ABSTRACT: The use of modal testing through ambient vibration to assess the changes in rehabilitated structures and to enable the evaluation of the actual effectiveness of different rehabilitation strategies emerge as an extremely important topic, since the current knowledge in this field is short and therefore demands an improvement in order to achieve more cost-effective designs without lowering the required safety levels. This paper presents the ambient vibration tests conducted on centenary steel bridges before and after their rehabilitation, with the purpose of evaluating the changes in their dynamic properties as a result of the adopted strengthening strategies. The implemented testing programs, experimental setups, data processing and modal identification techniques are described. Field data validated numerical models for the analysis of the structural changes produced by the strengthening process are presented. Significant conclusions were drawn by comparing the experimental and numerical results between the pre and post-rehabilitation conditions, namely in what concerns the vibration levels experienced by the structures, their stiffness variation and suitability of the adopted modeling methodologies.

KEY WORDS: Centenary steel bridges; Rehabilitation assessment; Ambient vibration testing; Modal analysis; Numerical modeling.

1 INTRODUCTION
The use of nondestructive testing tools is emerging as a valuable solution to assist the condition assessment of existing structures, both in terms of its load-carrying capacity and serviceability [1]. Moreover, data obtained through these means are essential for the quantification of parameters and identification of mechanisms that are to be integrated in the numerical models that support reliable and objective structural evaluation [2].

While some researchers have pointed out the intrinsic difficulties in performing vibration testing in large bridges, such as non-stationary excitation [3], non-linear boundary or continuity conditions and non-ideal connections/interfaces between structural members and components [2], it is unquestionable that dynamic testing stands as an innovative way of great potential in the structural identification of bridges for condition assessment [4-6]. It can play a decisive role in the rehabilitation of centenary steel bridges, both in supporting the design and in validating their performance for the new structural condition. Several examples of dynamic tests performed in old steel bridges have been reported in technical literature related to repair, rehabilitation, upgrade or strengthening projects [7-13].

2 THE BRIDGES AND THEIR REHABILITATIONS
The first bridge, named as Pinhão Bridge, was commissioned in 1907 and stands as a vital link in the road infrastructure that serves the Douro vineyard region where the Porto wine is produced. The superstructure is constituted by three truss spans of about 68.60 m between supports and a skewed deck plate girder span measuring no more than 10 m in length (see Figure 1a)). The simply supported main girders of the longer spans have a semi-parabolic arch shape at the upper level, varying its height from 2.67 m at the supports to 8.86 m at the centre of the span. The bridge deck is formed by a concrete slab resting on a steel grid of stringers and crossbeams, and in its present condition it carries a single roadway lane and two side walkways, 4.60 m and 0.62 m wide, respectively.

Figure 1. The bridges under analysis: (a) Pinhão Bridge; (b) Luiz I Bridge.
The Luiz I Bridge is the second structure under analysis, and has been in continuous operation since its completion in October 1886. This bridge constitutes a unique example of bridge engineering, worldwide known (see Figure 1b)), being characterized by having a single parabolic double-hinged arch supporting simultaneously two decks at different levels, 172 m long and 45.1 m high. The 391.25 m long upper deck is formed by two 5 m high truss girders, comprising 13 spans that rest on five metallic piers and two masonry piers. The two lattice girders of the lower deck are suspended from the arch by four tie-trusses spaced at 36 m intervals, crossing a total span length of 174 m. The loads applied on both decks are transferred to the abutments and piers through bearings that allow longitudinal displacements and rotations, except in the piers supported on the arch.

For both bridges, the main elements pertaining to the original structure have I, T or box shaped sections, built by joining several plates and angles through riveted connections, a typical technique of the steel construction of that time. Since their construction, both structures have experienced some rehabilitation and maintenance operations or minor changes in order to make possible the crossing of new types of vehicles. However, recently these bridges have undergone extensive rehabilitation and strengthening processes. In the case of the Pinhão Bridge, these construction works aimed at endowing the structure with the necessary resistance to carry the current loads, whereas for the Luiz I Bridge the prime objective was to allow the integration of its upper deck in the infrastructure of the Porto Metro Network.

