Engineering Structures 172 (2018) 919–928

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Optimization of cable pre-tension forces in long-span cable-stayed bridges considering the counterweight

Chaolin Song, Rucheng Xiao, Bin Sun*

Department of Bridge Engineering, Tongji University, Shanghai 200092, China

ARTICLE INFO

Keywords:
Optimization
Cable-stayed bridges
Cable pre-tension forces
Counterweight

ABSTRACT

Over the past 20 years, many long-span cable-stayed bridges with asymmetric spans have been constructed, and the counterweight is always used to balance the self-weight of the main span. This paper presents an optimization method to determine the cable pre-tension forces in long-span cable-stayed bridges considering the counterweight. This method includes: finite element (FE) model, formulation of the optimization problem and optimization algorithm. FE model is established considering the geometrical nonlinearity. The optimization problem is formulated with the objective of minimum weighted total bending energy. In addition, the constraints for the cable pre-tension forces, the bending moment of the girder and the tower, the load of the counterweight, the bearing reactions of the transition piers and auxiliary piers are all implemented in the optimization model. The optimization algorithm solves the optimization problem through the variable-step search along each design variable including the cable pre-tension forces, the load and the range of the counterweight. The efficiency and the accuracy of the proposed method are demonstrated by an application example and the results exhibit the importance of considering counterweight in the design of asymmetric cable-stayed bridges.

1. Introduction

Cable-stayed bridges are widely used for large span constructions because of aesthetic and economic grounds. Since the first modern cable-stayed bridge was built in 1955, the number of cable-stayed bridges and their span-length have been increased rapidly [1–3]. Over the past 20 years, cable-stayed bridges have opened up a new era with spans over 1000 m, such as the Ostrov Russkiy Bridge with a main span of 1104 m, the Hutong Yangtze Bridge with a main span of 1094 m, the Sutong Yangtze Bridge with a main span of 1088 m and the Stonecutter Bridge with a main span of 1018 m [4].

Despite all its advantages, there have been several concerns over the use of cable-stayed bridges. Cable-stayed bridges are usually statically indeterminate structures, and their structural behavior is greatly influenced by cable pre-tension forces. Therefore, the determination of cable pre-tension forces is critical in the design procedure. For long-span cable-stayed bridges, advanced analysis techniques with greater accuracy and precision are required to determine the optimum cable pre-tension forces.

Early, model tests have been used for determining the cable pre-tension forces [5,6]. With the development in computers and numerical methods, many papers concerning the shape-finding of cable systems have appeared [7–10].

Lazar et al. [11] were among the first to study the optimization of cable force in cable-stayed bridges. The load-balance method was used to determine the pre-tension cable forces of cable-stayed bridges. Firstly, the influence matrix of the bending moments due to a unit force applied successively along each stay cable of the bridge is determined. Then, a system of equations is written to express that the bending moments caused by pre-tension forces of stay cables shall be opposite in sign to the bending moment due to the dead load and equal in absolute value. By solving this system of equations, the pre-tension forces in cables are determined.

Wang et al. [12] proposed the zero displacement method to determine the cable pre-tension forces and the initial configuration of the bridge. The configuration of zero deflections along the girder is taken as the target and the cable forces are obtained by iterative calculation. Zhang [13] improved the zero displacement method using a Kriging surrogate model. The improved method is easier to converge and more time-saving. Nevertheless, the moment distribution is not rational when the vertical profile of the girder is significant.

Wang et al. [14] presented four methods to determine the cable pre-tension forces, namely: minimizing the summation of squares for vertical displacements along the girder (MSSVD), minimizing maximum moment of the girder (MMM), continuous beam method (CBM) and simple beam method (SBM). The MSSVD and MMM both use sequential

* Corresponding author.
E-mail address: sunbin@tongji.edu.cn (B. Sun).

https://doi.org/10.1016/j.engstruct.2018.06.061
Received 9 March 2018; Received in revised form 12 June 2018; Accepted 17 June 2018
0141-0296/

unconstrained minimization techniques (SUMTs) to solve the problem. However, it is not applicable to the situation where the design variables exceed 10. The simple beam method (SBM) is recommended by the author. The cable pre-tension forces are determined by considering the girder of a cable-stayed bridge equivalent to the behavior of the continuous beam on elastic support. Nevertheless, it is difficult to control bending moments at girder-tower junctions and because the cable forces are determined based on the equilibrium of forces, the nonlinearities are not considered.

Chen et al. [15] presented the force equilibrium method. In this method, the cable forces are considered as independent variables for achieving the target bending moments along the girder. The approximate influence coefficients are calculated and the cable pre-tension forces are obtained iteratively. However, this method has the same problems with the SBM.

Janjic et al. [16] presented the unit load method (ULM). The ULM determines the adequate factors that should be multiplied to the applied unit loads to achieve a desired bending moment distribution. The ULM can take time-dependent effects and geometrically nonlinear behavior into account. Asgari et al. [17] improved the ULM through the application of an inverse problem based on the ULM. The improved method costs shorter simulation time and results in less stresses in bridge members.

