DYNAMIC ANALYSIS AND STRUCTURAL EVALUATION OF GÓIS FOOTBRIDGE

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ABSTRACT

The results of a campaign of dynamic tests performed in a timber footbridge for evaluating its structural performance is presented in this paper. First of all, the modal parameters of the footbridge were determined through ambient vibration tests. In a second step, the scaling factors for mode shapes were calculated using the mass change method. Finally, the vibration level of the footbridge under pedestrian excitation was evaluated.

1- INTRODUCTION

The structural evaluation of bridges is an important topic in the safety of these structures. Footbridges are not the exception even when their carrying loads could be apparently less demanding than those present in railway and highway bridges. Uncertainties on the structural behaviour of a timber arch bridge are abundant since these structures have not been deeply studied so far. For instance, it is widely accepted that the connections between timber elements play an important role in the structural behaviour of this kind of footbridges. However, there is not enough quantitative information about this role. It is evident that a correct estimation of the stiffness of these connections will reduce the uncertainty in their structural behaviour. As a result, advanced techniques like model updating procedures have been recently applied to footbridges for their structural evaluation. Živanovic et al. (2007) have described the complete process of the model updating process applied to a lively footbridge. da Silva et al. (2007) developed four different load models of pedestrians walking for analyzing the dynamic behaviour of a footbridge located in Rio de Janeiro, Brazil. The acceleration responses determined by simulations were not proved in this footbridge by experimental dynamic tests. Gentile and Gallino (2008) carried out experimental dynamic tests on a historic suspension footbridge in Italy. A structural model of the footbridge was done and its modal parameters were matched with the experimental modal parameters determined from the ambient vibration tests (AVTs). A state of the art of the serviceability conditions of footbridges under pedestrian excitation was carried out by Živanovic et al. (2005). Important aspects in this issue as load models of pedestrians walking, footbridge numerical structural modelling, human perception of vibration bridges, pedestrian-footbridge dynamic interaction, among others were discussed in that work. In this paper, the structural behaviour of a timber arch bridge was evaluated. For accomplishing this objective, a simplified static analysis of the bridge was done. Afterwards, experimental dynamic tests were carried out on the bridge under AVTs and the modal identification of the structure was obtained. With the purpose of calculating the scaling factors of mode shapes, a second campaign of AVTs were performed on the footbridge with additional masses located in strategic points. In addition to the AVTs, accelerations were acquired when pedestrians were walking, running and jumping. Finally, a Finite Element (FE) model of the footbridge was prepared. The characteristics of this model were updated matching its frequencies and mode shapes with those calculated from the dynamic tests. Stiffness values of the bridge connections were adjusted to match the frequencies and mode shapes.

2- FOOTBRIDGE DESCRIPTION

The Góis timber arch footbridge consists of two parallel glued laminated timber (glulam) three hinged arches (25 x 50 cm^2) with suspended deck of 30.6 m span between arches. Two longitudinal glulam beams (11.5 x 46.7 cm^2), simply supported in the concrete abutments and suspended by the arches, complement the main structure of the deck. The deck, with 2.15 m wide and 31 m long, is formed with solid timber boards supported by a grid of glulam stringers (9 x 23.3 cm^2) and cross beams (11.30 x 30 cm^2). Twelve glulam hangers (16.7 x 18.5 cm^2) connect the deck with each arch. Steel strut gives lateral support to the arch (ϕ=2.0 cm) and the deck (ϕ=1.6 cm and 1.2 cm). Moreover, two arches are connected each one by cross beams located in the position of the hangers. Finally, arch and deck are connected by drift pins. Figure 1 shows the footbridge analysed and its more relevant connections.

3 – VISUAL INSPECTION

The structural condition of the Góis timber arch footbridge was firstly evaluated by the visual inspection technique. The main conclusion of this inspection was the poor details of this structure. For example, in most structural elements, the bad construction details are not able to keep the outer faces of wood dry.