The rehabilitation of the Pinhão Bridge comprised: i) the strengthening of the deck floor system by attaching a new steel grid on its top; ii) the replacement of the old concrete-steel composite slab by a reinforced concrete one; iii) the addition of reinforcement angles and plates to the chords, verticals and to the end diagonals; iv) the replacement of the remaining diagonals by pairs of shear connected C-sections; v) the replacement of the original roller and pin bearings by modern pot and disk bearings, respectively; and vi) hydroblasting of the whole steel surface followed by its painting. Regarding the Luiz I Bridge, the major operations undertaken were: i) the complete replacement of the upper deck bridge floor system by a suitable metallic profile grid, capable of properly transmitting the new railway traffic loads to the truss girders; ii) the replacement of the I-beams that constitute the upper deck girders at the arch crown due to insufficient strength and to enable the direct support of the railway sleepers; iii) the strengthening of the upper deck girders, suspension ties, arch diagonals and bracing elements all over the bridge with addition of steel profiles; iv) the cleaning and lubrication of all original roller bearings, except for the supports at the upper deck abutments which were replaced by disk bearings due to their severe damaged condition; and v) the removal of the old coating through hydroblasting and subsequent 3-layer epoxy painting.

3 SCOPe AND OBJECTIVES

The rehabilitation and strengthening projects of the bridges were carried out after the favorable recommendations obtained in viability studies, which were performed to analyze the feasibility of the structures to carry the current vehicles in accordance with the contemporary standards (Pinhão Bridge) or to withstand the action of new vehicles (Luiz I Bridge). Under the scope of these studies, dynamic analyses were conducted through the complete modal identification of the structures, based on the development of 3D numerical models and on the execution of ambient vibration tests. Therefore, these first tests were mandatory to support the rehabilitation projects by enabling the bridges condition assessment and by supplying in-situ data for the validation of the models to be used in the selection and optimization of alternative intervention strategies.

After the completion of the construction works, second field tests sought to determine the changes introduced in the bridges dynamic properties and to verify the behaviors predicted at the design stage. Moreover, the effectiveness of the implemented strengthening schemes in terms of stiffness variation of the decks, either vertical or transverse, was also appraised. Complementarily, the updated models enable the simulation of the structural responses in the new service stages, and consequently turn possible their integration into health monitoring systems to identify alterations in the bridges behavior over the time.

4 AMBIENT VIBRATION TESTING

The ambient vibration tests conducted before and after the rehabilitation and strengthening of the bridges, henceforth generally termed as Test 1 and Test 2, respectively, sought to measure their vibration response under natural excitation, mostly induced by the wind and traffic, in order to identify the modal parameters for both conditions.

4.1 Testing program

For the Pinhão Bridge acceleration time series were recorded at 7 measurement sections of the three main spans (see Figure 2a)), at both sides of the roadway, so that vertical and torsional mode shapes could be properly identified. Four strong motion recorders were used to collect the accelerations, two of them permanently stationed at section 3, serving as reference, and the remaining two were successively placed at the other six sections. Additionally, a supplementary setup was performed by simultaneously measuring the accelerations at the mid-span sections of all spans. This has allowed to unveil an unexpected global mode shape of the whole bridge.

For the Luiz I Bridge 28 measurement sections were adopted to record the structural vibrations (see Figure 2b)), 19 in the upper deck and 9 in the lower deck. However, in Test 1 the accelerations of the lower deck were only collected at the
suspension ties’ joints and during Test 2 no measurements were taken at section 19. All sections of the upper deck were instrumented with two seismographs, one positioned at each side of the deck, upstream and downstream. On the lower deck a single device recorded the upstream accelerations, even though for Test 2 the vibration response has also been occasionally acquired from the downstream side to identify likely torsion modes. For the entire duration of the tests two reference stations were set on the upper deck at section 15, and for Test 2 an additional reference was introduced at the upstream side of section 23 during the test of the lower deck. The remaining recorders acted as moving sensors.