Negrão et al. [18] and Simões et al. [19] proposed that cable pre-tension forces could be determined by minimizing a convex scalar function. Martins et al. [20–22] improved the method to include the time-dependent effects, the construction sequence and the geometrical nonlinearities. In recent years some modern methods (such as genetic algorithm (GA) and support vector machines) have been applied to solve the optimization problem [23–25].

For long-span cable-stayed bridges, the side span and the main span are usually asymmetric. Therefore, the counterweight is used to balance the self-weight of the main span [26]. So far, to the knowledge of the authors, there are few comprehensive studies carried out to combine the counterweight design with the optimization of cable pre-tension forces. Most optimization methods of cable pre-tension forces concern symmetric cable-stayed bridges and do not take the influence of the counterweight into account.

In this paper, an optimization method to determine the cable pre-tension forces considering the counterweight is proposed. The paper is organized as follows. Section 2 describes the formulation of the optimization problem. Section 3 introduces the algorithm to solve the optimization problem. Section 4 presents an application example to demonstrate the efficiency and accuracy of this method. Section 5 provides concluding remarks.

2. Optimization problem formulation

The determination of the cable pre-tension forces is posed as optimization problem. This involves the definition of the design variables, the design objectives and the design constraints.

2.1. Design variables

Normally for long-span cable-stayed bridges, the side-to-main span ratio is relatively small. Hence massive counterweights are commonly installed at the side span to keep the tower straight and to reduce the bending moment in side span (see Fig. 1).

In this paper, the counterweight is assumed to be distributed uniformly in the area from the transition pier. The design variables are the cable pre-tension forces, the load and the range of the counterweight. The global design variable vector is

$$[x_1, x_2 \cdots x_n, x_{n+1}, x_{n+2}]^T \tag{1}$$

where x_i is the pre-tension force of the stay cable i, x_{n+1} is the load of the counterweight and x_{n+2} is the range of the counterweight.

2.2. Objective function

The objective function determines the desired reasonable state of the cable-stayed bridge. If the maximum deflection of the girder is defined as the objective function, the desired state is "zero displacement" configuration. If the moment distribution in critical sections is defined as the objective function, the desired state is target bending moment distribution along the girder and the tower. In this paper, the weighted total bending energy of the girder and the tower is defined as the objective function.

$$U = \alpha \times \sum_{a=1}^{n} \left(\frac{L_a}{4E_a I_a}(M_{ai}^2 + M_{aj}^2) \right) + \beta \times \sum_{b=1}^{m} \left(\frac{L_b}{4E_b I_b}(M_{bi}^2 + M_{bj}^2) \right) \tag{2}$$

where α is the weighted factor of the girder, n is the total number of the girder element, L_a is the length of the girder element a, $E_a I_a$ is the bending stiffness of the girder element a, M_{ai} and M_{aj} are the bending moment at the ends of the girder element a, β is the weighted factor of the tower, m is the total number of the tower element, L_b is the length of the tower element b, $E_b I_b$ is the bending stiffness of the tower element b, M_{bi} and M_{bj} are the bending moment at the ends of the tower element b.

It should be noted that the distribution of the bending moment can be adjusted by changing the weight factor of the objective function. If the bending moment in a specific area is expected to be reduced, the weighted factor of the element in this area is supposed to be increased [30,31].

2.3. Design constraints

Four design constraints are considered to avoid unreasonable state, which are reflected by penalty function.

The cable pre-tension forces should be limited to a reasonable range to makes the cross sectional area of each cable determined fall within a feasible region.

$$T_{\min} \leqslant T_i \leqslant T_{\max} \quad i = 1 \cdots n \tag{3}$$

where T_i is the pre-tension force of the stay cable i, $T_{\min}$ is the minimum allowable cable pre-tension force, $T_{\max}$ is the maximum allowable cable pre-tension force and n is the total number of the stay cables.

The moment distribution along the girder and the tower should also be limited to an acceptable range, which ensures that the tower is mainly under axial compression and that the stress of the girder does not exceed the allowable stress.

$$\begin{cases} |M_b| \leqslant M_{b,\max} \\ |M_t| \leqslant M_{t,\max} \end{cases} \tag{4}$$

where $|M_{\mathrm{b}}|$ is the maximum absolute moment of the girder, $|M_t|$ is the maximum absolute moment of the tower, $M_{b,\max}$ is the maximum allowable moment for the girder and $M_{t,\max}$ is the maximum allowable moment for the tower.

The load of the counterweight per unit length should be allowable. Otherwise the girder may not have enough space to accommodate the counterweight.

$$0 \leqslant w \leqslant w_{\max} \tag{5}$$

where w is the load of the counterweight per unit length and $w_{\max}$ is the maximum allowable load of the counterweight per unit length.

The bearing reactions of the transition piers and the auxiliary piers should be reasonable to ensure that the bearing reactions are within the region of bearing capacity for the commonly used bears.

$$0 \leqslant R_{\min} \leqslant R_i \leqslant R_{\max} \quad j = 1 \cdots m \tag{6}$$

where R_i is the bearing reaction of the transition pier or the auxiliary pier j, $R_{\min}$ is the minimum allowable bearing reaction, $R_{\max}$ is the maximum allowable bearing reaction and m is the total number of the transition piers and the auxiliary piers.