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Role of Cable Forces in the Model Updating of Cable-Stayed Bridges / Aloisio, A., Pasquale Pasca, D., Rosso, M.M., Briseghella, B.. - In: JOURNAL OF BRIDGE ENGINEERING. - ISSN 1084-0702. - 28:7(2023).
[10.1061/JBENF2.BEENG-6168]

Availability:

This version is available at: 11583/2984679 since: 2023-12-22T11:13:14Z

Publisher:

ASCE-AMER SOC CIVIL ENGINEERS

Published

DOI:10.1061/JBENF2.BEENG-6168

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

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ABSTRACT

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

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

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

41 However, there can be exceptions, and in some instances, ambient vibration data could be
42 used to update the inertial and structural stiffness features reliably. It could be the case of cable-
43 stayed bridges (Zárate and Caicedo 2008), where an experimental estimate of the cables' natural
44 frequencies and the deck's modal parameters can be obtained. The natural frequencies of the cables
45 can return an indirect assessment of the cable forces using suitable mechanical models of the cable
46 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.

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

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

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

98 Cable-stayed bridges, especially those built in the last decade, possess peculiar aesthetic and
99 structural features. Cable-stayed bridges play a crucial role in infrastructure networks (Virlogeux
100 1999). However, they also characterize the urban landscape with their unique structural shape
101 (Wilson and Liu 1991; Astaneh-Asl and Black 2001; Ni et al. 2019). There are multiple examples
102 of cable-stayed bridges, but only some of them possess curved decks. The most known examples
103 of cable-stayed bridges with a curved deck are: the Rhine bridge near Schaffhausen in Switzerland
104 (Deger et al. 1996), the Safti Link bridge in Singapore (Brownjohn and Xia 2000), the twin bridges

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

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

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

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

- 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.
- 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.

PROBLEM FORMULATION

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

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

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

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

$$180 \quad \hat{\mathbf{x}} = \min_{\mathbf{x} \in \Omega} \{\mathbf{g}(\mathbf{x})\} \quad (1)$$

181 where $\hat{\mathbf{x}}$ and \mathbf{x} collect the optimized and unknown parameters respectively, \mathbf{g} the objective functions
182 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.

$$184 \quad \mathbf{g}(\mathbf{x}) = \begin{cases} \sum_{i=1}^{n_c} \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2, & \text{Cable forces} \\ \sum_{i=2}^{n_m} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2 + \sum_{i=2}^{n_m} (1 - \text{diag}(\text{MAC}(\Phi_i^m, \Phi_i^c))), & \text{Modal parameters} \end{cases} \quad (2)$$

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

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

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

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

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

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

222 **CASE STUDY**

223 This section briefly describes the bridge and the experimental tests for characterizing its dynamic
224 response.

225 **Bridge description**

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

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

251 **Dynamic characterization of the deck**

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

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

267 **Dynamic characterization of the cables**

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

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

$$278 \quad f_n = \frac{1}{2L} \left(\frac{T}{\rho} \right)^{0.5} \quad (3)$$

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

$$283 \quad T = \rho \left(2L \frac{\partial f_n}{\partial n} \right)^2 \quad (4)$$

284 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
 285 forces from the two experimental campaigns. As remarked in the original technical report, the recent
 286 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 (Mehrabani 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.

294 FINITE ELEMENT MODELLING OF THE BRIDGE AND PRELIMINARY ANALYSES

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

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

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

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

331 **SENSITIVITY ANALYSIS OF THE CABLE FORCES**

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

$$337 \quad g_1 = \sum_{i=1}^{n_c} \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2 \quad (5)$$

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

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

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

- 351 1. The Young's modulus of the tower ($E_{c,t}$). The cable forces might be significantly reduced
352 if the tower is more deformable than expected by design. If so, the tower does not behave
353 as almost fixed support for the cables. Therefore, Young's modulus of concrete has been
354 chosen as a synthetic representation of the tower deformability. The bounds [30, 50] GPa
355 were assumed in the analyses.
- 356 2. The mass of the steel deck (ρ_s). The FE model accurately reproduces the geometry of the
357 steel deck by using finite elements. However, the approximation related to finite elements
358 might lead to an over or underestimation of the structural mass. Therefore, the weight per
359 unit of volume of steel has been chosen to represent the mass of the steel deck. Indeed, ρ_s
360 should be a known parameter since steel is manufactured in a workshop. As envisioned, the
361 analysis shows that the parameter is negligible and can be excluded from the updating. The
362 bounds [75, 80] kN/m³ were assumed in the analyses. The variability does not represent
363 the possible variation of the steel mass but comprises the possible uncertainties related to
364 the steel deck modeling.
- 365 3. The mass of the concrete deck (ρ_c). The FE model attempts to reproduce the geometry of the

366 concrete deck accurately. However, it is conventionally assumed that the concrete specific
367 weight is equal to 25kN/m^3 . Still, the weight of a reinforced concrete slab depends on the
368 proportions of concrete and steel, which might cause variations of the conventional value
369 assumed in calculations. Additionally, the uncertainties in the actual size of the concrete
370 deck might propagate to the cable forces. Therefore, the authors selected the specific weight
371 of concrete as a synthetic representative of the mass of the concrete deck. The uncertainties
372 in the concrete deck's actual weight may depend on both the slab size and the specific weight
373 of the concrete. The bounds $[24, 30]\text{ kN/m}^3$ were assumed in the analyses.

- 374 4. The Young's modulus of the concrete deck ($E_{c,d}$). The paper proposes a two-stage FE
375 model update for cable-stayed bridges. In the first stage, the mass of the deck is estimated
376 from the experimental values of the cable forces. In the second stage, the deformability
377 of the deck is estimated from the experimental modal parameters. This approach works
378 if the cable forces do not significantly depend on the deck stiffness. Therefore, Young's
379 modulus of the concrete deck is used to prove that the cable forces are not affected by the
380 deck stiffness. The bounds $[30, 50]\text{ GPa}$ were assumed in the analyses.
- 381 5. The vertical stiffness of the bearings (k_a). The vertical stiffness of the bearings can be a
382 crucial parameter affecting the cable forces. For example, the cable forces might increase
383 significantly if the bearings are too deformable. The bounds $[10, 500]\text{ kN/mm}$ were assumed
384 in the analyses.

