Wind-Induced Vibration of Stay Cables: Summary of FHWA Study

by

Nicholas P. Jones[1] Thusitha Kumarasena[2], Peter Irwin[3], and Andreas Felber[4]

ABSTRACT

A study sponsored by the US Federal Highway Administration was recently completed in order to develop a set of consistent design guidelines for mitigation of excessive cable vibrations on cable-stayed bridges. In order to accomplish this goal, the Project Team started with a thorough review of existing literature to determine the state of knowledge and identify any gaps to be filled in order to enable the formation of a consistent set of design recommendations. This review indicated that while the rain/wind problem seemed to be known in sufficient detail, galloping of dry inclined cables was a critical wind-induced vibration mechanism in need of further experimental research. A series of wind-tunnel tests was performed to study this mechanism. Analytical and experimental research was performed to study mitigation methods, covering a range of linear and non-linear dampers and cross-ties. The study also included studies on live load induced vibrations and establishing driver/pedestrian comfort criteria.

Based on the above, design guidelines for mitigation of wind-induced vibrations of stay cables were developed. This paper summarizes the important findings from that study. Much of the text that follows is extracted in summary form from the report that was coauthored by all the project investigators listed above.

KEYWORDS: bridge engineering, long-span bridges, stay-cable vibration, wind engineering, wind-rain vibration.

1.0 INTRODUCTION

Cable-stayed bridges have become the structural form of choice for medium to long span bridges over the past several decades. With their increasingly widespread use, some cases of serviceability problems with large amplitude vibrations of stay cables due to environmental conditions have surfaced. A significant correlation had been observed between the occurrence of these large amplitude vibrations and occurrences of rain combined with wind, leading to the adoption of the term "rain/wind vibrations." However, a few instances of large amplitude vibrations have also been reported (in the literature) without rain.

In 1999, the Federal Highway Administration (FHWA) commissioned a study team led by HNTB Corporation and three subconsultants, Johns Hopkins University, Rowan Williams Davies & Irwin, Inc. (RWDI), and Buckland and Taylor, Ltd. to investigate wind-induced vibration of stay cables. The Project Team represented expertise in cable-stayed bridge design, academia, and wind engineering.

By this time, a substantial amount of research on this subject had already been conducted by researchers and cable suppliers in the U.S. and abroad. This work has established water rivulet formation and its interaction with the wind flow as the root cause of the rain/wind vibrations. With this understanding, various surface modifications had been proposed and tested with the aim of disrupting this water rivulet formation. Recently developed mitigation measures such as "double

[1] Professor and Head, Dept. of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana IL 61801, USA (formerly Johns Hopkins University.)
[2] HNTB Corporation, 75 State St., Boston, MA 02109 and 352 Seventh Ave., 6th Floor, New York, NY 10001-5012
[3] Rowan Williams Davies & Irwin, Inc., 650 Woodlawn Road West, Guelph, Ontario N1K 1B8
[4] Buckland and Taylor, Ltd., Suite 101, 788 Harborside Drive, North Vancouver, BC V7P3R7

helix" surface modifications as well as traditional measures such as external dampers and cable cross-ties have been provided to many of the newer bridges. However, the lack of a uniform criteria or a consensus in some of the other key areas such as large amplitude galloping of dry cables has made the practical and consistent application of the known mitigation methods difficult.

The objective of this FHWA sponsored study was to develop a set of uniform design guidelines for vibration mitigation for stay cables on cable-stayed bridges. The project was subdivided into the following distinct tasks:

- Task A: Develop electronic database of reference materials
- Task B: Develop electronic database of inventory of U.S. cable-stayed bridges
- Task C: Analysis, evaluation and testing
- Task D: Assessment of mitigation approaches
- Task E: Identification of research needs
- Task F: Project documentation

The initial phase of the study consisted of a collection of available literature on stay cable vibration. Due to the large volume of existing literature, the information was entered into two electronic databases. These databases were developed to be user friendly, have search capabilities, and also facilitate entering of new information as it becomes available. The databases have been turned over to FHWA for future maintenance. It is expected that these will be deployed over the internet for use by the engineering community.

The Project Team conducted a thorough review of the existing literature to determine the state of knowledge and identify any gaps that must be filled to enable the formation of a consistent set of design recommendations. This review indicated that while the rain/wind problem is known in sufficient detail, galloping of dry inclined cables was the most critical wind-induced vibration mechanism in need of further experimental research. A series of wind-tunnel tests was conducted at the University of Ottawa propulsion wind tunnel to study this mechanism.

The Project Team obtained additional funds from Canada's National Engineering and Sciences Research Council (NSERC) for the testing at the University of Ottawa, effectively doubling the original funding for the wind tunnel testing task. The Project Team also supplemented the study by incorporating work of its key team members on other ongoing, related projects.

Analytical research was also performed covering a wide spectrum of related issues, such as the behavior of linear and nonlinear dampers and cable cross-ties. The research also included brief studies on parametric excitation and establishing driver/pedestrian comfort criteria with respect to stay cable oscillation.

