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Using the vibration envelope as damage-sensitive feature in composite beam structures

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Abstract

A novel approach of damage detection in composite steel-concrete composite beams is suggested. Based on the idea of using the envelope's profile deflections and rotations induced by a moving load, this approach can lead to a practical cost-effective alternative to the traditional use of accelerometers and laser vibrometers. A parametric study has been undertaken, quantifying the sensitivity of the dynamic response of a realistic composite bridge to the presence of damage at different levels of partial steel-concrete interaction and velocity of the moving load. When compared to shifts in the natural frequencies, it has been verified that the proposed approach generally enjoys a higher sensitivity (so damage can be detected at an early stage), is more effective closer to the ends of the bridge (where shear studs are more likely to be damaged), and displays an ordered set of results (which would reduce the possibility of a false damage). Further work is required to assess the effects of uncertainties and the adoption of more refined models for the moving load.

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Composite bridges, Damage detection, Moving load problem, Structural health monitoring, Non-destructive evaluation

1. Introduction

Composite steel-concrete beams are widely used in structural engineering, offering the advantages of construction efficiency, durability and improved economy [1–3]. Their performance is strongly influenced by the flexibility of the connection between concrete slab and steel, which generally allows a partial interaction between the two materials. In bridge engineering applications, faster trains and augmented traffic have significantly increased the number and amplitude of loading cycles experienced on a daily basis by composite bridges. This higher demand accelerates the occurrence of damage in the shear connectors, which in turn affects the overall integrity of the structure.

Conventional approaches of damage detection (including ultrasonic, thermal, eddy current and X-ray testing) were termed as cumbersome and expensive, and their application is often limited to the evaluation of local structural performance [4], while visual inspections represent an unreliable solution [5] (also because shear connectors are often inaccessible). Vibration-based damage detection methods have therefore emerged, as they allow identifying meaningful changes in the dynamic characteristics of the composite beam due to alterations in the mechanical properties of the structure [6], with little or no need for the user to know a priori where the damage might be located. Accelerometers have been extensively employed for this purpose,

22 although their application to large structural systems like composite bridges
23 may be difficult because of long cabling, number of sensors and installation
24 time. Laser doppler vibrometers (LDVs) can be used as a viable non-contact
25 alternative to accelerometers, especially when targets are difficult to access,
26 but large displacements can adversely affect measurements [7] and the simul-
27 taneous acquisition of vibration at multiple points would make very expensive
28 the dynamic testing.

29 In the general framework of structural health monitoring, vibration-based
30 methods can be classified into “model based methods”, which iteratively up-
31 date the numerical model of the structure to match some dynamic character-
32 istics experimentally measured, and “non-model based methods”, which di-
33 rectly compare changes in these characteristics, without any numerical model
34 being required [8]. In both cases, various dynamic characteristics can be ex-
35 ploited as damage-sensitive feature (DSF), including: natural frequencies and
36 modal shapes [9]; modal beam curvatures [10]; frequency response function
37 (FRF) [11]; modal flexibilities [12]; modal strain energy [13].

38 An early review of different methods of damage detection using natural
39 frequencies can be found in Ref. [14]. However it has become apparent that
40 environmental factors affect eigenfrequencies, which can then mask changes
41 due to damage events [15]. It was also argued that damage does not equally
42 affect all modal frequencies [4, 16].

43 Pascual et al. [17] suggested the use of operating deflection shapes (ODSs)
44 for assessing the presence of damage, while Limongelli [18] proposed an in-
45 terpolation damage detection method (IDDM), in which the deviation of the
46 deformed shape from a smooth behaviour is used as DSF. Zhang et al. [19]

47 proposed the global filtering method (GFM) as detection algorithm for beam-
48 and plate-like structures, using ODS curvatures extracted from the dynamic
49 response to moving loads.

