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1	The role of cable forces in the model updating of cable-stayed bridges
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12 ABSTRACT

This paper presents and discusses the feasibility of complete model updating of cable-stayed 13 bridges using experimental estimates of the cable forces and modal parameters. The procedure 14 is applied to the model updating of a curved cable-stayed bridge in Venice (Italy). Conventional 15 optimization problems of mass and stiffness using ambient vibration data are prone to ill-posedness 16 and ill-conditioning. Generally, the scholar must assume one of the two to achieve a trustworthy 17 optimization. This paper demonstrates that it is possible to assess a large set of parameters 18 affecting the mass and stiffness of a cable-stayed bridge following a step-wise procedure based on 19 ambient vibration tests. Preliminary variance-based sensitivity analysis supports the reduction in 20 the number of parameters to be calibrated. Then, the selected parameters are tuned using a meta-21 heuristic optimization algorithm. In the considered case study, the sensitivity analyses highlight 22 the significance of the following: the concrete mass, the vertical stiffness of the bearings, and the 23

²⁴ concrete Young's modulus of the deck and the tower. However, optimizing all the unknowns using
²⁵ a single objective function does not lead to optima within the search domain. Therefore, the authors
²⁶ show that a three-step optimization is required in the considered case study to achieve convergence
²⁷ within the parameters space. As a result, all the twelve modes of the calibrated model perfectly
²⁸ match the experimental ones, with the Modal Assurance Criterion (MAC) higher than 0.9. In
²⁹ addition, the cable forces of the calibrated model present a good match with the experimental ones,
³⁰ with an average percentage error equal to 11%.

31 INTRODUCTION

Finite element (FE) updating of large-scale structures is a challenging task using traditional 32 optimization methods (Friswell and Mottershead 1995). Classical parametric FE updating is based 33 on comparing experimental and simulated modal parameters. The main drawback of parametric 34 optimization using the modal parameters extracted from ambient vibration data is indeterminacy 35 when assessing the stiffness, and mass matrix simultaneously (Simoen et al. 2015). Operational 36 modal analysis (OMA) returns unscaled mode shapes, which cannot be used to simultaneously 37 estimate the elastic and inertial features of a structural model (Rainieri and Fabbrocino 2014). The 38 optimization problem would be ill-posed and requires the selection of either the inertia or stiffness 39 as unknown parameters. 40

However, there can be exceptions, and in some instances, ambient vibration data could be used to update the inertial and structural stiffness features reliably. It could be the case of cablestayed bridges (Zárate and Caicedo 2008), where an experimental estimate of the cables' natural frequencies and the deck's modal parameters can be obtained. The natural frequencies of the cables can return an indirect assessment of the cable forces using suitable mechanical models of the cable dynamics (Irvine 1981; Zhao et al. 2020).

⁴⁷ Mechanical intuition suggests that the cable forces, if the cable almost behaves like a linear taut ⁴⁸ string (Graff 2012), largely depend on the mass of the suspended structure. At the same time, the ⁴⁹ modal parameters are affected by both the mass and the structural stiffness. Therefore, a step-wise ⁵⁰ model updating could be carried out in cable-stayed bridges using the cable forces and the modal ⁵¹ parameters, respectively.

Optimizing parameters affecting both the inertial and stiffness features represents a problem 52 analogous to estimating scaling factors of mode shapes in operational modal analysis. Since the 53 forces in OMA are unknown, the mode shapes cannot be mass normalized, and only the un-scaled 54 mode shapes can be determined for each mode (Parloo et al. 2001; Parloo et al. 2002; Parloo et al. 55 2003; Parloo et al. 2005; Brincker and Andersen 2003). Therefore, scaling the mode shapes to the 56 mass matrix would allow a well-posed optimization problem when estimating the mass and stiffness 57 matrices from OMA. Since the majority of existing structures are not cable-stayed, several scholars 58 devised alternative and more general strategies to get information about the structural mass based 59 on modifications of the structure by changing the stiffness and the mass during OMA (Bernal and 60 Gunes 2002; Bernal 2004; Brandt et al. 2019). The most known scaling techniques is the mass-61 change method (Parloo et al. 2001; Parloo et al. 2002; Brincker and Andersen 2003; Lopez Aenlle 62 et al. 2005), although there are another approaches based on exogenous inputs (López Aenlle 63 et al. 2007; Parloo et al. 2005) or moving loads (Tian et al. 2019; Tian et al. 2021; Sheibani 64 and Ghorbani-Tanha 2021). For instance, the mass-change method involves attaching masses to 65 the points of the structure where the mode shapes of the unmodified structure are known. The 66 mass-change method has also been used in FE-model updating, where the modal parameters of 67 the modified structure are used as additional information to calibrate the mass, and the stiffness 68 matrices of the system (Shahverdi et al. 2005). This approach has been validated by experimental 69 testing of lab-tested structure scale models (Lopez Aenlle et al. 2005), bridges (Parloo et al. 2005), 70 buildings (Brincker et al. 2004), and mechanical systems (Parloo et al. 2001). In cable-stayed 71 bridges, the additional information required for a complete FE model updating is provided by the 72 cable forces, determined from their vibration response by assuming a specific dynamic response 73 model. Achieving an almost complete model updating is particularly relevant for structural health 74 monitoring (Arangio and Bontempi 2015; Li and Ou 2016) and in particular for the identification 75 of damage (Talebinejad et al. 2011; Babajanian Bisheh et al. 2019; Ni et al. 2008). 76

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The cable-stayed bridge selected for the current analysis is the bridge of Porto Marghera

in Venice (Italy) (Briseghella et al. 2021; Gentile and Siviero 2007; De Miranda et al. 2010; 78 Briseghella et al. 2010; Fa et al. 2016) to prove the feasibility of the proposed approach: assessing 79 an almost complete FE model updating from ambient vibration data. In the first step, the authors 80 carried out a variance-based sensitivity analysis (Saltelli and Sobol' 1995) of the cable forces to 81 the bridge's significant inertial and stiffness parameters. The analyses proved that cable forces are 82 mainly affected by the mass of the deck and the support deformability. This evidence enabled 83 uncoupling the model updating in separate phases. There are multiple examples of FE model 84 updating of cable-stayed bridges, but most of them are based on the sole experimental modal 85 parameters (Zhang et al. 2001; Brownjohn and Xia 2000; Xiao et al. 2015; Zhu et al. 2015; Bursi 86 et al. 2014; Lin et al. 2020a; Lin et al. 2020b; Ding and Li 2008; Pinqi and Brownjohn 2003; 87 Park et al. 2015; Park et al. 2012; Ding and Li 2008). However, the FE updating of the cable 88 forces is also a crucial task. As remarked by (Martins et al. 2020), 80% of research studies on 89 cable-stayed bridges focus on cable forces optimization and control (Correia et al. 2020; Ferreira 90 and Simoes 2011; Feng et al. 2022; Kim and Adeli 2005). Still, there are few pieces of research 91 on the experimental evaluation of the cable forces (Cho et al. 2010; Haji Agha Mohammad Zarbaf 92 et al. 2017; Nazarian et al. 2016; Feng et al. 2017; Haji Agha Mohammad Zarbaf et al. 2017), 93 and a fewer on the model updating using both the modal parameters and the estimated cable forces 94 (Hua et al. 2009). Nonetheless, the experimental assessment of the cable forces and the subsequent 95 calibration of the forces predicted by the FE model is imperative to achieve a reliable prediction of 96 the structural response. 97

Cable-stayed bridges, especially those built in the last decade, possess peculiar aesthetic and structural features. Cable-stayed bridges play a crucial role in infrastructure networks (Virlogeux 1999). However, they also characterize the urban landscape with their unique structural shape (Wilson and Liu 1991; Astaneh-Asl and Black 2001; Ni et al. 2019). There are multiple examples of cable-stayed bridges, but only some of them possess curved decks. The most known examples of cable-stayed bridges with a curved deck are: the Rhine bridge near Schaffhausen in Switzerland (Deger et al. 1996), the Safti Link bridge in Singapore (Brownjohn and Xia 2000), the twin bridges

close to the Milan-Malpensa airport (Gentile and Martinez Y Cabrera 2004), the Katsushika bridge 105 in Japan (Siringoringo and Fujino 2007), the Térénez bridge in France (Halpern and Billington 106 2013), the Yabegawa River Bridge (Kim and Lee 2012) and the Ponte del Mare bridge in Pescara 107 (Italy) (Bursi et al. 2014; De Miranda et al. 2010). The deck curvature, despite its appreciated 108 aesthetic peculiarities (Bonelli et al. 2010), adds complications in the construction phase and 109 when assessing the actual structural behavior compared to cable-stayed bridges with a straight deck 110 (Daniell and Macdonald 2007; Wen et al. 2016; Zhang et al. 2017; Martins et al. 2020). The selected 111 bride also possesses an inclined tower to reduce the eccentricities of the cable forces. Therefore, 112 especially for curved cable-stayed bridges, the experimental modal analysis is a determinant aspect 113 for assessing the reliability of the structural model. 114

In (Briseghella et al. 2021), the authors discussed the dynamic characteristics of the bridge of Porto Marghera. Briseghella et al. (Briseghella et al. 2021) used the bridge's finite element (FE) model, calibrated to the experimental modal parameters, to assess the effect of its geometric configurations on its dynamic response. Specifically, the analyses aimed at determining the sensitivity of the natural frequencies to cable arrangement, deck curvature, and cross-section of the tower.

