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|  | **10th International Conference on Short and Medium Span Bridges**  **Quebec City, Quebec, Canada,**  **July 31 – August 3, 2018** |  |

**DESKTOP WIND LOAD AND VIBRATION ASSESSMENTS OF PEDESTRIAN BRIDGES**

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**Abstract:** Light-weight and slender pedestrian bridges, such as long-span girder, cable-stayed or suspension bridges, are vulnerable to wind loads and wind-induced vibrations and have required wind tunnel testing even in the preliminary design stage. Wind tunnel tests are often time consuming and expensive. This paper introduces desktop assessment methods for the wind loading and wind-induced vibration of bridges including buffeting, vortex-shedding, flutter and wind-induced cable vibrations based on international guidelines and theory. As examples, several cable-stayed bridges are analyzed. This paper will provide useful tools for wind-induced vibration assessment, as well as provide a comprehensive understanding on theoretical backgrounds of wind-induced vibrations, which are an important part of pedestrian bridge design. This paper will also help bridge engineers to find economical solutions without conducting wind tunnel testing, especially in preliminary design stages.

1. **INTRODUCTION**

Pedestrian bridges have become lighter and more slender for their aesthetics and financial viability, made possible through the development of high-performance construction materials. These bridges include not only long-span steel or concrete girder bridges but also cable-supported bridges, such as cable-stayed or suspension bridges. Due to the complexity of potential wind-induced issues, wind assessment of bridges such as these is normally performed based on wind tunnel tests, even in the preliminary design stage. The above wind tunnel test-based assessments are usually time consuming and expensive. Due to decades-long research on wind-induced dynamic response of slender line-like structures such as bridges (Jeong et al. 2005; King et al. 2006; Jeong and King 2006, 2008; Jeong and Ferraro 2018), many vibration sources and corresponding responses have been identified, and their theoretical solutions became available with a degree of accuracy required at preliminary design stage.

This paper introduces desktop assessment methods and formula for the wind-induced vibration of bridges including i) buffeting, ii) vortex-shedding, iii) flutter, iv) wind-induced cable vibrations, such as rain-induced vibration, dry galloping, and parametric resonance, as well as v) design of cable dampers based on international guidelines and theory. Several examples of cable-stayed bridges are analyzed in this paper. Wind loads and aerodynamic stability are assessed based on the proposed desktop approaches, using either empirical formula or a simple one or two-degree numerical model. From one section to another, the same variables are sometimes defined differently to match the definitions of specific codes for the convenience of the readers.

1. **WIND BUFFETING**

Natural wind flow over earth surfaces with different exposures, such as water, open, suburban or urban exposures, develops a turbulent boundary layer wind. Wind shedding off upstream structures also creates turbulence, all of which generates wind buffeting forces on structures. The lighter and more slender the structure is, the larger the influence of wind-buffeting.

Wind buffeting force consists of along-wind (drag), across-wind (lift) and torsion (pitching) on line-like structural members such as the bridge deck, piers or pylons. Among the wind loading components, since lift and torsional components are largely affected by wind-structure interactions in terms of quasi-steady or transient aerodynamic damping (Jeong and King 2008), aerodynamic damping effects should be considered in terms of flutter derivatives. Generalized wind buffeting analysis considering all three directional components as well as aerodynamic damping effects can give accurate results (Jeong et al. 2005; King et al. 2006; Jeong and King 2006, 2008) when flutter derivatives of the section are available. However, the analysis requires flutter derivatives measured from wind tunnel testing, and the analysis itself is relatively complicated multi-mode coupled analysis. Vertical and torsional motions are related to stability issues such as flutter or vortex-shedding, which will be addressed later in this report, whereas drag-directional buffeting force has a larger contribution to overall wind loading, especially on the bridge girder.

In this section, a method will be introduced to calculate along-wind directional wind buffeting force and dynamic response based on the ASCE 7 gust factor approach that is simple and familiar to structural engineers.