50 When compared to other structural and mechanical systems, limited at-
51 tempts have been made to apply damage detection methods to shear connec-
52 tors in composite bridges, with the implementation of vibration-based meth-
53 ods being further restricted by modelling uncertainties of the connectors and
54 low sensitivities. Queiroz et al. [1] investigated full and partial shear connec-
55 tions using nonlinear springs in the FE (finite element) model of composite
56 beams, demonstrating that partial interaction effects should be considered in
57 the analysis. Xia et al. [8] introduced a local identification approach based
58 on the vertical vibration of slab and girders, which does not require baseline
59 data. Dilenia and Morassi [20–22] proposed an Euler-Bernoulli beam model
60 to describe the dynamic response of damaged composite beams based on
61 frequency shifts, showing that damage at interior connectors tends to cause
62 lower variations in the modal frequencies, while Liu and De Roeck [23] per-
63 formed a parametric study, investigating the behaviour of shear connectors
64 during train passages. It was shown that train speed influences the global be-
65 haviour of the bridge, and that the longitudinal shear force are not uniformly
66 distributed along the span, with critical regions located near the supports.

67 While all the above studies use the dynamic response in terms of acceler-
68 ations and/or displacements at a few locations (analysed in the time domain
69 and/or in the frequency domain), a radically different approach of dam-
70 age detection and quantification is envisaged in the present research, which
71 consists of analysing the envelope’s profile of vehicle-induced deflections in

Figure 1: Sketch of the structural problem.

the composite bridge. Instead of considering the whole time history of the dynamic response (and/or its frequency-domain counterpart), the proposed approach only uses the maximum and minimum values of displacements and rotations. Coupled with recent advances in the field of digital image analysis and processing (e.g. deblurring techniques for long-exposure imageries, recently developed by McCarthy et al. [24, 25] for structural dynamics applications), this can lead to an alternative non-contact high-sensitivity method of structural health monitoring for composite bridges, capable of assessing at an early stage the presence and severity of damage.

A set of encouraging preliminary results are presented in this paper, proving the concept in the simple case of a single moving force, although further investigation will be required to assess the effects of uncertainties in the dynamic problem (e.g. random stiffness and random damping of the track [26, 27]) and to extend this approach to more advanced models for the moving load (e.g. moving masses and moving oscillators [28, 29]).

2. Envelope-based damage measure

Let us consider the vehicle-induced vibration of a composite steel-concrete bridge, whose sketch is shown within Figure 1. If a set of moving forces

is adopted to represent the dynamic load and the structure is assumed to respond within the linear range, the equations of motion for the FE model can be written as:

$$\mathbf{M} \cdot \ddot{\mathbf{u}}(t) + \mathbf{C} \cdot \dot{\mathbf{u}}(t) + \mathbf{K} \cdot \mathbf{u}(t) = \mathbf{g} + \mathbf{f}(t), \quad (1)$$

where $\mathbf{u}(t)$ is the array collecting the DoFs (degrees of freedom) of the model; $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are the matrices of mass, equivalent viscous damping and elastic stiffness; while $\mathbf{g}$ and $\mathbf{f}(t)$ are the load vectors associated with the dead and moving forces, respectively. Interestingly, $\mathbf{f}(t)$ depends on the time t not because the magnitude of the applied forces varies, but because they move along the bridge. It is worth mentioning here that, for the sake of simplicity, Eq. (1) does not include the inertia effects due to the moving mass and any vehicle-bridge dynamic interaction phenomena, as they would require time-dependent mass, stiffness and damping coefficients [30, 31]. Such refinements of the model would be outside the scope of this work, which is aimed at assessing whether the envelope of the deformations caused by a moving load is sensitive enough to be used in a damage identification scheme instead of changes in the modal frequencies.

Once the governing equations are numerically integrated, the dynamic response of the bridge in terms of displacements and rotations can be expressed as linear combination of the DoFs:

$$\theta(t) = \mathbf{a}_\theta^\top \cdot \mathbf{u}(t), \quad (2)$$

where $\theta(t)$ is the generic response parameter (e.g. deflection at midspan, slope at the supports or the curvature at a given position along the bridge);

Figure 2: Modification of the envelope E_θ in a damaged structure (a) and geometrical representation of the damage measure $D_\theta = \tan(\alpha)$ (b)

111 $\mathbf{a}_\theta$ is the array collecting the associated influence coefficients; and the super-
 112 scripted symbol $\top$ stands for the transpose operator.