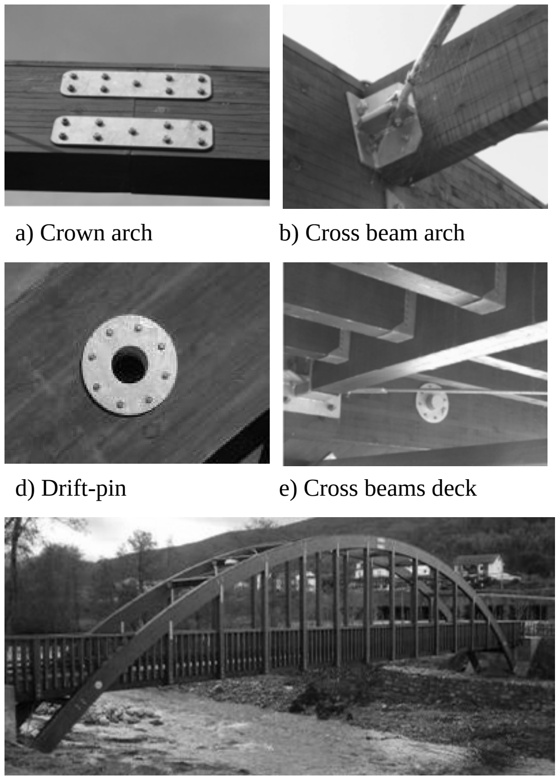


Fig 1 – Timber arch footbridge of Góis.

In addition to these facts, significant delamination problems of the glulam elements were observed. The delamination is more serious in the glulam arch elements, because of the higher perpendicular tension to the grain in these elements. This delamination is the consequence of a faulty production of the glulam arch elements. At the moment, and because the delamination is not so deep that can split the glued laminated timber elements of the arches, this problem cannot jeopardize by itself the structural safety of the footbridge. However, it will cause a fast decay of the wood material.

4 – FINITE ELEMENT MODEL

The finite element model of the footbridge done using SAP2000 computer program [Computers and Structures, (2006)] was formulated using the following assumptions: main local rotation was released for all the connections. Exception was the handrail connection in which any degree of freedom (DOF) was released. Besides this consideration, the timber handrail was modelled and supports in arches and deck were considered as hinged. The deck boards were not modelled but their masses were added to the cross beams. Timber elements were modelled with a Young's modulus of 11.60 GPa in longitudinal direction and 0.39 GPa in both transversal directions, according to NP EN 1194 (1999) for GL24h. Steel struts were modelled as cable elements of two nodes without considering presstressing forces. The Young's modulus of the steel struts was assumed as 199.9 GPa. Numerical frequencies and mode shapes were obtained using the Ritz Vector procedure. The model consists of 702 nodal points, 1043 bar elements and 54 cable elements. In an initial step, this finite element model was used for determining the maximum solicitations caused by the application of the design forces in the static analysis. Moreover, natural frequencies and corresponding mode shapes calculated with this model were calculated and compared with those obtained from the preliminary dynamic tests.

5 – STATIC ANALYSIS

A static analysis over the Góis timber arch footbridge was carried out with the aim to check the structural performance of the footbridge under ultimate and serviceability limit state conditions. For this purpose, the Eurocode 5: "Design of timber structures" (2004) was used. Demands caused by the applied forces were calculated using a structural model done in SAP2000 [Computers and Structures, (2006)]. Design loads were obtained according to Eurocode 1 (2004). The static analysis was performed on the three main elements of the footbridge: arch, hanger and deck cross beam. The values given by the NP EN 1194 (1999) for GL24h were adopted for the mechanical properties of the glulam elements of the structure. Results of the revision of the three main elements by combined bending and axial stresses, buckling and maximum deflections caused by the design forces showed that the analyzed structure is safe considering the load combinations proposed by Eurocode 1 (2004) for the Ultimate and Serviceability Limit States, ULS and SLS, respectively, and the requirements established for the SLS by Eurocode 5 (2004). A detailed calculation of the static analysis of the footbridge can be consulted in Salgado et al (2007).