Based on the above investigations, design guidelines for mitigation of wind-induced vibrations of stay cables were developed. It must be noted that this is the first time such design guidelines have been proposed. They are meant to provide a level of satisfactory performance for stay cables with respect to recurring large amplitude stay oscillations due to common causes that have been identified to date, and are not intended to eliminate stay cable oscillations altogether (as this would be impractical).

It is expected that these guidelines can be suitably refined based on future observations on actual performance of stay cables in bridges around the world as well as developments in stay-cable technology. With the widespread recognition of the mitigation of stay cable vibration as an important design issue among long-span bridge designers, all new cable-stayed bridges are more than likely to incorporate some form of mitigation discussed in this document. This would provide ample future opportunity to measure real life performance of bridges against the design guidelines contained herein.

As a precautionary note, the state of the art in mitigation of stay cable vibration is not an exact science. These new guidelines are only intended for use by professionals with experience in cable-stayed bridge design, analysis, and wind engineering, and should only be applied with engineering judgment and due consideration of special conditions surrounding each project.

2.0 COMPILATION OF EXISTING INFORMATION

2.1 Reference materials

An extensive literature survey was initially performed to form a baseline for the current study. An on-line database of references was created so that all members of the Project Team could add or extract information as necessary. The database had 198 references and includes the article titles, authors, reference information, and abstracts when attainable and has built-in search capabilities.

2.2 Inventory of U.S. cable-stayed bridges

An inventory of cable-stayed bridges in the United States was created to organize and share existing records with the entire Project Team. This database includes information on geometry, cable properties, cable anchorages, aerodynamic detailing, site conditions, and observed responses to wind for 26 cable-stayed bridges. The inventory is stored in Microsoft Access (Microsoft Office) database format, which allows for easy data entry and retrieval.

This electronic database of U.S. cable-stayed bridges, along with the reference database, has been turned over to FHWA. They are expected to be launched over the internet for use by the engineering community.

3.0 ANALYSIS, EVALUATION, AND TESTING

3.1 Mechanics of wind-induced vibrations

There are a number of mechanisms that can potentially lead to vibrations of stay cables. Some of these types of excitation are more critical or probable than others but all are listed here for completeness:

- Rain/wind induced vibrations of cables
- Galloping of single cables inclined to the wind
- Aerodynamic excitation of overall bridge modes of vibration involving cable motion
- Vortex excitation of an isolated cable or groups of cables
- Wake galloping of groups of cables
- Galloping of cables with ice accumulations
- Motions due to buffeting by wind turbulence
- Motion due to fluctuating cable tensions

The first three of these will be discussed briefly below, as they formed the focus of much of the investigation.

3.2 Rain/Wind induced vibrations

The combination of rain and moderate wind speeds can cause high amplitude cable vibrations at low frequencies. This phenomenon has been observed on many cable-stayed bridges and has been researched in detail.

Rain/wind induced vibrations were first identified by Hikami and Shiraishi (1987) 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 in Georgia, the Cochrane Bridge in Alabama, 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. These vibrations occurred typically when there was rain and moderate wind speeds (18-34 mph, 8-15 m/s) in the direction angled 20° to 60° to the cable plane with the cable declined in the direction of the wind. The frequencies were low, typically less than 3 Hz. The peak amplitudes were very high, in the range of 10-in to 3-ft (0.25-m to 1.0-m), with violent movements resulting in clashing of adjacent cables observed in several cases.

Wind tunnel tests have shown that rivulets of water running down the upper and lower surfaces of the cable in rainy weather were the essential component of this aeroelastic instability (Hikami & Shiraishi, 1987; Matsumoto et al., 1989). The water rivulets changed the effective shape of the cable, and moved as the cable oscillated causing cyclical changes in the aerodynamic forces which led to the wind feeding energy into oscillations. The wind direction causing the excitation was approximately 45° to the cable plane. The particular range of wind velocities that caused the oscillations appears to be that which maintained the upper rivulet within a critical zone on the upper surface of the cable.

It is noteworthy that some of the rain/wind vibrations that have been observed on cable-stayed bridges have occurred during construction when both the damping and mass of the cable system are likely to have been lower than in the completed state, resulting in a low Scruton number. For the Meiko-Nishi Bridge, the Scruton number was estimated at 1.7. The grouting of the cables adds both mass and damping, and often sleeves of visco-elastic material are added to the cable end regions which further raises the damping. The available circumstantial evidence indicates that the rain/wind type of vibration primarily arises as a result of some cables having exceptionally low damping, down in the $\zeta = 0.001$ range.

