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Optimal prestress design of composite cable-stayed bridges

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ABSTRACT

The present study considers the optimal pre-tensioning design of lattice structures forming composite cable-stayed bridges. With reference to a model problem, a target bending moment distribution over the longitudinal beams is identified, with the aim of achieving an optimized use of the material composing the bridge. Next, a procedure for the optimization of cable forces is developed, in order to achieve the desired bending moment distribution through the application of a self-equilibrated state of stress induced by optimal cable pre-tensioning. Results indicate that the given design approach is suitable for the optimization of the pre-tensioning sequence of arbitrary composite cable-stayed bridges.

1. Introduction

The use of composite materials in long-span bridges is attracting increasing interest, as these materials exhibit higher stiffness-to-weight and strength-to-weight ratios compared with traditional structural materials. Composite materials are extremely lightweight and display extreme fatigue resistance and high durability in any type of environment. Unlike traditional technologies, their qualities also included speed of execution and complete reversibility. Other benefits include low energy consumption during manufacturing, construction and execution processes [1–6]. The use of hybrid Fiber Reinforced Polymer (FRP) cables in long-span cable-stayed bridges is an area of particular interest [7–12]. Their excellent durability means that composite materials also entail low maintenance costs. Such properties are very useful for extending the service life of bridge structures, as bridges are usually exposed to severe environmental conditions [13]. While the cost of composite materials is generally greater than that of traditional structural materials, the extended life of a composite structure results in reduced long-term structural costs. Low maintenance costs are one of the most important considerations in Bridge Life Cycle Cost Analysis (BLCCA).

Several examples of long-span cable-stayed bridges using composite materials have been developed in recent years, including Sutong Bridge (1088 m span) in China, Stonecutters Bridge (1018 m) in Hong Kong, and Tatara Bridge (890 m) in Japan [7]. The Sutong Bridge is a cable-stayed bridge with double-plane and twin-pylon. Two auxiliary piers and one transitional pier were erected in each side span. The main span of the bridge is 1088 m making it the world's longest main cable-stayed bridge span. The stay cables are arranged in double inclined cable planes and made of a parallel wire strands consisting of 7 mm wires. Other pedestrian and vehicular bridges have been constructed in the past using FRP cables, including Aberfeldy footbridge (UK, 1992 – one of the finest such examples), Herning Bridge (Denmark, 1999) and I-5/Gilman Bridge (USA, 2002), the latter being a composite cable-stayed bridge with an eccentric type pylon [8,9].

The construction of cable-stayed bridges is characterised by a series of phases in which geometry, boundaries, and loads vary significantly, causing changes in the state of stress [14–19]. The optimization of the construction process via the regulation of the initial forces in cables is important for the optimal control of the whole structural behaviour [20]. One of the most common problems when dealing with cable-stayed bridges concerns the computing of the initial cable forces and the pre-tensioning sequence, which are needed to obtain the designed configuration [21]. An optimal pre-tensioning sequence is useful for controlling stress and strain during and after the construction phases. The literature offers several different approaches [18,19], and the best solution remains an open issue. In fact, there are no closed form analytical solutions

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that allow for the computing of the pre-tensioning sequence given a final design configuration. Only iterative algorithms are available [18,19], although these require several cable tightening operations that lead to technological, structural, and economic problems.

The absence of closed form solutions is due to the large number of parameters in characterizing cable stress distribution in CSBs. The structural behaviour of CSBs depends on geometry, statics, material properties, construction process, and technology. The development of tools to control the behaviour of these structures is therefore an open issue.

The present work deals with the formulation of a procedure for the optimization of cable pre-tensioning forces that is suitable for any kind of CSBs. The proposed approach allows for the optimization of the bending moment distribution in the deck under relevant pre-tensioning force values. We employ the 'influence matrix method' to compute the optimal pre-tensioning sequence that guarantees the achievement of the designed bending moment distribution (BMD), which is statically equivalent to another target distribution.

The proposed procedure falls under the so-called 'force equilibrium methods' [20,22] methods that act directly on the internal forces and indirectly on the elastic deformations. It can be applied to any construction sequence and can be generalized to account for time-dependent phenomena [23]. It is worth noting that the overall structural system of a cable-bridge, which is composed of the bridge deck, the supporting piers and the suspending cables, may be regarded as a composite lattice structure whose mechanical performance and structural weight can be profitably optimized by experimenting with different optimal design of cables' prestress.