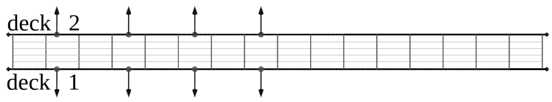
6 – DYNAMIC TESTS

Dynamic tests consisted of acquiring acceleration response on four different conditions. In the first case, a preliminary dynamic test was performed. In a second step, acceleration response was acquired with ambient vibration test (AVT). In the third case, accelerations were acquired on AVT with additional masses over the deck, and in the last case, dynamic test was performed with deliberated pedestrian excitation. For the dynamic tests, eight accelerometers (model PCB 393B12) with a sensitivity of 1000 mV/g, a Data Acquisition System, DAQ, with 16 channels and a portable computer were used. Modal parameters for the four cases were obtained using ARTEMIS Extractor program [Structural Vibration Solutions, 2004]. Here, the Stochastic Subspace Identification (SSI) method in the Canonical Variate Analysis (CVA) Estimation and the Enhanced Frequency Domain Decomposition (EFFD) method were used. In the preliminary dynamic test sensors were located along the first half of the deck in vertical position in a first step and in the transversal position later (see Figure 2a). The tests were performed on 20th of March 2007. Scattered clouds with no rain were present during the test (Wunderground, 2007). These preliminary tests showed that the frequency range was delimited from 2 to 10.5 Hz. The sampling frequency was set to 100 Hz and the recorded time was set to 900 seconds.

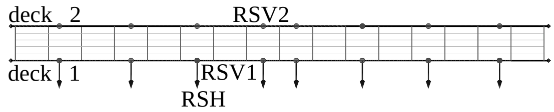
Moreover, comparison between natural frequencies and mode shapes from the finite element model and from the preliminary test showed important differences. Therefore, it was concluded that the structural model needed to be updated mainly in its connection stiffness.

In the second step, a more robust campaign of ambient vibration tests (AVTs) during 30 March 2007 and 2 April 2007 was performed. Overcast weather condition with occasionally light rain in the afternoon and average wind speed of 12 km/h happened these days (Wunderground, 2007). In our opinion, wind speeds higher than those here reported will not cause higher level of vibrations than caused by pedestrian excitation. During these tests including those done with additional masses, three sensors were kept at fixed positions. Two at the vertical position referred as RSV1 and RSV2 and the last one at transversal position referred as RSH in Figures 2a and 2b. The remaining sensors were roved six times to cover all the measuring points. Dynamic acquisition was performed twice for each sensor layout, first time for the AVT tests and second time with additional masses. The identified modal parameters of the footbridge (AVTs) are given in Figure 3.

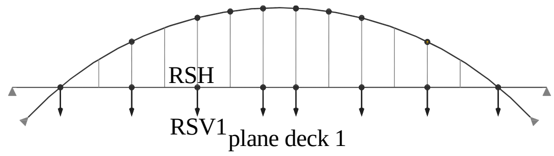
The first mode shape is in transversal direction with movements of the arch and deck. This clearly indicates that the footbridge is more flexible in this direction than in the vertical direction. The second mode is rotational in vertical direction with significant movement of the arch elements in transversal direction. Moreover, a slight movement in vertical direction of the deck was also detected for the second mode. The third mode represents the first vertical mode shape, with mainly contribution of the deck. The fourth mode represents the transversal movement of the deck. Arches presented only minimal movement. The fifth mode represents the second vertical mode, having mass contribution of the arch and deck. It is important to mention that crown arch connections had a main contri-



a) Sensor layout for the preliminary tests

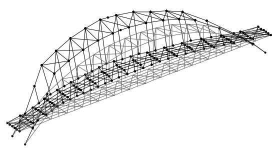


a) Sensor layout along the footbridge deck (AVTs)

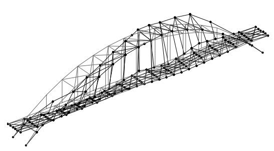


b) Sensor layout along plane deck 1 (AVTs)

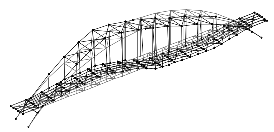
Fig 2 – Sensor layout adopted for the dynamic tests



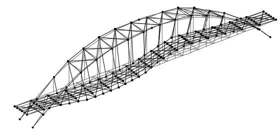
a) f_1=2.44 Hz, ξ_1=1.40 %



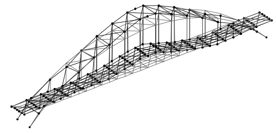
b) f_2=4.48 Hz, ξ_2=2.31 %



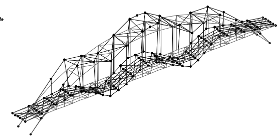
c) f_3=5.15 Hz, ξ_3=2.06 %



d) f_4=7.81 Hz, ξ_4=0.97 %



e) f_5=8.10 Hz, ξ_5=1.79 %



f) f_6=22.99 Hz, ξ_6=2.05 %

Fig 3 – Main mode shapes of Góis footbridge (AVTs).

bution in the movement of the fifth mode. The sixth mode illustrates the third vertical mode of the footbridge with mass contribution of the deck and arch.