Since some bridges have been built without experiencing problems from rain/wind vibration of cables it appears probable that in some cases the level of damping naturally present is sufficient to avoid the problem. The rig test data of Saito et al. (1994), obtained using realistic cable mass and damping values, are useful in helping to define the boundary of instability for rain/wind oscillations. Based on their results it appears that rain wind oscillations can be reduced to a harmless level using the following criteria for the Scruton number (PTI, 2001):

$$\frac{m\zeta}{\rho \mathrm{D}^2} > 10$$

This criterion can be used to specify the amount of damping that must be added to the cable to mitigate rain/wind vibrations.

Since the rain/wind oscillations are due to the formation of rivulets on the cable surface, it is probable that the instability is sensitive to the surface roughness. Several researchers have tried using small protrusions on the cable surface to solve the problem. Flamand (1994) has used helical fillets 1/16 inch (1.5 mm) high on the cables of the Normandie Bridge. This technique has proven successful, with a minimal increase in drag coefficient. This type of cable surface treatment is becoming a popular design feature for new cable-stayed bridges, including the Leonard P. Zakim Bunker Hill Bridge (MA), U.S. Grant Bridge (OH), Greenville Bridge (MS), William Natcher Bridge (KY), Maysville-Aberdeen Bridge (KY), and the Cape Girardeau Bridge (MO).

3.3 Galloping of dry inclined cables

Galloping of single dry, inclined cables is a theoretical possibility. Results from one experimental study (Saito et al., 1994) seem to suggest that this could be a concern for cable-stayed bridges. Theoretical formulations predict that this galloping may occur at high wind speeds with possible large-amplitude vibrations and that many existing cable-stayed bridges are susceptible, but there is no evidence of their occurrence in the field.

Single cables of circular cross-section do not gallop when they are aligned normal to the wind. However, when the wind velocity has a component that is not normal to the cable axis, an instability with the same characteristics as galloping has been observed. For a single inclined cable the wind acts on an elliptical cross-section of cable. An ellipticity of 2.5, corresponding to an angle of inclination of the cable of approximately 25°, can occur in the outer-most cables of long-span bridges. (Ellipticity is defined as the maximum width divided by the minimum width - e.g., a circle has an ellipticity of 1.0.) There is the potential for galloping instability if the level of structural damping in these cables is very low.

Saito et al. (1994) conducted a series of wind tunnel experiments on a section of bridge cable mounted on a spring suspension system. Their data suggest an instability criterion given approximately by the following:

$$(U / f \mathrm{D})_{\mathrm{CRIT}} = 40\sqrt{\mathrm{S_c}}$$

This data was for cases where the angle between the cable axis and wind direction was 30° to 60°. The above criterion is a difficult condition to satisfy, particularly for the longer cables of cable-stayed bridges with a typical diameter of 6 to 8-in (150 to 200-mm). Further experimental research was necessary to confirm the results of Saito et al. (1994) and to extend the range of conditions studied. All of their experiments used low levels of damping, so it was important to investigate whether galloping of an inclined cable is possible at damping ratios of 0.005 and higher.

Based on existing information, it was apparent that galloping of dry inclined cables presented the biggest concern and biggest unknown for wind-induced vibration mitigation. The Project Team therefore focused the wind-tunnel test program on this subject.

The model was developed to be similar to that used in the test carried out by Saito et al. (1994). A 22-ft (6.7-m) long cable consisted of an inner steel pipe covered with a smooth polyethylene tube with an outside diameter of 6.3-in (160 mm). The effective mass per cable length was 40.9 lb/ft (60.8 kg/m). The end supports at the upwind end were maintained out of the wind flow above the wind tunnel, and at least 19.2-ft (5.9-m) of the 22-ft (6.7-m) length of the cable was directly exposed to the wind tunnel flow.

Testing was performed for various levels of structural damping, cable frequency ratios, surface roughness and at various angles of wind flow. The cable model orientation was changed against the mean wind flow direction for several configurations.

Limited-amplitude oscillations were observed under a variety of conditions. The limited-amplitude vibrations occurred within narrow wind speed ranges only, which is characteristic of vortex excitation of the high speed type described by Matsumoto (1998). For the typical cable diameters and wind speeds of concern on cable-stayed bridges the Reynolds number is in the critical range where large changes in the airflow patterns around the cables occur for relatively small changes in Reynolds number. The excitation mechanism is thus likely to be linked with these changes. The maximum amplitude of the response depended on the orientation angle of the cable. For wind blowing along the cable, for cables with a vertical inclination angle, the increase of surface roughness made the unstable range shift to lower wind speeds.

The results of this testing showed a deviation from the criterion described earlier. While significant oscillations of the cable occurred (double amplitudes up to 1D), it is not conclusive that this was dry inclined cable galloping. In fact, as indicated above they had similar characteristics to Matsumoto's (1998) high speed vortex excitation. Divergent oscillations only occurred for one test setup at very low damping, and the vibrations had to be suppressed since the setup only allowed for amplitudes of 1D. Large vibrations were only found at the lowest damping ratios ($\zeta<0.001$). Above a damping ratio of 0.003, no significant vibrations (> 0.4-in ; >10-mm) were observed.