* 1. **Drag-directional Buffeting Force on Structures**

Based on gust factor approach (Davenport 1964; Solari 1993a, 1993b), a peak value can be calculated by using the gust factor () as follows:

[1]

where = average value. A peak wind load in terms of design wind pressure () can similarly be expressed as follows by using a gust factor () for a flexible structure:

[2]

where = velocity pressure evaluated at the girder height (ASCE 7-10 Eq. 27.3-1); = drag coefficient of the bridge girder. The gust factor () can be calculated as Eq. 26.9-10 in ASCE 7-10 with some appropriate selections of parameters including, length (, width *L*, height and, height about local surface for horizontally aligned structures such as a bridge deck. The dynamic along-wind load and response calculations are explained in detail for a bridge girder as shown in the example below.

* 1. **Example – A Generic Bridge Girder**

In this example, wind buffeting drag loads on a generic bridge with the following properties are calculated in terms of equivalent pressure. Bridge length, = 100 m; bridge cross-section height (size), = 3 m; cross section width, = 15 m; elevation to the girder, = 30 m; natural vibration frequency in horizontal direction, =0.2 Hz; structural damping ratio, = 0.01; wind exposure = C (open exposure according to ASCE 7-10) and surface roughness length, =0.03 m; 3-second gust wind speed at 10 m height in open exposure, = 40.23 m/s (= 90 mph); drag force coefficient, =1.2. Table 1 below summarizes the intermediate variables defined in Sections 26.9.5 and 27.3.2 of ASCE 7-10 as well as wind load in terms of equivalent pressure, on the bridge girder defined in Equation [2] above.

Table 1: Code-Based Along-Wind Load Assessment

|  |  |  |  |
| --- | --- | --- | --- |
| Parameter | ASCE 7-10 | Parameter | ASCE 7-10 |
|  | 274.32 m |  | 0.28 |
|  | 0.105 |  | 0.94 |
|  | 0.154 |  | 0.46 |
|  | 0.65 |  | 0.84 |
|  | 0.2 |  | 1.52 |
|  | 152.4 m |  | 3.4 |
|  | 0.2 |  | 3.787 |
|  | 4.57 m |  | 1.33 |
|  | 30 m |  | 1.26 |
|  | 31.0 m/s |  | 1.0 |
|  | 0.167 |  | 0.85 |
|  | 190 m |  | 1062 Pa |
|  | 1.23 | \* | 1697 Pa |
|  | 0.12 |  |  |

\* Equivalent static along-wind directional wind load (pressure) on bridge deck.

Mean and dynamic wind-induced motion of the bridge girder can also be calculated based on structural statics and dynamics (Solari 1993b). The mean deflection, , dynamic deflection, , peak acceleration, at the mid-span of the girder can be calculated as follows:

[3]

[4]

[5]

[6]

[7]

where = density of air 1.226 kg/m3; for horizontal structures like a bridge deck (Solari 1993b); (Solari 1993b); = mass per unit length of the deck; and = standard deviation of displacement and acceleration of the bridge deck respectively. When = 20,000 kg/m and = kg, mean and peak displacement and acceleration can be calculated as follows by using above equations: = 0.0725 m; = 0.205 m; =0.039 m; = 0.0616 m/s2; =0.233 m/s2.

1. **FLUTTER INSTABILITY**

Since the devastating failure of the first Tacoma Narrow Bridge in 1940, bridge engineers have been aware of the aeroelastic instability that can occur for flexible structures with a bluff-shaped cross-section, such as a long span bridge deck under strong or even moderate winds. Flutter instabilities usually occur as vertical-torsional coupled flutter between the lowest vertical and torsional modes. A simple formula that has been known for coupled-flutter estimation proposed by Selberg (1961) is presented as follows:

[8]

Here, note that corresponds to the width of the cross-section, which is defined differently from the previous section; and = mass and mass moment inertia per unit length respectively; and represent vertical and torsional frequencies respectively.

More sophisticated flutter instability analysis can be performed by 2-DOF coupled flutter analysis. The main source of the instability has been identified as aerodynamic damping and stiffness, which can be represented by flutter derivatives. Flutter derivatives, measured from wind tunnel testing, depend solely on cross-sectional geometry of the girder. This means that a catalogue of flutter derivatives of measured cross-sections can be used for a new bridge with a similar cross-section. Based on the authors experience, even theoretically well-defined flutter derivatives such as those of flat-plate can also be used for bridge sections with reasonable accuracy.