In a third step, dynamic tests considering additional masses located along the deck were performed on the footbridge with the purpose of calculating the scaling factors of the mode shapes obtained from the previous AVT modal analysis. The Mass Change Procedure (MCP) proposed by López et al. (2005) assumes that the frequency shift is small enough (1 to 2 %) and the mode shapes obtained with additional masses are not significantly altered. In addition to this method, scaling factors were also estimated using the Mass Matrix Method (MMM). With this procedure mass matrix was determined from a numerical model with the same degrees of freedom (DOFs) as the experimental mode shapes. Comparison of these methods, one experimental (MCP) and the other numerical (MMM) is given in the Table 1.

Table 1. Scaling factors.

Mode	α_{MCP}	α_{MMM}	$\Delta\alpha$ (%)
1	3.44	4.20	18.13
2	2.90	3.35	13.38
3	2.16	4.60	52.96
4	2.27	5.61	59.53
5	2.78	4.74	41.46
6	1.99	4.50	55.86

In the comparison of the scaling factors calculated with MCP and MMM methods, differences were evident and trend to increase for higher mode shapes. Bigger differences were expected for higher mode shapes according to López et al (2005). However, differences for the first two mode shapes are also big. That may be attributed to not accurate estimation of the mass matrix and frequency shifts caused by pedestrian crossing the footbridge during the dynamic acquisition. An important part in the structural evaluation of a footbridge consists of determining if the structure vibrates too much under certain load excitation. As a part of the last series of dynamic tests carried out on this foot-bridge, three different cases under pedes-trian excitation were performed. Firstly, three pedestrians walked on the bridge with synchronized step. In the second case, pedestrians synchronised run on the footbridge and in the last case, the pedestrians jumped at the same time in the midspan of the footbridge. These cases were considered to cause the most demanding level of vibration on the footbridge under low pedestrian density. In addition to these cases, acceleration response for high pedestrian density was calculated for the case of pedestrian walking according to the procedure proposed by Grundmann [Fib, (2005)]. This method found that the acceleration response for a pedestrian density of 1.0 pers/m^2 was resulted of multiply the acceleration response of 3 pedestrians by 2.85.

6.1 – Limits of comfort vibration

The vibration evaluation of the Góis timber arch footbridge was performed comparing the results obtained from the dynamic tests under pedestrian excitation with comfort limits recommended in Codes, Standards and several publications. Živanovic et al (2005) and Fib (2005) presented a summary of several studies related to the limit of comfort for bridges and other structures. In these studies, vibration evaluation is commonly based on limit values of frequencies and accelerations. Comparison of different codes for frequency limits indicated that Góis Footbridge natural frequencies are outside the pedestrian loading frequencies. The vertical limit frequency was found to be 4.5 compared with 5.15 Hz determined from the AVT. In transversal direction the limit frequency was found to be 2.5 Hz compared with 2.44 Hz determined from the AVT. Vertical acceleration limits showed that most of the references determined the level of vertical acceleration at 7.0 %g. In the case of transversal acceleration limits, most of the methods proposed a vertical acceleration limit of 2.0 %g. In the case of vandalism loads like people jumping trying to excite the natural frequency of the footbridge, some authors [Fib (2005)] proposed a maximum limit of acceleration between 70 to 80 %g. Maximum acceleration values determined from experimental dynamic tests were larger than the comfort limit for both directions (7 %g vertical and 2 %g transversal) when pedestrians were passing on the midspan of the footbridge as illustrated in Figures 4 and 5. This behaviour is because two facts: firstly, accelerations increased when pedestrians approached the midspan of the footbridge, and secondly, the accelerometers are also excited by the noise and by the local impact forces caused by the pedestrian steps. This last assumption may be responsible for the instantaneous sharp increment of accelerations. Similar behaviour was determined for the case of pedestrians running. In the case of pedestrians jumping, higher accelerations than the recommended limit values were detected. However, displacements deter-mined from the acceleration responses were below the limit values recommended by Nakamura (2003).