This testing suggests that if even a low amount of structural damping is provided ($\zeta>0.003$), then vortex shedding and inclined cable galloping vibrations are not significant. This damping corresponds to a Scruton number of approximately 3, which is less than the minimum of 10 established for suppression of rain/wind vibrations. Therefore dry cable instability should be suppressed by default if enough damping is provided to mitigate rain/wind vibrations.

3.4 Deck-stay interaction due to wind

Measurements of both deck and stay movements have been taken at the Fred Hartman Bridge that suggests that in some instances the deck is driving the cable to vibrate with large amplitude in its fundamental mode. Vortex-induced vibration of the deck is thought to be the driving mechanism for this motion. Further studies are continuing to identify additional occurrences of this behavior for corroboration, and to better understand the underlying mechanisms their consequences. It is emphasized that this appears to be a relatively uncommon event.

4.0 MITIGATION APPROACHES

Space limitations preclude the full presentation of the various studies of mitigation undertaken. Presented here in summary form are some of the important results for dampers and cross ties.

Additional resources can be found in the references and bibliography.

4.1 Dampers

4.1.1 Linear viscous damper

Free vibrations of a taut cable with an attached linear viscous damper were investigated in detail. An analytical formulation of the complex eigenvalue problem was used to derive an equation for the eigenvalues that is independent of the damper coefficient. This "phase equation" reveals the attainable modal damping ratios ζ_i and corresponding oscillation frequencies for a given damper location ℓ_1/L, affording an improved understanding of the solution characteristics and revealing the important role of damper-induced frequency shifts in characterizing the response of the system.

When the damper-induced frequency shifts are small, an asymptotic approximate solution has been developed, relating the damping ratio ζ_i in each mode to the nondimensional damper coefficient, κ.

For damper locations resulting in small frequency shifts (e.g., dampers near the end of the stay), the expression can be written as:

$$\frac{\zeta_i}{(\ell_1/L)} \cong \frac{\pi^2\kappa}{(\pi^2\kappa)^2+1} \qquad (1)$$

where κ is a nondimensional parameter grouping defined as

$$\kappa \equiv \frac{c}{mL\omega_{o1}} i \frac{\ell_1}{L} = \frac{1}{\pi}\frac{c}{\sqrt{Tm}} i \frac{\ell_1}{L} \qquad (2)$$

Equation 1 gives the damping attainable in a particular mode of vibration i as a function of the damper coefficient c, mass per unit length m, cable length L and fundamental circular frequency ω_{o1} as well as normalized damper location ℓ_1/L.

In Figure 2, the normalized damping ratio $\zeta/(\ell_1/L)$ has been plotted against the nondimensional damping parameter, κ, for the first five modes for a damper location of $\ell_1/L=0.02$, and it is evident that the five curves collapse very nearly onto a single curve in good agreement with the approximation of Equation 1. It is important to note that because the mode number is incorporated in the nondimensional damping parameter κ, the optimal damping ratio can be achieved in only one mode of vibration in the case of a linear damper. This is a potential limitation, because it is currently unclear how to specify, a priori, the mode in which optimal performance should be achieved for effective suppression of stay-cable vibration, and designing a damper for optimal performance in a particular mode may potentially leave the cable susceptible to vibrations in other modes.

When the damper is far from the end of the stay, different regimes of behavior can be observed, potentially significantly altering the performance of the damper. For such situations, the reader is referred to the publication Main and Jones (2002a).

4.1.2 Linear damper with a friction threshold

An asymptotic solution for the damping ratios in the case of a taut cable with a linear viscous damper was extended to the nonlinear case of a viscous damper with a friction threshold using an "equivalent viscous damper" formulation. Relevant nondimensional parameter groupings were identified, and the solution characteristics were explored. It is observed that this formulation does a poor job of estimating the amplitude at which the damper is completely locked due to friction, and the results of this analysis should only be considered accurate for relatively small values of the friction parameter μ,

$$\mu \equiv \left(\frac{F_o}{T}\right)\left(\frac{Ai}{L}\right)^{-1} \qquad (3)$$

for which the assumption of nearly sinusoidal oscillation is appropriate (F_0 is the friction threshold). The observed effect of the friction threshold is to reduce the optimal value of the viscous damping parameter κ and ultimately to reduce the attainable damping ratios from the values for a purely viscous damper. Figure 3 shows an example of this effect.

4.1.3 Power-law damper

Details of a power-law damper solution are omitted here and can be found in Main and Jones (2002b).

4.2 Cross ties

One of the methods that is sometimes adopted to counteract the undesired oscillations is to increase the in-plane stiffness of stays by connecting them together by means of a set of transverse cables, defined as cross-ties (or “aiguilles”). These cables are also used to reduce the cable sag variations among the stays of different length, ensuring a more uniform axial stiffness on the consecutive stays. From the dynamic perspective, the properties of the single cable, considered as a separate element, are modified by the presence of the lateral constraints that influence its oscillation characteristics. Similarly, a connection of simple suspended elements is transformed into a more complex cable network; a closed-form solution to the dynamic problem is more elusive.