This paper represents an extension of the previous research (Briseghella et al. 2021) to discuss the feasibility of simultaneously calibrating an extended set of parameters affecting both the mass and stiffness features from ambient vibration data. Dynamic identification and FE model updating is a standard practice in structural engineering (Sehgal and Kumar 2016). However, the peculiarity of the Porto Marghera bridge also adds originality to this piece of research. The main aspects of novelty and originality are:

- 126 127
- Assessing the sensitivity of the cable forces and modal parameters to the structural parameters using a variance-based sensitivity analysis (Asgari et al. 2013).
- Proposing a step-wise procedure for the model updating of cable-stayed bridges using
 ambient vibration data and testing the updating procedure on the bridge of Porto-Marghera
 using two meta-heuristic optimization algorithms: the particle swarm optimization (PSO)
 and the differential evolution (DE).

5

- The authors found a significant discrepancy between the experimental and simulated cable
 forces in the preliminary FE model. The paper discusses the role of the bearing and tower
 stiffness in affecting the cable forces. These effects cannot be appreciated from the modal
 parameters of the deck.
- 136

• Highlighting the numerical issues related to the simultaneous identification of all the parameters from a single or multi-objective optimization problem. (Brincker et al. 2000).

The paper has the following organization. The second section briefly introduces the case study and the outcomes of operational modal analysis. The third section describes the FE model of the bridge and preliminary analysis to show the initial discrepancies with the experimental data. The fourth section discusses the sensitivity analysis of the cable forces to the inertial and stiffness parameters. The fifth section shows the results of the variance-based sensitivity analysis of the natural frequencies of the deck's stiffness in terms of Young's moduli of steel and concrete. The last section presents the global optimization results based on a step-wise approach.

145 **PROBLEM FORMULATION**

Ambient vibration tests of cable-stayed bridges can be used to obtain estimates of the modal 146 parameters of the deck, the tower, and the stay-cables, which are the main components of a 147 cable-stayed bridge. In OMA, the forces are unknown, therefore, the mode shapes cannot be mass 148 normalized, and only the unscaled mode shapes can be determined for each mode (Parloo et al. 2001; 149 Parloo et al. 2002; Parloo et al. 2003; Parloo et al. 2005; Brincker and Andersen 2003). However, in 150 cable-stayed bridges, the forces can be effortlessly estimated from elementary mechanical models 151 of the cables (Irvine 1981). Therefore, cable-stayed bridges represent a peculiar case where ambient 152 vibration tests yield both the modal parameters and some forces acting in the structure. This paper 153 shows that the augmented information due to cable forces might allow the complete model updating 154 of the bridge model in terms of inertial and stiffness parameters. In conventional model updating 155 from OMA, the scholar must select either the mass or stiffness matrix to be updated to avoid an 156 ill-posed mathematical problem. As illustrated in Fig.1, the augmented information compared to 157

traditional OMA allows the formulation of two objective functions in terms of cable forces and
 modal parameters, respectively. Fig.1 illustrates the procedure followed in this paper to understand
 whether the cable forces and the modal parameters can be used to achieve an almost complete FE
 model updating.

The authors are aware that each cable-stayed bridge is a stand-alone case. Therefore, it is 162 challenging to generalize a model update using the cable forces, natural frequencies, and unscaled 163 modal parameters. However, although each cable-stayed bridge might deserve minor adjustments 164 to the procedure, this paper proves that it is possible to calibrate most of the parameters of the FE 165 model, affecting both the mass and stiffness matrices. Specifically, except for the cables, each cable-166 stayed bridge consist of three main constituents: the tower, the deck, and the bearings. Intuition 167 suggests the stiffness of the tower, bearings, and deck, and the mass of the deck influence both the 168 modal parameters and the cable forces. It also indicates that the tower mass little affects the modal 169 parameters and the stay-cables being a self-sustained structure. 170

Rigorously, a variance-based sensitivity analysis can highlight the most significant parameters 171 affecting each cable force and modal parameter. As discussed in the body of the paper, the outputs 172 of the sensitivity analysis prove that it is challenging to estimate all parameters at once since 173 some parameters are more influential than others on a cable force or modal parameter. Therefore, 174 the optimization problem formulation cannot be generalized and deserves a case-by-case analysis. 175 However, as illustrated in Fig.1, the outputs from a sensitivity analysis and OMA can be used to 176 select the parameters and properly formulate an optimization problem. Eq.(1) displays the general 177 expression for the optimization problem, where x collects all the involved parameters affecting the 178 deck, tower, and bearings. 179

$$\hat{\boldsymbol{x}} = \min_{\boldsymbol{x} \in \Omega} \{ \boldsymbol{g}(\boldsymbol{x}) \}$$
(1)

where \hat{x} and x collect the optimized and unknown parameters respectively, g the objective functions and Ω is the input space parameters. The objective function can be written by manipulating two ¹⁸³ functions containing the squared difference between the two types of input parameters.

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$$\boldsymbol{g}(\boldsymbol{x}) = \begin{cases} \sum_{i=1}^{n_c} \left(\frac{T_i^m - T_i^c}{T_i^m}\right)^2, & \text{Cable forces} \\ \sum_{i=2}^{n_m} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m}\right)^2 + \sum_{i=2}^{n_m} \left(1 - \text{diag}(\text{MAC}(\Phi_i^m, \Phi_i^c))\right), & \text{Modal parameters} \end{cases}$$
(2)

Where T_i^m and T_i^c are the measured and calculated cable forces, ω_i^m and ω_i^c are the measured and 185 calculated natural pulsations, MAC is the Modal Assurance Criterion, Φ_i^m , and Φ_i^c are the measured 186 and computed mode shapes, n_c is the number of cables, while n_m is the number of modes. The 187 sensitivity analysis might indicate that some experimental parameters should be excluded from 188 the objective function. Following a common approach in FE model updating, (Friswell and 189 Mottershead 1995), meta-heuristic algorithms are used to solve the optimization. These techniques 190 are mainly based on mimicking natural phenomena with simple iterative stochastic search rules in 191 a phenomenological perspective, without a solid mathematical framework ensuring convergence to 192 the global optima and its existence (Martí et al. 2018). Due to their intrinsic nature, does not exist 193 a single unique method since the No-Free Lunch theorem (Wolpert and Macready 1997) affirms 194 that there is no ideal algorithm to deal with any problem. However, their successful capability to 195 handle complex problems without requiring any gradient-based information often represents the 196 only means to deal with such situations (Martí et al. 2018). Indeed, meta-heuristic algorithms can 197 accomplish the solution estimate of the optimal Pareto front for hard computational and multi-198 objective problems (Jones et al. 2002). 199

The general problem in Equation 2 is presented as a multiobjective optimization, where two objective functions to are optimized simultaneously. No single solution exists for a nontrivial multiobjective optimization problem that simultaneously optimizes each objective. The goal may be to find a representative set of Pareto optimal solutions and/or quantify the trade-offs in satisfying the different objectives. To simplify the problem, the authors conducted two separate singleobjective optimizations in the following sections, as shown in Equations 5 and 8.

In this paper, the authors attempt to achieve the almost complete model updating of a cable-

stayed bridge in Porto Marghera, selected as a case study, using experimental estimates of cable
forces and modal parameters. The sensitivity analysis of each experimental parameter will support
the formulation of the objective functions. However, despite multiple attempts, the optimization
does not achieve convergence when a single objective function comprising all unknowns and
experimental data is used. Despite several attempts, the authors will find that optimization is
successful only if the optimization is split into three companion optimizations.

One for the tower stiffness using selected cable forces and modal parameters, one for the deck stiffness using selected modal parameters, and another for the vertical stiffness of the bearings using the deck mass and selected cable forces. By bearing, the authors intend the bridge component between the abutment and the deck. The selection of the cable forces and modal parameters for each optimization is based on the results of the sensitivity analysis.

This paper reveals the information obtained from a sensitivity analysis is necessary to achieve a mindful formulation of the objective function. Each cable stayed-bridge is a stand-alone case. However, the investigations prove that, under certain choices of objective functions, the problem can be considered well-posed and leads to the optimal set of parameters.

222 CASE STUDY

This section briefly describes the bridge and the experimental tests for characterizing its dynamicresponse.

