EFFECT OF PYLON SHAPE ON ANALYSIS OF CABLE-STAYED BRIDGES

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ABSTRACT:
The paper presents finite element approach for the geometric nonlinear aerostatic analysis of self anchored cable-stayed bridges with different pylon configurations along with vehicular interaction. The example bridges are supported by three different pylon configurations such as H shape, A shape and Inverted Y shape. For above mentioned bridges, linear and non-linear analysis was carried out for a wind speed of 55 m/s for both self anchored and partially earth anchored (bi-stayed) bridges. The results showed that these factors have significant influence on the aerostatic behavior and should be considered in the aerostatic analysis of long span cable stayed bridges. Analysis results will be useful for the designers to consider the shape of pylon at the initial stage of design. Again the results indicate the significant influence of pylon shapes on aerostatic behavior of such long span bridges

KEYWORDS: Finite Element, Aerostatic Analysis, Cable-stayed Bridge, Non-linear Analysis, Bi-stayed Bridge.

INTRODUCTION
Man’s achievements in structural Engineering are most evident in the World’s largest bridge spans. Today the suspension bridge reaches a free span of almost 2000m (Akashi-Kaikyo Bridge, Japan) while its cable-stayed counterpart can cross almost 1000m (Tatara Bridge, Japan, Normandie Bridge, France). Cable-Stayed bridges, in particular, have become increasingly popular in the past decade in United States, Japan and Europe as well as one third-world counties. There is still place for innovation in Cable-Stayed Bridge techniques and the increase in their span length occurring during this last decade of the twentieth century is remarkable. The Cable-Stayed Bridge seems to be a developing bridge type at the moment. The German engineer F. Dischinger (1949) rediscovered the stayed bridge, while designing a suspension bridge across the Elbe River near Hamburg in 1938. He recognized that the inclined cables of the early cable-stayed bridges were never subject to any initial tension, thus cables started to perform properly only after considerable deformations of the whole structure. In earlier development, this behavior led to the misconception that this type of bridge was unacceptably flexible and consequently unsafe. During World War II, approximately 15,000 bridges were destroyed in Germany. The demand to rebuild these bridges was urgent. The requirements of efficient use of materials and speedy construction made cable-stayed bridges the most economical design for the replacements.

The first modern cable-stayed bridge, the Strömsund Bridge designed by F. Dischinger, was completed in Sweden in 1955. The design and construction of this bridge represent the beginning of a new era of modern cable-stayed bridges. The rapid growths of modern cable-stayed bridges throughout the world afterwards is due to the many advances in bridge engineering leading towards better understanding of the behavior and performance and recognizing the advantages of this type of bridges in terms of economy, ease of fabrication and construction, aesthetics and the different possibilities in structural
arrangements, etc... Nowadays, the cable-stayed bridge has been recognized as a very efficient and competitive design for bridges of span ranging from 200m to 800m. For a span length between 200m and 400m, the reinforced concrete girder design for the longitudinal bridge member is generally considered more economical. For a span length between 400m to 600m, the composite deck cross-section can be considered, whereas for 600m to 800m, the steel box girder or composite deck design is preferable.

**Finite Element Modeling**

With modern commercial finite element programs it is possible to accurately predict both static and dynamic structural behavior of cable-stayed bridges. The discretized finite element model provides a convenient and reliable idealization of structure. Thanks to rapid computer developments and the wealth analysis studies on nonlinear problems available. Finite deformation theory with a discrete finite elements model is one of the most powerful tools used in the analysis and design on cable-stayed bridges. Application of the finite deformation theory can include the effect of all nonlinear cable-stayed bridge sources such as cable sags, large deflections and axial force and bending moment interactions. Another advantage of the finite element method lies in the capability of in-depth dynamic analysis. In the analysis and design of cable-stayed bridges, the dead load often contributes most of bridge load. In the finite element analysis, the dead load influence is included through static analysis under dead loads before the live load or dynamic analysis is carried out. The objective of the static analysis process is to achieve the deformed equilibrium configuration of the bridge due to dead loads where the structural members are “pre-stressed”. The initial tension in the cables due to the dead load is determined by on-site testing. In addition, the geometric nonlinear effect has been studied by including the stress stiffening and large deflection.