6.2 – Evaluation of lock-in effect

This phenomenon has gained attention of the researchers since the Millennium Footbridge underwent for this phenomenon. However, there is still a lack of knowledge about the interpretation and analysis of this effect. Findings in these studies led that this effect can arise for accelerations below those limit recommended for pedestrian comfort. There-fore, this effect is more demanding than the comfort limits before calculated. Lock-in effect leads inevitably to high amplitudes in the footbridge. Hence, in authors opinion, a displacement evaluation of the dynamic response should be also done.

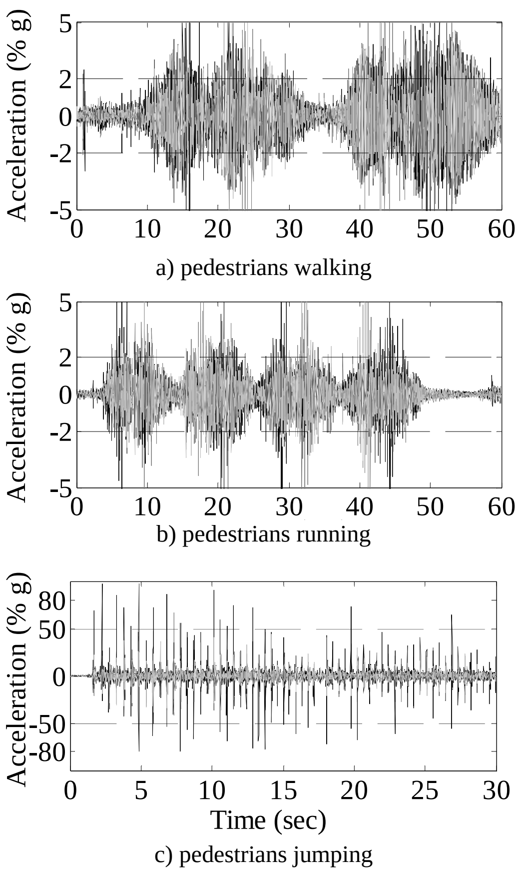


Fig 4 – Transversal acceleration response for pedestrian excitation.— —Comfort limit values.

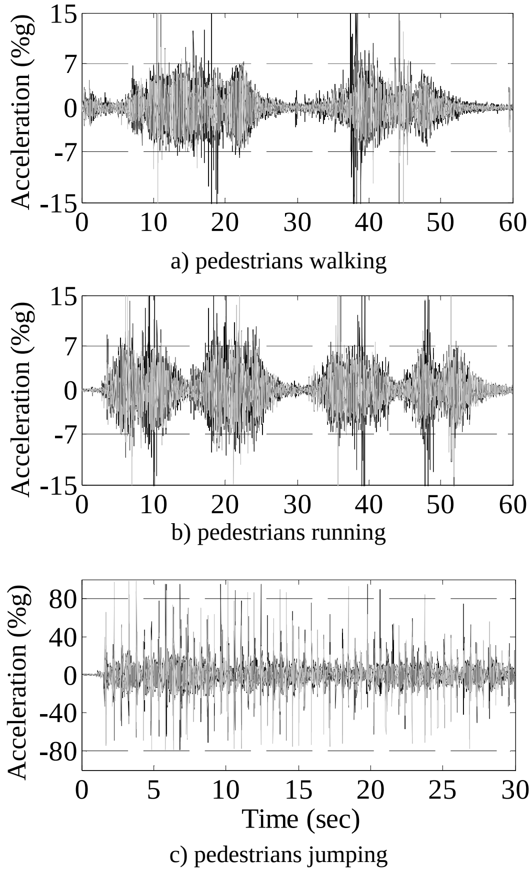


Fig 5 – Vertivalal acceleration response for pedestrian excitation. — — Comfort limit values.

Bachmann (2002) recommended limit values of accelerations and displa-cements to diminish the change of the lock-in effect. This author proposed a maximum vertical and transversal accele-ration of 4 %g and 0.8 %g, respectively. With regard to maximum displacements, 10 mm and 2 mm were recommended by the vertical and transversal direction, respectively. Acceleration limits are evidently not fulfilled according to the history of accelerations of Figures 4 and 5. Displacement response of the footbridge under pedestrian excitation calculated by double integration of the acceleration response indicated that the maximum displacements were 7 mm (less than 10 mm) for the vertical direction and 2 mm (equal to the maximum value). Therefore, maximum displacements in the footbridge are below the recommended values.