Bridge description

The Porto Marghera bridge, connecting the city of Mestre to the Commercial Harbor of Venice-226 Marghera, Italy (Fig.2), has a total length of 387m, divided into six spans (42m + 105m + 126m +227 30 + 42m + 42m). The first spans present a straight alignment, and the others a curved one with a 228 175m radius. Fig.3 shows the plan, elevation and typical cross-sections of the deck (De Miranda and 229 Gnecchi-Ruscone 2010). The bridge is characterized by an inclined L-shape prestressed concrete 230 tower, a single set of cables with a spatial arrangement and a curved steel-concrete composite 231 deck (Briseghella et al. 2010). The two main spans have a cable-stayed structure with the stays 232 arranged on a single plane, attached by the cross-section center. The bridge has two traffic lanes and 233

pedestrian walkways with a 23.70m total width. The deck consists of a composite concrete-steel 234 continuous girder embracing all six spans. There are two cross-sections of the deck, depicted 235 in Fig.3. The first one, adopted by the end spans, consists of four double-T steel girders, while 236 the second one, by the central spans, consists of two outer double-T steel girders and one central 237 girder with a box section. Transverse crossbeams made by double-T beams spaced every 5.25m 238 stiffen the girder. Steel girders and crossbeams have a 1.90m height and are connected to a cast-239 in-place concrete slab with a 25-27 cm thickness. The cast-in-place prestressed concrete inclined 240 tower represents the bridge landmark and played a determining role in the conceptual and executive 241 design of the bridge. The tower has about 75m in height, and a triangular cross-section characterizes 242 its geometric layout. The cross-section base enlarges upward to provide a more suitable anchorage 243 for the stays. The tower prestressing aimed at reducing the dead loads' eccentricity due to the curved 244 deck layout. Despite the classic static scheme conception, typical of cable-stayed bridges with a 245 central tower, numerous elements present a considerable architectonic impact and originality: the 246 curvilinear layout of the suspended deck, the suspension scheme with a central curtain of stays, 247 and the inclined tower with variable cross-section. Furthermore, the remarkable size of the deck 248 made in the open profile (23.7m) and the mentioned bridge singularities supported the dynamic 249 identification of the bridge for the experimental assessment of its dynamic response. 250

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Dynamic characterization of the deck

The Laboratory of Vibrations and Dynamic Monitoring of Structures of Politecnico di Milano 252 carried out the dynamic identification of the Porto Marghera bridge in two experimental campaigns 253 during Autumn 2010 and Spring 2011 (Talebinejad et al. 2011; Yang et al. 2018). The two 254 studies identified the modal parameters of the bridge and the natural frequencies and damping 255 of the cable stays. The experimental modal analysis of the bridge was carried out using the 256 Frequency Domain Decomposition (Brincker et al. 2000). The natural frequencies of the stays 257 derived from the direct inspection of the auto spectra of the recorded signals (Bendat 1993). The 258 details of the experimental tests and the results of dynamic identification are thoroughly discussed 259 in (Briseghella et al. 2021). Concisely, the analysis identifies 12 and 11 modes in the 0-6Hz range 260

during the experimental campaigns in 2010 and 2011 respectively. The authors selected the 11 experimental modes detected in 2011 for the sensitivity analysis and the following optimization. Tab.1 shows the experimental modal parameters. Fig.4 shows a representation of the mode shapes. The pictures include the tower and the stays, although the displayed modal deformations refer to the sole deck. As remarked in (Briseghella et al. 2021), the bending-torsional modes disavow the assumption of transversal non-deformability of the profile.

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Dynamic characterization of the cables

Each cable stay was instrumented with a single sensor, placed at approximately 9.0m to the road surface. Additional details about the experimental setup are presented in (Briseghella et al. 2021; Zhao et al. 2020). Fig.6 and Tab.2 display the natural frequencies of the 18 stay cables identified in 2010 and 2011. Each figure reports the results of two symmetric stay cables, following the numbering in Fig.5. Fig.6 plots the natural frequencies and interpolating line n- f_n , where n is the mode order, and f_n is the associated natural frequency.

The interpolating line n- f_n displayed in Fig.6 does not exhibit a significant discrepancy to the single natural frequencies. Therefore, a simplified mechanical model of a fixed-fixed vibrating string can be used to derive the cable forces. The *n*-th natural frequency of a linear fixed-fixed string can be written as:

$$f_n = \frac{1}{2L} \left(\frac{T}{\rho}\right)^{0.5} \tag{3}$$

where *L* is the cable length, *T* is the cable force, ρ is the mass per unit of length of the cable. By assuming *L*, and ρ , the cable force can be estimated from the interpolating line *n*-*f_n*. Specifically, the cable force can be derived from the slope of the interpolating line as follows (Irvine 1981; Caetano et al. 2007):

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$$T = \rho \left(2L\frac{\partial f_n}{\partial n}\right)^2 \tag{4}$$

where $\frac{\partial f_n}{\partial n}$ is the slope of the interpolating lines shown in Fig.6. Tab.3 and 4 list the estimated cable forces from the two experimental campaigns. As remarked in the original technical report, the recent publication (Briseghella et al. 2021) and past research on the cable force identification (Gentile and ²⁸⁷ Cabboi 2015), there is a certain discrepancy between the forces of two symmetric cables. However, ²⁸⁸ there is a minor deviation to the n- f_n interpolation line. Therefore, the cable bending stiffness, sag ²⁸⁹ extensibility, and intermediate springs do not play a significant role in affecting the cable forces ²⁹⁰ (Mehrabi and Tabatabai 1998). The elementary model of a taut string can be considered valid ²⁹¹ in this case study and the observed differences between the experimental and numerical values ²⁹² reasonably depend on the structural parameters of the deck, tower, and supports. In the following ²⁹³ sections, the authors will attempt to understand the possible reasons for the detected differences.

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FINITE ELEMENT MODELLING OF THE BRIDGE AND PRELIMINARY ANALYSES

The FE model is developed in SAP200 and consists of 8014 nodes and 6600 elements (namely 295 2946 beams, 18 trusses, and 3636 solid elements), as depicted in Fig.7. The model is linear and 296 does not reproduce any geometrical or mechanical non-linearity (Adeli and Zhang 1995; Song 297 et al. 2007). In detail, four-node shell elements reproduce the concrete slab, while solid elements 298 the tower. Rigid links, without mass, connected the concrete slab and the grid of steel stringers 299 and transverse cross-beams. Additionally, rigid links reproduced geometrical offsets between the 300 structural members and strut-and-tie bracings of the deck. The piers are modeled by 3D beam 301 elements while the cables are modeled by cable elements. The bearings, not included in the 302 original bridge model (Briseghella et al. 2021), are modeled by vertical linear spring. The model 303 reproduces the curvature of the deck and the tower inclination. The initial values for the material 304 properties are the following. The weight per unit volume and the Poisson's ratio of the concrete 305 were assumed to be 25.0 kN/m³ and 0.2, respectively. An additional weight equal to 1 kN/m² 306 represented the deck slab, including the asphalt pavement and walkways. Young's modulus and 307 steel weight was assumed to be 205 GPa and 785 kN/m³ respectively. 308

Fig.8, Tab.6 and Tab.5 highlight the starting point of the model updating. Fig.8 plots all the eleven modes with an indication of the experimental and numerical natural frequencies and the MAC before calibration. Analogously, Tab.5 compares the experimental estimates of the cable forces and the FE model predictions before the updating. The numerical estimates of the cable forces (the cables are modeled as cable elements in SAP 2000) were obtained from static analysis

under dead loads. The experimental forces were measured when the bridge was closed to vehicular 314 traffic and only dead loads were acting on it. The authors do not adopt cable models with geometric 315 nonlinearity for two main reasons. (i) The experimental estimates of the cable forces exhibit an 316 almost exact agreement with the natural frequencies obtained from the classical linear model for 317 a taut string. This observation proves that a nonlinear cable model might not be necessary for the 318 current research objective. (ii) Secondarily, the authors are only considering ambient vibration 319 tests, where the vibration amplitude of the cables is so low that geometric nonlinearities do not 320 manifest. There is a significant gap in the starting point in terms of modal parameters and cable 321 forces. The modal parameters exhibit an acceptable agreement before optimization. The cable 322 forces are enormously biased. Therefore, a sensitivity analysis of the modeling parameters and 323 cable forces is required to understand which parameters need to be updated. 324

Tab.7 shows the mass participation ratios of the considered eleven modes. The first six modes present quite relevant mass participating ratios, thus producing global mode shapes. On the contrary, the remaining modes approach zero percent of mass ratios, thus evidencing local modes as depicted in Fig.4. It is worth noting that modes 1, 3, 4, and 10 are clearly characterized by a single-direction mobilized mass, whereas modes 2, 5, and 6 are characterized by mixed mass participation ratios.

331 SENSITIVITY ANALYSIS OF THE CABLE FORCES

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Before estimating the optimum values of the modeling parameters, sensitivity analyses provided a quantitative assessment of their effect on the chosen objective function and the mass and stiffness parameters. The authors chose as objective functions the 18 force values of the cables (nine on the Mestre side and nine on the Venice side) and an error function defined as the difference between the estimated and the numerical cable forces:

$$g_1 = \sum_{i=1}^{n_c} \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2$$
(5)

where g_1 is the cost function, T_i^m the measured cable force, T_i^c the simulated cable force.

The analyses assessed the sensitivity of the objective functions to the parameters influencing the cable forces. The main parts of a cable-stayed bridge are the deck, the cables, the tower, and the supports. Each part possesses mass and stiffness properties. The cable forces are optimized by the designer to ensure the meeting of given design criteria generally in terms of stress and/or displacement. The physical parameters mainly affecting the actual value of cable forces are the mass and stiffness of the deck and the stiffness of the tower and the supports.

The tower's mass and bearings are self-sustained and have not been included. The geometry and material properties of the cables are known with high precision. Therefore, the authors will assume that they can be assumed as known in the sensitivity assessment. Young's modulus of steel is also known with significant accuracy and has not been included in the sensitivity assessment. The above considerations led to selection of five parameters representative of the deck's mass and stiffness, the tower's stiffness, and the bearings.

1. The Young's modulus of the tower $(E_{c,t})$. The cable forces might be significantly reduced if the tower is more deformable than expected by design. If so, the tower does not behave as almost fixed support for the cables. Therefore, Young's modulus of concrete has been chosen as a synthetic representation of the tower deformability. The bounds [30, 50] GPa were assumed in the analyses.

