Engineering Structures 167 (2018) 166–174

Contents lists available at ScienceDirect

Engineering Structures

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

An analytical study on the bending moment acting on the girder of a long-span cable-supported bridge suffering from cable failure

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ARTICLE INFO

Keywords:
Long-span cable-supported bridge
Progressive collapse
Analytical approach
Cable-loss scenario
Least squares method

ABSTRACT

In this study, the structural behavior of a long-span cable-supported bridge after the sudden rupture of one of its axial cables is of concern. Cable failure produces large bending moments on the girder of the bridge. Henceforth, the focus of this study is to find the “maximum bending moment” on the girder due to the cable failure. For this purpose, an analytical approach based on differential equations of the system will be used. Then, an approximation function for a simplified bridge model in a cable-loss scenario will be derived. The use of the least squares method is also applied to minimize the error of the approximation function. The proposed approximation function has been checked by numerical models, and its good accuracy has been proven. The results show that by increasing the ratio of the bending stiffness of the girder to the axial stiffness of the cables, cable failure produces a larger bending moment on the girder.

1. Introduction

Progressive collapse is defined as the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it [1]. It is characterized by a distinct disproportion between the triggering event and the resulting widespread collapse [2].

The two most important guidelines that address progressive collapse, the General Services Administration guideline (GSA [3]) and the Unified Facilities Criteria (UFC [4]), are exclusively designed for buildings. Standards addressing progressive collapse and cable-loss scenarios in bridges are few and far between. According to Post-Tensioning Institute (PTI [5]), the sudden loss of any one cable must not lead to the rupture of the entire structure. In addition, regarding the simplified design method (i.e. linear static analysis), a dynamic amplification factor (DAF) of two must be applied. However, recent research proves that the suggested DAF is only safe for the design of cables, and that it is not safe for the design of pylons or girders with negative moments [6–10].

Recently, the issue of cable failure in bridges has been studied in some research experiments [11–15]. In [14,15], the collapse behavior of a cable-stayed bridge in a cable-loss scenario has been investigated. It was shown that the initial failure of three adjacent short cables, which were responsible for the stabilization of the bridge girder in compression, caused the lack of bracing in the girder. The girder began to buckle in the vertical direction as a result of high normal forces, and finally an instability type of collapse occurred in the girder. The different types of collapse and their specifications are explained comprehensively in [16,17]. A parametric study has also been conducted on the dynamic response of cable-stayed bridges to the sudden failure of a cable. It was shown that the sudden failure of a cable produced large bending moments on the deck and pylons [9,10].

In this study, a simplified bridge model is considered. Then, an analytical approach based on differential equations of the system is used, and an approximation function for a long-span cable-supported bridge is derived. It is shown that the proposed approximation function and the results of numerical models are in a good agreement.

2. Simplified bridge model

To use the analytical approach, a simplified bridge model is considered. In Fig. 1, the simplification procedure is depicted. The simplified model consists of a beam suspended from tension elements. As shown in Fig. 1, the simplified model considers a unique axial stiffness in each cable. To make the mathematical procedure straightforward, a reference axial stiffness (K) is used and the stiffness of the cables is expressed as a multiple of the reference stiffness ($K_i = \delta_i K$).

In this study, only a part of the bridge is considered. Therefore, there are interferences in the border regions. The borders to account for the additional regions of the girder are investigated on the one hand as fixed supports, and on the other hand as hinged supports. By doing so, two extreme values limiting the real behavior of actual systems are

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https://doi.org/10.1016/j.engstruct.2018.04.017
Received 30 July 2017; Received in revised form 5 April 2018; Accepted 6 April 2018