In terms of modal participation, flutter involves multiple vertical and torsional modes; however, the flutter instability occurs for a couple of vertical and torsional modes, such as between first symmetrical vertical and first symmetrical torsional modes, or between first anti-symmetrical vertical and first anti-symmetrical torsional modes, etc. Therefore, multiple chances of flutter instability of a bridge can be assessed by calculation of multiple vertical-torsional combinations separately. This means that instead of multi-mode coupled flutter analysis, 2-DOF coupled flutter analysis is sufficient to identify the flutter instability of a bridge deck, especially for medium span size bridges such as pedestrian bridges. The next section will describe the 2-DOF flutter analysis.

* 1. **2-DOF Coupled Flutter Analysis**

When wind blows perpendicular to the bridge span, 2-DOF coupled flutter between the vertical and torsional modes can be expressed as follows (Simiu and Scanlan 1996):

[9a]

[9b]

where and = damping coefficients in vertical and torsional directions respectively; ; ; , , , , , , and are flutter derivatives of the bridge section. Flutter instability will occur when the damping of the coupled system represented as [9a] and [9b] becomes negative for a given wind speed of . The absolute value of the valid two eigen values of the matrix [10] are () and () and the two real parts of the eigen values divided by () and () respectively, represent opposite sign of the total system damping.

[10]

* 1. **Example – 2D Coupled Flutter Analysis of a Cable-Stayed Bridge**

A 2D-coupled flutter analysis is performed using the matrix [10] above for wind speed, varying 5 m/s to 80 m/s. The bridge dimension and properties follow: span length, = 1,000 m; mass per unit length, =33,300 kg/m; mass moment of inertia of the cross section, =9,440,000 kg-m2/m; bridge deck width, = 53 m; frequencies of the first symmetric vertical and the first symmetric torsional modes = 0.1 Hz and 0.2 Hz, respectively; damping ratios of , = 0.0061 for both modes. The dimensions and mass properties of the example are the same as those of Stonecutters Bridge in Hong Kong, however, the frequencies are adjusted (reduced from the original ones) to demonstrate the flutter instability. In the analysis, flutter derivatives for flat plate which is used in the example are shown in Figure 1. Figure 2 illustrates the total system frequencies and system damping for a range of wind speeds calculated from eigen value analysis of the matrix [10] above. As shown in the figures, due to the contribution of aerodynamic stiffness and aerodynamic damping, the total system frequency and damping increases as wind speed increases. The system damping becomes negative for wind speeds above 61 m/s, which corresponds to flutter wind speed.



Figure 1: Flutter Derivative of Flat Plate



Figure 2: Total System Frequencies and Damping of the Bridge for Various Wind Speeds

1. **VORTEX SHEDDING**

Slender cylindrical structures such as bridge girders, piers, pylons and arches with non-streamlined cross sections are easily susceptible to wind-induced vibration due to the resonance between the structural frequency and periodic vortices shed off of the structure. Vortex shedding occurs in the across-wind direction even at moderate wind speeds. This is very common to wind-induced vibration and an important consideration for the wind design of slender structures such as pedestrian bridges.

Accurate vortex-shedding period and amplitude can be expressed as Strouhal Number () and dynamic lift coefficient (), respectively, which are available in literature for select generic cross-sections and bridge sections (Eurocode 2004; Simiu and Scanlan 1996). However, due to their unique cross sections, values for many bridge girders are not commonly available as for those of arches or pylons. Nonetheless, once the information is available, vortex-induced vibration can reasonably be estimated based on Eurocode 1 (2004) based on our experience on many slender structures.