2. Model problem

Our study refers to a composite-material re-design of the At Tannumah Bridge (ATB), which is a part of a highway viaduct connecting Basra city centre to the At Tannumah area in Iraq, passing over the Shatt al Arab River. The original design of such a bridge was developed by the Studio 'De Miranda Associati' in Milan, Italy.

The present bridge concept includes a semi-fan, a central suspended span, and a self-supported cable-stayed system (Fig. 1). In the longitudinal direction, the bridge's mid-span is symmetric. It includes two towers and two sets of cables. The deck is made of three spans – a central span of 150 m and two lateral spans of 75 m each. The deck is assembled from semi-precast elements with lengths of 12.50 m each. The towers are made of pre-built sections of reinforced high-strength concrete (class C45/55), as per Eurocode 2 [24].

In the original design, the deck is made of steel welded beams (class S355) [23] and a reinforced concrete slab (class C25/30) [23] of 26 cm thickness. The connection between the concrete slab

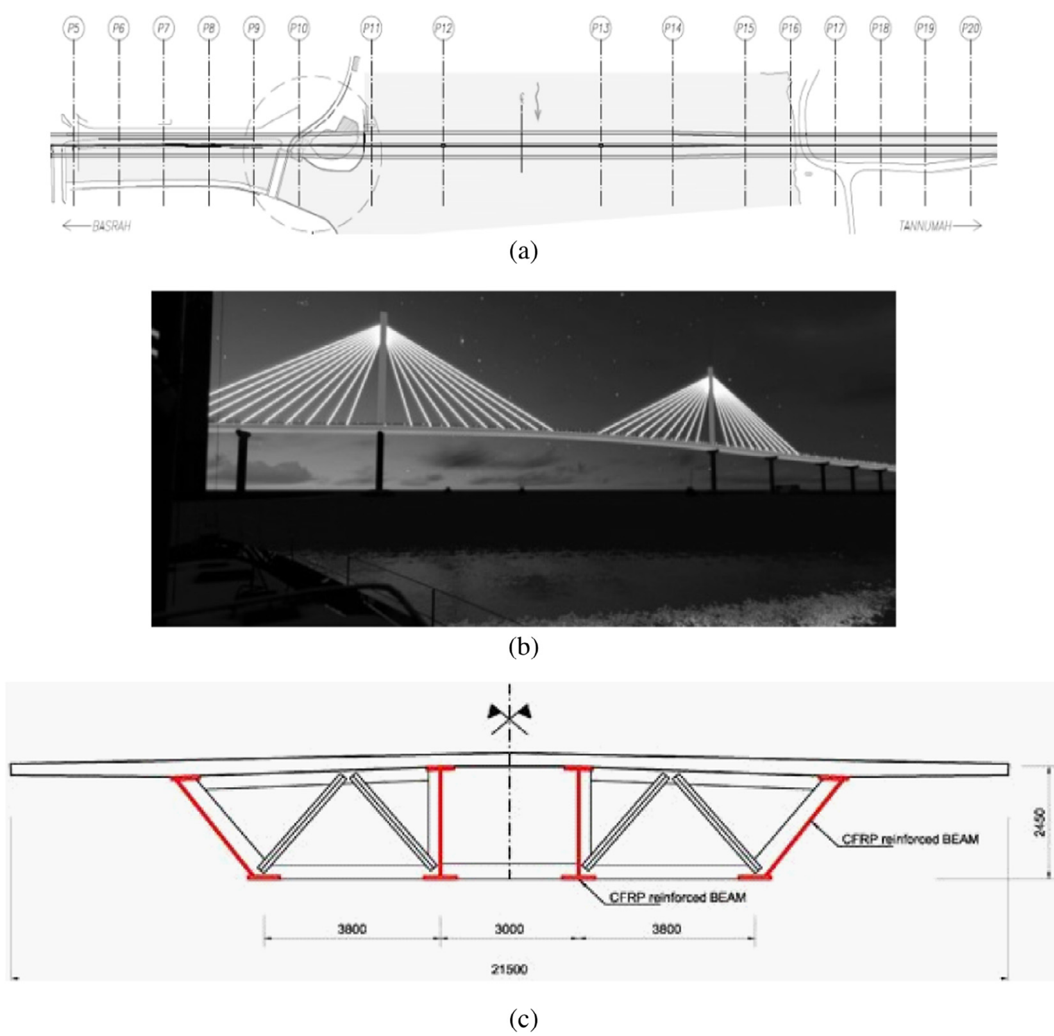

Fig. 1. The At Tannumah Bridge: (a) top view, (b) 3D view, (c) deck cross section [16] (courtesy of Studio 'De Miranda Associati', Milan, Italy).