**Study Undertaken**

To better understand the effect of pylon system with conventional system, parametric studies are carried out. The varying parameters for the parametric study are listed as below. A long span bridge of total span of 1200m was considered to study the effect of various pylon shapes. Three cases of typical pylon arrangement in modern cable-stayed bridge are considered i.e. the H shape, A shape and inverted Y shape. In each case, linear and nonlinear analysis was carried out for a wind speed of 55m/s. The deck cross section of Normandie cable-stayed bridge has considered. The displacement aerostatic load, for all model, has been calculated by taking drag coefficient $C_D = 1.20$, coefficient of lift $C_L=0.38$ [6]. The cross sectional property of deck is listed in Table 1. Hollow rectangular sections of steel are utilized for modeling the pylons. The c/s properties are as given in Table 2. The results of parametric study are listed in Fig. 1 to 10.

<table>
<thead>
<tr>
<th>Geometrical Properties of Pylon</th>
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<tbody>
<tr>
<td>Bridge Span in m.</td>
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<tr>
<td>c/s Area (m²)</td>
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<tr>
<td>Torsional Constant</td>
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<tr>
<td>Moment of Inertia (m⁴)</td>
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<tr>
<td>Section Modulus (m³)</td>
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<tr>
<td>Plastic Modulus (m³)</td>
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<td>Radius of Gyration (m)</td>
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<tr>
<th>Girder Properties</th>
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<tr>
<td>c/s Area (mm²)</td>
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<tr>
<td>Torsional Constant</td>
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<tr>
<td>Moment of Inertia(2-2) (mm⁴)</td>
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<tr>
<td>Moment of Inertia(3-3) (mm⁴)</td>
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<td>Section Modulus(2-2) (mm³)</td>
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<td>Radius of Gyration(2-2) (m)</td>
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Fig 1: Nonlinearity Effect on Cable Force

Fig 2: Nonlinearity Effect on Deck Force

Fig 3: Nonlinearity Effect on Deck Mt.

Fig 4: Nonlinearity Effect on Pylon Force

Fig 5: Nonlinearity Effect on Pylon Mt.

Fig 6: Nonlinearity Effect on Pylon Disp.
Nonlinear Analysis

This type of nonlinearity is based on the deformations of an elastic body and is possible in many instances. Problems involving deformation that are large are called geometrically nonlinear. In dealing with the nonlinear behaviour of the deformable bodies, such as beams, plates and shells, the relationship between the extensional strains and the shear strains on one hand, the displacement components on the other hand are taken to be nonlinear, resulting in nonlinear strain-displacement relations. As a direct consequence of this nonlinearity, the governing differential equations will turn out to be nonlinear. This is true in spite of the fact that the relationship between curvatures and displacement components can be assumed to be linear. Normally, an iterative procedure [4] is required to solve the nonlinear equilibrium problem. In this paper, the Newton-Raphson method is employed with the linear solution as a first approximation. For successive iterations, the actual strain and stress are determined by taking into account both the linear and appropriate nonlinear contributions of the previous approximation. The tangent stiffness matrix and the external and internal force vectors are formed by using the usual assembly procedure for the current structural configuration. The improved trial solution for the \((i+1)^{th}\) iteration is obtained as
\[ q_{i+1} = q_i + \Delta q_{i+1} \quad \ldots(1) \]

where the incremental displacement vector, \( \Delta q_{i+1} \), is computed from equation:

\[ B = B_0 + B_L(q) \quad \ldots(2) \]

The \( B_0 \) is the usual linear S-D relationship matrix. Conversely, \( B_L \) is the large displacement S-D relationship matrix, and is here a linear function of the displacement vector \( q \).

