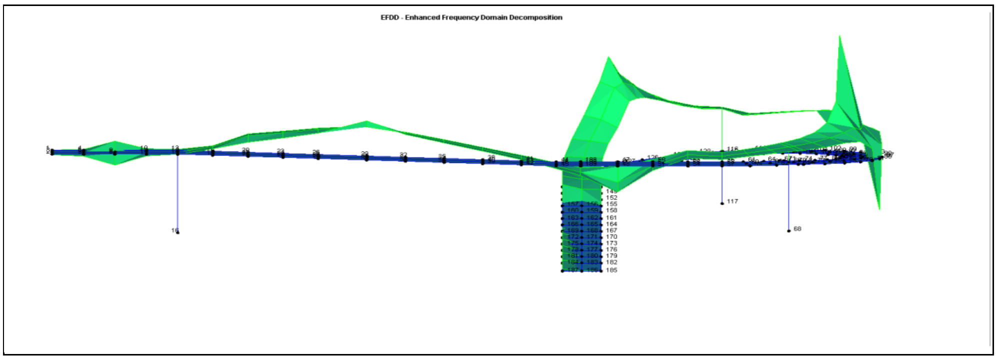

Figure 10b: Shape of the 2nd vibration mode after restoration.

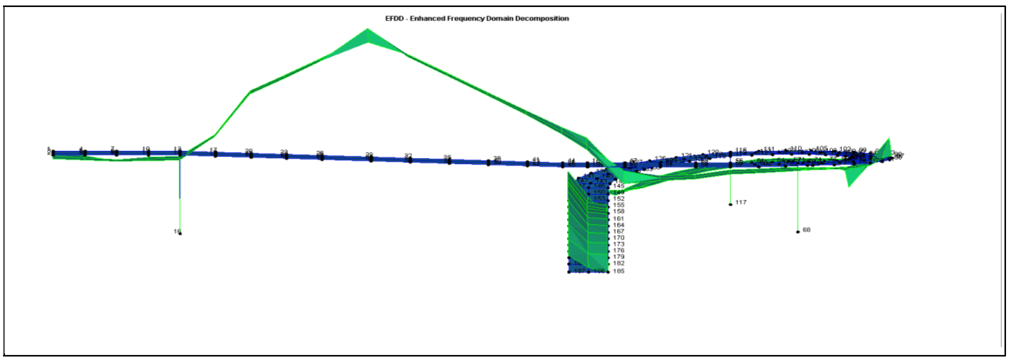

Figure 10c: Shape of the 3rd vibration mode after the restoration.

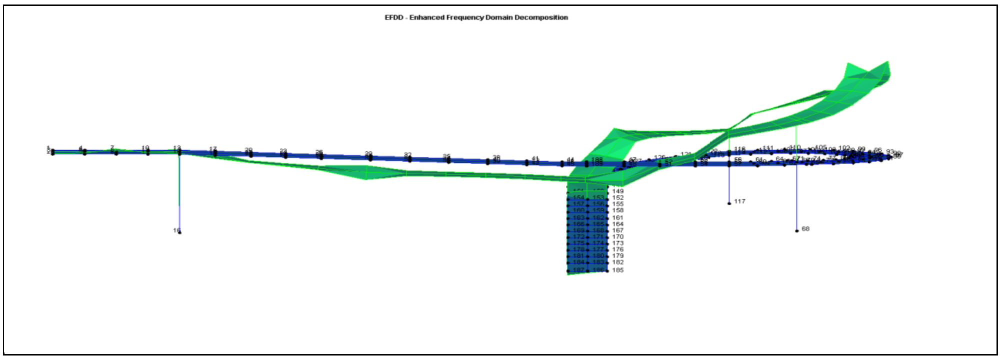

Figure 10d: Shape of the 4^{th} vibration mode after the restoration.