113 It is now possible to introduce the envelope of the dynamic response $\theta(t)$
 114 as the interval $[\Theta_1, \Theta_2]$ defined by its extreme values within the selected
 115 observation time interval $[0, T]$:

$$\Theta_1 = \min_{0 \leq t \leq T} \{\theta(t)\} ; \quad \Theta_2 = \max_{0 \leq t \leq T} \{\theta(t)\} , \quad (3)$$

116 such that $\Theta_1 \leq \theta(t) \leq \Theta_2$ for $0 \leq t \leq T$, and the amplitude of the envelope
 117 is (see Figure 2(a)):

$$E_\theta = \Theta_2 - \Theta_1 . \quad (4)$$

118 Alternatively, the amplitude of the envelope can be evaluated as:

$$E_\theta = \left(A_\theta^{(+)} + A_\theta^{(-)} \right) \theta_f . \quad (5)$$

119 where $A_\theta^{(+)}$ and $A_\theta^{(-)}$ are the dynamic amplification coefficients for the re-

120 sponse parameter $\theta(t)$, given by:

$$A_{\theta}^{(+)} = \max \left\{ \frac{\theta(t) - \theta_g}{\theta_f} \right\} ; \quad A_{\theta}^{(-)} = \max \left\{ -\frac{\theta(t) - \theta_g}{\theta_f} \right\} ; \quad (6)$$

121 θ_g is the static response due to the dead load:

$$\theta_g = \mathbf{a}_{\theta}^{\top} \cdot \mathbf{K}^{-1} \cdot \mathbf{g} ; \quad (7)$$

122 and θ_f if the reference value of the static response due to the moving load, i.e.
 123 the largest response obtained when the moving forces are applied statically
 124 at different positions on the bridge; formally:

$$\theta_f = \begin{cases} \theta_f^{(+)} & \text{if } |\theta_f^{(+)}| \geq |\theta_f^{(-)}| ; \\ \theta_f^{(-)} & \text{otherwise ;} \end{cases} \quad (8)$$

125 where $\theta_f^{(+)} = \max\{\theta_i\}$ and $\theta_f^{(-)} = \min\{\theta_i\}$ are the maximum and minimum
 126 values of the static responses $\theta_i = \mathbf{a}_{\theta}^{\top} \cdot \mathbf{K}^{-1} \cdot \mathbf{f}(t_i)$, ideally obtained by freezing
 127 the dynamic load vector at different time instants $t = t_i$, with $0 \leq t_i \leq T$.

128 If damage occurs, the dynamic response changes and in general the ex-
 129 tremes values defining the envelope will be different; that is: $\tilde{\Theta}_1 \neq \Theta_1$ and
 130 $\tilde{\Theta}_2 \neq \Theta_2$ (in which the over-tilde denotes the quantities affected by the dam-
 131 age).

132 A dimensionless damage measure (DM) D_{θ} can therefore be introduced
 133 as:

$$D_{\theta} = \frac{|\Delta\Theta_1| + |\Delta\Theta_2|}{E_{\theta}} , \quad (9)$$

134 in which the variation in the extremes of the envelope are:

$$\Delta\Theta_1 = \tilde{\Theta}_1 - \Theta_1 ; \quad \Delta\Theta_2 = \tilde{\Theta}_2 - \Theta_2 . \quad (10)$$

Figure 2(b) shows that the DM of Eq. (9) can be graphically interpreted as the tangent of the angle α formed by the amplitude of the envelope E_θ on the horizontal axis and the variations of the extreme values $\Delta\Theta_1$ and $\Delta\Theta_2$ on the vertical axis, that is: $D_\theta = \tan(\alpha)$.

For comparison purposes, a more traditional DM can be adopted, based on the reduction in the modal frequencies of the structure. Let $\omega_1, \omega_2, \dots, \omega_m$ be the first m undamped modal circular frequencies of the undamaged composite bridge, solution of the classical real-value eigenproblem $\omega_i^2 \mathbf{M} \cdot \phi_i = \mathbf{K} \cdot \phi_i$, ordered from the lowest to the highest; and let $\tilde{\omega}_1, \tilde{\omega}_2, \dots, \tilde{\omega}_m$ be the corresponding frequencies in a given damage scenario. The dimensionless DM associated with the i th modal frequency can be defined as:

$$S_i = \frac{\omega_i - \tilde{\omega}_i}{\omega_i}. \quad (11)$$

Depending on the dynamic characteristics of the structure, as well as on location and type of damage, different modal frequencies are differently affected by the damage. For this reason it is worth considering an overall DM for the first m modal frequencies which can be realistically determined with a dynamic test on the composite bridge:

$$\bar{S}_m = \max \{S_1, S_2, \dots, S_m\}. \quad (12)$$

3. Numerical investigations

In order to assess the potential for the proposed envelope-based measure D_θ (see Eqs. (9) and (10)) to be used as DSF in civil engineering structures, and specifically in composite steel-concrete bridges, a parametric study has

Figure 3: FE model of the composite bridge used as test structure.