The analysis method developed was applied to the study of a cable network that was modeled after the Fred Hartman Bridge, a twin-deck cable-stayed bridge over the Houston Ship channel, with central span of 380m and side spans of 147m.

The investigated system corresponds to the south-tower central-span portion, a set of twelve stays with a three-dimensional arrangement (Figure 4). Stay “24S” is assumed as reference element; all other stay quantities are normalized with respect to this element.

A careful study of the solution patterns showed two categories of roots (Figure 5). The first are associated with “global modes”, in which the whole set of cables is involved in the oscillation and with a substantial increment in the generalized mass of the mode, with respect to the individual cable modes. The second category is one of “localized modes”, in which the response of network is not global but the maximum amplitudes are located in the intermediate segments of specific cables only. Moreover the overall characteristics of these modal forms (both symmetric or antisymmetric) can be different from the solution for individual cables, and influenced by the presence and the location of the transverse connectors. The wavelength of these modes is essentially governed by the distance between two consecutive connectors (being almost coincident with the nodes of the modal shape). The high density of solutions is related to the fact that these modes are the components of high-order modes of the individual cables with different magnitude, for which the location of the connector on each cable represents a node of the individual mode shape. These modal patterns carry important implications for design.

More details can be found in Caracoglia and Jones (2002a-c)

5.0 DESIGN GUIDELINES

The preceding investigations and analyses led to the develop of a set of guidelines for mitigation against stay-cable vibration. These are divided into guidelines for new bridges, and for existing bridges in sections 4.1 and 4.2 below.

5.1 New cable-stayed bridges

1. General:
A sufficiently detailed cable vibration analysis (including modal analysis of the cable system) must be performed as part of the bridge design to identify the potential for cable vibration. The following factors must be examined: the dynamic properties of the cables, dynamics of the structural system, geometry of the cable layout, cable spacing, exposure conditions, and estimated Scruton Numbers (Sc).

2. Mitigation of rain/wind mechanism:
At a minimum, providing an effective surface treatment for cable pipes to mitigate rain/wind vibrations is highly recommended. One common method is the use of double-helical beads. The effectiveness of the surface treatment must be based on the tests applicable to the specific system, provided by the manufacturer.

3. Additional mitigation:
Depending on the outcome of the vibration study (Item 1), the provision of at least one of the following major cable vibration mitigation measures (in addition to surface treatment) is

recommended:

- Additional damping (using external dampers)
- Cable cross-ties

4. Minimum Scruton Number
Following are minimum desired Scruton Numbers (Sc):
$m\zeta / \rho D^2 > 10$ for regular cable arrangements

$m\zeta / \rho D^2 > 5$ for cable pipes with effective surface treatment suppressing rain/wind vibrations (see note below)

Note: Limited tests (Larose & Smitt, 1999) on cables with double-helix surface treatments have suggested that $m\zeta / \rho D^2 > 5$ may be acceptable. However, it is felt that such reductions should be made only for regularly spaced single cable arrangements. In general it is recommended to keep the Scruton Number as high as possible by providing external dampers and/or cross-ties. For unusual geometry or double stay arrangements where parallel stays are placed within close proximity to one another, careful case-by-case evaluation of these limits are recommended.

5. External dampers:
Manufacturer warranties should be provided for all damping devices. Most dampers used in bridges are proprietary items and design details should be provided by the manufacturer.

A damper can be tuned to yield optimal damping in any one selected mode. For other modes the level of damping will be less than this optimal value. Rain/wind vibrations occur predominantly in mode 2. Therefore, if a damper is to be tuned to a particular mode to mitigate rain/wind vibrations, it appears logical to select mode 2.
There are many types and designs of dampers, and linear dampers have been shown to be effective through their widespread use in the past. However, recent analytical studies show that non-linear dampers can be used to provide a more optimal condition than linear dampers, as these are effective over a larger range of modes. In particular, the damping performance of square-root dampers is independent of the mode number and is only affected by the amplitude of vibration.

With some dampers (such as dash-pot type), an initial static friction force must be overcome before engaging of the viscous element. Field experiments have shown the presence of this stick-move-stick-move behavior associated with such dampers. This may effectively provide a fixed node instead of the intended damping for the cable at low amplitude oscillations, and should be considered. The viscoelastic type dampers where an elastomeric element is permanently engaged between the cable and the supporting elements, theoretically, are free of such initial frictional thresholds. On the other hand, there are also damper designs that rely on friction as the energy dissipation mechanism and the static friction threshold for such dampers may be higher than for the other types.

Another factor needing consideration is the directionality of the damper. The cable vibrations observed in the field indicate both vertical and horizontal components of motion. Some damper designs are antisymmetric and provide damping against cable motion in any direction. Other dampers (e.g. dash-pot types) provide damping against motion only along the axis of the damper. It is possible to arrange two or more such dampers so that the combination is effective in all directions. As the majority of the observed motion due to rain/wind vibrations is in the vertical direction, it may be sufficient to provide damping against only the vertical motion. However, this has not been clearly established. It is recommended that damping be made effective against cable movement in any direction.

