Analysis of Cable-Stayed Bridges Subjected to Severe Wind Loading

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Abstract The excitation forces acting on cable-stayed concrete bridges due to winds are highly dependent upon the underlying structural dynamics of the bridge structures. The Sydney Lanier Bridge in Bruckswick, Georgia is used as an example in order to study the dynamic characteristics of a representative cable stayed bridge. This bridge is cable-stayed with a post-tensioned concrete deck, double towers, and double cable planes. The study is conducted by performing a modal analysis using finite elements, and concentrating on the global bridge response and the cable vibrations. An additional benefit of modal analysis is that damage in bridges is generally reflected in the changes of the modes and frequencies of vibration. Therefore these results are potentially useful for wind-induced damage assessment. This study is part of a larger on-going study supported by the Georgia Department of Transportation to examine the hurricane vulnerability of coastal bridges.

1 Introduction

While cable stayed bridges are widespread in Asia and Europe, they are only beginning to be used in North America. So far, they are mostly located on the Mississippi River, Ohio River, and the eastern seaboard. Cable supported precast/post-tensioned concrete bridges are flexible structures which may be easily excited by wind loading. The excitation forces acting on cable supported concrete bridges are aerodynamic by nature; however, they are highly dependent upon the underlying structural dynamics of the bridge structures. For a concrete bridge girder, aero-elastic instability may develop as torsional divergence or flutter. Torsional divergence occurs when the total aero-elastic stiffness becomes negative, and any small perturbation of the concrete girder's at-rest state can become magnified and result in a sudden failure [2-3]. Flutter, on the other hand, is due to negative aerodynamic damping and is characterized by a rapid build-up of intense torsional and vertical vibrations potentially leading to collapse [3].

Cable-stayed bridges are more vulnerable to sizeable environmental forces such as hurricanes and severe wind storms due to their lightness, flexibility, and low damping coefficients and may potentially be unsafe during strong wind events. There are two cable-stayed bridges on the east-coast of Georgia: the Eugene Tallmadge Bridge located in Savannah, GA and the Sidney Lanier Bridge in Brunswick, GA. These two bridges are similar in that each has two main support towers and two planes of tension cables supporting a post-tensioned concrete bridge-deck. Due to the location of theses bridges near the coast, they are subjected to high wind loading and are in the direct path of hurricanes. The need to model and understand their performance during storms and hurricanes is pertinent. These analysis and models will help further wind-resistant design of cable-stayed bridges.

The ultimate goal of this study is to understand and model the behavior of the two cable-stay bridges in Georgia and to propose criteria for the Georgia Department of Transportation to use in closing the bridges to traffic for beyond-design basis wind events. The first stage of this research, which is being presented here, is to develop a model and study dynamic mode shapes of one of the bridges. A model of the Sidney Lanier Bridge has been built in ANSYS and run to determine the bridge's modes of vibration, and a possible way to reduce cable vibrations. Both bridges will eventually be studied.

2 Methodology

The study presented herein is focused on the understanding of the underlying structural dynamics of the cable-stayed bridges (i.e., global response) and cable dynamics. This was accomplished first by a review of published literature on the subject of bridge, wind, and vibration analyses in order to define critical behaviour and second by analytically determining the dynamic characteristics of the bridge and the cable vibrations. The literature review was focused on providing a conceptual understanding of the important dynamic characteristics of bridges relevant to their behaviour during wind events. The analytical study was done using finite element analysis with the intent of computing those dynamic parameters needed to evaluate bridge dynamic behaviour during wind.

3 Review of Bridge Behavior Associated with Wind Loads

Cable-stayed bridges are subjected to the excitation forces of the wind. These aerodynamic forces result in vibrations of the bridge-deck and cables which can cause torsional divergence, flutter, galloping, and ultimately collapse [2 – 6]. Flutter may govern the failure mode as the wind speed increases. In case of the Stonecutters Bridge [4], a vibration problem was caused when the downwind bridge girder was buffeted by vortices shed by the upwind girder. This vibration problem has since been overcome by providing a heavier composite section. Flutter instability can also cause a galloping phenomenon, typically observed with slender bridge deck sections, such as caused the collapse of the Tacoma Narrow Bridge which had been nicknamed "Galloping Gertie" due to the extreme vertical deck movement during a wind storm. Galloping can occur for all speeds above a critical value and thus must be considered for the beyond-design basis wind assessment.