* 1. **Example - Vortex-induced Vibration of a Pedestrian Bridge Girder and an Arch**

In this section, vortex-induced vibration amplitudes of a rectangular-shaped bridge girder and an circular-shaped arch of proposed arch-supported pedestrian bridge as part of Knight Campus development at University of Oregon in Eugene, are calculated based on Eurocode 1. The structural properties and geometric information of the bridge girder follows: bridge girder length, = 57 m; cross section height, = 4.27 m; cross section width, = 4.27 m; mass per unit length, = 4,587 kg/m; the first vertical natural frequency, = 1.383 Hz; damping ratio = 0.005; Strouhal Number, = 0.12; 50-year return period mean wind speed at girder height, =32 m/s. The structural properties and geometric information of the arch follows: arch length, = 57 m; girder cross section height, = 0.5588 m; girder cross section width, = 0.5588 m; mass per unit length, = 599 kg/m; the first vertical natural frequency, = 0.854 Hz; damping ratio = 0.005; Strouhal Number, = 0.18; 50-year return period mean wind speed at girder height, =34 m/s. For the given properties, maximum displacement, , m, of the bridge girder and the arch due to the vortex shedding can be calculated as Table 2.

Table 2: Code-Based Vortex-induced Vibration Assessment (Eurocode 1)

|  |  |  |  |
| --- | --- | --- | --- |
| Parameter | Bridge Girder | Parameter | Arch |
|  | 12.66 |  | 15.35 |
|  | 0.1 |  | 0.1 |
|  | 49.2 m/s |  | 2.7 m/s |
|  | N.A. |  | 9.8 × 104 |
|  | 1.1 |  | 0.7 |
|  | 0.0 |  | 0.7 |
|  | 13.4 |  | 102 |
|  | 12.0 (assumed) |  | 12.0 (assumed) |
|  | 0.6 |  | 0.172 (1st iteration) |
|  | 0.0 |  | 3.85 × 10-3 (1st iteration) |
|  |  |  | 6.0 (2nd iteration) |
|  |  |  | 0.0785 (2nd iteration) |
|  |  |  | 1.76 × 10-3 (2nd iteration) |

\* Variables follows the definitions in Eurocode 1.

In the case of the bridge girder, as shown in Table 2, since the critical vortex shedding wind speed, is greater than the 50-year mean hourly wind speed at girder height (the design wind speed), , vortex-shedding will not occur during the lifetime of the bridge. However, due to the much smaller cross-section, the critical vortex shedding wind speed of the arch is much lower than the design wind speed, which results in frequent vortex-induced vibration of the arch. The number of excitations, for the fatigue design of the arch during the lifetime of 50 years, can be evaluated as follows:

[11]

where = 1.6 × 109s; = 0.3 typically; = 6.4 m/s. Therefore, the number of excitation, = 1.19 ×108. Despite the large number of excitations, vortex shedding effects in this example are ignorable due to the small amplitudes.

1. **CABLE WIND-INDUCED VIBRATIONS**

Due to high slenderness ratio, flexibility, structural nonlinearity, inclination, and sags, cables are vulnerable to wind-induced vibration and interact with winds in a unique way compared with other structures. Problematic wind-induced vibration on cables include: rain-wind vibration, galloping of ice coated cables, dry galloping of inclined cables, wake-buffeting of an array of cables, as well as vortex-shedding and buffeting. Other than wind-induced vibration, sometimes parametric resonance due to the support-excitation from the deck can cause a problem for cables.

* 1. **Rain-Wind Vibration**

Rain-wind vibration (RWV) is the most common cable vibration. In the case of cable-stayed bridges, approximately 95% of problematic vibrations are rain-wind vibrations. RWV occurs on inclined cables such as stay cables. When rain flows down an inclined cable, the bead of water along the cable breaks the axi-symmetry of the cable cross-section, which can lead to galloping instability based on Den Hartog Criteria (Caetano 2007). Analysis models are available in various levels of complexity (Yamaguchi 1990; Geurts 1998, 1999).

Code and industry recommendations to prevent RWV occurrence on cables can be checked relatively simply by Scruton Number, as follows:

[12]

where = cable diameter. Note that the definition of Scruton Number is different by compared to the definition used for vortex-induced vibration described in Section 4.1. PTI (2014) recommendations on RWV of stay cables are to keep the Scruton Number of the cables larger than 10 and 5, respectively, for smooth cables and roughened surfaces such as helical fillet or dimples.