In both tests, and for both bridges, the accelerations were recorded along three orthogonal directions oriented according to the natural reference axes of the structures (longitudinal, vertical and transverse). All measuring devices integrate one tri-axial force balance accelerometer with linear behavior from DC to 100 Hz, an 18-bit A/D converter, a 1-day test autonomy battery, a memory card for data storage, and an external GPS sensor to enable an independent and synchronized operation. However, in Test 1 of the Luiz I Bridge the resolution of the converter was only 16 bits and no GPS sensors were used. Since each unit was autonomous and pre-programmed by a laptop, the need for cables and hard labor in preparing the tests was overcome. The acceleration time series were collected in setups of 6 to 16 minutes, with sampling frequencies ranging between 50 and 100 Hz, thus making possible the identification of the frequency content of interest for both bridges (below 20 Hz).

4.2 Modal identification

The experimental identification of the most relevant modal parameters was firstly performed using the Peak-Picking method (PP), and afterwards these estimates were confirmed with the results produced by applying more sophisticated identification techniques [14, 15].

Before the application of the method, for each instrumented bridge section, three combined signals were calculated from the records collected at the upstream and downstream sides of each section: half-sum of vertical acceleration components, half-difference of vertical acceleration components and half-sum of transverse acceleration components. In this way, an adequate separation of the natural frequencies in the bandwidth of interest was assured.

The adopted identification process can be summarized in the following steps: i) determination of the normalized power spectral density functions (NPSD) for each pre-combined signal in each instrumented section; ii) calculation of the coherence functions in correspondence to the simultaneous measurements at different locations; iii) estimation of the average normalized spectra (ANPSD) of all signals of the same type; iv) identification of the natural frequencies from the peaks in the spectra; and v) identification of the mode shapes by evaluating the transfer functions relating the response at each section with the one collected at the reference sections. In order to obtain a suitable frequency resolution (at least 0.02 Hz), in this process each record was properly divided into time segments with adequate overlapping.

5 NUMERICAL MODELING

For each bridge two base finite element (FE) models were developed to simulate their behavior before and after the rehabilitation, which will be henceforth generically termed as Models A and Models B, respectively. These models use a 3D mixed meso-micro level modeling approach by means of frame and shell elements, in which all nodes present six degrees-of-freedom. Although a detailed description of the models can be found in [12], herein a summarized version is presented.

![Figure 3. FE models of the bridges: (a) Pinhão Bridge; (b) Luiz I Bridge.](image)

The models replicating a single main span of the Pinhão Bridge hold a deck modeled through shell elements, shear connected to the grid constituted by the crossbeams and stringers (see Figure 3a)). These structural members were simulated by simple frame elements or by a combination of these with shell elements, depending on the level of discretization required to analyze the data from the static field tests [16]. Similar procedure was adopted in modeling the girders’ U-chords, whereas for the diagonals and verticals, as well as for the bracings and transverse sway trusses, frame elements were used. The two supports at one end of the span only enable the longitudinal rotation, while in the other end the longitudinal displacements are also permitted.

In what concerns the models created for the Luiz I Bridge, for the vast majority of the structural members the modeling was accomplished on the basis of frame elements (see Figure 3b)). However, shell elements were also adopted in the following cases: i) to improve the connection between some important elements of the bridge wherein a higher accuracy in replicating the force transmission mechanisms was needed; and ii) to properly simulate the upper deck’ I-girders response at the two spans over the arch crown. The supports of both decks at the abutments only prevent vertical and transverse displacements. However, restrictions of the decks longitudinal displacements had to be considered, which were caused by the extreme degradation of the steel bearings and by the very nature of the expansion joints at the decks ends [17]. These constraints were simulated by springs, whose stiffness coefficients were estimated through a calibration procedure by
matching the relevant numerical modal parameters to the corresponding experimental ones. The compatibility of the longitudinal rotations and displacements between the upper deck and the piers was disabled, except for those two supported by the arch. With respect to the bearing conditions of the arch, these were replicated by pinned supports.

Sensitivity analyses aimed at identifying the factors that control the effective stiffness of the bridges, and in turn the natural frequencies associated to the vibration modes, were conducted. For this purpose, sub-models were generated by sequentially changing key quantities. The variables under analysis were: i) level of restriction of the decks longitudinal displacements; ii) effect of the concrete slabs; iii) contribution of the decks’ stringers and crossbeams; iv) transverse bending stiffness of the decks; and v) level of continuity at the joints between the floor systems and the girders.