Even for aerodynamically stable girder sections which have been optimized to obtain better flutter performance, another kind of self-excited vibration, vortex-induced vibration, may happen at lower wind speed range [3]. While galloping is not commonly encountered in cable-supported bridges, vortex shedding is a highly anticipated phenomenon. In order to suppress vortex induced oscillations of the bridge girder, a guide vane has sometimes been provided below the deck section [4].

Furthermore, rain-wind induced cable vibrations have been reported for a relatively low wind velocity because a raindrop falling on the windward surface of the cable slides down the surface whereas a drag force of flow acting on the raindrop overcomes the gravity and friction forces for a wind velocity greater than 9 m/s. The vibration appears to cease at a wind velocity of around 14 m/s. [2].

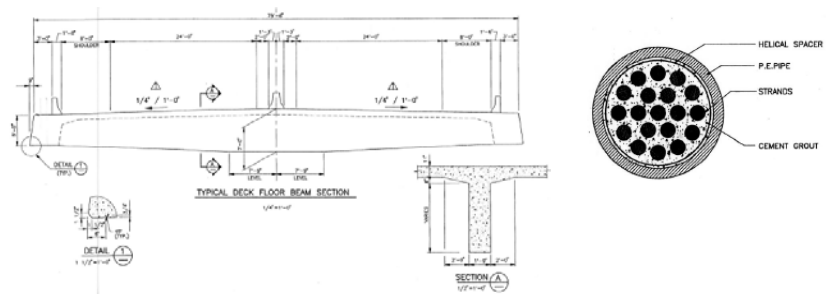
Rain/wind-induced vibrations were first identified by Hikami and Shiraishi on the Meiko-Nishi cable-stayed bridge. Since then, these vibrations have been observed on other cable-stayed bridges, including the Fred Hartman Bridge in Texas, the Sidney Lanier Bridge and the Talmadge Memorial Bridge in Georgia, the Faroe Bridge in Denmark, the Aratsu Bridge in Japan, the Tempohzan Bridge in Japan, the Erasmus Bridge in Holland, and the Nanpu and Yangpu Bridges in China [11]. The mitigation methods that are proven to be effective include dampers, cable crossties, and cable surface modification [11].

4 Analysis of a Cable-Stayed Bridge in Georgia

A modal analysis using a three-dimensional finite element model was performed using "ANSYS". The analysis determined the natural frequencies and displacement mode shapes. The bridge represented in Figs. 1 and 2 is composed of a central span of 381 m and a total span length of 760 m. The wide bridge deck is partially supported at the 142 m high towers and suspended by stay cables. These cables arranged in a total of 44 pairs of anchors at the top of each towers.

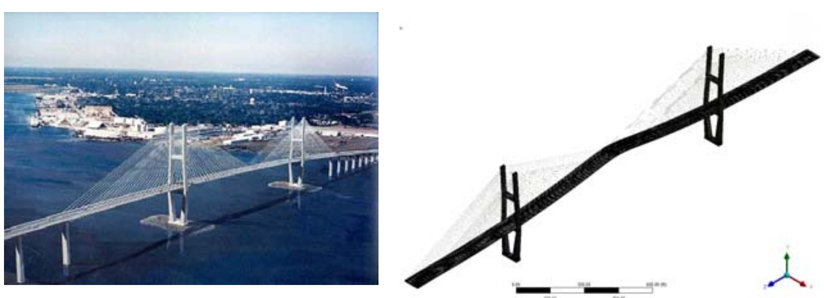
4.1 Finite Element Analysis Model

The modal parameters of a structure are functions of its physical properties such as geometry, mass, stiffness, and damping [5]. Thus, efforts were made to characterize the geometry of the bridge deck, span, and cables with accuracy in a finite element analysis model, based on available structural drawings. Two different element types were considered for the cables: (1) beam elements to study cable vibrations and (2) truss elements to study the global dynamic behaviour of the bridge.



(a) Bridge deck section (b) Cable section

Figure 1 – Typical cross sections of cables and bridge deck [1].



(a) Sydney Lanier Photo [12]. (b) 3D FEA model (ANSYS)

Figure 2 - The Sydney Lanier Bridge in Brunswick, GA and its FEA model.

4.1.1 Bridge Deck and Towers

The typical bridge girder section is presented in Fig. 1(a). The concrete girder and towers were modelled using 8 node solid elements as illustrated in Fig. 2(b). The element size was 1.2 meters resulting in a fine mesh of elements. A 5.5% linear grade profile is assumed from the deck ends to the mid-span, as shown in Fig 2(b) although a crest vertical curve at the mid-span is specified in the bridge drawings.