155 been carried out with the idealised FE model of Figure 3, created and vali-
 156 dated with the commercial software SAP2000 [32].

157 Based on an existing structure [6], a single-span simply-supported com-
 158 posite bridge of length $L_b = 50$ m has been analysed; the mass density
 159 is $\rho_s = 7,850$ kg/m³ for the steel and $\rho_c = 2,500$ kg/m³ for the concrete;
 160 $E_s = 206$ GPa and $E_c = 31$ GPa are the corresponding Young’s moduli; three
 161 values of elastic stiffness have been considered for the shear-type connection
 162 between steel and concrete, namely $K_i = 0.077$ GPa for “soft” interaction
 163 and $K_i = 0.77$ GPa, $K_i = 7.7$ GPa for “medium” and “stiff” interaction
 164 respectively; cross sectional areas, $A_s = 7.7$ m² and $A_c = 5.6$ m², and sec-
 165 ond moments, $I_s = 11.95$ m⁴ and $I_c = 0.0747$ m⁴, fully define the geometry
 166 of steel girder and concrete slab, respectively; $d_b = 1.5$ m is the distance
 167 between the centroids of the two components.

168 A single concentrated force $F = 10$ kN has been used as test load, repre-
 169 senting a vehicular movement from left to right, with velocities $V = 250$ and
 170 300 km/h (see Figure 3).

171 The planar FE model of the objective structure has 201 DoFs and consists
 172 of two parallel chords, each one discretised with $N = 40$ beam elements, plus

173 $N - 1$ elastic springs for the shear connectors (which are assumed to be
 174 uniformly distributed), while a rigid constraint is applied to the transverse
 175 movement of the two chords. As a result, concrete slab and steel girder
 176 experience the same amount of deflection but different axial displacements,
 177 and the interlayer slip depends on the stiffness of the shear connectors (e.g.
 178 [33, 34]).

179 3.1. Dynamic amplification

180 In a first stage, the dynamic amplification has been computed for increas-
 181 ing values of the velocity V of the moving force. Figure 4 confirms that the
 182 dynamic response of the bridge tends to increase with V , for both midspan
 183 deflection (denoted with δ_M) and rotation at the right end (denoted with
 184 φ_R). In each graph, the top curves $A^{(+)}$ refer to movements with the same
 185 sign as the corresponding static quantities (i.e. downward displacements at
 186 midspan and counterclockwise rotation at the right support of the bridge),
 187 while the bottom curves $A^{(-)}$ refer to the maxima with opposite sign. Al-
 188 though the pair of $A^{(+)}$ and $A^{(-)}$ has a very similar trend in the two graphs,
 189 there are some differences, e.g. the right rotation tends to show higher values
 190 of dynamic amplification for $V > 400 \text{ km h}^{-1}$, while relative maxima of the
 191 dynamic amplification can occur at different velocities, e.g. $V = 280 \text{ km h}^{-1}$
 192 for $A_{\delta_M}^{(+)}$ and $V = 310 \text{ km h}^{-1}$ for $A_{\varphi_R}^{(+)}$. Interestingly, the dynamic amplifica-
 193 tion is also seen to increase with the flexibility of the connection, particularly
 194 at higher values of V .

195 Figure 5 shows the envelopes E_{δ_M} and E_{φ_R} , normalised with respect to the
 196 corresponding reference values of the static response (see Eq. (5)). It appears
 197 that the envelope is highly sensitive to the velocity V of the moving force.

Figure 4: Dynamic amplification factors for (a) midspan displacement, (b) right support rotation

198 In both graphs, for instance, a relative valley and a relative peak appear for
 199 velocities close to $V = 250$ km/h and $V = 300$ km/h, and these values have
 200 therefore been used in the next set of dynamic analyses with moving forces.

