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Optimization of stay cables in cable-stayed bridges using finite element, genetic algorithm, and B-spline combined technique

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ABSTRACT

Traditionally, a trial-and-error procedure is carried out to design cross-sectional areas of stay cables in cable-stayed bridges. This design process is monotonous, expensive, time-consuming, and incapable of finding the optimum design solution. The aim of this study is to develop a robust design optimization technique in order to achieve the minimum cross-sectional areas of stay cables. The developed optimization technique integrates finite element method, B-spline curves, and genetic algorithm. The capability and efficiency of the proposed optimization technique is tested and assessed by applying it to a practical sized cable-stayed bridge.

1. Introduction

Because of their aesthetic appeal, ease of erection, efficient utilization of structural materials, and other notable advantages, cable-stayed bridges have found wide applications all over the world in the last few decades [1]. Bridges of this type have recently entered a new era with main spans exceeding a value of 1000 m. In modern long-span cable-stayed bridges, such as the Sutong Bridge in China (2088 m), a large number of stay cables would be required in order to achieve reasonable distribution of bending moments along the bridge deck. The unit cost of stay cables is relatively high compared to other construction materials; therefore, there is a need for the development of an optimization technique to determine the minimum cost of stay cables in cable-stayed bridges.

In the current practice, the design process of stay cables is performed in two stages. The first stage involves the determination of initial post-tensioning cable forces, which are evaluated corresponding to zero vertical deflections of the deck and zero horizontal deflections of the pylons' tops under only self-weight of the bridge. These forces are required to determine the initial configuration of the bridge. In the second stage, the cross-sectional areas of stay cables are determined under the combined effect of self-weight, initial post-tensioning cable forces, and live load cases. To date, this design stage is based on a trial-and-error procedure, which depends on the designer's experience and skills [2,3]. A set of cross-sectional areas of stay cables is first assumed. Structural analysis for the bridge is then carried out in order to obtain the bridge deflections and stresses. If the deflections and stresses satisfy the requirements imposed by design codes, the assumed cross-sectional areas of stay cables are adopted. Otherwise, the cross-sectional areas are modified and the structural process is repeated until all the design criteria are met. The previous iterative design procedure is expensive, tedious, and time consuming. Moreover, it does not guarantee that the final solution will be the best of all the possible design solutions that satisfy the requirements of design codes

There have been many studies concerning the determination of the optimum post-tensioning cable forces under self-weight [4–15]; however, there have been only a few attempts to determine the optimum cross-sectional areas of stay cables under self-weight, initial post-tensioning cable forces, and live load cases. One of the first attempts was conducted by [16]. In their study, a convex scalar function was used to minimize the cost of a box-girder deck cable-stayed bridge. The proposed function combines dimensions of the cross-sections of the bridge and post-tensioning cable forces. This method is very sensitive to the constraints, which should be imposed very cautiously to obtain a practical output [8]. In the research done by [3], the optimization module implemented in MATLB (*fmincon*), together, with the commercial finite element software, ABAQUS, are employed to evaluate the minimum cost of stay cables for cable stayed bridges.

It should be noted that the two previous studies are based on direct search optimization techniques. The drawback of these

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Nomenclature

A_i	cross-sectional area of the cable i
A_{max}	maximum cross-sectional area of the cable
C	the objective function
E_{eq}	equivalent modulus of elasticity of cables
E_{cs}	modulus of elasticity of cables
H	pylon height
L	total length of the bridge
L_i	length of the cable i
L_c	horizontal projected length of the cable
LC	total number of live load cases
M	main span length
N_{cables}	total number of stay cables
N_{Area}	total number of cross-sectional areas of stay cables
N_{Pylons}	number of pylons
S	side span length
T	tension in the cable
w_{cs}	weight per unit length of cables
σ_i^k	tensile stress in stay cable i for the live load case k
σ_{Max}	maximum allowable tensile stress in stay cables
δ_i^k	vertical deflection of the deck at the cable position i for the load case k
δ_{Max}	maximum allowable vertical deflection of the deck
Δ_j^k	horizontal deflection of the tops of the pylon j for the load case k
Δ_{Max}	maximum allowable horizontal deflection of the tops of pylons
γ_{cable}	unit weight of stay cables

direct techniques is that they begin the search procedure with a guess solution, which is often chosen randomly in the search space. If the guess solution is not chosen close enough to the global minimum solution, the optimization technique will be trapped in local minima. As a result, the final solutions of these previous studies may not be the global minimum [3]. On the other hand, the cross-sectional areas of stay cables are considered as discrete design variables in both studies. With the increase in the number of stay cables, the number of design variables becomes quite large leading to potential numerical problems in the optimization technique. In addition, the increase in the number of stay cables makes the final distribution of the cross-sectional areas of stay cables non-smooth. Hence, the resulting values from these methods may be impractical in such cases.