In conclusion, for the evaluation of lock-in effect of a footbridge, maximum displacement of the structure should be checked as well as the maximum accelerations. In the case of Góis timber arch footbridge, pedestrian walking did not cause lock-in effect. This fact was verified for the low amplitude of the displacements calculated from the performed dynamic tests.

7 – MODEL UPDATING

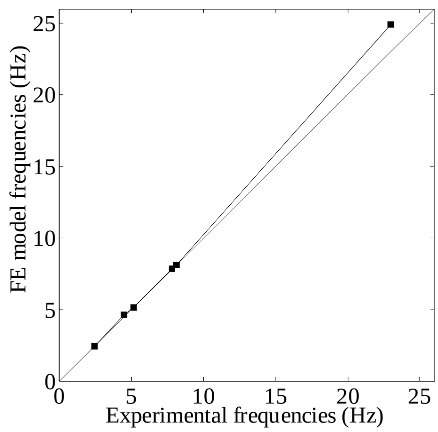
The finite element model of Góis timber arch footbridge was updated by tuning the stiffness values of its main connections. Nine critical connections capable of modifying the dynamic behaviour of the structure were identified. Six degrees of freedom (DOFs) for each connection were considered. Force in cables, support restraints in deck and arch were also selected as variables for the model updating process. A sensitivity analysis of the connection stiffness values was carried out releasing one DOF by one and comparing the calculated natural frequencies and mode shapes with those obtained with the non-released structure. Results of this analysis showed that twenty eight stiffness values of the selected connections had to be modified. Moreover, the longitudinal DOF of the deck supports u_1 showed an important influence on the fourth mode shape. In total, 30 sensitive structural parameters of the footbridge (including cable force) were updated using information from the six natural frequencies and the six Modal Assurance Criterion (MAC) [Allemang, 2003] values between the experimental mode shapes and the updated finite element model. Therefore, more than one solution is possible due to have more updating variables (30) than equations (12).

7.1 – Model updating results

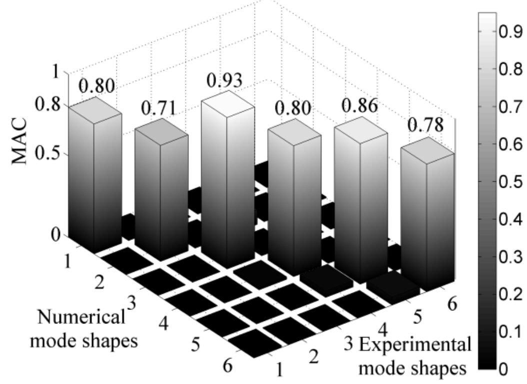
Finite element model of Góis timber arch footbridge was updated matching its natural frequencies and mode shapes with those calculated based on the experimental modal analysis. The comparison was done using the differences between natural frequencies (Δ_{freq}) and the MAC method. Results related to the correlation between modal parameters are shown in Figure 6.

Good correlation of frequencies was obtained for all the modes with a maximum difference for the mode 6 of 8.37%. For the comparison of mode shapes, a MAC value greater than 0.80–0.85 is considered a good match while a MAC value less than 0.40 is considered a poor match [Gentile and Gallino (2008)]. Only modes 2 and 6 with MAC values equal to 0.71 and 0.78 could be considered not very well correlated. However, visual comparison of both modes indicated good correlation between all the modes. Therefore, according to the comparison parameters and taking into consideration the complexity of the model updating process which involved 30 different variables, it was concluded that the modal parameters of Góis timber arch footbridge were matched properly.

The best match of modal parameters was obtained with the connections stiffness given in Table 2. Moreover, supports in arches were considered as hinged and a longitudinal stiffness of 7.25 MN/m was applied to the longitudinal translation for the deck supports.



a) Frequency pair



b) MAC values

Fig 6 – Correlation of modal parameters after the model updating procedure.

Table 2. Connection stiffness values after the model updating process.