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

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

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

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

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

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

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

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

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

447 evidences the presence of a maximum for the ρ_c value.

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

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

465 SENSITIVITY ANALYSIS OF THE MODAL PARAMETERS

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

$$468 \quad f(\mathbf{x}) = f_1(\mathbf{x}) + f_2(\mathbf{x}) \quad (6)$$

$$469 \quad f_1(\mathbf{x}) = \sum_{i=1}^{n_m} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2 \quad (7)$$

$$471 \quad f_2(\mathbf{x}) = \sum_{i=1}^{n_m} (1 - \text{diag}(\text{MAC}(\Phi_i^m, \Phi_i^c))) \quad (8)$$

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

477
478 The main aspects arising from the observation of the sensitivity analysis might be itemized as
479 follows based on the results collected in Tabs.10,11,12:

- 480 • Tab.10-The vertical stiffness of the bearing is the most influential parameter in Eq.(6).
481 However, as shown in Tab.10, the highest sensitivity to the bearing stiffness is given by the
482 cost function in terms of mode shapes ($\approx 97.2\%$). Conversely, the cost function in terms
483 of natural frequencies is more affected by the mass of the deck (ρ_c), with an approximate
484 61.4% indicator. In the selected ranges of variation, the weight per volume of steel does not
485 sensibly affect the objective functions with a sensitivity indicator less than 0.1%.
- 486 • Tab.11-The sensitivity analysis of each natural frequency confirms the minimal effect of
487 the weight per unit of volume of steel. Conversely, there are several cases where Young's
488 modulus of the tower and the deck and mass per unit of volume of concrete are more
489 influential than the bearing stiffness. This fact can be mainly observed for modes V2,
490 V3, T1, and T2. Specifically, V2 exhibits a marked dependence on $E_{c,t}$ ($\approx 49.6\%$), ρ_c
491 ($\approx 25.4\%$), and k_a ($\approx 74.1\%$). Modes V3, T1, and T2 also significantly depend on Young's
492 modulus of the deck, with sensitivity indicators approximately equal to 16.3%, 58.3%,
493 and 48.8%, respectively. These are the only cases where Young's modulus of the deck
494 is influential. Therefore, V3, T1, and T2 are the only modes that can be ideally used to
495 estimate Young's modulus of the deck. Except for the mentioned modes, the other modes
496 exhibit a prevalent dependence on the bearing stiffness.
- 497 • Tab.12-The results in natural frequencies are pretty similar to those in MAC. The main
498 difference stands in the role of the mass per unit of volume of concrete. In comparison,
499 ρ_c and k_a are the most influential parameters for the natural frequencies, except for V2,

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.

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

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

521 **Discussion**

522 The selected plots show the complexity of a possible model update driven by the modal pa-
523 rameters. There are two main reasons. (i) The preliminary model of the bridge without updating
524 already exhibits an excellent agreement with experimental data. Therefore, the parameter calibra-
525 tion should lead to the near identity between the experimental and numerical modal parameters.

526 However, the model updating using both the mass and stiffness parameters would be indeterminate,
527 and the scholar should arbitrarily assume one parameter possibly associated with lower uncertainty.
528 (ii) The presence of subspaces and discontinuities in the plots of either the natural frequency or the
529 mode shape prove the possible limits of a meta-heuristic optimization algorithm. The optimization
530 outcome depends on the subspace where the algorithm might fall in the search process. Therefore,
531 a FE model updating using the cable forces has several advantages compared to one based on the
532 modal parameters.

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

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

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

547

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

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

- 561 1. Optimization of ρ_c and k_a , after assuming a specific value for Young's moduli of the tower
562 $E_{c,t}$ and the deck $E_{c,d}$. The objective function depends on all the cable forces except for T1,
563 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.

569 The global optimization algorithms, the differential evolution (DE) (Storn and Price 1997) and the
570 particle swarm optimization (PSO) (Kennedy and Eberhart 1995), are used for mutual validation.
571 Also, to perform the model updating, the script is written in Python using SAP2000 OAPI with the
572 Python module Scipy (to run DE) and PySwarms (Miranda 2018) (to run PSO). Since no significant
573 difference is observed in comparing the outcomes of the two optimization algorithms, the authors
574 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:

578
$$\hat{\mathbf{x}}_1 = \min_{\mathbf{x}_1 \in \Omega_1} \{g_1(\mathbf{x}_1)\} \quad (9)$$

579
$$g_1(\mathbf{x}_1) = \sum_{i=3}^9 \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2 \quad \mathbf{x}_1 = \{\rho_c, k_a\}^T, \hat{E}_{c,t} = \hat{E}_{c,d} = 30\text{GPa} \quad (10)$$

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 \times symbol) among the admissible intervals

586
$$\Omega = [\rho_c^l, \rho_c^u] \times [k_a^l, k_a^u] \quad (11)$$

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

590
$$\hat{\mathbf{x}}_2 = \min_{\mathbf{x}_2 \in \Omega_2} \{g_2(\mathbf{x}_2)\} \quad (12)$$

591
$$g_2(\mathbf{x}_2) = \sum_{i=1}^2 \left(\frac{T_i^m - T_i^c}{T_i^m} \right)^2 + \sum_{i=2}^2 \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2 + \sum_{i=2}^2 (1 - \text{diag}(\text{MAC}(\Phi_i^m, \Phi_i^c))) \quad (13)$$

593
$$\mathbf{x}_2 = \{E_{c,t}\}^T, \hat{E}_{c,d} = 30\text{GPa}, \{\hat{\rho}_c, \hat{k}_a\}^T \text{ in Tab.13 and 14} \quad (14)$$

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

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

599 or inequality constraints.

$$600 \quad \hat{\mathbf{x}}_3 = \min_{\mathbf{x}_3 \in \Omega_3} \{g_3(\mathbf{x}_3)\} \quad (15)$$

$$601 \quad g_3(\mathbf{x}_3) = \sum_{i=1,3}^{12} \left(\frac{\omega_i^m - \omega_i^c}{\omega_i^m} \right)^2 + \sum_{i=1,3}^{12} (1 - \text{diag}(\text{MAC}(\Phi_i^m, \Phi_i^c))) \quad (16)$$

$$602 \quad \mathbf{x}_3 = \{E_{c,d}\}^T, \{\hat{\rho}_c, \hat{k}_a, \hat{E}_{c,t}\}^T \text{ in Tab.13 and 14} \quad (17)$$

603
604
605 The objective functions includes all mode shapes except for the second one related to the
606 tower deformation.