**Aerostatic Analysis**

Bridges are frequently built on exposed sites and are subject to severe wind conditions. Aerostatic loads on bridge superstructures depend on the type of bridge, such as slab-stringer, truss, arch, cable-stayed, or suspension. Other parameters that affect aerostatic loads on bridge superstructures are the wind velocity, angle of attack, the size and shape of the bridge, the terrain, and the gust characteristics. General discussions on aerostatic loads and their effects on structures have been presented by several researchers. Aerostatic loads form a major component of lateral loads that act on all structure. In general, they are a component of the so-called environmental loads to which all structures are subjected. Because of the increasing span, cable-stayed bridges have become increasingly sensitive to aerostatic instability. On the other hand, experimental observations suggest that the aerostatic instability of long-span bridges (suspension bridges and cable-stayed bridges) can occur under the action of static aerostatic loads. Therefore, the aerostatic stability analysis of long span cable-stayed bridges under the displacement-dependent aerostatic loads is of considerable importance.

**Aerostatic Load**

Under the wind effect, the bridge is subjected to, and acts to resist Drag Force, Lift Force and Pitching Moment.

Consider a section of bridge deck in a smooth flow, as shown in Fig. 6. Assuming that under the effect of the mean wind velocity \( V \) with the angle of incidence \( \alpha_0 \), the torsional displacement of deck is \( \theta \). Then the effective wind angle of attack is \( \alpha = \alpha_0 + \theta \). The components of wind forces per unit span acting on the deformed deck can be written in wind axes as

- **Drag Force**
  \[ F_Y(\alpha) = \frac{1}{2} \rho V_Z^2 C_Y(\alpha) D \quad \ldots(3) \]

- **Lift Force**
  \[ F_Z(\alpha) = \frac{1}{2} \rho V_Z^2 C_Z(\alpha) B \quad \ldots(4) \]

- **Pitching Moment**
  \[ M(\alpha) = \frac{1}{2} \rho V_Z^2 C_M(\alpha) B^2 \quad \ldots(5) \]

Where, \( C_Y(\alpha) \), \( C_Z(\alpha) \), \( C_M(\alpha) \) is the coefficients of drag force, lift force, and pitch moment respectively in local bridge axes, \( B \) is the deck width; \( D \) is the vertical projected area, \( \rho \) is the air density and \( V_Z \) is the design wind speed. The above mentioned wind forces are the function of the torsional displacement of structure. They vary as the girder displaces. Therefore, the three components of aerostatic loads are displacement dependent.

**Live Load on Cable-Stayed Bridges**

During the parametric study, moving loads on cable-stayed bridges are taken as per IRC-6:2000 guidelines. This code defines the type of vehicle and number of vehicle for bridges. If span increases than number of vehicles will increases.

As per IRC-6:2000, class 70R, which is hypothetical vehicle, and class A vehicles are considered. The deck is divided in four lanes according to IRC-6:2000 norms.

**CONCLUSIONS**

The finite element method is discussed to find out the efficient pylon configuration from comparison of analysis results during aerostatic analysis of cable stayed bridges. The following conclusions are deduced from the parametric results:

- Nonlinearity effect is predominant in long span bridges and gives approximately 20 to 60% higher results than linear analysis.
• In case of bi-stayed bridges also, the nonlinearity effect is considerably high.
• Apart from few results, for all types of bridges, the load combination DL+MFOUR+WP is found the governing load combination.
• The displacement due to wind is reduced by approximately 20 to 35% when live load due to IRC loads is considered.
• The concept of anchoring of top cable to the earth proved effective in reducing the forces in cables.
• The results show that both in self-anchored and bi-stayed bridges, the forces and moments are found minimum in A frame pylon configuration due to tripod action.
• The costs of cables are very high in case of cable-stayed bridges which can be reduced using the concept of partially earth anchored bridges.

REFERENCES