6 ANALYSIS OF RESULTS

6.1 Vibration levels

6.1.1 Pinhão Bridge

As both ambient vibration tests were conducted without significant restrictions to the traffic on the bridge, it was possible to assess the level of vibration induced by the vehicles. A reduction to less than one third was identified, with the peak vertical acceleration presenting a decrease from approximately 1.5 m/s² to 0.5 m/s². The records of the transverse accelerations revealed that the level of vibration for the new condition became considerable lower, not exceeding in general one sixth of the vertical accelerations. This difference in the vibration response of the structure was caused by the excessive vibration of the slender flat plates that constituted the diagonals in the previous condition, which had been well noticed during the viability study.

6.1.2 Luiz I Bridge

The peak value of the vertical accelerations acquired in Test 2 was around 2.0 m/s², much higher than 1.2 m/s² recorded in Test 1. This difference was attributed mainly to the change of the dynamic loading on the upper deck (road to rail traffic), and to the higher acquisition frequency adopted in Test 2, which has allowed to capture significant high frequency contributions, and therefore less relevant in terms of pedestrian comfort. In addition, it is also noteworthy the fact that for both bridge conditions the magnitude of the vertical accelerations was almost 4 times that of the transverse accelerations.

6.2 Natural frequencies

6.2.1 Pinhão Bridge

Table 1 lists the average natural frequencies identified before and after the bridge rehabilitation for the three main spans, and compares those values with the estimates calculated from Models A and B, respectively. It is worth pointing out that for some vibration modes detected after the rehabilitation no correspondence could be found in the experimental data of the first test.

On the one hand, the results reveal that for all vibration modes the corresponding frequency value remained unchanged or increased, which combined with the widespread increment of the mass, close to 22 %, clearly indicates a stiffening of the structure. On the other hand, and in general, the correlation between the experimental and numerical frequencies is very good, except for the 6th vibration mode, a pure torsional one, whose deformed configuration can be described in simple terms as corresponding to the anti-symmetric vertical bending of the main girders with a small lateral deformation. After performing a numerical sensitivity study, it was found that this discrepancy was caused by the overestimation of the rotational rigidity of the connections at the common joints of the verticals with the crossbeams and upper truss girders [18].

Table 1. Summary of natural frequencies of the Pinhão Bridge.

<table>
<thead>
<tr>
<th>Mode Type</th>
<th>Before rehabilitation</th>
<th>After rehabilitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st T</td>
<td>1.721</td>
<td>1.684</td>
</tr>
<tr>
<td>2nd V</td>
<td>2.779</td>
<td>2.752</td>
</tr>
<tr>
<td>3rd T</td>
<td>3.310</td>
<td>3.267</td>
</tr>
<tr>
<td>4th T</td>
<td>4.273</td>
<td>4.359</td>
</tr>
<tr>
<td>5th V</td>
<td>5.460</td>
<td>5.341</td>
</tr>
<tr>
<td>7th V</td>
<td>8.293</td>
<td>8.403</td>
</tr>
<tr>
<td>8th V</td>
<td>10.604</td>
<td>10.814</td>
</tr>
</tbody>
</table>

Note: Vibration modes are numbered according to the order of the identified modes after the rehabilitation; T – Transverse mode; V – Vertical mode; T* – Torsional mode; Δa = Numerical / Experimental - 1; Δa = Experimental/Experimental[1] - 1; [1] – Model A; [4] – Model B.

6.2.2 Luiz I Bridge

In this section an analysis of the natural frequencies identified in both tests within the range of 0-3.6 Hz is performed, which are listed in Table 2. Comparing the values for the same vibration modes, small differences are found, not reaching 6.6 %, except for the 6th mode in which the reduction exceeded 17 %. Nevertheless, a slight increase of the natural frequencies appears to be the tendency. A switch of the relative position of the 4th and 5th modes is also detected, potentiated by the close proximity of the values.

Table 2. Summary of natural frequencies of the Luiz I Bridge.