201 3.2. Damage sensitivity

202 3.2.1. Modal frequencies

203 Stiffness reduction at a given location of the structure generally causes
 204 the modal frequencies to drop, which in turn can indicate the presence of
 205 damage at a global level. However the same amount of damage at different
 206 locations may induce different amount of frequency changes. A parametric
 207 study has then been carried out to quantify the effectiveness of such variations
 208 as detection feature of a damage occurring at the interface between concrete
 209 slab and steel girder in the objective composite bridge. Figure 6 shows the
 210 colour maps of the sensitivity matrices $\mathbf{S}$ for two values of the elastic stiffness

Figure 5: Envelope of dynamic amplification factors for (a) midspan displacement, (b) right support rotation

211 of the shear connectors, assuming in both cases that the localised damage
 212 corresponds in the FE model to a 90% reduction in the stiffness of the j th
 213 shear spring. The generic coefficient $f_{i,j}$ of each matrix is the dimensionless
 214 frequency shift S_i of Eq. (11) for a damage occurring at the j th position
 215 $x_j = j \Delta x$, in which $j = 1, \dots, N - 1$ and $\Delta x = L_b/N = 125$ cm is the size
 216 of the FE discretisation.

217 The two colour maps lend themselves to the following considerations:

- 218 1. Due to the symmetry of the problem, the maps are symmetric with
 219 respect to the midspan position ($j = 20 = N/2$);
- 220 2. The sensitivity tends to increase with the level of partial interaction;
- 221 3. For each mode i , the sensitivity is higher when the location x_j of the
 222 damage is close to a point of contraflexure in the associated modal
 223 shape, e.g. close to the ends of the bridge (i.e. $j = 1$ and $j = N - 1$),
 224 where the shear force is larger;

Figure 6: Damage sensitivities $f_{i,j}$ for the natural frequencies associated with the first eight flexural modes of vibration in case of medium (a) and stiff (b) partial interaction.

4. Conversely, the sensitivity of a given mode i reduces when the location x_j of the damage is close to a zero-value point in the shear force diagram of the associated modal shape (that is, if the shear force is relatively small, the effect of a damage in the shear studs at that position will be relatively negligible).

As a consequence, the first mode of vibration only shows a good level of sensitivity if the damage occurs near to the supports of the bridge, while higher modes of vibration reveal damage at additional positions. Furthermore, different modes have different sensitivity levels for the same damage position (e.g. a damage at midspan affects second and fourth mode of vibration, as shown by a warm spot in the colour maps, but does not affect first and third mode). Therefore, different modes of vibration are required to detect the presence of damage, meaning that a large number of sensors

238 may be required for practical applications.

239 A further observation is that the mode number i with the highest sen-
240 sitivity S_i to damage in the shear connectors at $x = x_j$ may vary with the
241 level of partial interaction between the concrete slab and shell girder. Indeed,
242 while for the case of medium interaction (Figure 6(a)) the first mode shows
243 the highest sensitivity values for position index $j \leq 5$ (and $j \geq 35$), modes
244 $i = 2, 3$ and 4 appear to be more sensitive in the case of stiff interaction
245 (Figure 6(b)).

246 3.2.2. Envelope of deflections

247 In a second stage, it has been numerically verified that the envelope E_{δ_i}
248 of the deflection $\delta_i(t)$ at the output position $x = x_{i-1}$ can be used as sensitive
249 feature for a localised damage in the shear connector at the position $x = x_j$,
250 with $1 \leq i \leq N + 1$ and $1 \leq j \leq N - 1$. Considering all the possible
251 combinations of output position and damage position in the FE model, the
252 relevant sensitivity matrix $\mathbf{D}$ has been obtained, where the generic element
253 $d_{i,j}$ is the dimensionless DM D_{δ_i} of Eq. (9), in which: $\theta(t) = \delta_i(t)$; damage
254 occurs at the j th shear spring; the observation time interval is $[0, L_b/V]$,
255 which corresponds to the time required by the moving force F to cross the
256 bridge.

257 A set of $N - 1$ time-history analyses was therefore required (i.e. one
258 analysis for each damage location), and this was repeated four times (as two
259 levels of partial interaction K_i , medium and stiff, and two velocities of the
260 moving force V were studied). Including the undamaged scenarios, a total of
261 158 dynamic analyses were carried out, whose results are summarised with
262 the four colour maps of Figure 7, in which warmer colours show where the

263 sensitivity to the damage is higher.