Damper mounting details may transfer lateral forces due to damper action onto components of the cable anchorage. Such forces must be considered in the design of the cable anchorages.

6. Cable cross ties:
If used, provide clear and mandatory specifications for cable cross-ties. Experience shows that cross-ties, when properly detailed and installed, can be an effective method for suppressing undesirable levels of cable vibrations. Reported failures of cross-ties have been generally traced to improper details and material selection.

The use of cross ties creates local modes, which must be considered in design. The frequency of the first plateau of local modes should be kept as high as possible. Symmetric configurations of the restrainers with respect to intermediate-length cables is preferred to increase the frequency interval (lower limit in particular) corresponding to local modes, since they minimize the longest segment length.

Cable cross-ties must be provided with initial tension sufficient to prevent slack of the cross-ties during design wind events. The level of tension depends on the dynamic properties of the cable system and the design wind event. The initial cross-tie tensions must be established based on rational engineering analysis. Also, the tie to cable connection must be carefully designed and detailed for the transfer of the design forces.

7. User tolerance limits:
A preliminary survey on sensitivity of bridge users to stay cable vibrations has indicated that the comfort criteria for cable displacement can be described using the following maxima (within 0.5 to 2.0 Hz range):

- 0.5 D (Preferred)
- 1.0 D (Recommended)
- 2.0 D (Not to Exceed)

While this aspect may need further study, the above can be used as a guide when such displacements can be computed and/or needed as input for design of such elements as dampers and cross-ties. The displacement limits need not be considered for extreme events.

5.2 Retrofit of existing bridges

If an existing bridge is found or suspected to exhibit episodes of excessive stay cable vibration, an initial field survey and inspection of the cable system should be performed to assemble the following information:

- Eye-witness accounts, video footage of episodes
- Condition of the stay cable anchorages and related components, noting any visible damage and/or loose, displaced components

A brief field instrumentation and measurement program can be used to obtain such parameters as the existing damping levels of the cables. Instrumentation of cables to record the vibration episodes, wind direction, wind velocity, and rain intensity during their occurrences could also provide some confirmation of the nature of cable vibrations.

The mitigation methods available for retrofit of existing bridges follow closely those provided for the new bridges. However, the application of surface treatment may be difficult, not practical, or cost prohibitive on existing structures. The addition of cross-ties and/or dampers is recommended.

A split-pipe with surface modifications can be installed over the existing cable pipe if this is found to be practical and cost effective. In many of the older bridges for cables using polyethylene (PE) pipes, ultraviolet (UV) protection to cable pipes is provided by wrapping the PE pipe with Tedlar tape. These cables require periodic re-wrapping as part of routine maintenance. The newer high-density polyethylene (HDPE) cable pipes are manufactured with a co-extruded outer shell that provides the needed UV resistance, thus providing a split-pipe as a secondary outer pipe has the added benefit of eliminating the need for future Tedlar taping for the UV protection.

In addition, any damaged cable anchorage hardware must be properly retrofitted or replaced. It is recommended that the original cable supplier be contacted to ensure the replacement of cable anchorage components and that the addition of mitigative devices are compatible with the original design of the stay anchorage area.

6.0 RECOMMENDATIONS FOR FUTURE RESEARCH AND DEVELOPMENT

The design guidelines provide a concise approach to suppress wind-induced vibrations in cable-stayed bridges, and are based on the existing knowledge base and further investigations performed through this project. While the design recommendations are empirical, the mitigation methods discussed (dampers, cable cross-ties, and surface modification) are proven to be effective through both past experience and research. Future research in the following areas clarifying some of the remaining key issues would strengthen the design guidelines.

This is the first time a set of design guidelines have been proposed for the mitigation of stay cable vibration. It is expected that future adjustments based on actual cable performance and advances in cable technology may require further refinements to the design guidelines.

1. Additional wind-tunnel testing of dry inclined cables.

2. Further investigation of characteristics of deck-induced vibration of stay cables.

3. Development of mechanics-based model of rain/wind induced vibrations.

4. Develop a mechanics-based model for stay-cable vibration enabling the prediction of anticipated vibration characteristics.

5. Predict the performance of stay cables after mitigation using the model.

6. Perform a detailed quantitative assessment of various alternative mitigation strategies.

7. Develop improved understanding of inherent damping in stays and that provided by external devices.

8. Improved understanding of cross-tie solutions.

9. Refine recommendations for effective and economical design of stay-cable vibration mitigation strategies for future bridges.

7.0 CONCLUDING REMARKS

This project has been successful in producing a set of guidelines and recommendations for stay-cable vibration mitigation based on information available at the time of its conclusion. While this does include information based on a review of the literature and a significant amount of research on the characterization of field measurements and damper performance, the guidelines may be improved through the future research items proposed.