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

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

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

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

644 Initial updating attempts excluded the bearing stiffness and always led to minor improvements
645 in the matching between cable forces. However, the agreement's progress in modal parameters
646 is negligible and worsens in some cases. This fact proves that the geometric features are more
647 influential on the modal parameters than the chosen updated parameters. The average frequency
648 error is approximately 1% before and after updating. The same for the average MAC, which
649 keeps constant at 90%. The values of the optimum parameters are consistent with the engineering
650 judgment. For example, the optimum concrete mass is 24kN/m^3 , while Young's modulus of the
651 deck is 40GPa. The optimum Young's modulus of the tower is higher than the values expected for
652 concrete, being equal to 61.1GPa. This value proves that the tower exhibits a higher stiffness. Higher

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

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

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

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

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

688 CONCLUSIONS

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

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

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

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

736 **DATA AVAILABILITY STATEMENT**

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

739 **ACKNOWLEDGEMENTS**

740 The authors would like to thank the technical personnel (M. Antico and M. Cucchi) of the
741 VIBLAB Laboratory of Vibrations and Dynamic Monitoring of Structures, Polytechnic of Milan, for
742 their valuable help in conducting the field tests. Moreover, the collaboration of Dr. Fulvio Busatta
743 (University of Cape Town) in implementing the first Finite Element Model and of Prof. Airong
744 Chen (Tongji University) in performing a Risk Analysis of the Bridge is highly acknowledged. This
745 work was supported by the National Natural Science Foundation of China [grant No. 51778148].

746 **DISCLOSURE STATEMENT**

747 No potential competing interest was reported by the authors.

748 **FUNDING**

749 The authors gratefully acknowledge the financial support from the National Natural Science
750 Foundation of China (Grant No. 51778148).

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

No	f_{2010} [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 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.

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

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

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

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

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

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

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 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 with respect to the mentioned directions X, Y and Z.

Mode		Mass participation ratios [%]					
No	Label	Ux	Uy	Uz	Rx	Ry	Rz
1	V1	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	M1	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	T3	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).

$E_{c,t}$	ρ_s	ρ_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	$E_{c,t}$	ρ_s	ρ_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	$E_{c,t}$	$E_{c,d}$	ρ_c	ρ_s	k_a
$f(\mathbf{x})$	0.3%	0.5%	3.7%	0.0%	94.6%
$f_1(\mathbf{x})$	0.6%	0.9%	61.4%	0.1%	41.6%
$f_2(\mathbf{x})$	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	$E_{c,t}$	$E_{c,d}$	ρ_c	ρ_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	$E_{c,t}$	$E_{c,d}$	ρ_c	ρ_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%

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

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

Parameter	Unit	L.B.	U.B.	Optimum
ρ_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 15. Reaction forces on the Venice and Mestre side in case of rigid and flexible supports.

Label	Reaction [kN]		Relative difference	Displacement [mm]
	$k_a \rightarrow \infty$	$k_a = \hat{k}_a$		
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

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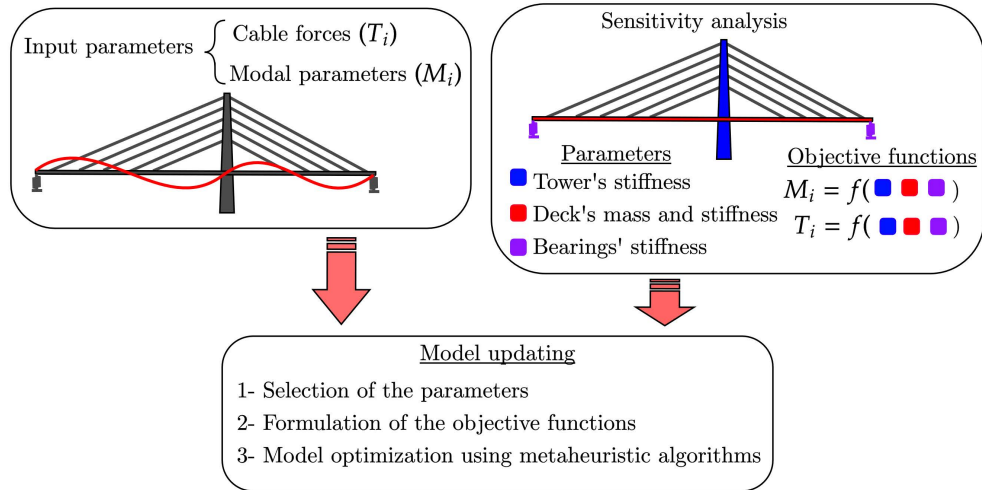
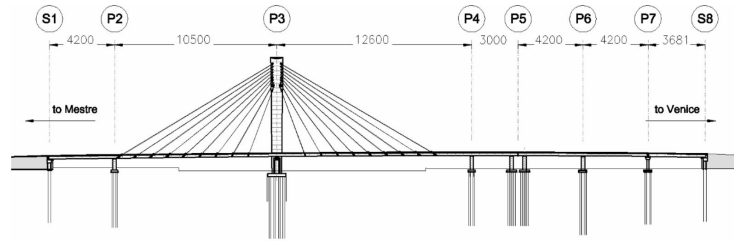


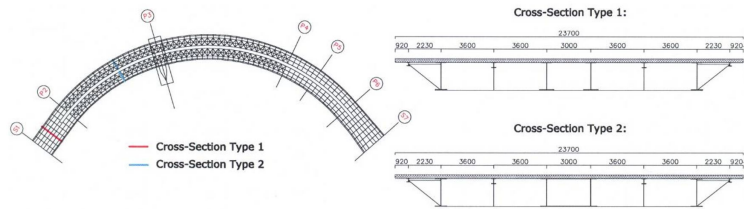
Fig. 1. Illustration of the followed procedure.