3.3.3 Modal Assurance Criterion (MAC)

The validation of the results to verify the reliability of a structural finite element model can be obtained by comparing experimental or dynamic testing. Dynamic modal tests, for example, give the possibility to identify the frequencies and the natural modes which are the fundamental parameters for this analysis; the differences between the numerical model and the experimental data allow therefore an evaluation of the quality of the data obtained with the FEM model and also the experimental set-up. The techniques for quantifying the comparison between the mode shapes are useful in the numerical-experimental correlation, but in general to compare any set of modal shapes with another. The most used index for this type of analysis is modal assurance criterion (MAC) [13]:

$$MAC\ (\psi_{test}, \psi_{FE}) = \frac{|\{\psi_{test}\}^{*T}\{\psi_{FE}\}|^2}{(\{\psi_{test}\}^{*T}\{\psi_{test}\})(\{\psi_{FE}\}^{*T}\{\psi_{FE}\})} \quad (1)$$

where:

- $\{\psi_{test}\}$ is the vector, which represents an experimental vibration mode;
- $\{\psi_{FE}\}$ is the vector, which represents a numerical vibration mode.

MAC is therefore defined as a scalar constant included in the range [0,1] and it expresses the degree of correlation between two modal vectors that should be compared.

It is expected that a numerical vector and an experimental vector, which describe the same vibration mode of the structure, show a value of MAC equal to 1, while if they refer to two different modes they show a value close to 0. It is then important to consider that each degree of freedom (d.o,f.) of each mode contributes to MAC:

- big on phase displacements of a d.o.f. between two correlated nodes give a positive contribution;
- big out-of-phase displacements give instead a negative contribution;
- small displacements give less important contributions.

Considering for example a set of *n* experimental modes and a set of *m* numerical modes it is possible therefore to build an *n x m* matrix which clearly indicates the superposition degree of each numerical mode with each experimental mode.

Examining the analysis of MAC before restoration of the footbridge results are reported of the comparisons obtained from MAC considering the different techniques utilized in the OMA. Comparing the values of the pre-restoration frequencies of the vibration modes among the FDD techniques (Table 6) it is possible to note the presence of anomalies (not perfect diagonality of the matrices) that is due to the scarcity of experimental data and to the presence of white noise inside the starting data-set. It is possible to compare the vibration modes assumed by the model in the different vibration modes with the different techniques of modal identification.

Regarding the modes relative to Subspace Stochastic Identification the experimental results of the techniques unweighted principal component and principal component are compared. It is possible to note the presence of more complete and uniform results respect to the FFD techniques because there is the presence of values close to unity along the diagonal. This means that the modes find out with the two techniques considered are compatible and therefore reliable (Table 7), except than for EFDD Mode 5 and the CFDD Mode 2, respectively, of frequency equal to 10.97 Hz and to 3.75 Hz. These modes will not be taken in consideration because they are not compatible with the other detected modes.

Frequency (Hz)	3.65	4.21	4.65	5.38	7.24	7.68
1.63	0.33	0.18	0.03	0.88	0.48	0.66
2.11	0.25	0.13	0.06	0.66	0.43	0.61
2.72	0.63	0.29	0.12	0.66	0.26	0.33
3.65	0.84	0.21	0.14	0.13	0	0.01
3.84	0.28	0.12	0.05	0.95	0.5	0.74
4.21	0.21	1	0.01	0.08	0.23	0
4.65	0.02	0.1	0.32	0	0	0
5.38	0.13	0.08	0.04	1	0.61	0.84
6.01	0.17	1	0.04	0.89	0.57	0.79
7.24	0	0.1	0.02	0.76	0.91	0.77
7.68	0.01	0.01	0	0.8	0.58	0.92

Table 6: MAC comparison between EFDD – CFDD techniques pre-restoration.

Frequency (Hz)	3.65	3.75	4.66	7.3	7.7
3.65	1	0.18	0.05	0.12	0
4.66	0.04	0.03	1	0	0
7.29	0.12	0.25	0	0.98	0.22
7.69	0	0.01	0	0.14	1

Table 7: MAC comparison between SSI UPC – SSI PC techniques pre-restoration

Comparing the unweighted principal component and the canonical variate Analysis it is evinced the non-compatibility between CVA Mode 3 and the modes UPC and between UPC 6 and the CVA ones; this is denoted by very low values of MAC, between 0.02 and 0.23 well defined from almost unity values present along the principal diagonal.