Connection	Stiffness (MN or MN-m/m				
	N	V_2	V_3	M_2	M_3
Crown Arch	21	21	1	N/R	0.10
Top hanger	5	5	0.5	0.05	0.05
Bottom Hanger	10	10	10	1	1
Bottom handrail	10	10	10	1	1
Cross beam deck	0.35	N/R	N/R	0.15	0.15
Stringer deck	N/R	N/R	N/R	0	0
Cross beam arch	N/R	N/R	N/R	0.05	0.05
Steel strut	N/R	N/R	N/R	0	0

Steel struts and stringers in the deck were fully released in bending moment. The remaining connections were partially released in bending moment. A stiffness value of 0.35 MN/m was assigned to the deck cross beam connection in direction u_1. This value was considered for matching the mode 3. The crown arch, hanger and handrail connections were partially released in all the five considered DOFs. From this group, top hanger was the weakest connection. The same stiffness values were determined for bottom hanger and bottom handrail connections. In the case of crown arch connections, a value of 21 MN/m was determined for the translational directions.

8 - CONCLUSIONS

A campaign of dynamic tests was carried out with the purpose of evaluating the structural condition of the Góis Footbridge through dynamic parameters calculated from Ambient Vibration Tests. For accomplishing this task, the scaled mode shapes were obtained using the mass change method. The level of vibration of the footbridge under pedestrian excitation was also evaluated. Revision of vibration level of the footbridge showed that accelerations were bigger than limit values for all the evaluated cases. However, local effect caused by pedestrians may have sharply increased acceleration values. Evaluation of the dynamic displacements of the footbridge under pedestrians jumping led to values below the limits. It can be concluded that Góis Footbridge is not prone to harmful dynamic effects provoked by the dynamic interaction pedestrian-footbridge. Recommendations for the vibration limit values of footbridges done by Codes and Standards can be considered conservative for the case studied. A finite element (FE) model of the footbridge was updated with the modal parameters obtained from the AVTs. Connection stiffness values were the main parameters to be determined. Good correlation was obtained between the modal parameters from the FE model and experimental tests. Damage localisation on this footbridge using its dynamic parameters and simulation of damage scenarios with the updated FE model is under investigation and will be presented in further communications.

REFERENCES

Allemang, R.J., The modal assurance criterion, Sound and Vibration, August, pp. 14-21, 2003.

Bachmann, H., Lively footbridges – a real challenge, Proceedings International Conference Footbridge, Paris, France, 2002.

Computers and Structures, Inc., SAP2000 Advanced 10.0.5, structural analysis program, Berkeley, CA, USA, 2006.

da Silva JGS, Vellasco, P.C.G & Andrade, S.A.L., Vibration analysis of footbridges due to vertical human loads, Computers and Structures, 2007.

Eurocode 1: Actions on Structures, EN 1991:2004, European Committee for Standardization, Brussels, Belgium, 2004.

Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings, EN 1995-1-1:2004, European Committee for Standardization, Brussels, Belgium, 2004.

Fib, Fédération internationale du betón, Guidelines for the design of footbridge, Task Group 1.2, Bulletin 32, Lausanne, Switzerland, 2005.

Gentile, C. & Gallino, N., Ambient vibration testing and structural evaluation of an historic suspension bridge, Advances in Engineering Software, pp. 356-366, 2008.

López, A.M., Brincker, R., Fernández, C.A. & Villa, G.L., Scaling Factor Estimation by the Mass Change Method, Proc. 1st Int. Conf. IOMAC, Copenhagen, Denmark, 2005.

Nakamura, S., Field measurements of lateral vibration on a pedestrian suspension bridge, The Structural Engineer 81 (22), pp. 22–26, 2003.

NP EN 1194, Estruturas de Madeira. Madeira lamelada colada. Classes de resistência e determinação dos valores característicos. CT 14 (CTIMN), IPQ, 1999.

Salgado, R., Branco, J.M., Cruz, P.J.S. & Ayala, G., Dynamic analysis and structural evaluation of a timber arch bridge, internal report. 07-DEC/E-28, ISISE, University of Minho, Portugal, 2007.

Structural Vibration Solutions A/S, ARTEMIS Extractor Pro, release 3.43, Modal Analysis Program, Aalborg, Denmark, 2004.

Wunderground, The Weather Underground, Inc., 2007, http://www.wunderground.com (Accessed 20, 30 March 2007 and 2 April 2007).

Živanovic, S., Pavic, A. & Reynolds, P., Vibration serviceability of footbridges under human-induced excitation: a literature review, Journal of Sound and Vibration (279), pp. 1–74, 2005.

Živanovic, S., Pavic, A. & Reynolds, P., Finite element modelling and updating of a lively footbridge: The complete process, Journal of Sound and Vibration (301), pp. 126–145, 2007.