8.0 ACKNOWLEDGEMENTS

This research was supported through an FHWA-sponsored project (Harold Bosch technical contact) on stay-cable vibration awarded to a team of investigators including HNTB Corporation, New York, Johns Hopkins University, Baltimore, Maryland; Rowan Williams, Davies and Irwin, Ontario, Canada, and Buckland and Taylor of Vancouver, BC, Canada. The Fred Hartman and Veterans' Memorial Bridges project were funded by the Texas Department of Transportation (TxDoT) through Texas Tech University (TTU) and the University of Texas at Austin (UTA) in a project managed by Whitlock, Dalrymple Poston and Associates, Inc. (WDP) of Austin, Texas.

All components of the research team were assisted by various staff and other personnel. The first author would like in particular to acknowledge the significant contributions of the following graduate students and post-doctoral fellows: Joseph Main, Luca Carcoglia; Ender Ozkan; Delong Zuo.

9.0 REFERENCES

Flamand, O. (1994). Rain-wind induced vibration of cables. Proceedings of the International Conference on Cable-stayed and Suspension Bridges (AFPC), Deauville, France, Oct. 1994, 2, 523-531.

Hikami, Y. and Shiraishi, N. (1988). “Rain-wind induced vibrations of cables in cable stayed bridges”, Journal of Wind Engineering and Industrial Aerodynamics, Vol. 29, p. 409-418

Matsumoto, M., Shiraishi, N., & Shirato, H. (1989). Inclined-cable aerodynamics: Structural design, analysis & testing proceedings. Proceedings of the ASCE Structures Congress, San Francisco.

Matsumoto, M. (1998). Observed behaviour of prototype cable vibration and its generation mechanism. Bridge Aerodynamics, Larsen &

Esdahl (eds), Balkema, Rotterdam, 189-211.

PTI Guide Specification (2001). Recommendations for stay cable design, testing and installation. Post-Tensioning Institute Committee on Cable-Stayed Bridges, 4th ed.

Saito, T.; Matsumoto, M.; and Kitazawa, M. (1994). "Rain-wind excitation of cables on cable-stayed Higashi-Kobe bridge and cable vibration control", Proceedings of Conference on Cable-Stayed and Suspension Bridges, p. 507-514

10.0 BIBLIOGRAPHY

Listed below is a partial list of publications that resulted from the research effort described herein.

Caracoglia, L. & Jones, N. P. (2002a) "Analytical method for the dynamic analysis of complex cable structures." Proceedings, ASCE Engineering Mechanics Division Conference. Columbia University, NY.

Caracoglia L., & Jones N. P. (2002b) "In-plane dynamic behavior of cable networks I: formulation and basic solutions." Journal of Sound and Vibration. (In review)

Caracoglia L., & Jones N. P. (2002c) "In-plane dynamic behavior of cable networks II: Prototype prediction and validation." Journal of Sound and Vibration. (In review)

Jones, N. P. & Zuo, D. (2002) "Measurement of Stay Damping - Sunshine Skyway Bridge, Tampa, FL" Report to Lendis Corporation and FHWA.

Jones, N. P. & Main, J. A. (2002) "Evaluation of Mitigation Strategies for Stay-Cable Vibration." Proc. Structures Congress. ASCE, Reston, VA.

Main, J. A. & Jones, N. P. (2002) "Analytical Investigation of the Performance of a Damper with a Friction Threshold for Stay-Cable Vibration Suppression." Proceedings, ASCE Engineering Mechanics Division Conference. Columbia University, NY.

Main, J. A. & Jones, N. P. (2001) "Analysis and design of linear and nonlinear dampers for stay cables." Proceedings Fourth International Symposium on Cable Dynamics, Montreal, Canada, AIM.

Main, J. A., & Jones N. P. (2002a) "Free Vibrations of a Taut Cable with Attached Damper. I: Linear Viscous Damper." Journal of Engineering Mechanics, ASCE, 128(10), 1062-1071.

Main, J. A., & Jones N. P. (2002b) "Free Vibrations of a Taut Cable with Attached Damper. II: Nonlinear Damper." Journal of Engineering Mechanics, ASCE, 128(10), 1072-1081.

Main, J. A., & Jones, N. P. (2001) "Measurement and mitigation of stay-cable vibration." Proceedings Asia-Pacific Conference of Wind Engineering. Kyoto University, October.

Main, J. A., & Jones, N. P. (2001) "Stay-cable vibration in cable-stayed bridges: Characterization from full-scale measurements and mitigating strategies." Journal of Bridge Engineering. ASCE, 6(6), 375-387.

Main, J. A., Jones, N. P. & Yamaguchi, H. (2001) "Characterization of rain/wind induced stay-cable vibrations from full-scale measurements." Proceedings Fourth International Symposium on Cable Dynamics. Montreal, Canada, AIM.