2. The mass of the steel deck (ρ_s). The FE model accurately reproduces the geometry of the 356 steel deck by using finite elements. However, the approximation related to finite elements 357 might lead to an over or underestimation of the structural mass. Therefore, the weight per 358 unit of volume of steel has been chosen to represent the mass of the steel deck. Indeed, ρ_s 359 should be a known parameter since steel is manufactured in a workshop. As envisioned, the 360 analysis shows that the parameter is negligible and can be excluded from the updating. The 361 bounds [75, 80] kN/m³ were assumed in the analyses. The variability does not represent 362 the possible variation of the steel mass but comprises the possible uncertainties related to 363 the steel deck modeling. 364

365

3. The mass of the concrete deck (ρ_c). The FE model attempts to reproduce the geometry of the

concrete deck accurately. However, it is conventionally assumed that the concrete specific 366 weight is equal to 25kN/m³. Still, the weight of a reinforced concrete slab depends on the 367 proportions of concrete and steel, which might cause variations of the conventional value 368 assumed in calculations. Additionally, the uncertainties in the actual size of the concrete 369 deck might propagate to the cable forces. Therefore, the authors selected the specific weight 370 of concrete as a synthetic representative of the mass of the concrete deck. The uncertainties 371 in the concrete deck's actual weight may depend on both the slab size and the specific weight 372 of the concrete. The bounds [24, 30] kN/m³ were assumed in the analyses. 373

4. The Young's modulus of the concrete deck $(E_{c,d})$. The paper proposes a two-stage FE model update for cable-stayed bridges. In the first stage, the mass of the deck is estimated from the experimental values of the cable forces. In the second stage, the deformability of the deck is estimated from the experimental modal parameters. This approach works if the cable forces do not significantly depend on the deck stiffness. Therefore, Young's modulus of the concrete deck is used to prove that the cable forces are not affected by the deck stiffness. The bounds [30, 50] GPa were assumed in the analyses.

5. The vertical stiffness of the bearings (k_a) . The vertical stiffness of the bearings can be a crucial parameter affecting the cable forces. For example, the cable forces might increase significantly if the bearings are too deformable. The bounds [10, 500] kN/mm were assumed in the analyses.

It must be remarked that the bounds of the parameters have been chosen so that the minimum 385 never falls close to the bounds. The authors conducted several trial optimizations by progressively 386 increasing the difference between the lower and upper bounds, initially close to the expected 387 values corresponding to the physical variables the modeling parameters represent. Beyond a given 388 increment of the bounds, the optimum parameters did not change and always fell within them. 389 Therefore, after several attempts, the authors selected the limits specified above for running the 390 optimization discussed in this paper. The boundaries which requested more attention and effort were 391 those for the bearing stiffness, which are those characterized by the highest uncertainty since there 392

was no preliminary experimental estimate of them. Therefore, the authors centered the boundaries 393 at the nominal value of the bearing stiffness according to the producer and then increased the 394 boundaries so that the minimum clearly fell within them. The analysis allowed decomposing the 395 variance of the model's output (objective function and natural frequencies) into fractions that can 396 be attributed to the chosen mechanical parameters (Pasca et al. 2021). The first step was setting 397 the inputs sampling range and generating the model inputs according to Saltelli's sampling scheme 398 (Saisana et al. 2005). $(N \cdot (2D + 2))$ model inputs were generated, where N = 100 is the number of 399 samples, and D = 2 is the number of input parameters). After running all the model inputs, the first 400 order was calculated. The authors computed the first-order sensitivity indexes (S_1) , which do not 401 consider interactions among input variables (Sobol' 1990). Instead, they contribute to the output 402 variance of the chosen objective function of a given modeling parameter (Young's modulus, mass 403 density, e.g.) (Aloisio et al. 2020; Aloisio et al. 2021; Aloisio et al. 2022a; Aloisio et al. 2022b). 404 S_1 measures the effect of varying each parameter alone, averaged over variations of the other input 405 parameters. Theoretically, the summation of all indexes is one. However, the sensitivity analysis 406 is based on a Montecarlo approach. Therefore, the sum of all indexes tends to be one, but it is not 407 precisely one due to the statistical approach followed for their estimation. Tab.8,9 list the sensitivity 408 indicators of the objective function and cable forces respectively, where the rows refer to different 409 parameters. 410

The objective function in Eq.(3), later used for the optimization of cable forces, comprises the 411 modeling errors of all cables, although each cable is more affected by a specific set of variables 412 among the chosen five. The results show that the most significant variable is the vertical stiffness 413 of the bearings. In the original model developed by the authors (Briseghella et al. 2021), the 414 bearings were assumed as fixed supports since their stiffness did not cause a significant effect on the 415 modal parameters. Conversely, the cable forces are significantly influenced by k_a , as proved by the 416 sensitivity indicator, reaching approximately 94%. In this preliminary phase, the authors assumed 417 a range of stiffness to have a maximum vertical displacement of 1mm. The other significant 418 parameters, with sensitivity indicators of approximately 5%, are Young's tower modulus and the 419

concrete deck's mass. The authors expected the importance of the two parameters, compared to the
steel mass and Young's modulus of the concrete deck, which have a minor influence on the cable
forces. The results in Tab.8 indicate that the vertical stiffness will be the most influential parameter
in the FE updating of the cable forces.

Parallelly, Tab.9 displays the contribution of each of the five parameters to the force values of the nine cables on the Mestre and Venice sides. The results are almost identical for the two sides, despite minor discrepancies. The tower deformability influences the force in the shortest cable close to the tower. The sensitivity indicator reaches almost 85%. The mass of the concrete deck influences its value by nearly 15%, while the bearings, quite distant from the cables, do not sensibly affect the force value.

The sensitivity indicators start modifying in favor of the concrete deck's mass and the subsequent cables' bearing stiffness. The middle cables exhibit the highest sensitivity to the deck mass, reaching more than 90% in some cases. Conversely, the cables closest to the bearings (especially the ultimate three) exhibit the highest sensitivity to the bearing supports, close to 90%.

In conclusion, the outcomes of the sensitivity analyses on the single cable forces highlight three trends for the $E_{c,t}$, ρ_c , and k_a . First, the tower deformability exhibits the highest effects on the shortest cable. Then its effects decrease to the ninth cable with an almost null effect. The mass of the concrete deck shows a sensitivity growth starting from the shortest to the middle cables. Then, the effects decrease for the cables close to the supports. The sensitivity indicators to the bearings' stiffness grow to a maximum for those cables closest to the supports.

Fig.9,10 and 11 provide a graphical illustration of the outcomes of the sensitivity analysis by means of a scatter plot of the objective functions in the considered space of parameters. The representation has been limited to $E_{c,t}$, ρ_c , and k_a , which play the most significant role.

Fig.9 shows two representations of the scatter plot of the objective function in Eq.(3). The objective function reduces significantly if the k_a is lower than 100 kN/mm. For higher values of k_a , the objective function tends to stabilize. This effect is reasonable. Due to a lower deformation, the supports tend to behave rigidly and play a minor role in the cable forces. The alternate dots' color

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evidences the presence of a maximum for the ρ_c value.

Fig.10,11 illustrate the effects of $E_{c,t}$, ρ_c and k_a on each cable force on the Mestre and Venice 448 sides. The plots confirm the results in Tab.9. The shortest cables exhibit a prevalent dependence 449 on the stiffness of the tower. If Young's modulus $E_{c,t}$ grows to values higher than 30MPa, the 450 objective function reduces. Conversely, the objective function stabilizes at a higher value as the 451 tower's stiffness lowers. If the tower does not deform, it behaves as a rigid support for the cables. 452 The second and third cables start manifesting an inversion of the objective function dependence 453 since the weight of the concrete deck leads to a higher dispersion of the dots towards lower values 454 of the objective function. Starting from the fourth cable, the dependence of the objective function 455 on k_a starts displaying. Specifically, moving from the fourth to the ninth cable, the surfaces tend to 456 reduce in the vertical scatter related to the concrete specific weight and exhibit a clear dependence 457 on k_a . Expressly, the curves referred to as the ninth cables have a nonlinear trend, where the 458 objective function reduces for lower values of the bearing stiffness. 459

The plots in Fig.10,11 are proxies for assessing the role of the parameters in finding the optimal set associated with the minimum of the objective function in Eq.(3). The optimal set of parameters is associated with higher values of Young's modulus of the tower ($E_{c,t} > 30$ MPa), a lower value of the concrete mass of the deck ($\rho_c < 25$ kN/m³), and a lower value of the bearing stiffness ($k_a < 100$ kN/mm).