and the steel beams is made with metallic bolts. The original S355 steel welded beams are replaced with S235 steel welded beams wrapped with Carbon Fiber Reinforced Polymer (CFRP) layers (Fig. 1c) in the present study. This composite redesign of the longitudinal beams uses a kind of steel (S235) with 1/3 strength, as compared with that used in the original ATB design (S335). The S235 steel is employed in such a way as to obtain overall strength and stiffness equal to those of the original S335 beams, under approximately the same weight (Fig. 1c). In this study, the steel cables of the original ATB design (cables with diameter of 22 cm made of a set of 110 braided steel wires with diameter of 16 mm each) are replaced with equivalent hybrid basalt and carbon cables (B/CFRP), using the principle of cable replacement given in [8]. In the new design, BFRP wires are arrayed in the centre of the cable, while hybrid B/CFRP wires are arrayed in the outer layer of the cable. A viscoelastic material is inserted in the gap between the center and outer layers [8].

Fig. 1 shows different views of the ATB, while Fig. 2 illustrates the layout of the structural model that we employed to describe such a bridge. The elastic problem of the CSB model in Fig. 2 has been solved by assuming a linear elastic response for all the bridge elements, using the numerical algorithm detailed in [19] and references therein.

Hereafter, we refer to the bridge model S_0 (Fig. 3a) that corresponds to assumed zero pre-tensioning forces in the cables (bridge unstressed under zero external loads) as 'initial'. We shall see in the next section that such a model induces a highly non-uniform moment distribution over the longitudinal bridge axis, which is not ideal for the optimal use of the material (assuming uniform cross section of the deck along the span).

Similarly, we have named the realization S_d of the bridge model in Fig. 3(a) that corresponds to pre-tensioning forces inducing a bending moment distribution over the deck identical to that exhibited by a low-stiffness re-design of the ATB deck ('optimal' bending moment distribution) as 'optimal design' [25]. The model deriving from a low stiffness re-design of the deck is hereafter referred to as 'auxiliary', and denoted by S_a (Fig. 3b).

It is shown in [25], where different alternative designs of the ATB are compared, the above optimal bending moment distribution is nearly uniform over the span, determining an optimized use of the material composing the deck. The proposed optimal design procedure allows us to obtain a target bending moment distribution through the application of a self-equilibrated state of stress induced by optimal cable pre-tensioning. Our final goal is to experiment with the pre-tensioning forces of the cables of the model in Fig. 2(a), in order to achieve on the real bridge the same bending moment distribution shown by the auxiliary bridge model S_a in Fig. 2(b).

3. Initial and optimal bending moment distributions

Figs. 4 and 5 show the bending moment and cable force distributions for the initial and auxiliary bridge models defined in the previous section. It is worth noting that the bending moment distribution over the longitudinal beams of the initial model (M_0) is rather non-uniform along the deck, featuring a peak value for the deck-pier junction that is more than five times larger than the moment at the middle of the span (Fig. 4a). In the same model, the axial force carried by the most-stressed cable (cable # 7) is 3.28 times larger than the axial force carried by cable # 1 (Fig. 4b).

The optimal bending moment distribution (M_d) is illustrated in Fig. 5, with M_d showing a much more uniform distribution over the span than M_0 (Fig. 4a).

As stated previously, M_0 was computed on the real model of the ATB prescribing zero pre-tensioning forces in all cables. Such moment distribution on the initial portions of the two branches