* 1. **Galloping**

Galloping instability even without rain or ice coating on cable surfaces, known as ‘Dry Galloping’, has occurred on notable bridges including the Faro Bridge, Helgeland Bridge and Oresund Bridge (Macdonald and Larose 2008a), among others. Although some researchers (Macdonald 2006; Macdonald and Larose 2008a, 2008b) explain galloping based on quasi-steady theory and full-scale wind tunnel measurement of a stay cable, more research is required to understand the underlying mechanism of the instability.

Current code- and theory-based recommendations on dry galloping can be summarized as follows:

[13] (FHWA 2007)

[14] (Macdonald and Larose 2008a, 2008b)

where equals 35 and 25 respectively for regular cables and cables within a range of spacing from 2 to 6; ; = cable damping; = mass per unit length of the cable; = cable natural frequency; = viscosity of air.

* 1. **Buffeting and Vortex-Induced Vibration of Cables**

Buffeting or vortex-induced vibration are usually not problematic wind loading conditions in cable design. Buffeting wind loading responses of cables can be similarly evaluated as those of girders, presented in Section 2 above. However, due to the large deflections, significantly larger aerodynamic damping creates favourable effects both in along-wind and across-wind directional cable vibrations. The aerodynamic damping, which is also known as quasi-steady damping on a cable, can be evaluated as follows for the circular cross-section such as in cables:

[15] (along-wind direction);

[16] (across-wind direction).

As shown above Equations [15] and [16], in the case of circular cross-sections such as in cables, along-wind directional aerodynamic damping is twice as high as that of the across-wind direction.

Vortex-induced vibration and its wind loading effects can also be evaluated similar to other structures based on Eurocode 1, which is explained in detail in Section 4 above. However, in the case of cables, due to their small cross-section and flexibility, higher modes should also be investigated for vortex-induced vibration. For higher modes, all calculations are the same except for the effective correlation length, . generalized for higher modes, which can be expressed as follows as derived from mathematical calculation by assuming mode shapes to be sinusoidal functions:

[17] .

where = mode number; = cable diameter; = cable length.

* 1. **Cable Dampers**

Dashpot cable dampers are commonly used to mitigate cable vibrations determined to be excessive. Design of the dashpot-type cable dampers are limited by installation height of the supporting point. In this section, Universal Curve-based approaches (Pacheco et al. 1993; Krenk 2000) are introduced for optimal cable damper design.

A dashpot damper installed at location, of a taut cable will provide no support, or rigid support, when the damping coefficient of the damper is zero or infinity, respectively, and will not provide any damping effects in either case. We can almost intuitively understand that optimal damping will lie between the two extremes. Based on research (Pacheco et al. 1993), the optimal damping, of the damper can be calculated as follows for mode :

[18]

where = mass per unit length of the cable; = angular velocity of the cable first mode. Figure 3 illustrates the Universal Curve that can be used for the optimal design of cable dampers, which can be determined by higher damping (higher y-values) over the modes of interest of the cable. Practical examples of cable damper design using the Universal Curve can be found in the literature (Pacheco et al. 1993; Krenk 2000).



Figure 3: Universal Curve for Stay Cables (Pacheco et al., 1993)

1. **SUMMARY AND CONCLUSION**

Desktop wind assessment methods based on theory and existing codes have been introduced for pedestrian bridges as well as for medium and long-span road and rail bridges. These methods provide useful tools for preliminary design of bridges and their components including arches and cables, in some cases eliminating the need for wind tunnel testing.

**References**

ASCE. 2010. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-10.

Caetano, E.S. 2007. Cable Vibrations in Cable-Stayed Bridges, *International Association for Bridge and Structural Engineering (IABSE).*

Davenport, A.G. 1964. Notes on the Distribution of the Largest Value of a Random Function with Application to Gust Loading. *Proc. Institution of Civil Engineering*, London, U.K., 187-196.