264 In comparison with the results of Figure 6, higher values of sensitivity
265 have been computed, meaning that the envelope of displacements is poten-
266 tially more effective than the modal frequency shift as DSF (that is, the
267 maximum frequency sensitivity $f_{i,j}$ in Figure 6 is about 0.1, while the sensi-
268 tivity of E_{δ_i} in Figure 7 is about 0.6, more than five times higher). Clearly
269 the actual performance of the method will depend on the velocity V of the
270 moving force, which therefore needs to be carefully selected. For instance,
271 at relatively low value of V , say, $V < 100\text{km/h}$ in the case study, very lit-
272 tle dynamic effects are expected, and therefore any attempt to identify the
273 presence of damage in the bridge could become difficult for the presence of
274 noise in the measurements and other forms of uncertainties.

275 Additionally, the sensitivity coefficients $d_{i,j}$ are higher at the ends of the
276 bridge, i.e. for $j \leq 5$ or $j \geq 35$, and tend to decrease with the distance
277 between damage position and output position, i.e. with $|i - j|$. While the
278 first feature is acceptable from an engineering point of view, since damage
279 in shear studs is unlikely to happen toward the middle of the bridge, where
280 lower levels of shear stress are expected, the second feature is highly desirable,
281 as it makes easier the localisation of the damage by looking at the position
282 where the maximum variation in the envelope of displacements is observed.

283 Interestingly, the effects of damage on the envelope E_{δ_i} are more localised
284 in the case of stiff concrete-steel interaction, while comparatively the varia-
285 tion in the velocity V has a less significant impact on the sensitivity coeffi-
286 cients $d_{i,j}$.

287 3.2.3. *Envelope of rotations and curvatures*

288 Further sensitivity analyses were carried out on the test bridge using the
289 rotation φ_i and the curvature χ_i at the generic abscissa $x = x_{i-1}$ as DSFs
290 (with $i = 1, \dots, N + 1$). While the rotation φ_i was obtained directly from
291 the dynamic analyses (being a DoF of the FE model), the curvature was
292 computed as $\chi_i = M_s(x_{i-1})/E_s I_s$, $M_s(x)$ being the bending moment in the
293 steel girder at the generic abscissa x . The results in terms of rotation's
294 sensitivity coefficients $r_{i,j}$ and curvature's sensitivity coefficients $q_{i,j}$ are pre-
295 sented in Figures 8 and 9, respectively, being $r_{i,j} = D_{\varphi_i}$ and $q_{i,j} = D_{\chi_i}$ for a
296 concentrated damage occurring at the j th shear spring in the FE model.

297 The same trends predicted by the envelope of displacements are verified
298 for the case of rotations. In particular, similar sensitivity levels have been
299 computed for the rotations at the ends of the bridge, and the localisation
300 tends to improve with the rigidity of the inner layer. Interestingly, a rota-
301 tions' sensitivity to damage increases at midspan position with respect to the
302 envelope of the displacements.

303 The results for the envelope of the curvatures are quite different. In
304 particular, their sensitivity shows large peaks closer to the ends of the bridge,
305 while reduced values are noted elsewhere. Moreover, increased sensitivity is
306 also observed for the stiffer shear connectors, with minimal differences due
307 to the velocity of the load.

308 3.2.4. *Comparison*

309 In order to assess the relative performance of different DSFs for the com-
310 posite bridge under consideration, the maximum value attained by the vari-
311 ous sensitivity coefficients has been computed for each damage position j , e.g.

312 $f_j = \max \{f_{i,j}, i \leq 6\}$ for the frequency shifts $d_j = \max \{d_{i,j}, i \leq N + 1\}$
 313 for the envelope of displacements. The semi-logarithmic plots of Figure 10
 314 compare the four DSFs f_j , d_j , r_j (envelope of rotations) and q_j (envelope
 315 of curvatures) for two velocities of the moving load and two levels of partial
 316 steel-concrete interaction. It appears that the curvature (dotted blue lines) is
 317 highly sensitive to damage occurring close to the ends of the bridge. Unfortu-
 318 nately, it is particularly difficult to track curvature changes using non-contact
 319 measurements on a bridge structure, and for this reason the envelope of the
 320 curvature appears as the least practical approach.