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



(a) Elevation.



(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)).

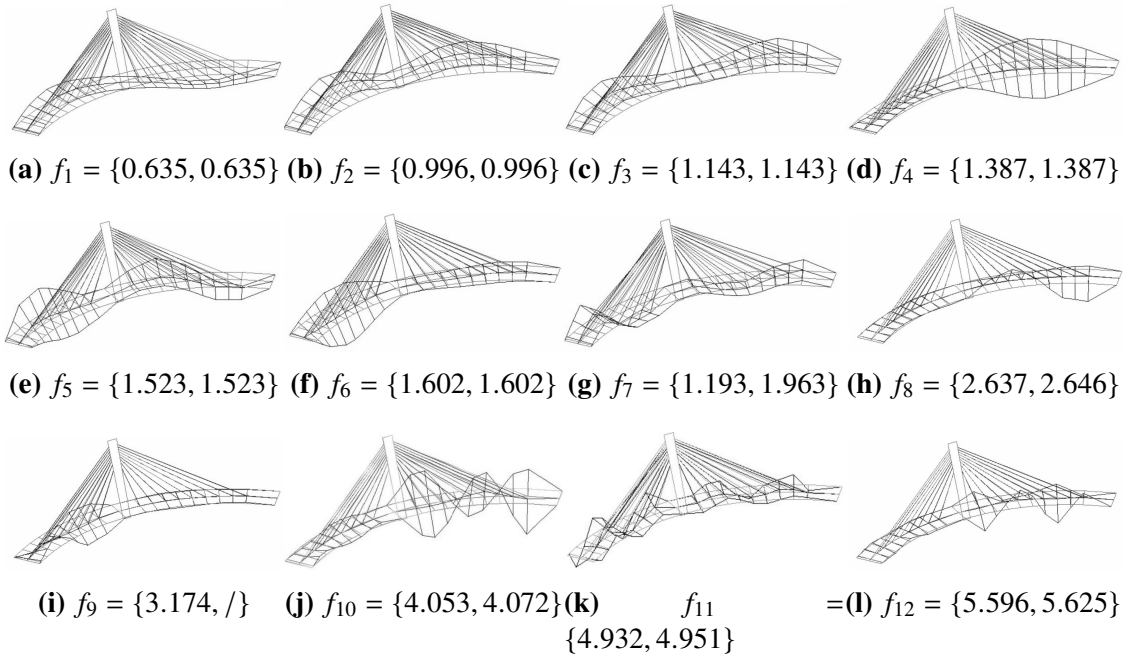


Fig. 4. Experimental mode shapes detected in the three experimental campaigns. The sub captions indicate the natural frequencies of each mode corresponding to the dynamic identifications carried out 2010 and 2011 respectively.

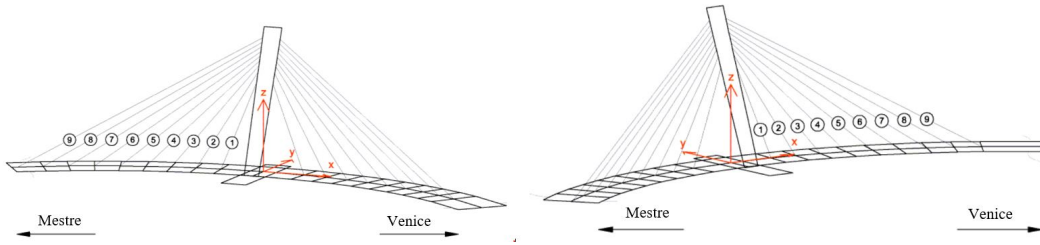


Fig. 5. Numbering of the stay cables.