<table>
<thead>
<tr>
<th>Mode Type</th>
<th>Before rehabilitation</th>
<th>After rehabilitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st T</td>
<td>0.757</td>
<td>0.759</td>
</tr>
<tr>
<td>2nd T</td>
<td>0.903</td>
<td>0.908</td>
</tr>
<tr>
<td>3rd T</td>
<td>1.343</td>
<td>1.400</td>
</tr>
<tr>
<td>4th T</td>
<td>1.660</td>
<td>1.657</td>
</tr>
<tr>
<td>5th V-L</td>
<td>1.636</td>
<td>1.636</td>
</tr>
<tr>
<td>6th T</td>
<td>2.124</td>
<td>2.016</td>
</tr>
<tr>
<td>7th T</td>
<td>---</td>
<td>2.042</td>
</tr>
<tr>
<td>8th T</td>
<td>---</td>
<td>2.626</td>
</tr>
<tr>
<td>9th T</td>
<td>---</td>
<td>3.227</td>
</tr>
<tr>
<td>10th V</td>
<td>2.295</td>
<td>2.274</td>
</tr>
<tr>
<td>11th V</td>
<td>---</td>
<td>2.806</td>
</tr>
<tr>
<td>12th V</td>
<td>---</td>
<td>2.949</td>
</tr>
<tr>
<td>13th V</td>
<td>3.125</td>
<td>3.126</td>
</tr>
<tr>
<td>14th V</td>
<td>3.369</td>
<td>3.399</td>
</tr>
</tbody>
</table>

Note: Vibration modes are numbered according to the order of the identified modes after the rehabilitation; T – Transverse mode; V – Vertical mode; L – Longitudinal mode; Δa = Numerical / Experimental - 1; Δa = Experimental/Experimental[1] - 1; [1] – Model A; [4] – Model B.
One peculiar aspect that is highlighted from the identification of the natural frequencies is the fact that the value of the 1st vertical bending mode (5th mode) remained unaltered, in spite of the significant operations performed on the upper deck. From a structural point of view, this fact is a fortunate coincidence, since it was not specifically planned by the rehabilitation design. In addition, given the close proximity of the natural frequencies and the frequency resolution achieved for both tests (of about 0.02 Hz) the estimated values may fall within the same measuring interval. Still, with the use of alternative identification methods a small difference could be found [15].

Table 2 also presents the natural frequencies calculated from the numerical Models A and B (fourth and seventh columns), as well as the percentage variation from the field estimates, Δ. In general, with regard to the estimates supplied by the models of this work, the correlation with the experimental values can be classified as very good, being the average of the absolute deviations of 1.35% and 1.99%, respectively for the pre and post-rehabilitation phases. Consequently, the constructed models proved to be extremely accurate in estimating the natural frequencies of the bridge, with model A showing a clear improvement with respect to the one developed for the viability study [19].

Another aspect of interest in the numerical results is the change in the ordering of the vibration modes. The close proximity of the natural frequencies of the 9th and 10th modes (8th transverse and 2nd vertical bending modes) after the rehabilitation made their numerical calculation more difficult, resulting in the inversion of the modes. More importantly, the ordering of the vibration modes from the 8th to the 12th position is completely altered between the two models.

6.3 Mode shapes

6.3.1 Pinhão Bridge

The vibration modes supplied by the Model A are illustrated in Figure 4, where the deformed configuration of the girders is represented in elevation (left-hand side) and that of the chords is depicted in plan view (right-hand side). It should be noted that the experimental components are plotted only for the lower level of the north span and that vertical coordinates are displayed for the vertical bending and torsional modes whereas for the transverse modes only the lateral displacements are represented.

The analysis of the correlations between the identified and computed parameters was performed either by a graphical comparison of the experimental modal coordinates with the numerical mode shapes or using the Modal Assurance Criterion (MAC). For both bridge conditions all vibration modes present a MAC indicator above 0.96, with an average value higher than 0.98, which reveals a very good correlation.