4.1.2 Cables

Figure 1(a) shows typical cable cross section. The cable composite cross-section was transformed into an equivalent cross-sectional area of steel. The cables consist of nineteen to fifty 15.2 mm diameter seven wire low relaxation strands (grade 270).

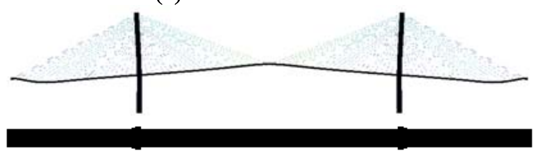
4.1.3. Boundary Conditions

The Sydney Lanier Bridge has pile foundations to support the two towers. The analytical models for pile foundations that have been developed range from very complex finite element models which explicitly treat the soil-structure interaction to a more simplified approach which treat the effects of the soil with a set of simple rotational and translational springs [7]. The soil-structural interaction method can be computationally expensive and thus not appropriate for the large number of wind analysis of cable-stayed bridges required by this study. A simpler approach, which allows for more economical analysis time while considering the effects of the surrounding soil [6], has been used to include the effects of the soil with simple rotational and translational springs. Although these spring constants are of our future interest, the rotational stiffness at the top of each pile is not considered in this study as it is reported to have a negligible contribution [6]. Thus, it is assumed that pile foundations are constructed in competent soil due to the inherent reserve capacity of the foundation. Competent soil limits the lateral translation and rotation of the pile group, resulting in low moment and shear demands in the piles [6]. Therefore, a fixed pile cap base is assumed to develop no moment at the tower support.

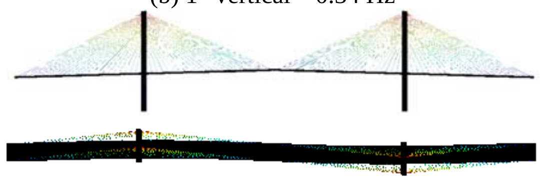
4.2 Analysis Results

The following two sections present the results from the modal analysis. The global and local structural responses from the mode shapes are illustrated in sections 4.2.1 and 4.2.2, respectively.

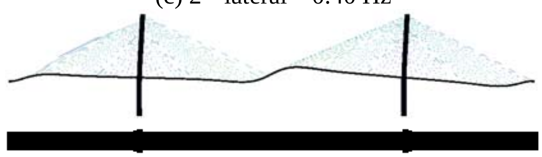
(a) 1st lateral – 0.31 Hz



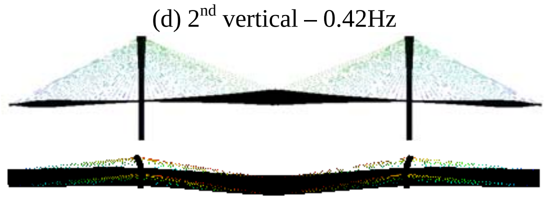
(b) 1st vertical – 0.34 Hz



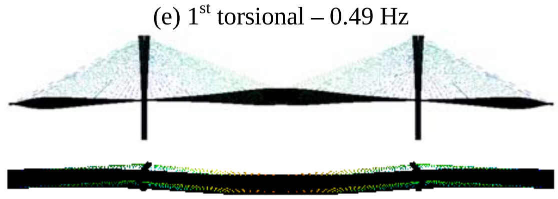
(c) 2nd lateral – 0.40 Hz



(d) 2nd vertical – 0.42Hz



(e) 1st torsional – 0.49 Hz



(f) 2nd torsional – 0.56 Hz

Figure 3 – Displacement mode shapes associated with global dynamic behaviour, modelling cables as truss elements: (a) 1st mode at 0.31 Hz; (b) 2nd mode at 0.34 Hz; (c) 3rd mode at 0.40 Hz; (d) 4th mode at 0.42Hz; (e) 5th mode at 0.49 Hz; (f) 6th mode at 0.56 Hz.