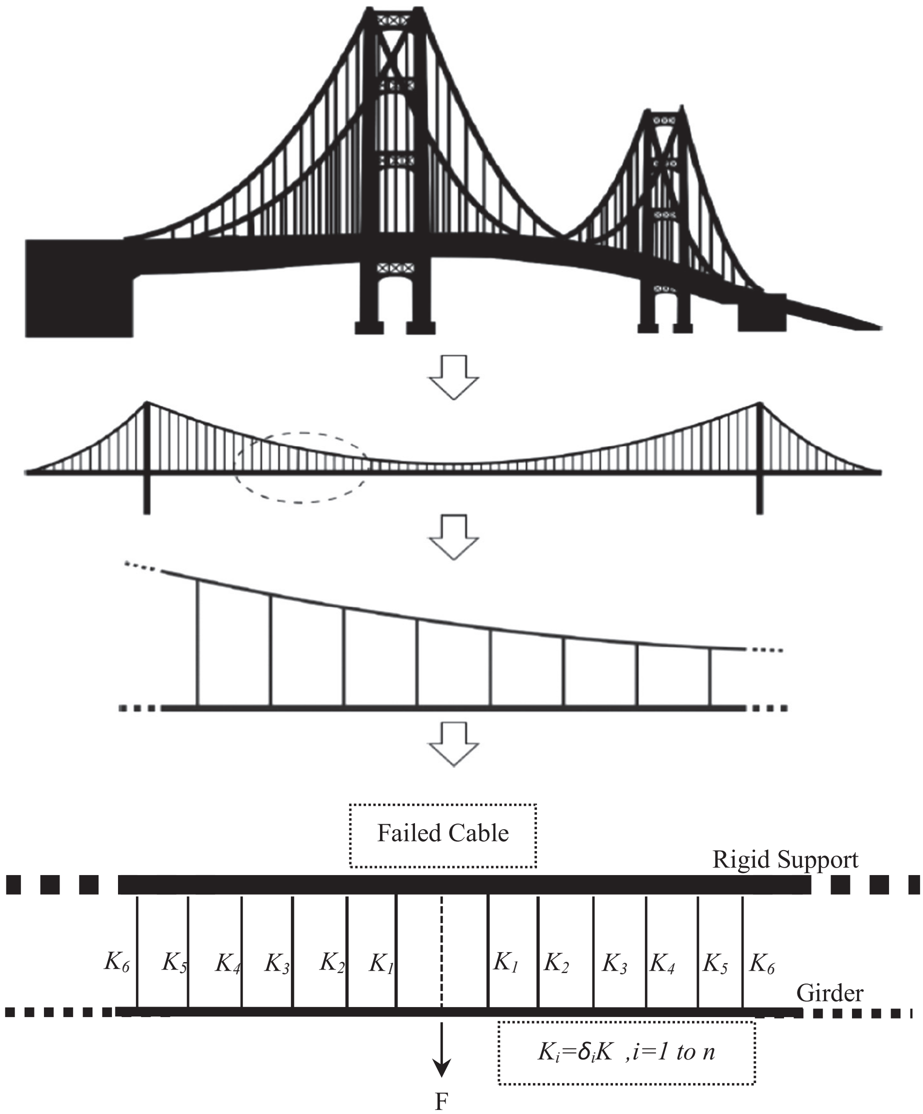

Fig. 1. From bridge to model, based on [18].

determined. The investigation of these two extreme conditions for long-span systems showed relatively similar results regarding the maximum bending moment due to the cable failure. It should be noted that the critical section of the girder is in the center of the system and far from border regions. Therefore, to make the analytical approach easier, a hinge is assumed at the border regions. The main target of this study is to develop an analytical method for the analysis of a long-span cable-supported bridge suffering from a cable failure. For this purpose, a conceptual approach is applied. Hence, some differences between an accurate bridge model and the simplified model used here are unavoidable. For instance, assuming rigid upper cable supports does not exactly correspond to the actual structures. It should be mentioned that in some cases torsion can be neglected. For example, in mono cable plane systems with box girder or systems with two cable planes with edge girders the torsion effect is negligible.

It is assumed that the stiffness of the girder is the same in all cross sections. The axial stiffness of the cables should be determined with consideration of the entire structural system of the actual bridges. The target is to find a general equation for the "maximum bending moment" of the girder due to the cable failure. Therefore, the number of cables is variable.

In the simplified model, the distance between two adjacent cables is L, the axial stiffness of the cable is K_i and the bending stiffness of the girder is $K_b = 12EI/L^3$. The failed cable is in the center and the whole system is symmetrical. The load which was carried by the failed cable is F, and the absorbed load in other cables due to the cable rupture is F_1 to F_n (corresponding to K_1 to K_n). The calculated forces in cables and consequently the bending moment on the girder are increased cable force and increased bending moment due to the cable rupture.

3. Analytical approach for the determination of the "maximum bending moment" of the girder due to the cable loss

The simplified system in Fig. 1 is a symmetrical system and could be solved by the superposition principle and boundary conditions taking into account the symmetry of the system. The elastic behavior of the girder is expressed as follows:

$$M(x) = -EI\frac{d^2v}{dx^2} \tag{1}$$

where EI is the flexural stiffness of the girder, I is moment of inertia of the girder, v is the vertical displacement and x is the distance of the section from the left end of the beam. The bending moment, $M(x)$, is a function of x and could be found as follows:

$$0 \leqslant x \leqslant L \quad M(x) = F_n x \tag{2}$$