The assessment of the variation experienced by the mode shapes was performed by calculating the MAC indicator, either correlating the modal parameters provided by Models A and B, MAC(n), or comparing the experimental mode shapes obtained from both tests in the north span, MAC(e). Although both indicators reveal a declining trend as the modes order increases, values of MAC(n) remain always above 0.98, whereas MAC(e) exhibits lower levels for the 4th, 5th and 7th vibration modes (yet, no less than 0.93). In spite of some level of degradation of the bridge prior to the rehabilitation, the results have shown that no significant change in the deformed configuration of the mode shapes has occurred due to the rehabilitation process. Last but not least, a major feature highlights from the comparison of the results supplied by both tests, which is the identification of a global vertical bending vibration mode encompassing all the three main spans after the bridge rehabilitation, contrary to the expected decoupled behavior. In order to investigate the role played by the new expansion joints and by the bolt-bars linking the bridge main spans, a global model was constructed by assembling three models of type B connected together via double hinged bars, linking the slabs in the alignment of each of the five stringers, as well as the end verticals at three different levels. A combined analysis of the experimental and numerical results led to the conclusion that almost all bolt-bars were blocked due both to the misalignment of the eye supports and to the small clearance as result of the new coating. This anomaly was confirmed after a thorough survey of the bridge in the new condition.
6.3.2 Luiz I Bridge

Mode shapes provided by the numerical Models A and B are shown in Figure 5, represented in the left- and right-hand sides, respectively. The deformed shape of the bridge is defined by the upper chords of the upper deck and by the lower chords of the lower deck. Transverse bending modes are represented by plan views whereas the vertical bending modes are presented in elevation. Simultaneously, the modal coordinates extracted from both tests are plotted over the numerical mode shapes in order to enable an easy and immediate evaluation of the results.

In general, the experimental modal components compare well with the mode shapes estimated by the models, exhibiting a slightly better match for the post-rehabilitation condition, particularly with respect to transverse modes, which is a consequence of the upgrade in the measuring devices and improvement of the testing parameters and data processing. These conclusions are also supported by the MAC values for the pre and post-rehabilitation conditions, labeled as MAC(1) and MAC(2), respectively, presented beside each mode shape in Figure 5. However, it should be pointed out that for some vibration modes the similarity level is less good as a consequence of the lower quality of the field test data. These are the cases of the 5\textsuperscript{th} transverse bending mode (6\textsuperscript{th} mode) before the rehabilitation and of the 6\textsuperscript{th} vertical bending mode (14\textsuperscript{th} mode) in the new condition. Nevertheless, if these two modes are excluded, the average value of MAC(1) is 0.96 and of MAC(2) is 0.98, which proves once again the reliability of the numerical models developed in this work for the accurately replication of the bridge modal parameters.

Additionally, the changes produced in the structure response regarding its stiffness can also be inferred by a careful examination of the mode shapes. The transverse bending modes after the rehabilitation reveal a less smooth deformed shape of the upper deck near the steel piers. Since the strengthening of the steel piers was very limited this fact suggests a clear decrease of the transverse bending stiffness of the upper deck, which will be properly analyzed in the following section.

6.4 Stiffness variation

In any rehabilitation process of a bridge involving strengthening works both the structure mass and stiffness will be altered to a greater or lesser degree depending on the strategy adopted. If for the former its quantification is easily accomplished through the amounts of material removed from or added to the bridge, with respect to the latter its variation can only be accurately assessed by conducting field tests. In the case of modal testing the parameters that can be directly estimated are modal properties, such as the natural frequencies and mode shapes, both depending mainly on the relation between mass and stiffness. Therefore, knowing the variation of the structure mass and its spatial distribution, as well as the deformed shape of the vibration modes, it is possible to estimate the stiffness shifts produced for the main components, in the vertical and transverse directions, which may turn the execution of static field tests not mandatory.

The evaluation of the stiffness variation based on modal data requires a judicious selection of the vibration modes, but also an accurate quantification of the mobilized mass. The expression for the modal calculation of this parameter, $\Delta K$ (modal stiffness), between two structural conditions is given by the following equation:

$$\Delta K = (\Delta \phi)^2 \times \Delta M$$

where $\Delta \phi$ and $\Delta M$ are correspondingly the proportional variations of the natural frequency and modal mass, computed as a post to pre-rehabilitation condition ratio.