The objective of the current study is to present a powerful optimization design technique in order to achieve the optimum cross-sectional areas of stay cables, which is directly proportional to the cost of the material. The proposed study focuses on the second design stage, where self-weight, initial post-tensioning cable forces, and live load cases are applied to the bridge. The proposed optimization technique involves interaction between three numerical schemes: finite element method (FEM), B-spline curves, and Real Coded Genetic Algorithm (RCGA). The novelty of this combined technique lies in the adoption of the B-spline curves to represent the distribution of cross-sectional areas of stay cables along the bridge length, which significantly reduces the number of design variables. In addition, RCGA, which is a global optimization method, is capable of finding the global optimal solution.

The remainder of the paper is organized as follows. In the next section, the geometry, finite element modeling, and design loads of the bridge chosen for the study are described. In Section 3, a description of the design variables, design constraints, objective function, and optimization technique is presented. A detailed description of the optimization design technique that involves a combination between the FEM, B-spline curves, and RCGA is presented in Section 4. In Section 5, detailed presentation and discussion of the numerical optimization results are given. Finally, Section 6 presents the main conclusions drawn from the study.

2. Description, finite element modeling, and design loads of the bridge

2.1. Description of the bridge

The geometry of the bridge chosen for this study is similar to the Quincy Bayview Bridge, located in Illinois, USA [17]. The length of the main span (M) is 285.6 m, with two side spans (S) of 128.1 m. Therefore, the total length of the bridge (L) is 541.8 m, as shown in Fig. 1a.

The deck superstructure is supported by double planes of stay cables in a semi-harp type arrangement, where forty cables are anchored into each transverse H frame shaped pylon. As such, eighty stay cables support the whole bridge, where forty cables support the main span and twenty cables support each side span.

The bridge has two lanes of traffic having a width of 12.2 m measured between centers of the cables. The typical cross-section of the bridge deck (Fig. 1b) consists of a precast concrete deck having a thickness of 0.23 m and a width of 14.20 m. Two steel main girders are located at the outer edge of the deck. These girders are interconnected by a set of equally spaced floor steel beams. The distance between each pair of floor steel beams is 9.0 m.

The pylons consist of two concrete legs, interconnected with a pair of struts. The upper strut cross beam connects the upper legs and the lower strut cross beam supports the deck. The lower legs of the pylon are connected by a 1.22 m thick wall, which is placed as a web between the two legs, as shown in Fig. 1c.

2.2. Finite element modeling of the bridge

In general, the elastic stay cables are assumed to be perfectly flexible and to resist a tensile force only. The inclined stay cables of cable-stayed bridges will sag into a catenary shape due to their self-weight [18]. The tension stiffness of a cable, which varies depending on the sag, is modeled by using one straight truss element with an equivalent modulus of elasticity. The concept of an equivalent cable modulus of elasticity was first proposed by [19]. The equivalent cable's modulus of elasticity used to account for the sag effect is given by:

$$E_{eq} = \frac{E_{cs}}{1 + \frac{(w_{cs}L_c)^2 A_i E_{cs}}{12T^3}} \tag{1}$$

where E_{eq} is the equivalent modulus of elasticity; E_{cs} is the cable material effective modulus; L_c is horizontal projected length of a cable; w_{cs} is the weight per unit length of the cable; A_i is the cross-sectional area of the cable i; and T is the tension in the cable.

On the other hand, three-dimensional frame elements are used to model the deck and the pylon. The deck is modeled using a single spine passing through its shear center. The axial stiffness of the deck and the moments of inertia about the vertical and transverse axes are obtained by converting the concrete slab to an equivalent steel section, using the ratio of the two elastic moduli. The finite element model of this bridge is shown in Fig. 1d.