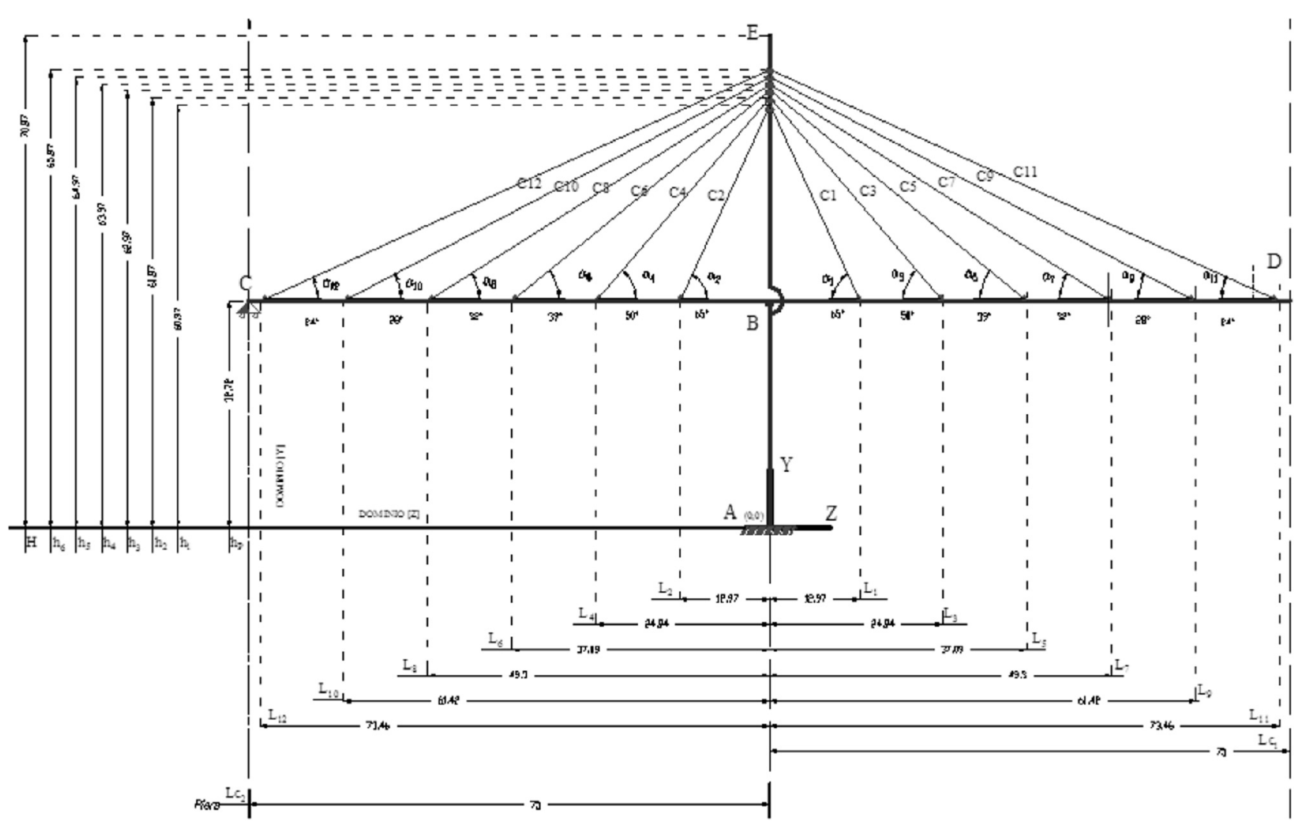

Fig. 2. Lattice structure of the adopted bridge model.