An even better diagonality condition is found by comparing the CVA and PC techniques where it is possible to notice the lack of compatibility between the CVA Mode 7 with a frequency of 10.96 Hz and the vibration modes found with the Principal Component technique; a good compatibility is found between CVA Mode 1 with a frequency of 2.84 Hz and PC Mode 2 of 3.75 Hz with a MAC value equal to 0.84.

Let's consider now the mode comparation method for the model of the footbridge after the restoration. As for the case prior to the restoration, we will first analyze the situation concerning the comparison of results between the FDD Peak-Picking techniques and then the SSI ones.

Analyzing the vibration modes identified with the deterministic techniques (SVD Subspace) it is found a perfect compatibility between the EFDD modes and the CFDD ones, evidenced by a unit MAC on the diagonal. It has a strong enough compatibility between EFDD Mode 2 (3.05 Hz) and CFDD Mode 1 (3.06 Hz) and viceversa with a MAC = 0.45 (Table 8).

Frequency (Hz)	3.06	3.5	4.11	6.45
3.06	1	0.45	0	0.29
3.5	0.45	1	0.01	0.38
4.11	0	0.01	1	0.08
6.46	0.27	0.36	0.08	1

Table 8: MAC comparison between EFDD – CFDD techniques post-restoration.

A situation much closer to the theoretical one is when SSI-UPC and SSI-PC are considered and the three vibration modes experimentally identified are compatible with each other. Abnormality occurs in the results of the comparison of vibration modes related to UPC-CVA and PC-CVA. The modes compatible are the first two identified by ARTeMIS Extractor with the technique SSI-UPC (respectively of frequency 3.056 Hz and 4.09 Hz) and the first and the third of the Canonical Variant Analysis. (Table 9)

Frequency (Hz)	3,05	4,09
3,06	1	0,02
4,09	0,02	1

Table 9: MAC comparison between SSI-UPC – SSI-PC techniques post-restoration.

4 Critical comparison of the results

The original structure of Postiguet walkway was analyzed with SAP2000 v.14 software [14]. The geometric structural model was built in order to represent in detail the geometric and mechanical characteristics of the component elements. The geometric model consisted of a FE model with 10838 nodes, 83 frame elements and 10401 shell elements. In this way we tried to approximate the structural response to the real expected results.

For the mechanical characteristics of the steel constituting the walkway an elastic modulus E1=2.059.000 MPa and a shear modulus G12=792100 MPa have been assumed. These values of the Mechanical properties have been assumed after an updating process of a preliminary numerical model of the footbridge. It is possible to observe that the stiffness assigned to the material is almost ten times higher than usual; with this value are calibrated other aspects of the overall stiffness of the structure. The global results of the modal analysis after a first calibration of the model are shown in Table 10.

Vibration modes	Ux	Uy	Uz	Frequency	Mode classification
	(%)	(%)	(%)	Hz	
1	69.00	14.00	0.00	2.01	Longitudinal
2	11.00	22.00	1.32	2.81	Transversal
3	6.56	3.02	2.55	4.00	Longitudinal
4	0.00	0.00	49.00	4.75	Vertical
5	0.62	4.47	25.00	4.96	Vertical
6	2.19	17.00	8.82	5.88	Transversal
7	1.09	5.76	0.68	7.24	Transversal
8	0.00	0.21	0.00	8.77	Transversal
9	0.19	0.00	0.00	8.77	Longitudinal
10	0.02	0.22	0.29	8.88	Vertical
11	0.05	0.14	0.00	10.75	Transversal
12	0.14	0.05	0.00	10.75	Longitudinal

Table 10: Vibration modes and participating modal mass.

Figures 11, 12 and 13 show the modal shapes assumed in the different vibration modes of the structure.

In Table 11 a comparison between the vibration modes of the structure from the numerical and the experimental models is shown.