6.4.1 Pinhão Bridge

As the natural frequencies obtained from both tests for the second mode are exactly the same and most of the bridge...
mass is displaced in the vertical direction (see Figure 4) it is possible to conclude that the stiffness of the structure for symmetric vertical loadings increased in the same proportion of the mass, i.e. its increment was approximately 22%. Furthermore, the variation of the modal mass in the vertical direction associated with the second mode shape, computed from Models A and B, is around 21.9%, thus validating this finding. This conclusion is also confirmed by the results collected during the static tests, from which a general increase of 20.5% for the vertical bending stiffness was inferred [16].

Although the first mode presents some torsion it is predominantly transverse, both lower and upper level masses are mobilized, and the deformed shape is symmetric. The ratio increase of the bridge transverse stiffness can then be approximately evaluated by equation (1), considering the corresponding natural frequencies of the vibration mode. The modal stiffening variation was estimated close to 45% by assuming $\Delta M = 1.18$. It is worth noting that the modal transverse displacements of the upper level are, in average, three times higher than at the lower level, which substantially reduces the modal mass variation. Static results from the numerical analysis aim at the same value, which proves the excellent correlation between experimental and numerical estimates.

6.5.2 Luiz I Bridge

In order to appraise the variation of the vertical bending stiffness experienced by the upper deck, the vertical vibration mode to be selected must have a configuration for which the deformation shape of the remaining elements is minimal. The most suitable candidate is therefore the 6th vertical vibration mode (14th mode) where the deformation is almost restricted to the four north spans. A stiffness reduction of 18% was estimated, a value very close to that inferred from the static load tests carried out on the bridge [12].

With regard to the change of the transverse bending stiffness of the upper deck, the task is made more difficult due to the fact that in all transverse modes the arch and piers are deformed. The selection of the candidate vibration mode met the following criteria: i) non-existence of rotation at the arch crown; ii) minimization of the transverse deformation of the arch and metallic piers; and iii) existence of at least one upper deck span whose deformation has inflection points and small transverse displacements at the supports. The most suited vibration mode is the 7th transverse (8th mode) and the best fitting span the 11th. The reduction of the transverse bending stiffness of the upper deck was estimated to be 58%. A static analysis of span 11, considered as a single simply supported bridge, points to a close value of stiffness decrease (54%) when a uniform transverse load is applied, thus proving the adequacy of the adopted procedure.

6.5 Parameters controlling the bridges dynamics

In order to evaluate the influence of several parameters in the bridges dynamics, sensitivity studies were carried out by performing modal analyses based on the models described in section 5. The conclusions are drawn bellow.

6.5.1 Pinhão Bridge

• The slab mainly affects the transverse modes and has a greater impact on the vertical bending modes as the order increases;
• The stringers influence both the transverse and vertical modes, even though their contribution to the latter is higher and more evident as the order of the mode rises;
• A decrease of 25% in the crossbeams stiffness has no significant effect on the vibration frequencies, even for the torsional mode where these elements are under in-plane bending;
• The flexural stiffness of the diagonals pertaining to the upper sway truss girders has no influence on the vertical bending modes, yet has some impact on the transverse modes and clearly influences the torsion mode;
• The rotational rigidity of the connections at the joints of the sway frames, which are formed by the verticals, crossbeams and upper truss girders, controls the natural frequency of the torsional mode with a lower influence in the transverse modes.

6.5.2 Luiz I Bridge

• Constraints on the longitudinal displacements of the decks ends only influence a limited number of vibration modes, the two transverse modes where the deformed shape of the lower deck is predominant (local modes) and the first vertical to which is associated a significant component of longitudinal movement on the upper deck (see Figure 5);
• There is no cross effect of the support conditions at both decks ends on the vibration modes influenced by them, e.g. restrictions to the longitudinal displacements of the upper deck do not generate changes in the natural frequencies of the vibration modes associated with the movement of the lower deck. This fact has enabled the calibration and/or update of the elastic coefficients of the longitudinal springs, on the basis of a small number of vibration modes (1st, 5th and 7th modes) by matching the numerical frequencies with the experimental values;
• The light-weight concrete slabs caused a general stiffening of the bridge in its pre-rehabilitation condition. The influence on the transverse modes associated to large deformation of the lower deck was very strong, but the impact on the remaining transverse modes was also very significant. For the new condition, the influence of the remaining lower deck concrete slab is substantially smaller, yet influencing in the same extent the transverse local modes of the lower deck and only slightly the vertical vibration modes;
• With respect to the upper deck, and for the new service condition, both the stringers of the new steel grid (main) and of the new floor system (secondary) have a minor influence on the global transverse modes, and also affect the first vertical mode which holds a significant longitudinal movement of the upper deck.