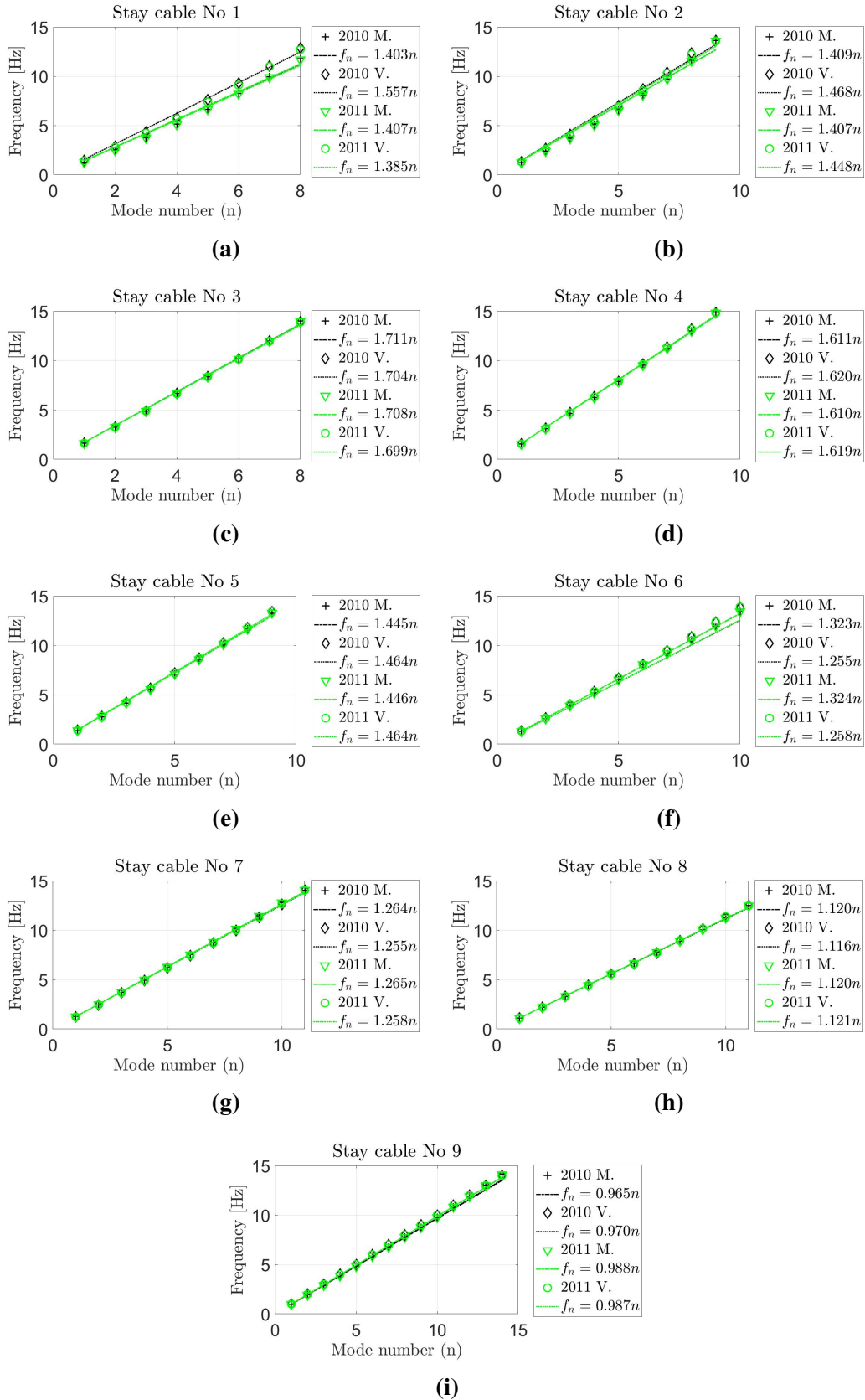


Fig. 6. Natural frequencies of the nine stay cables on the Mestre side and their correlation with the mode number.

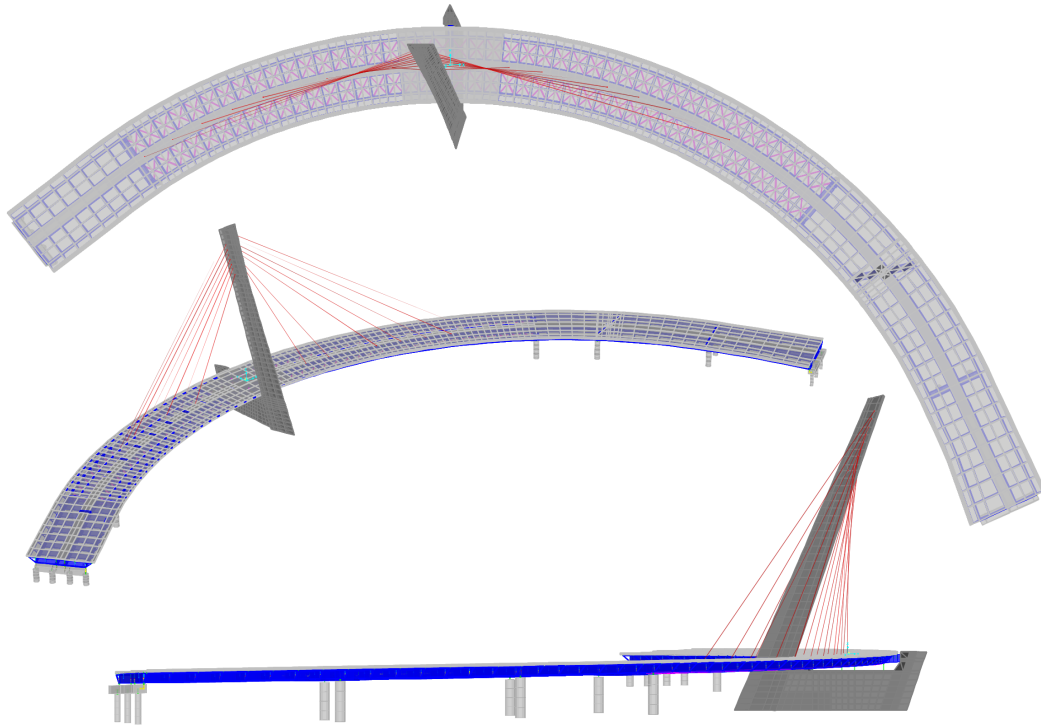


Fig. 7. FE model of the Porto Marghera bridge developed in SAP2000.