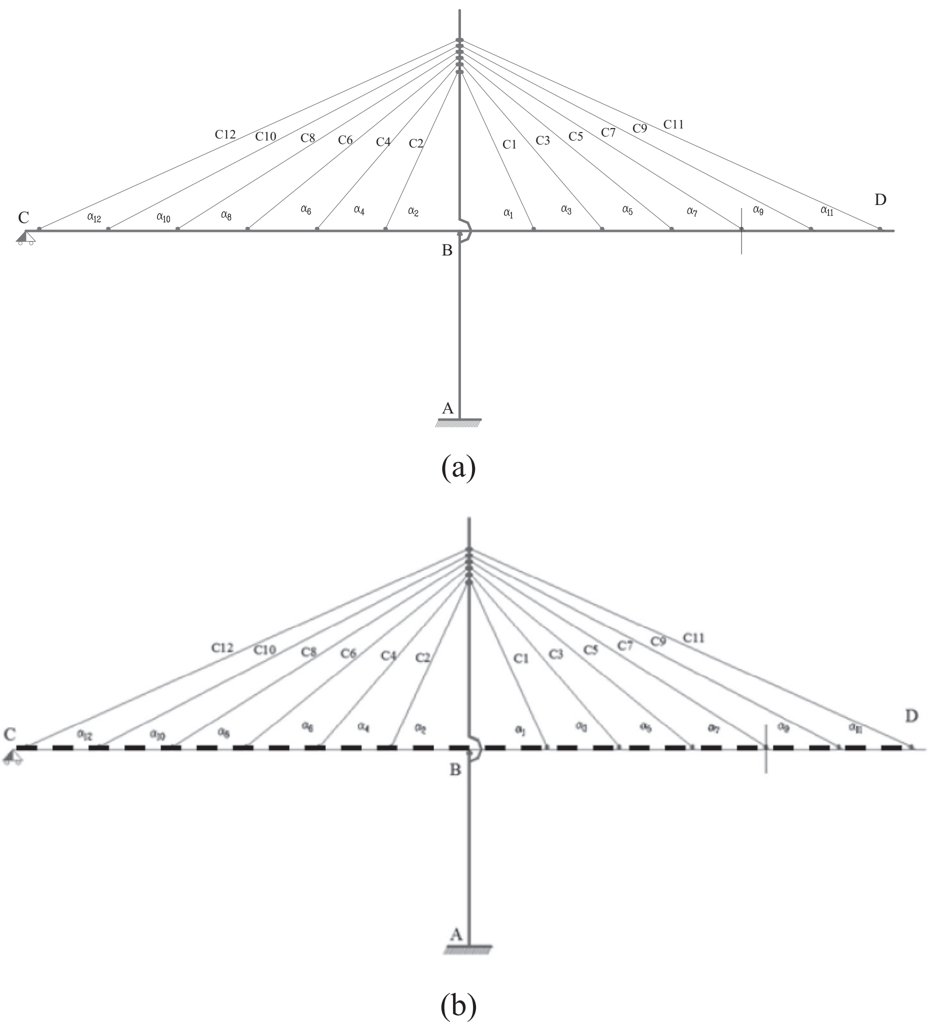

Fig. 3. (a) Initial bridge model S_0, (b) auxiliary bridge model S_a. The dashed thick line indicates a low stiffness deck.

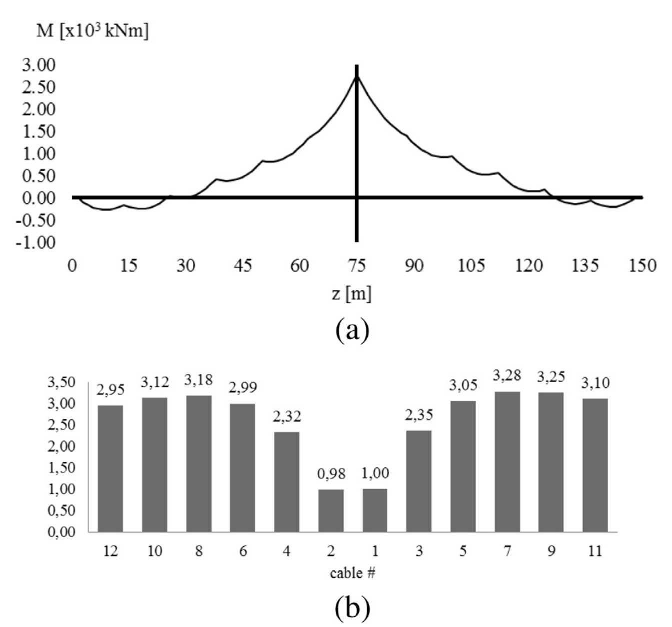

Fig. 4. (a) Bending moment distribution (M_0) and (b) normalized cable forces t_{0i} (divided by the axial force carried by cable #1 for the initial model (zero cable pre-tensions).

departing from the central pier (Fig. 4a) resembles that of a cantilever beam under uniform transverse loading. The M_d bending moment distribution was instead computed on a auxiliary bridge model with a low-stiffness deck and zero cable pre-tensions [25]. Such moment distribution over the initial portions of longitudinal beams departing from the pier resembles that of a multi-support continuous beam under uniform transverse loading (Fig. 5). The goal of the following section is to show how one can get the bending moment distribution M_d over the real bridge model (Fig. 2a), by playing with a suitable cable pre-tensioning.