7 SUMMARY AND CONCLUSIONS

This paper has presented a set of dynamic field tests conducted on centenary steel bridges before and after their rehabilitation. These tests provided a unique opportunity to evaluate the changes in the dynamic properties of the structures as a consequence of the adopted strengthening strategies and construction works carried out.

The data from the first tests were used to assist the viability studies of the projects, and subsequently also their designs,
whereas the measurements from the second tests helped to identify the changes produced in the behavior of the bridges for the new service conditions, as well as to confirm the adequacy of the rehabilitation plans. The procedures adopted for the ambient vibration tests, the instrumentation of the bridges, the acquisition systems and the testing sequences were comprehensively described. Attention was also paid to data processing and modal extraction techniques implemented for the systems identification. Three-dimensional FE models were constructed to support the modal analyses, and in turn experimental data served to validate and/or update the numerical models. Besides simulating the pre and post-rehabilitation conditions of the bridges, the base models have enabled the assessment of the influence of several structural parameters on the dynamic properties of both bridges.

The results have led to the following general conclusions: 1) modal parameters identified from both tests, either the natural frequencies or mode shapes, corresponded very well with the estimates supplied by the numerical models; 2) in general 3D frame systems are well suited to simulate the dynamic behavior of the bridges, even though hinged or semi-rigid connections for some joints might improve the accuracy of the modal estimates; and 3) detailed analyses to the changes produced in some natural frequencies has permitted to obtain excellent estimates for the variations of the bridges stiffness. Additionally, for the Pinhão Bridge: i) field data confirmed a large reduction of the vibrations at the deck level after the rehabilitation under similar traffic scenarios; ii) the natural frequencies of all identified vibration modes remained constant or increased with the rehabilitation process, whereas the mode shapes did not indicate any significant variance between the pre and post-rehabilitation conditions; iii) while the stringers stiffness influences both the vertical and transverse modes, the slab mainly impacts on the latter and a significant variation of the crossbeams stiffness has little effect on the vibration frequencies; and iv) a global vertical bending mode was identified after the bridge rehabilitation, which was confirmed by the numerical analysis and ascribed to the blocking of the bolt-bars. Regarding Luiz I Bridge these are the specific conclusions: i) measured natural frequencies for the same vibration modes have experienced small changes, presenting a slight tendency to increase after the bridge rehabilitation; ii) with respect to the mode shapes the deformed configuration of the upper deck became less smooth near the steel piers for the transverse vibration modes, particularly as the order increases; iii) constraints on the longitudinal displacements of both decks decisively contribute to control the natural frequencies of specific vibration modes, which has allowed a simple and reliable calibration procedure to quantify the stiffness constants adopted in the longitudinal springs of the models supports; and iv) the light-weight concrete slab of the decks clearly stiffen the bridge and the new floor system and steel grid of the upper deck have comparatively a smaller impact in the natural frequencies.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the Portuguese Scientific Foundation (FCT-MCES) to the first two authors through the grants with the references SFRH/BD/13138/2003 and SFRH/BD/24423/2005, respectively. The authors would also like to thank the bridges owners EP - Estradas de Portugal, E.P.E. and Metro do Porto, S.A. for their assistance and cooperation. Finally, the authors are grateful for the information provided by the designers of the rehabilitation projects, GEG – Gabinete de Estruturas e Geotecnia and GRID – Consultas, Estudos e Projectos de Engenharia, as well as the viability study of the Luiz I Bridge made available by the Instituto da Construção (FEUP).

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