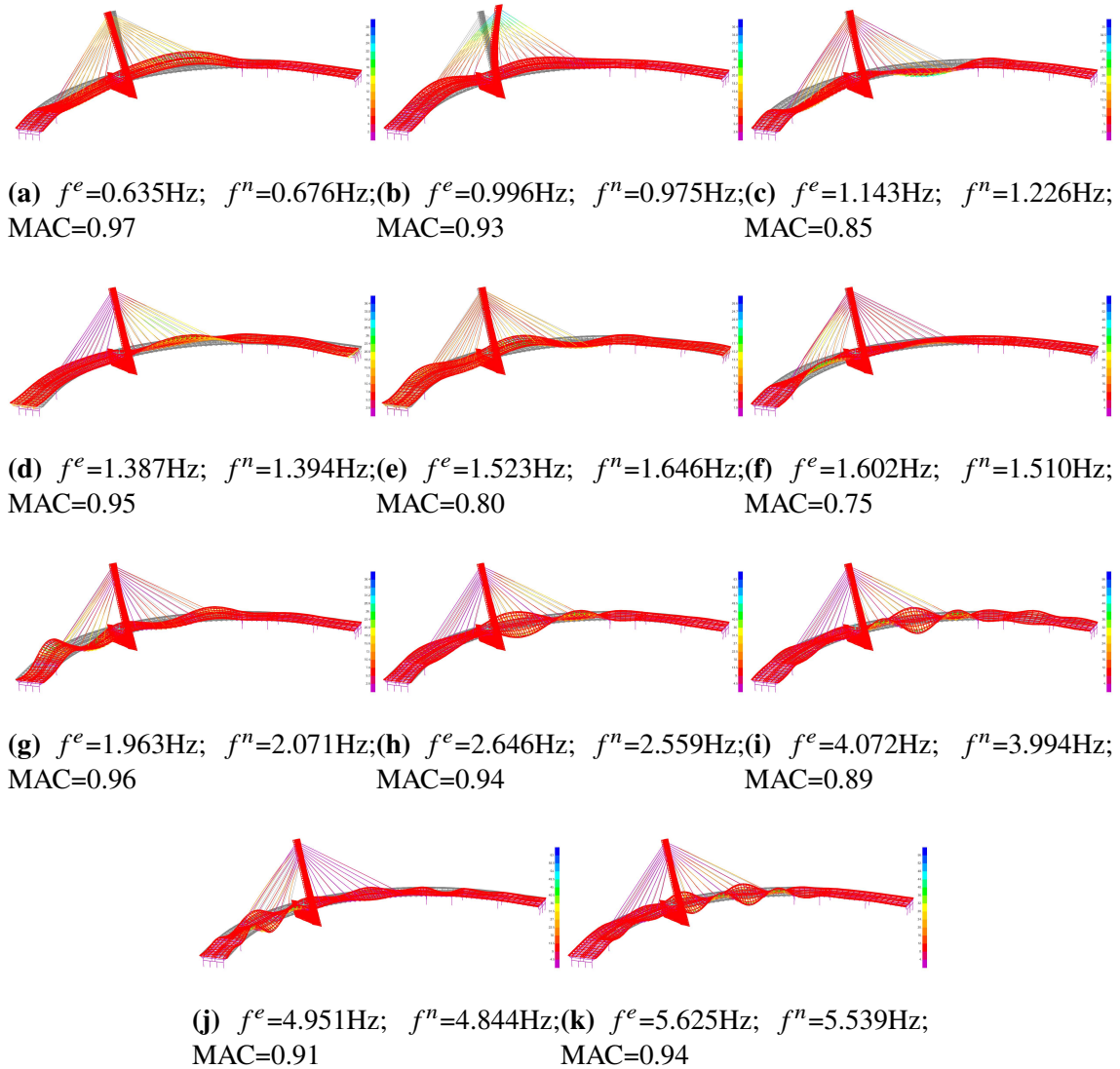


Fig. 8. Representation of a few selected numerical modes. f^e and f^n in the sub-captions indicate the experimental and numerical natural frequencies.

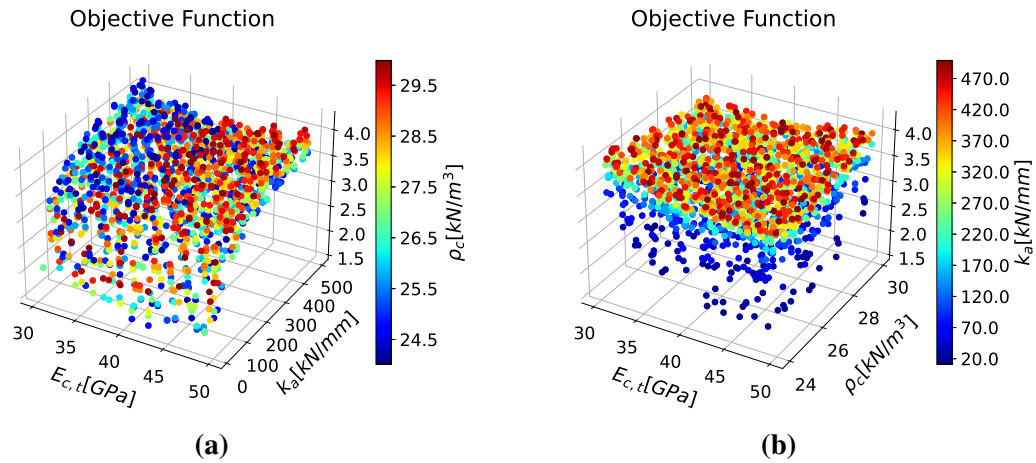


Fig. 9. Different views of the scatter plot of the sensitivity of the objective function in Eq. (3) to the concrete Young's modulus of the tower (E_c, t), the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c).