465 SENSITIVITY ANALYSIS OF THE MODAL PARAMETERS

To measure the distance between the estimated and the numerical modal parameters, the following objective function is used:

$$f(\mathbf{x}) = f_1(\mathbf{x}) + f_2(\mathbf{x})$$
 (6)

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$$f_1(\boldsymbol{x}) = \sum_{i=1}^{n_m} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2$$
(7)

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$$f_2(\boldsymbol{x}) = \sum_{i=1}^{n_m} \left(1 - \operatorname{diag}(\operatorname{MAC}(\Phi_i^m, \Phi_i^c)) \right)$$
(8)

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where $f(\mathbf{x})$ is the cost function, \mathbf{x} is the vector collecting all the modeling parameters, $f_1(\mathbf{x})$ and 473 $f_2(\mathbf{x})$ the cost functions in terms of natural frequencies and mode shapes, ω the natural pulsation, 474 the apex $(*)^m$ indicates a measured variable, the apex $(*)^c$ a calculated variable, Φ_i is the mode 475 shape vector, n_m is the number of modes, MAC is the Modal Assurance Criterion. 476

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The main aspects arising from the observation of the sensitivity analysis might be itemized as follows based on the results collected in Tabs.10,11,12: 479

٠ Tab.10-The vertical stiffness of the bearing is the most influential parameter in Eq.(6). 480 However, as shown in Tab.10, the highest sensitivity to the bearing stiffness is given by the 481 cost function in terms of mode shapes ($\approx 97.2\%$). Conversely, the cost function in terms 482 of natural frequencies is more affected by the mass of the deck (ρ_c), with an approximate 483 61.4% indicator. In the selected ranges of variation, the weight per volume of steel does not 484 sensibly affect the objective functions with a sensitivity indicator less than 0.1%. 485

Tab.11-The sensitivity analysis of each natural frequency confirms the minimal effect of 486 the weight per unit of volume of steel. Conversely, there are several cases where Young's 487 modulus of the tower and the deck and mass per unit of volume of concrete are more 488 influential than the bearing stiffness. This fact can be mainly observed for modes V2, 489 V3, T1, and T2. Specifically, V2 exhibits a marked dependence on $E_{c,t}$ (\approx 49.6%), ρ_c 490 ($\approx 25.4\%$), and k_a ($\approx 74.1\%$). Modes V3, T1, and T2 also significantly depend on Young's 491 modulus of the deck, with sensitivity indicators approximately equal to 16.3%, 58.3%, 492 and 48.8%, respectively. These are the only cases where Young's modulus of the deck 493 is influential. Therefore, V3, T1, and T2 are the only modes that can be ideally used to 494 estimate Young's modulus of the deck. Except for the mentioned modes, the other modes 495 exhibit a prevalent dependence on the bearing stiffness. 496

Tab.12-The results in natural frequencies are pretty similar to those in MAC. The main • 497 difference stands in the role of the mass per unit of volume of concrete. In comparison, 498 ρ_c and k_a are the most influential parameters for the natural frequencies, except for V2, 499

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500 V3, T1, and T2 modes. The sole bearing stiffness is the most significant parameter with 501 a sensitivity higher than 90%. Similarly, V2, V3, and T1 also show a clear dependence 502 on Young's modulus of the tower. In contrast, V3, T1, and T2 also depend on Young's 503 modulus of the deck.

On average, the sensitivity ranking of the selected parameters from the most influential to 504 the less is: bearing stiffness (k_a) , weight per unit of volume of concrete (ρ_c) , Young's modulus 505 of the tower $(E_{c,t})$, Young's modulus of the deck $(E_{c,d})$, and the weight per unit of volume of 506 steel. In general, the parameters are highly correlated since the sum of the sensitivity indicators 507 in Tabs.10,11,12 is much higher than 100%. This fact depends on including mass and stiffness 508 parameters in the sensitivity analysis. Figs.12-13 show selected scatter plots of the simulated data 509 as a function of the three most influential parameters, the bearing stiffness, Young's modulus of the 510 tower, and the mass per unit of volume of concrete. 511

Differently from the scatter plots of the cable forces' sensitivity analysis, the current ones manifest 512 the presence of subspaces where dots coalesce. The shape of the objective functions, the one 513 in Eq.(6) and those representatives of the frequency and MAC contributions stand on the same 514 hyper-surface. They all prove a lowering of the objective function as the bearing stiffness rises and 515 the concrete mass lowers. Other aspects cannot be interpreted from a direct inspection of the scatter 516 plots of the three objective functions in Fig.12. The dots associated with the same realizations but 517 corresponding to each mode aggregate peculiarly, exhibiting discontinuities like for V3 and VT1, 518 local minima like for M1 and T2, and stationary regions where the variation of the parameters is 519 not influential. 520

521 Discussion

The selected plots show the complexity of a possible model update driven by the modal parameters. There are two main reasons. (i) The preliminary model of the bridge without updating already exhibits an excellent agreement with experimental data. Therefore, the parameter calibration should lead to the near identity between the experimental and numerical modal parameters. However, the model updating using both the mass and stiffness parameters would be indeterminate,
and the scholar should arbitrarily assume one parameter possibly associated with lower uncertainty.
(ii) The presence of subspaces and discontinuities in the plots of either the natural frequency or the
mode shape prove the possible limits of a meta-heuristic optimization algorithm. The optimization
outcome depends on the subspace where the algorithm might fall in the search process. Therefore,
a FE model updating using the cable forces has several advantages compared to one based on the
modal parameters.

The simulated cable forces are in lousy agreement with the experimental ones. Therefore, a model update using the cable forces would be more valuable than one based on the modal parameters, which are already in excellent agreement. Specifically, the parameters affecting the cable forces are not highly correlated. Therefore, a lower number of parameter subsets is associated with a good matching with the experimental data. In contrast, the parameters affecting the modes are highly correlated. This fact leads to higher parameter subsets related to an excellent agreement with the experimental data.

The objective functions of cable forces have a regular trend without discontinuities. Conversely, the presence of multiple subspaces collecting the modal parameters might compromise the success of the search process of optimization algorithms.

The cable forces T1,M1 and T2,M2 can be used to estimate $E_{c,t}$ ($S_w > 50\%$), the remaining cable to estimate ρ_c ($S_w > 50\%$) and k_a ($S_w > 50\%$). The sole parameter left is $E_{c,d}$ which can be estimated from an objective function in terms of cable T1, and mode shapes excluding the second one, mainly affecting the tower deformability.

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548 **FE MODEL UPDATING**

⁵⁴⁹ Multiple attempts were carried out to test the feasibility of a global optimization algorithm ⁵⁵⁰ where all the parameters are updated simultaneously. However, all the efforts were unsuccessful. ⁵⁵¹ The parameters are highly correlated, and the objective functions, especially those dependent on ⁵⁵² the modal parameters, present many local minima. Therefore, simultaneously updating all the

parameters using meta-heuristic algorithms is challenging. Furthermore, the algorithms always 553 select the lower or upper bounds of the parameter domain. Thus, the following is the unique 554 updating procedure that provided optimum parameters within the bounds and in good agreement 555 with the experimental data. The main drawback of the procedure is the assumption of specific 556 parameters in the first optimization steps. However, as proved by the sensitivity analysis, different 557 choices of the assumed parameters, namely $E_{c,t}$ and $E_{c,d}$, do not modify the optimization outputs, 558 with differences lower than 5%. Therefore, the sole successful optimization will be described and 559 discussed in this section. The optimization process is based on the following steps: 560

- 1. Optimization of ρ_c and k_a , after assuming a specific value for Young's moduli of the tower $E_{c,t}$ and the deck $E_{c,d}$. The objective function depends on all the cable forces except for T1, M1, and T2, M2.
- ⁵⁶⁴ 2. Optimization of Young's modulus of the concrete tower $(E_{c,t})$, after assuming ρ_c and k_a ⁵⁶⁵ from the first step and Young's modulus of the deck $E_{c,d}$. The objective function depends ⁵⁶⁶ on the cable forces T1, M1 and T2, M2, and the second mode shape.
- ⁵⁶⁷ 3. Optimization of Young's modulus of the concrete deck $(E_{c,d})$, after assuming ρ_c , k_a and ⁵⁶⁸ $E_{c,d}$, from the previous optimization steps.

The global optimization algorithms, the differential evolution (DE) (Storn and Price 1997) and the particle swarm optimization (PSO) (Kennedy and Eberhart 1995), are used for mutual validation. Also, to perform the model updating, the script is written in Python using SAP2000 OAPI with the Python module Scipy (to run DE) and PySwarms (Miranda 2018) (to run PSO). Since no significant difference is observed in comparing the outcomes of the two optimization algorithms, the authors will only report the results from PSO. In detail:

⁵⁷⁵ 1. **Optimization of** ρ_c **and** k_a : This optimization can be formulated as unconstrained and ⁵⁷⁶ single-objective since there is one Objective Function (OF) $g(\mathbf{x})$ to be minimized and no ⁵⁷⁷ equality or inequality constraints. The problem can be formulated as follows:

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$$\hat{x}_1 = \min_{x_1 \in \Omega_1} \{ g_1(x_1) \}$$
(9)

$$g_1(\boldsymbol{x}_1) = \sum_{i=3}^{9} \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2 \, \boldsymbol{x}_1 = \{ \rho_c, k_a \}^T, \, \hat{E}_{c,t} = \hat{E}_{c,d} = 30 \text{GPa}$$
(10)

⁵⁸⁰ The objective functions include all the nine cable forces on the Venice and Mestre sides ⁵⁸¹ except for the two close to the tower mainly affected by $E_{c,t}$. The search domain is ⁵⁸² a multidimensional space Ω , based on the admissible intervals of values for each *j*-th ⁵⁸³ variable, defined by its lower and upper bounds $[x_j^l, x_j^u]$. This detects a box-type hyper-⁵⁸⁴ rectangular search space Ω which is typically defined as the Cartesian product (denoted by ⁵⁸⁵ the × symbol) among the admissible intervals

$$\Omega = [\rho_c^l, \rho_c^u] \times [k_a^l, k_a^u] \tag{11}$$

2. **Optimization of** $E_{c,t}$ This optimization can be formulated as unconstrained and singleobjective, since there is one Objective Function (OF) $g(\mathbf{x})$ to be minimized and no equality or inequality constraints.

$$\hat{x}_2 = \min_{x_2 \in \Omega_2} \{ g_2(x_2) \}$$
(12)

$$g_2(\mathbf{x}_2) = \sum_{i=1}^2 \left(\frac{T_i^m - T_i^c}{T_i^m}\right)^2 + \sum_{i=2}^2 \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m}\right)^2 + \sum_{i=2}^2 \left(1 - \operatorname{diag}(MAC(\Phi_i^m, \Phi_i^c))\right)$$
(13)

$$\mathbf{x}_2 = \{E_{c,t}\}^T, \ \hat{E}_{c,d} = 30$$
GPa, $\{\hat{\rho}_c, \hat{k}_a\}^T$ in Tab.13 and 14 (14)

The objective functions include two cables on the Venice and Mestre sides close to the tower and the second mode shape associated with the tower deformation.