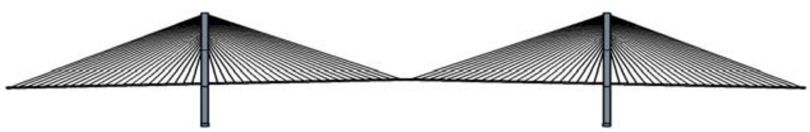
4.2.1 Global Bridge Behaviour

In this bridge eigenvalue analysis, global bridge response is studied by reviewing the displacement mode shapes and frequencies. Cable vibration modes are suppressed by modelling cables using truss elements in order to save computational effort. The analytical results for the global bridge behaviour are presented in Figure 3. The first 6 modes of vibration are presented, two lateral, two vertical, and two torsional.

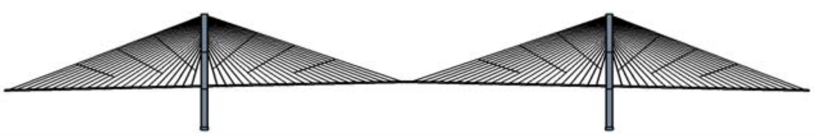
The frequencies range over these six modes varies only by a factor of two with the first two modes having nearly identical frequencies. This suggests that the response of the bridge due to wind will involve significant contribution from multiple modes.

4.2.2 Cable Dynamics

The cable movement and its eigen-frequencies may be predicted by simple taut string theory [2]. Based on the longest cable length of 204m, the frequency is estimated to be 0.15 Hz. The ANSYS analysis yields the frequency of approximately 0.18 Hz. This modal frequency of the Sydney Lanier Bridge is determined by idealizing stay cables as beam elements.



(a) Baseline model with no restrainer ties



(b) Tie configuration selected for the study - Yobuko [2]

(c) Enlarged FEA model to show the tie configuration.

Figure 4 – Cross-tie configurations considered in this study.

4.2.3 Effect of a Cross-tie Configuration on Cable Vibrations

The cable-tie configurations play a significant role in cable dynamics when no dampers are installed on the cables. Due to the low frequency of the cables (around 0.18 Hz from the analysis), rain-wind induced vibrations have been considered. Cross-tie restrainers to the stay cables provide additional mass and damping to individual stay cables and change the natural dynamics of the cables by restraining motion of some points and forcing the cables to respond in shorter and higher-frequency (>2Hz) segments [2]. Figure 4 presents the cross-tie scheme considered in this study that has been successfully employed in other bridges, either temporarily before providing damping devices or as a permanent retrofit solution to mitigate cable vibrations in conjunction with dampers.

In order to determine the global and local structural responses from the mode shapes, the differences of the mode shapes with respect to the baseline model are shown in Figure 4. The Yobuko tie configuration shown in Figures 4(b) and 4(c) leads to an increase of the cable frequencies from 0.18 Hz to approximately 2.0 Hz.

5 Discussions and Future Work

The following limitations are noted, especially regarding the original design drawings of the cable-stayed bridge and FEA model. First, because of several constraints, the authors were unable to fully evaluate as-built conditions such as the overlay thickness and the extent of pre-tensioning in the cables. Thus, the displacement mode shapes were obtained only through the finite element analysis presented herein. Furthermore, because the results in this study are preliminary in nature, statements regarding the dynamic responses are limited only to the studied parameters, super-imposed dead load, and geometry. If the dampers were to be installed on the bridge, the cable dynamics and its impact on the global response would need to be investigated.

One of the main conclusions of the study is that the cable restrainer configuration to alter cable vibrations may become significant when dampers are not provided in its initial construction, but if the bridge is instrumented with a relatively large number of dampers, the tie-configurations proposed in the literature might be complementary. Although boundary constraints have been suggested in this paper, a more thorough parametric study including varying bridge geometries and materials may be necessary to fully understand and quantify the cable vibrations. This constitutes an analytical investigation of various dampers in the finite element analysis model. The finite element analysis model will be refined over a 1-year period to more accurately reflect the mass, stiffness, and damping.

6 Conclusions

The Sydney Lanier Bridge was analyzed for dynamic behavior. The six lowest vibration mode shapes were identified as being symmetric lateral, vertical, and torsional modes. The natural frequencies associated with these modes vary by less than a factor of two. Modal analysis is well suited to understand the global dynamic response of both the bridge and cable vibrations as well as the benefits of methods for mitigating cable vibrations.

The results of the 3D model indicate that analytical evaluation of cable natural frequency is comparable to the simple truss/beam model based evaluation in available literature. The results of the 3D model indicate that the lowest frequency for cable vibration increases by a factor of around 10 when cable restrainers are implemented according to the pattern chosen.

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