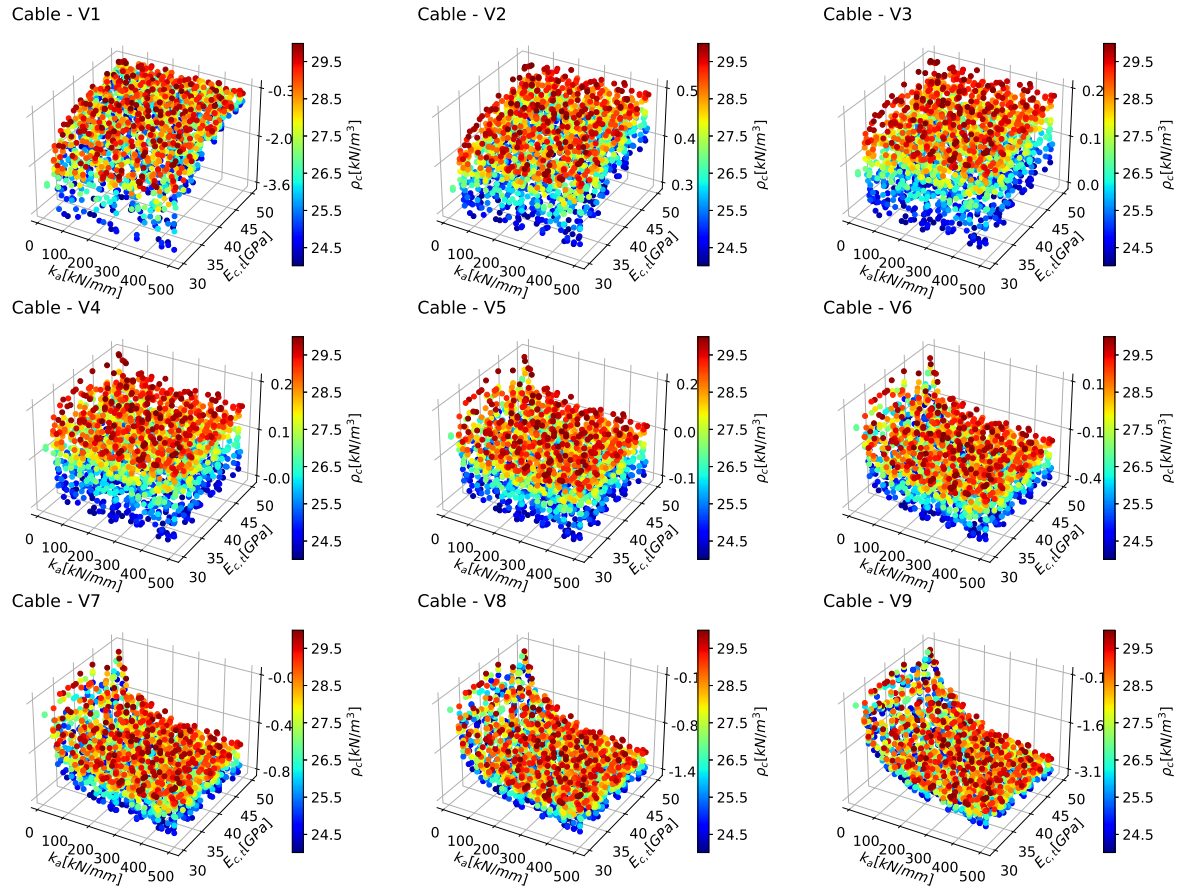


Fig. 10. Scatter plots of the sensitivity of the cable forces on the Venice side to the concrete Young's modulus of the tower ($E_{c,t}$), the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c).