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3. **Optimization of** $E_{c,d}$: This optimization can be formulated as unconstrained and singleobjective since there is one Objective Function (OF) $g(\mathbf{x})$ to be minimized and no equality or inequality constraints.

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$$\hat{x}_3 = \min_{x_3 \in \Omega_3} \{ g_3(x_3) \}$$
(15)

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$$g_3(\mathbf{x}_3) = \sum_{i=1,3}^{12} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2 + \sum_{i=1,3}^{12} \left(1 - \text{diag}(MAC(\Phi_i^m, \Phi_i^c)) \right)$$
(16)

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 $\mathbf{x}_3 = \{E_{c,d}\}^T, \ \{\hat{\rho}_c, \hat{k}_a, \hat{E}_{c,t}\}^T \text{ in Tab.13 and 14}$ (17)

The objective functions includes all mode shapes except for the second one related to the tower deformation.

It must be remarked that beyond its strictly physical meaning, concrete Young's modulus should 607 be also considered as a modeling parameter (Schlune et al. 2009). Indeed, in the FE model updating 608 procedures, the concrete Young's modulus is often assumed as a single parameter describing the 609 dynamical stiffness adaptation for all directions simultaneously, thus strongly affecting the simulated 610 global dynamics of the FE model (Schlune et al. 2009). Therefore, since it summarizes different 611 contributions to the global simulated dynamic response, it is affected by an intrinsic severe level 612 of uncertainty (Schlune et al. 2009). Firstly, these uncertainties may be related to modeling errors, 613 e.g. due to simplified assumptions when modeling complex structures, or from actual intrinsic 614 factors, such as the mesh discretization level (Park et al. 2012). Secondly, they may be also related 615 to model parameter errors, i.e. due to material and geometric properties uncertainties, as well as a 616 proper definition of their variation range boundaries (Brownjohn and Xia 2000). These boundaries 617 are normally set for the purpose of avoiding physically impossible updated parameter outcomes. 618 However, a trade-off between physically acceptable parameter values and the convergence level 619 is often required (Brownjohn and Xia 2000). In addition, the after-updating concrete stiffness 620 parameters are usually expected to increase because, by definition, the dynamic Young's modulus 621 of concrete is greater than the static one (Jaishi and Ren 2005). Another reasonable concomitant 622 cause is related to concrete long-term hardening phenomena (Schlune et al. 2009). Thus, in 623 (Daniell and Macdonald 2007) it is suggested to adopt an already significant value for the concrete 624 Young's modulus initial values, e.g. about 37 GPa. From this value, it is expected at least an 625

incremental variation at least of 15% (Park et al. 2012). However, after-updating values may also 626 reach considerably high values, e.g. about 53 GPa, as demonstrated in (Jaishi and Ren 2005). 627 In summary, due to epistemic uncertainty, the tuning parameters, generally Young's moduli, not 628 only express their intrinsic physical meaning, characterized by specific acceptable values. First, 629 they are modeling parameters that collect and compensate for modeling errors in the optimization 630 phase, while reducing the discrepancy between the simulated and experimental dynamic response. 631 Therefore, for the above-mentioned reasons, the authors selected wide boundaries for Young's 632 moduli for both the sensitivity analysis and the optimization. 633

The three optimizations led to the following values of the objective functions: 0.4306, 0.0347 and 1.0296 corresponding to Eq.(10), (13) and (16), respectively. Multiple identical repetitions of the optimization gave the same results. Tab.13 and 14 show the results of the three optimizations in a single table, displaying the values of the cable forces and modal parameters before and after the updating. Additionally, Tab.13 and 14 show the optimum parameters and the relative upper and lower bounds.

The updating reveals that, while the agreement between modal parameters does not improve meaningfully, the comparison in terms of cable forces enhances significantly. Except for cable two on both the Mestre and Venice side, the relative average error reduces from -17% to -6%. The key to successful updating is introducing the bearing stiffness.

Initial updating attempts excluded the bearing stiffness and always led to minor improvements 644 in the matching between cable forces. However, the agreement's progress in modal parameters 645 is negligible and worsens in some cases. This fact proves that the geometric features are more 646 influential on the modal parameters than the chosen updated parameters. The average frequency 647 error is approximately 1% before and after updating. The same for the average MAC, which 648 keeps constant at 90%. The values of the optimum parameters are consistent with the engineering 649 judgment. For example, the optimum concrete mass is 24kN/m³, while Young's modulus of the 650 deck is 40GPa. The optimum Young's modulus of the tower is higher than the values expected for 651 concrete, being equal to 61.1GPa. This value proves that the tower exhibits a higher stiffness. Higher 652

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stiffness might be related to modeling errors in the actual geometry and possible discrepancies
 between the design and real tower geometry.

As recalled by (De Miranda and Gnecchi-Ruscone 2010), the bearings of the considered 655 structure were produced by TENSA and consist of laminated neoprene pads. The stiffness of the 656 bearings significantly influences the global dynamic behavior in terms of torsional and horizontal 657 modes and cable forces. The bearings, modeled as linear springs, possess a high level of uncertainty. 658 As noted by (Petersen and Øiseth 2017; Petersen et al. 2018), the uncertainty depends not only 659 on the neoprene material itself but also on unknown effects related to the embedded steel plates 660 and pre-tensioning. Secondarily, the idealization of a bearing as a single node can also cause 661 errors (Zhu et al. 2019). The optimum value of the bearing stiffness is 1350kN/mm. The vertical 662 stiffness falls within the expected range of stiffness for this kind of support (Kaczinski et al. 2016; 663 Zhang and Xie 2019). The average expected deformation of the bearing without traffic loads equals 664 5.78mm. Tab.15 shows the reaction forces at the supports corresponding to fixed and deformable 665 supports. While the bearing stiffness significantly affects the cable forces and the modal parameters, 666 it does not influence the reaction forces. In particular, the reaction forces exhibit minor relative 667 discrepancies, approximately 1%. 668

The results of the updating are consistent with the ones discussed in (Briseghella et al. 2021). 669 Briseghella et al. found that the optimum matching is achieved when $E_{c,t} = 41.67$ Gpa and 670 $E_{c,d} = 33.74$ Gpa. The introduction of the bearing stiffness within the updating process lead to an 671 increment of the optimum values, equal to $E_{c,t} = 51.1$ Gpa and $E_{c,d} = 40$ Gpa, as shown in Tab.13 672 and 14. Still, it is challenging to understand the mechanical reasons behind the observed, despite 673 minor differences. Plausibly, the bearing stiffness adds higher deformability to the structure, which 674 is compensated by a stiffer deck and tower, a consequent higher $E_{c,d}$ and $E_{c,t}$. For optimization 675 tasks considering non-linear problems, derivative- free global algorithms are particularly suitable 676 (Hofmeister et al. 2019) 677

⁶⁷⁸ As already mentioned before, it must be remarked that Young's moduli of concrete of the ⁶⁷⁹ FE model should not be considered strictly physical quantities, but, indeed, modeling parameters

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(Schlune et al. 2009). Due to epistemic uncertainty, the FE model might not represent the actual 680 structure. Therefore, the tuning parameters, generally Young's moduli, not only express their 681 intrinsic physical meaning, characterized by specific acceptable values. First, they are modeling 682 parameters that collect and compensate for the modeling error in the optimization phase (Jaishi 683 and Ren 2005; Park et al. 2012). Therefore, it generally happens that the values of Young's moduli 684 might exceed or underestimate the expected values for concrete (Schlune et al. 2009; Brownjohn 685 and Xia 2000). Therefore, the authors selected wide boundaries for Young's moduli for both the 686 sensitivity analysis and the optimization (He et al. 2022). 687

688 CONCLUSIONS

This paper presents and discusses the almost complete finite element model updating of cable-689 stayed bridges using modal parameters and cable forces estimates. The optimization problem is 690 particularly challenging when dealing with large-scale structures with numerous degrees of freedom 691 using traditional model updating methods. For this reason, several scholars use surrogate models 692 to reduce computational costs, like the response surface (RS) method (Fang and Perera 2009; Fang 693 and Perera 2011; Horta et al. 2011). However, if a preliminary sensitivity analysis is carried 694 out to support the mindful formulation of the objective functions, the traditional model updating 695 based on meta-heuristic optimization algorithms represents a feasible approach. In this paper, the 696 authors achieve the almost complete model updating of a cable-stayed bridge following a step-wise 697 procedure supported by extensive variance-based sensitivity analyses. 698