321 Shifts in the natural frequencies (green solid lines) are very effective for
 322 stiff partial interaction and damage close to the ends of the bridge (see Fig-
 323 ures 10(b) and (d)). However, a sudden drop follows when moving towards
 324 the middle of the bridge. Importantly, while this DSF is independent of the
 325 load velocity, as it only uses modal information, its performance is highly de-
 326 pendent on the level of partial interaction, and indeed for the case of medium
 327 stiffness this is less effective approach (see Figures 10(a) and (c)).

328 The envelope of both displacements (red dashed lines) and rotations
 329 (black dot-dashed lines) appear as viable DSFs, with on average a slightly
 330 better performance for the rotations, although in practical applications it
 331 would be easier to get the displacements. It must also be stressed that, if an
 332 imagery type of approach is used to determine the envelopes (e.g. Refs. [24]
 333 and [25]), deflection and rotations can potentially be simultaneously tracked.
 334 It is also worth stressing here that in practical applications the envelope
 335 of both the undamaged and damaged bridge must be available under the
 336 same loading conditions, as correlating the two dynamic responses will allow

337 identifying the damage.

338 4. Conclusions

339 In this paper, a novel approach for damage detection in composite steel-
340 concrete bridges is suggested, in which the envelope of deflections and rota-
341 tions induced by moving loads are used as DSFs (damage-sensitive features).
342 While in traditional vibration-based approaches discrete-time signals of dis-
343 placements, strains or accelerations from field experimentation are collected
344 and analysed (either in the time domain or in the frequency domain), the
345 proposed approach only requires the extreme values of the dynamic response
346 to be known. As hardware and software for digital imagery continuously
347 progress, such information can potentially be acquired more economically
348 and more easily than in conventional methods. Moreover, since no special
349 sensors are needed, but just visible targets, more deflections and rotations
350 can be simultaneously monitored, which can improve the accuracy of damage
351 detection.

352 To prove the concepts, numerical analyses have been carried out on the
353 finite element model of a realistic composite bridge, assuming a single moving
354 force as dynamic excitation. In a first stage, it has been shown that the
355 dynamic effects associated with the moving load are significant, and tend to
356 increase with the flexibility of the shear connectors between concrete slab
357 and steel girder.

358 In a second stage, the effects of damage at different locations were quan-
359 tified for both medium and stiff partial interaction and for two velocities of
360 the moving force. In this way, any significant anomaly in the performance of

361 the proposed approach could have been spotted.

362 As expected, the results have demonstrated that the envelope of the dy-
363 namic response in terms of deflections and rotations tends to increase when
364 damage occurs. More importantly, about the same level of sensitivity to dam-
365 age was observed for shifts in the modal frequencies (which in the current
366 practice is often used as DSF) and variations in the envelope of deflections
367 and rotations, whose sensitivity did not suffer from significant changes when
368 the level of partial interaction and the velocity of the moving force were
369 varied. Additionally, the proposed approach was found to be most effective
370 closer to the ends of the bridge, where damage is more likely to happen, and
371 was shown to display an ordered set of results, that can potentially enhance
372 the predictiveness of any damage-detection algorithm.

373 Although these preliminary results are very promising, further numeri-
374 cal and experimental investigations need to be undertaken to fully develop
375 the method, explore its practical limitations and verify the application to
376 real structures. Moreover, due to the scalability of the imageries for the
377 extraction of the envelopes, this new approach could be potentially applied
378 to structures at different scales, from large civil-engineering buildings and
379 bridges to mechanical components and even nano-devices.

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Figure 7: Damage sensitivities $d_{i,j}$ for the displacement's envelope E_{δ_i} in case of medium (left) and stiff (right) partial interactions at velocities of the force $V = 250$ km/h (top) and 300 km/h (bottom).

Figure 8: Damage sensitivities $r_{i,j}$ for the rotation's envelope E_{δ_i} in case of medium (left) and stiff (right) partial interactions at velocities of the force $V = 250$ km/h (top) and 300 km/h (bottom).

Figure 9: Damage sensitivities $q_{i,j}$ for the curvature's envelope E_{δ_i} in case of medium (left) and stiff (right) partial interactions at velocities of the force $V = 250$ km/h (top) and 300 km/h (bottom).

(a)

(b)

(c)

(d)

Figure 10: Performance of different damage-sensitive features in case of medium (left) and stiff (right) partial interactions at velocities of the force $V = 250$ km/h (top) and 300 km/h (bottom).