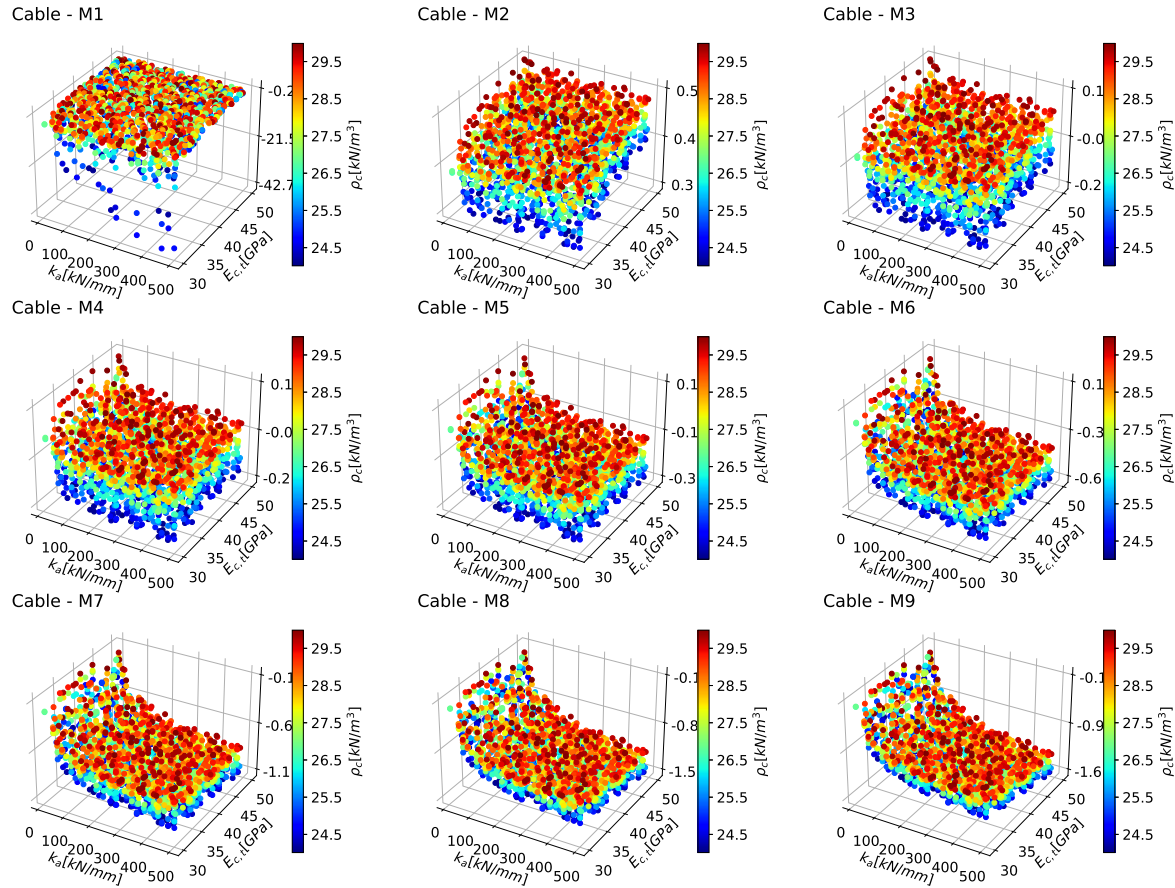


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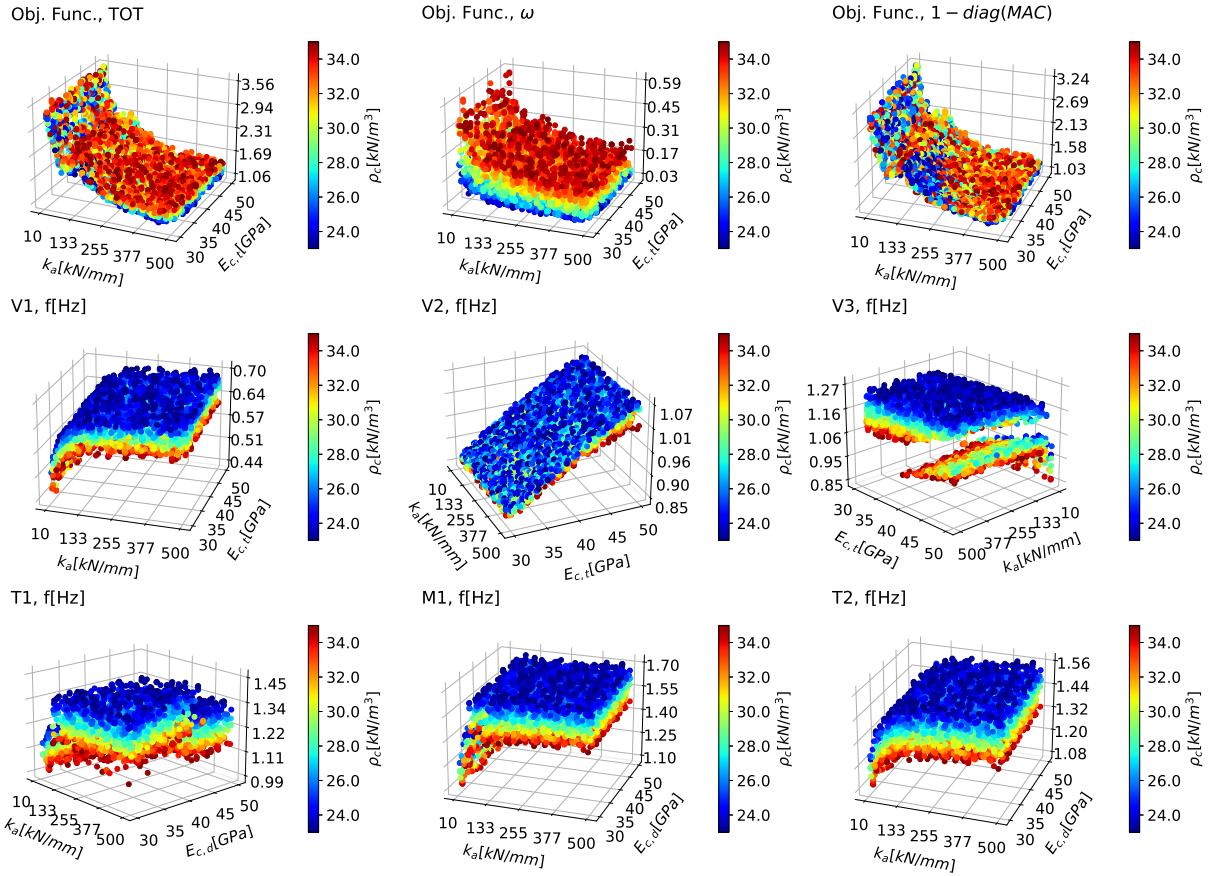


Fig. 12. Selected scatter plots of the sensitivity of the modal parameters to the concrete Young's modulus of the tower ($E_{c,t}$), the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c).

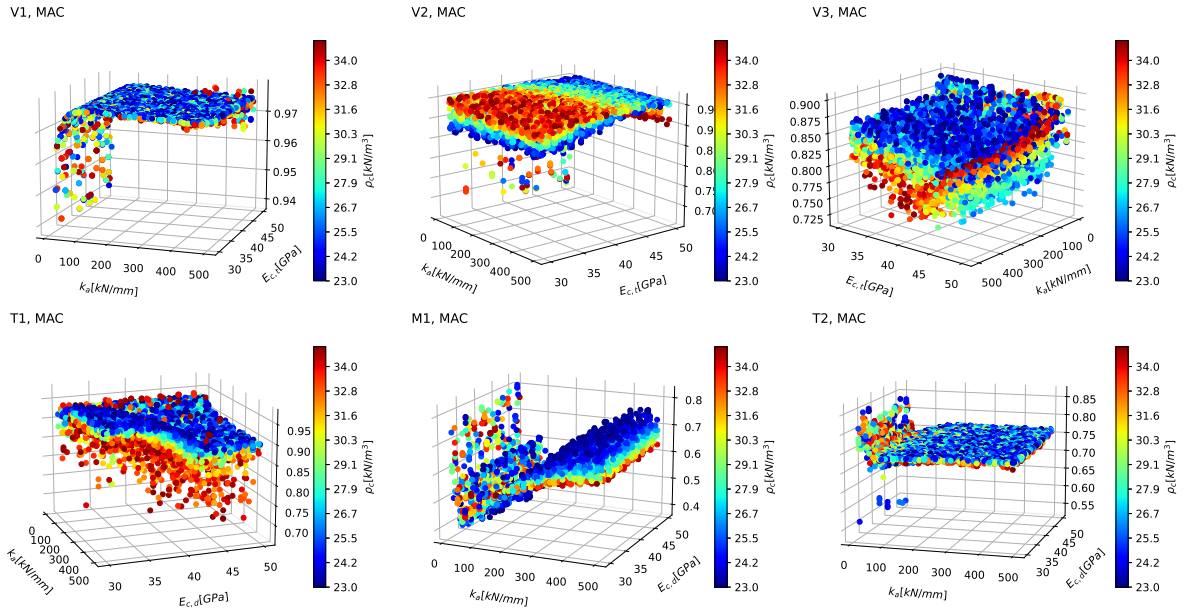


Fig. 13. Selected scatter plots of the sensitivity of the MAC to the concrete Young's modulus of the tower ($E_{c,t}$), the vertical stiffness of the bearings (k_a) and the mass of the concrete deck (ρ_c).