The procedure was applied to a cable-stayed bridge with a curved deck and inclined tower in 699 Porto Marghera (Italy). The authors used the particle-swarm (PSO) and differential evolution (DE) 700 algorithms to calibrate the model parameters from ambient vibration data collected on the deck 701 and cables. The availability of the cable forces estimates allows updating the inertial and stiffness 702 features, compared to more conventional FE updating where the sole modal parameters impose 703 the updating of either the mass or stiffness matrix to avoid ill-posedness and indeterminacy of the 704 optimization problem. The paper highlights the importance of preliminary sensitivity analyses to 705 formulate the optimization problem correctly. In the considered case study, preliminary sensitivity 706

analyses showed that the most influential parameters to be included in the update are: the concrete 707 mass (ρ_c), Young's modulus of the concrete deck ($E_{c,d}$), Young's modulus of the concrete tower 708 $(E_{c,t})$, and the bearing stiffness (k_a) . The sensitivity analyses demonstrated that ρ_c and k_a are mainly 709 affected by the cable forces, except for the cables close to the tower. The tower's deformability 710 $(E_{c,t})$ mainly influences the cables close to the tower. At the same time, the modal parameters 711 are mainly influenced by Young's modulus of the deck, except for the second mode related to 712 the tower deformation. Therefore, this evidence supported a three-step model updating, leading 713 to the progressive optimization of ρ_c and k_a , then $E_{c,t}$ and ultimately $E_{c,d}$. The updating in the 714 first two steps required the assumptions of specific parameter values. However, the optimization 715 results are not notably affected by different parameter choices, as confirmed by the sensitivity 716 analysis. The authors attempted the optimization of all the parameters simultaneously, following 717 multi-objective and single-objective approaches. However, all the endeavors were unsuccessful 718 since the algorithm always selected optimum values corresponding to the lower and upper bounds. 719 As evidenced by the sensitivity analysis, the chosen objective functions, especially the one in 720 modal parameters, present several local minima/maxima regions, which undermine the success 721 of global optimization, including all the parameters. Therefore, the only procedure which led to 722 values within the confidence bounds is the three-step one discussed in this paper. The analyses 723 also reveal that the agreement between modal parameters does not improve significantly. The 724 average percentage error remains equal before and after the update. Conversely, the cable forces 725 exhibited a noteworthy improvement. The key to this improvement is the introduction of bearing 726 stiffness. The sensitivity analysis highlighted the influence of the bearing stiffness on the modal 727 parameters and cable forces. The bearings consist of layered neoprene pads with an estimated 728 vertical stiffness equal to 1350kN/mm, consistent with the vertical stiffness of these structural 729 devices. This paper establishes that meta-heuristic optimization algorithms can be challenging to 730 use in FE model updating of cable-stayed bridges, especially when many parameters need to be 731 optimized. Therefore, the scholar must steer the optimization process by limiting the search space 732 and devising step-wise methods. A sensitivity analysis represents a necessary step to correctly 733

isolate the most relevant unknown parameters and suitably formulate the sets of objective functions
to be optimized.

736 DATA AVAILABILITY STATEMENT

All data, models, or code that support the findings of this study are available from the corre sponding author upon reasonable request.

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746 DISCLOSURE STATEMENT

⁷⁴⁷ No potential competing interest was reported by the authors.

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No	$f_{2010}[\text{Hz}]$	$f_{2011}[Hz]$	$\frac{f_{2010}-f_{2011}}{f_{2010}}$ [%]	MAC ₂₀₁₀₋₂₀₁₁
1	0.635	0.635	0.00	0.988
2	0.996	0.996	0.00	0.980
3	1.143	1.143	0.00	0.958
4	1.387	1.387	0.00	0.998
5	1.523	1.523	0.00	0.982
6	1.602	1.602	0.00	0.990
7	1.953	1.963	-0.51	0.988
8	2.637	2.646	-0.34	0.983
9	3.174	/	/	/
10	4.053	4.072	-0.47	0.954
11	4.932	4.951	-0.39	0.836
12	5.596	5.625	-0.52	0.835

TABLE 1. Comparison between the modal parameters estimated from experimental campaigns in 2010 and 2011.

Stay cable no.	2010	2011
1	1.250	1.230
2	1.211	1.211
3	1.621	1.621
4	1.543	1.543
5	1.387	1.387
6	1.289	1.289
7	1.250	1.250
8	1.113	1.094
9	0.977	0.977
1	1.445	1.406
2	1.309	1.289
3	1.641	1.641
4	1.563	1.563
5	1.426	1.406
6	1.328	1.328
7	1.230	1.230
8	1.094	1.113
9	0.938	0.938

TABLE 2. Fundamental natural frequencies in [Hz] of the stay cables estimated in 2010 and 2011. The first nine values refer to the Mestre side, the second nine to the Venice side.

Mestre side, stay cable n.									
Cable force	1	2	3	4	5	6	7	8	9
T ₂₀₁₀ [kN] T ₂₀₁₁ [kN]	455 458	755 757	2350 2359	3721 3715	3866 3842	4190 4199	4825 4828	5294 5289	4746 4771

TABLE 3. Cable forces identified from vibration data in the tests of June 2010 and April 2011 (Mestre side)

Venice side, Stay cable n.									
Cable force	1	2	3	4	5	6	7	8	9
T ₂₀₁₀ [kN] T ₂₀₁₁ [kN]	647 614	906 860	2414 2381	3771 3704	4005 3961	4324 4352	4588 4698	5275 5310	4512 4655

TABLE 4. Cable forces identified from vibration data in the tests of June 2010 and April 2011 (Venice side)

Cable	Exp. [kN]	Num. [kN]	Error
M1	458	221	51.7%
M2	757	1342	-77.3%
M3	2359	2411	-2.2%
M4	3715	3516	5.4%
M5	3842	3384	11.9%
M6	4199	2961	29.5%
M7	4828	2513	47.9%
M8	5289	2311	56.3%
M9	4771	1986	58.4%
V1	614	353	42.5%
V2	860	1596	-85.6%
V3	2381	2844	-19.5%
V4	3704	4084	-10.3%
V5	3961	3979	-0.5%
V6	4352	3578	17.8%
V7	4698	2821	40.0%
V8	5310	2309	56.5%
V9	4655	1139	75.5%

TABLE 5. Experimental numerical estimates of the cable forces before calibration.

No	Mode	f_e [Hz]	f_n [Hz]	$(f_e - f_n)/f_e \ [\%]$	MAC
1	V1	0.63	0.68	-6.49%	0.97
2	V2	1.00	0.97	2.12%	0.93
3	V3	1.14	1.23	-7.34%	0.86
4	T1	1.39	1.39	-0.54%	0.95
5	M1	1.52	1.65	-8.06%	0.80
6	T2	1.60	1.51	5.69%	0.76
7	V4	1.96	2.07	-5.49%	0.97
8	T3	2.65	2.56	3.31%	0.94
9	T5	4.07	3.99	1.91%	0.89
10	T6	4.95	4.84	2.17%	0.92
11	T7	5.63	5.54	1.53%	0.94

TABLE 6. Comparison between experimental and numerical modal parameters before model updating, where f_e and f_n are the experimental and numerical natural frequencies.

TABLE 7. Mass participation ratios of the modes before model updating. X, Y and Z indicate the longitudinal, transverse and vertical directions. U and R indicate the displacement and the rotation wth respect to the mentioned directions X, Y and Z.

Ν	lode		Mass	participa	tion rat	ios [%]	
No	Label	Ux	Uy	Uz	Rx	Ry	Rz
1	V 1	3.07	0.19	1.25	2.80	14.00	0.00
2	V2	1.29	8.29	6.33	9.86	0.02	0.43
3	V3	0.78	0.25	11.14	0.76	0.62	0.59
4	T1	2.85	16.58	0.35	0.16	0.01	0.28
5	M 1	9.56	2.98	0.07	0.03	0.01	17.40
6	T2	0.95	3.79	1.34	0.10	0.04	3.46
7	V4	0.38	0.00	0.70	0.31	0.14	0.29
8	Т3	0.01	0.00	0.00	0.06	0.08	0.00
9	T5	0.00	0.04	0.25	0.32	0.33	0.00
10	T6	0.00	0.00	0.04	1.78	3.19	0.00
11	T7	0.00	0.00	0.13	0.04	0.00	0.00

TABLE 8. Sensitivity indicators of the objective function in Eq.(3) to the Young's modulus of the tower (E_c) , the mass of the steel deck (M_s) , the mass of the concrete deck (M_c) , the Young's modulus of the deck $(E_{c,deck})$, and the vertical stiffness of the bearings (k_a) .

$E_{c,t}$	$ ho_s$	$ ho_c$	$E_{c,d}$	k _a
5.14%	0.34%	4.13%	1.21%	93.14%

TABLE 9. Sensitivity indicators of the cable forces labelled M1-M9 (Cables on the Mestre side) and V1-V9 (Cables on the Venice side) to the Young's modulus of the tower (E_c) , the mass of the steel deck (M_s) , the mass of the concrete deck (M_c) , the Young's modulus of the deck $(E_{c,deck})$, and the vertical stiffness of the bearings (k_a) .