4. Algorithm for the computation of the optimal pre-tension forces

This section is devoted to the formulation of a pre-tensioning design of the S_0 model, which ensures that the bending moment distribution over the longitudinal beams corresponds to M_d (Fig. 5). Let $\boldsymbol{X} = \{x_1, \cdots x_n\}^T$ denote the vector collecting the pre-tensioning forces of all cables (n = 12). By repeatedly solving elastic problems of the bridge model in Fig. 2(a), we compute the axial force carried by the j-th cable when the i-th cable is subject to a unit axial force (S_i system). Let d_{ij} denote such a force coefficient, and let **D** denote the $n \times n$ influence matrix collecting all such entries [25]. We are interested in solving the following linear problem [26]:

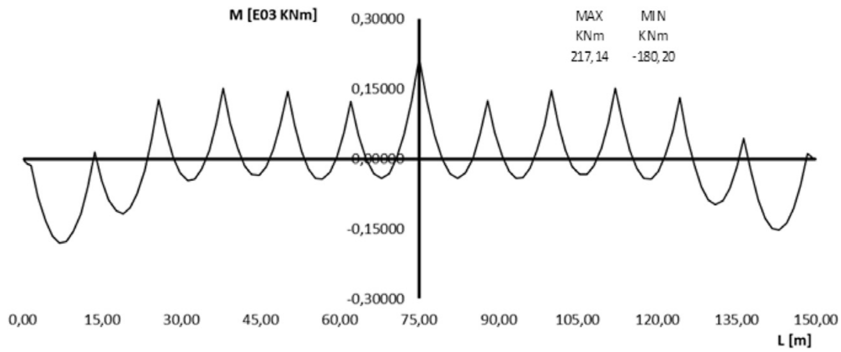

Fig. 5. Target bending moment distribution for the optimal model (M_d).

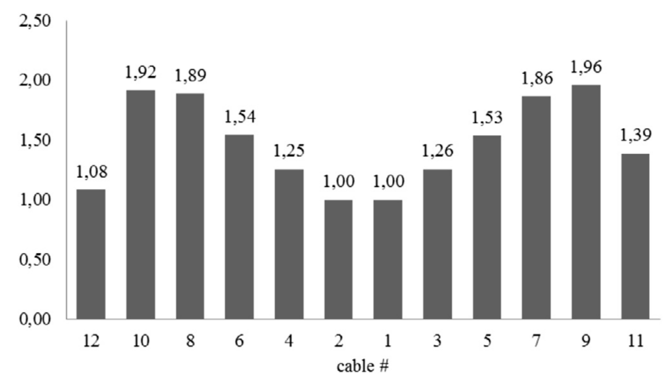

Fig. 6. Normalized cable forces t_{di} (divided by the axial force carried by cable #1) for the optimal model (including cable pre-tensions Δt_i).

$$\boldsymbol{D}^T \boldsymbol{X} = \Delta \boldsymbol{T} \tag{1}$$

where $\Delta \boldsymbol{T} = \{\Delta t_1, \cdots \Delta t_n\}^T$ is the vector with current entry $\Delta t_i = t_{di} - t_{0i}$, t_{di} and t_{0i} denoting the forces in the i-th cable, respectively, in correspondence of the optimal design and in the initial models. The algebraic system of Eq. (1) is obtained by solving the n + 2 elastic systems $S_0, S_a, S_1, \ldots, S_n$. Its solution allows us to determine the pre-tensioning forces Δt_i to be applied to the different cables in order to achieve the target bending moment distribution M_d (Fig. 6). The optimal cable force distribution illustrated in Fig. 6 shows a more uniform profile compared to that of the initial model (Fig. 4b), with a maximum cable force (in cable #9) that is 1.96 times larger than the force carried by cable #1.

5. Concluding remarks

We have presented an approach to the optimal pre-tensioning design of composite cable-stayed bridges whose aim is to achieve a target bending moment distribution over the deck. Such an approach allows designers to recover from construction errors that could compromise the structural safety of the bridge.

The proposed methodology is based on the matrix of influence method, and relies on the determination of the cable forces on n+2 elastic structural models, with n denoting the total number of cables. It can be applied to any construction phase and can be easily generalized to account for major dynamic effects (such as, e.g., wind and fluttering); time-dependent phenomena due, e.g., to material viscosity [23], and/or fracture damage [26]. Such a bridge design technique can also be applied to prestressed concrete structures, arch bridges with suspended decks, and tensegrity bridges [25,27,28]. Other fields of application of the current influence matrix approach include the study of the optimal prestress of tensile reinforcements for existing masonry and concrete structures [22,29–33]; the optimal design of cable-stayed bridges using innovative concretes reinforced with 3D printed rebars [34–36]; and the optimal pre-tensioning of mechanical metamaterials to be used as novel seismic isolation devices [37,38].

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