Eurocode. 2004. *Eurocode 1 -* *Actions on Structures* – Part 1-4. General Actions – Wind Actions, PrEN 1991-1-4.

FHWA. 2007. *Wind-Induced Vibration of Stay Cables*, U.S. Department of Transportation, FHWA Publication No. FHWA-HRT-05-083.

Guerts, C.P.W., Staalduinen, P.C., Reusink, J. 1998. Numerical Modeling of Rain-Wind-Induced Vibration: Erasmus Bridge, Rotterdam. *Structural Engineering International*, **2**: 129-135.

Guerts, C.P.W., Staalduinen, P.C. 1999. Estimation of the Effects of Rain-Wind Induced Vibration in the Design Stage of Inclined Cables. *Wind Engineering into the 21st Century, 10th ICWE*,A.A. Balkema, Rotterdam, Netherlands.

Jeong, U.Y., King, J.P.C. and Isyumov, N. 2005. A Systematic Finite Element-Based Buffeting Formulation. *Proc. 11th Americas Conference on Wind Engineering*, Baton Rouge, Louisiana.

Jeong, U.Y. and King, J.P.C. 2006. A Numerical Buffeting Analysis of Horizontally Curved Bridges under Three-Dimensional Wind Loading. *The 4th International Symposium on Computational Wind Engineering*, Yokohama, Japan.

Jeong, U.Y. and King, J.P.C. 2008. A Time Domain Wind Buffeting Formulation for Long Slender Structures. *The 4th International Conference on Advances in Wind and Structures*, Jeju, South Korea.

Jeong, U.Y. and Ferraro, V. 2018. Desktop Wind-Induced Vibration Assessment for Pedestrian Bridges. *ASCE / SEI Conference*, Fort Worth, TX.

King, JP.C., Jeong, U.Y. and Kong, L.Z. 2006. Wind Loads on Pedestrian Bridges. *Short and Medium Span Bridge Conference*, Montreal.

Krenk, S. 2000. Vibration of a Taut Cable with an External Damper. *ASME Journal of Applied Mechanics*, **67**: 772-776.

Macdonald, J.H.G. 2006. A Unified Approach to Aerodynamic Damping and Drag/Lift Instabilities and Its Application to Dry Inclined Cable Galloping. *Journal of Fluid and Structures*, **22**: 229-252.

Macdonald, J.H.G. and Larose, G. 2008a. Two-Degree-of-Freedom Inclined Cable Galloping – Part 1: General Formulation and Solution for Perfectly Tuned System. *Journal of Wind Engineering and Industrial Aerodynamics*, **96**: 291-307.

Macdonald, J.H.G. and Larose, G. 2008b. Two-Degree-of-Freedom Inclined Cable Galloping – Part 2: Analysis and Prevention for Arbitrary Frequency Ratio. *Journal of Wind Engineering and Industrial Aerodynamics*, **96**: 308-326.

Pacheco, B.M., Fujino, Y. and Sulekh, A. 1993. Estimation Curve for Modal Damping in Stay Cables with Viscous Damper. *Journal of Structural Engineering*, **119.6**: 1961-1979.

PTI. 2014. *Recommendations for Stay Cable Design, Testing and Installation*, PTI DC45. 1-12.

Simiu, E. and Scanlan, R.H. 1996. *Wind Effects on Structures,* The 3rd Edition, John Wiley & Sons, Inc., New York, NY, USA.

Selberg, A. 1961. *Oscillation and Aerodynamic Stability of Suspension Bridges*, ACTA Polytechnica Scandinavica, Civil Engineering and Construction Series 13.

Solari, G. 1993a. Gust Buffeting. I: Peak Wind Velocity and Equivalent Pressure. *Journal of Structural Engineering*, **119**: 365-382.

Solari, G. 1993b. Gust Buffeting. II: Dynamic Alongwind Response. *Journal of Structural Engineering*, **119**: 383-398.

Yamaguchi, H. 1990. Analytical Study on Growth Mechanism of Rain Vibration of Cables. *Journal of Wind Engineering and Industrial Aerodynamics*, **33**: 73-80.