Cable force	$E_{c,t}$	$ ho_s$	$ ho_c$	$E_{c,d}$	k _a
M1	87.13%	0.08%	12.57%	0.51%	0.48%
M2	38.14%	0.25%	61.29%	1.37%	0.69%
M3	9.06%	0.27%	86.05%	1.35%	4.79%
M4	1.32%	0.23%	78.93%	1.12%	19.18%
M5	0.01%	0.17%	55.23%	0.79%	43.59%
M6	0.27%	0.12%	31.74%	0.47%	66.41%
M7	0.97%	0.07%	16.75%	0.24%	80.52%
M8	2.10%	0.05%	9.23%	0.11%	86.86%
M9	4.22%	0.03%	6.82%	0.07%	87.17%
V1	82.85%	0.18%	17.14%	0.22%	0.19%
V2	30.49%	0.39%	68.93%	0.95%	0.83%
V3	6.76%	0.41%	93.46%	1.32%	0.01%
V4	1.18%	0.36%	94.99%	1.44%	3.79%
V5	0.12%	0.29%	78.15%	1.28%	21.13%
V6	0.00%	0.20%	48.68%	0.91%	50.04%
V7	0.08%	0.12%	22.88%	0.48%	75.34%
V8	0.21%	0.05%	8.28%	0.15%	89.66%
V9	0.44%	0.01%	2.06%	0.02%	95.70%

TABLE 10. Sensitivity indicators of the objective function (OF) in Eq.(6) to the Young's modulus of the tower $(E_{c,t})$, the mass of the steel deck (ρ_s) , the mass of the concrete deck (ρ_c) , the Young's modulus of the deck $(E_{c,d})$, and the vertical stiffness of the bearings (k_a) .

OF	$E_{c,t}$	$E_{c,d}$	$ ho_c$	$ ho_s$	k _a
$f(\mathbf{x})$	0.3%	0.5%	3.7%	0.0%	94.6%
$f_1(\boldsymbol{x})$	0.6%	0.9%	61.4%	0.1%	41.6%
$f_2(\boldsymbol{x})$	0.4%	0.7%	1.3%	0.0%	97.2%

TABLE 11. Sensitivity indicators of each natural frequency to the Young's modulus of the tower $(E_{c,t})$, the mass of the steel deck (ρ_s) , the mass of the concrete deck (ρ_c) , the Young's modulus of the deck $(E_{c,d})$, and the vertical stiffness of the bearings (k_a) .

Mode	$E_{c,t}$	$E_{c,d}$	$ ho_c$	$ ho_s$	k_a
1-V1	3.2%	0.3%	42.2%	0.1%	54.6%
2-V2	94.4%	0.0%	4.4%	0.0%	4.4%
3-V3	17.0%	5.3%	75.4%	0.6%	23.3%
4-T1	3.8%	17.8%	52.9%	0.7%	39.2%
5-M1	0.1%	0.7%	48.9%	0.1%	53.2%
6-T2	0.0%	1.2%	65.4%	0.1%	33.8%
7-V4	0.0%	0.5%	49.3%	0.1%	51.0%
8-T3	1.4%	3.9%	65.8%	0.1%	34.1%
9-T5	0.0%	5.3%	60.2%	0.6%	52.3%
10-T6	1.9%	4.3%	54.8%	0.0%	48.3%
11-T7	0.0%	2.0%	52.0%	0.8%	51.9%

TABLE 12. Sensitivity indicators of the MAC of each mode to the Young's modulus of the tower $(E_{c,t})$, the mass of the steel deck (ρ_s) , the mass of the concrete deck (ρ_c) , the Young's modulus of the deck $(E_{c,d})$, and the vertical stiffness of the bearings (k_a) .

Mode	$E_{c,t}$	$E_{c,d}$	$ ho_c$	$ ho_s$	k_a
1-V1	2.8%	2.6%	2.2%	0.0%	95.7%
2-V2	49.6%	0.2%	25.4%	0.0%	74.1%
3-V3	41.5%	16.3%	57.7%	0.2%	30.2%
4-T1	24.3%	58.3%	41.6%	0.4%	36.3%
5-M1	1.4%	3.8%	11.6%	0.0%	97.9%
6-T2	1.6%	48.8%	24.2%	0.1%	87.2%
7-V4	0.0%	0.0%	0.4%	0.0%	99.6%
8-T3	1.6%	0.6%	4.8%	0.2%	98.5%
9-T5	0.0%	0.9%	1.6%	0.0%	99.4%
10-T6	0.6%	0.6%	0.7%	0.0%	98.4%
11-T7	0.0%	0.6%	1.7%	0.0%	96.9%

Cable Label	Exp. [kN]	Num. [kN]	Error	Initial error	Mode Label	Freq. Exp. [Hz]	Freq. Num. [Hz]	MAC	Freq. error	Initial MAC	Initial freq. error
M1	458	408	11%	52%	V1	0.635	0.699	97.10%	-10.1%	97.41%	-6.5%
M2	757	1228	-62%	-77%	V2	0.996	0.975	93.79%	2.1%	93.06%	2.1%
M3	2359	2372	-1%	-2%	V3	1.143	1.226	82.41%	-7.3%	85.58%	-7.3%
M4	3715	3852	-4%	5%	T1	1.387	1.395	95.03%	-0.6%	95.26%	-0.5%
M5	3842	4271	-11%	12%	M1	1.523	1.650	78.13%	-8.3%	80.01%	-8.1%
M6	4199	4453	-6%	29%	T2	1.602	1.513	75.38%	5.5%	75.56%	5.7%
M7	4828	4540	6%	48%	V4	1.963	2.073	97.04%	-5.6%	96.91%	-5.5%
M8	5289	5041	5%	56%	T3	2.646	2.559	94.04%	3.3%	94.19%	3.3%
M9	4771	4618	3%	58%	T5	4.072	3.995	89.03%	1.9%	89.10%	1.9%
V1	614	530	14%	43%	T6	4.951	4.826	93.29%	2.5%	91.61%	2.2%
V2	860	1279	-49%	-86%	T7	5.625	5.539	94.48%	1.5%	94.48%	1.5%
V3	2381	2460	-3%	-19%							
V4	3704	3872	-5%	-10%							
V5	3961	4284	-8%	0%							
V6	4352	4563	-5%	18%							
V7	4698	4573	3%	40%							
V8	5310	5229	2%	57%							
V9	4655	4791	-3%	76%							

TABLE 13. Cable forces and modal parameters associated with the optimum set of parameters and percentage error before and after the updating.

Parameter	Unit	L.B.	U.B.	Optimum
$ ho_c$	kN/m ³	23	30	24
k_a	kN/mm	100	10000	1350
$E_{c,d}$	GPa	30	1	40
$E_{c,t}$	GPa	30	70	51.1

TABLE 14. Optimized estimated modal parameters with their upper (U.B) and lower (L.B.) bounds.

Label	Reaction [kN]		Relative difference	Displacement [mm]
	$k_a \to \infty$	$k_a = \hat{k}_a$		$k_a = \hat{k}_a$
Venice-1	9399.365	9302.627	1.03%	6.89
Venice-2	9055.16	8868.379	2.06%	6.57
Mestre-1	6705.512	6633.135	1.08%	4.91
Mestre-2	6480.927	6388.619	1.42%	4.73

TABLE 15. Reaction forces on the Venice and Mestre side in case of rigid and flexible supports.

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Fig. 1. Illustration of the followed procedure.



Fig. 2. View of the Porto Marghera Bridge (photographer: Bruno Briseghella).



(**b**) Plan and typical cross sections.

Fig. 3. Schematic plan view of the deck and typical cross-sections (dimensions in cm in (a) and mm in (b)).



Fig. 4. Experimental mode shapes detected in the three experimental campaigns. The sub captions indicate the natural frequencies of each mode corresponding to the dynamic identifications carried out 2010 and 2011 respectively.



Fig. 5. Numbering of the stay cables.



Fig. 6. Natural frequencies of the nine stay cables on the Mestre side and their correlation with the mode number.



Fig. 7. FE model of the Porto Marghera bridge developed in SAP2000.



(a) $f^e=0.635$ Hz; $f^n=0.676$ Hz;(b) $f^e=0.996$ Hz; $f^n=0.975$ Hz;(c) $f^e=1.143$ Hz; $f^n=1.226$ Hz; MAC=0.97 MAC=0.93 MAC=0.85



(d) f^e =1.387Hz; f^n =1.394Hz;(e) f^e =1.523Hz; f^n =1.646Hz;(f) f^e =1.602Hz; f^n =1.510Hz; MAC=0.95 MAC=0.80 MAC=0.75



(g) f^e =1.963Hz; f^n =2.071Hz;(h) f^e =2.646Hz; f^n =2.559Hz;(i) f^e =4.072Hz; f^n =3.994Hz; MAC=0.96 MAC=0.94 MAC=0.89



(j) f^e =4.951Hz; f^n =4.844Hz;(k) f^e =5.625Hz; f^n =5.539Hz; MAC=0.91 MAC=0.94

Fig. 8. Representation of a few selected numerical modes. f^e and f^n in the sub-captions indicate the experimental and numerical natural frequencies.



Fig. 9. Different views of the scatter plot of the sensitivity of the objective function in Eq. (3) to the concrete Young's modulus of the tower (E_c, t) , the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c) .



Fig. 10. Scatter plots of the sensitivity of the cable forces on the Venice side to the concrete Young's modulus of the tower $(E_{c,t})$, the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c) .



Fig. 11. Scatter plots of the sensitivity of the cable forces on the Mestre side to the concrete Young's modulus of the tower $(E_{c,t})$, the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c) .



Fig. 12. Selected scatter plots of the sensitivity of the modal parameters to the concrete Young's modulus of the tower $(E_{c,t})$, the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c) .



Fig. 13. Selected scatter plots of the sensitivity of the MAC to the concrete Young's modulus of the tower $(E_{c,t})$, the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c) .