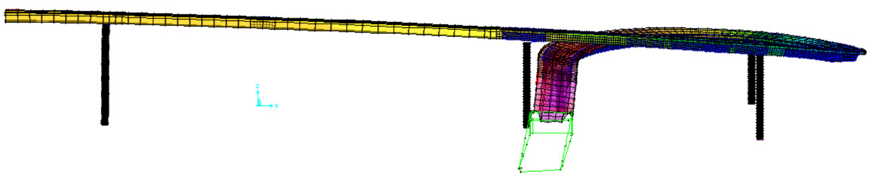

Figure 11: 2^{nd} vibration mode of the structure (transversal mode)

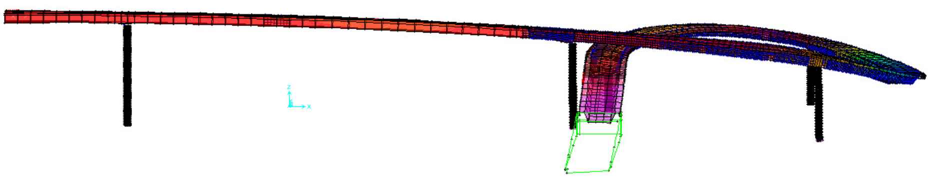

Figure 12: 3^{rd} vibration mode of the structure (longitudinal mode)

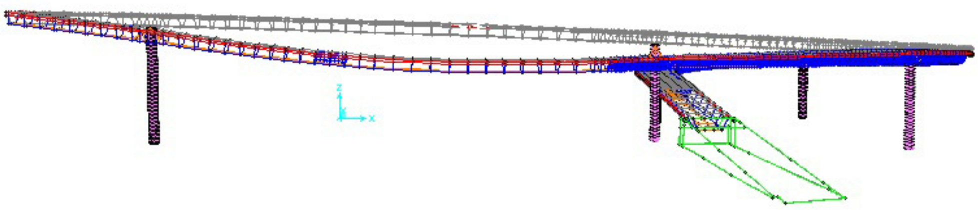

Figure 13: 4^{th} vibration mode of the structure (vertical mode).

Mode	Frequency f (Hz)		Mode classification
	Numerical model	Experimental model	
1	4	3.66	Longitudinal
2	4.75	4.65	Vertical
3	7.24	7.7	Transversal

Table 11: Comparison of the vibration modes

As shown in Tables 4 and 5 it can be noticed that variations of the values of the frequency and the damping coefficient in the two cases, before and after the remodeling, may be attributed to the following factors.

When modeling the structure before the pre-restoring phase the end section of the walkway (Benacantil upstream side) and the supports of the bridge were supposed perfectly fixed. It was not possible to determine both the effective stiffness of the join constraints and the loss of elasticity of the neoprene supports. Also during the design of the new shape of the walkway the support constraints have been changed and this helped to give the structure a new response to the dynamic loads as the stiffness of the entire structure is changed. In the new footbridge structure the most important changes concern the introduction of new coating materials such as fiberglass, the wooden decking and railings of glass material. Just the use of pine for the coating of the floor, as studied by Ciornei, Diaconescu, Glovnea [15] leads to an increase in the value of the damping coefficient to 0.36. If we assume a linear elastic behaviour of the structure, such as to apply the principle of superposition, this would explain the change in the frequency of the vibration modes of the structure (changes of total mass and stiffness) and the associated damping ratios.

5 Conclusions

The Postiguet footbridge is a classic example of a structure that is easily excitable by horizontal and vertical vibrations. With this paper we have investigated the dynamic behaviour of the structure under conditions prior to and following the structural reinforcement with fiber glass composite materials. The natural frequencies of vibration and the damping factors for the main bending vibration modes have been determined in the two situations studied.

The experimental analysis, with ARTEMIS 2011 software, and then the analysis of the numerical model in SAP 2000 have led to the same values of the dynamic parameters.

It has been identified the fundamental period of the structure to the vertical vibrations, is 3.65 Hz, and the corresponding damping factor $\nu = 1.7\%$ that in the condition after the reinforcement assumes the dynamic characteristics of 3.06 Hz for the frequency of the first mode of vertical vibration and $\nu = 1.92\%$.

This shows that structural changes to the pedestrian walkway and the new materials used for the coating have changed its dynamic response: there was an increase in the damping coefficient and a decrease in the values of natural frequencies of the free vibrations modes. This means that Postiguet footbridge is more resistant to vertical vibrations and therefore has a better behaviour with respect to the dynamic stresses of the structure.

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