Ozkan, E. & Jones, N. P. (2001) "Investigation of cable-deck interaction using full-scale measurements on a cable-stayed bridge." Proceedings 1st Americas Conference of Wind Engineering. Clemson University, June.

Ozkan, E., Main, J. A. & Jones, N. P. (2001) "Full-scale measurements on the Fred Hartman Bridge." Proceedings Asia-Pacific Conference of Wind Engineering. Kyoto University, October.

Ozkan, E., Main, J. A. & Jones, N. P. (2001) "Long-term measurements on a cable-stayed bridge." Proceedings IMAC-XIX. Society for Experimental Mechanics.

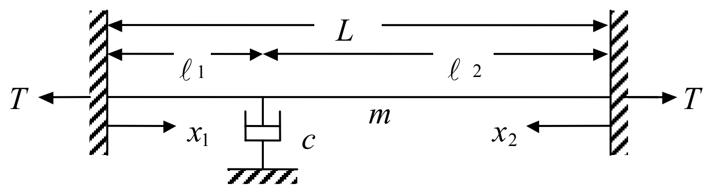

Figure 1:   Taut cable with a linear damper.

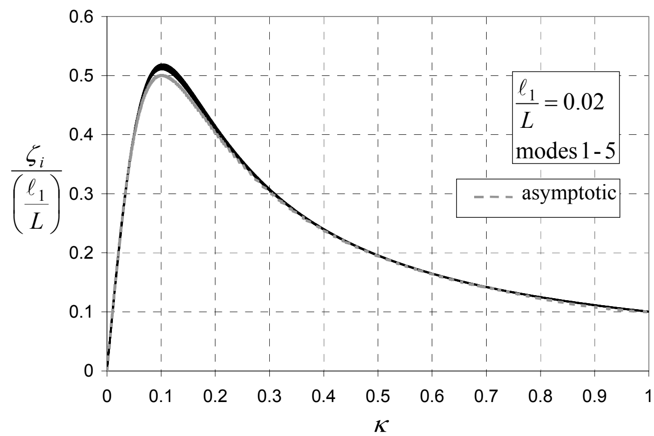

Figure 2: Normalized Damping Ratio versus Normalized Damper Coefficient.

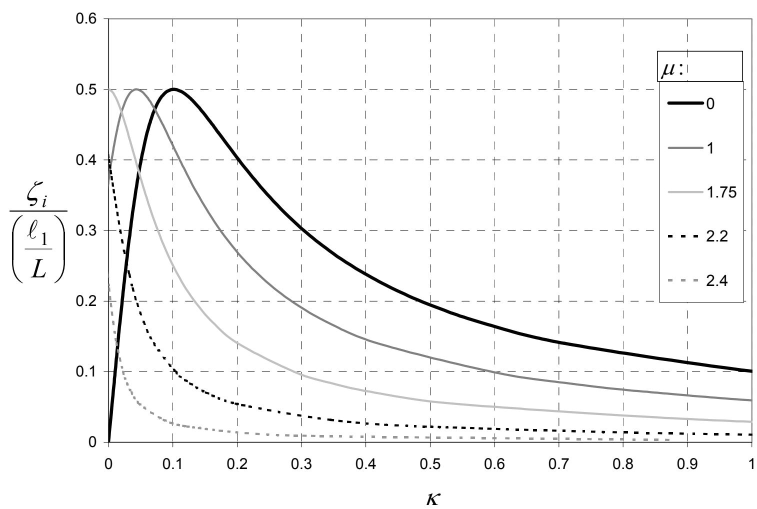

Figure 3: Normalized Damping Ratio vs. κ with varying μ.

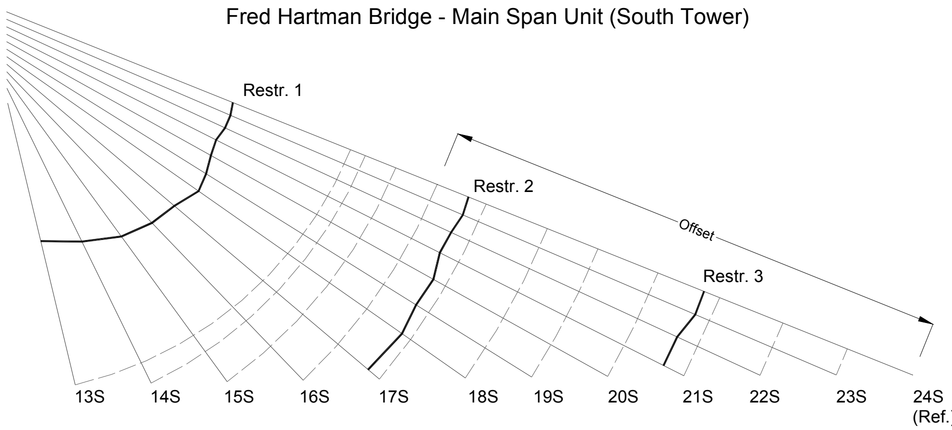

Figure 4: General problem formulation (original configuration).

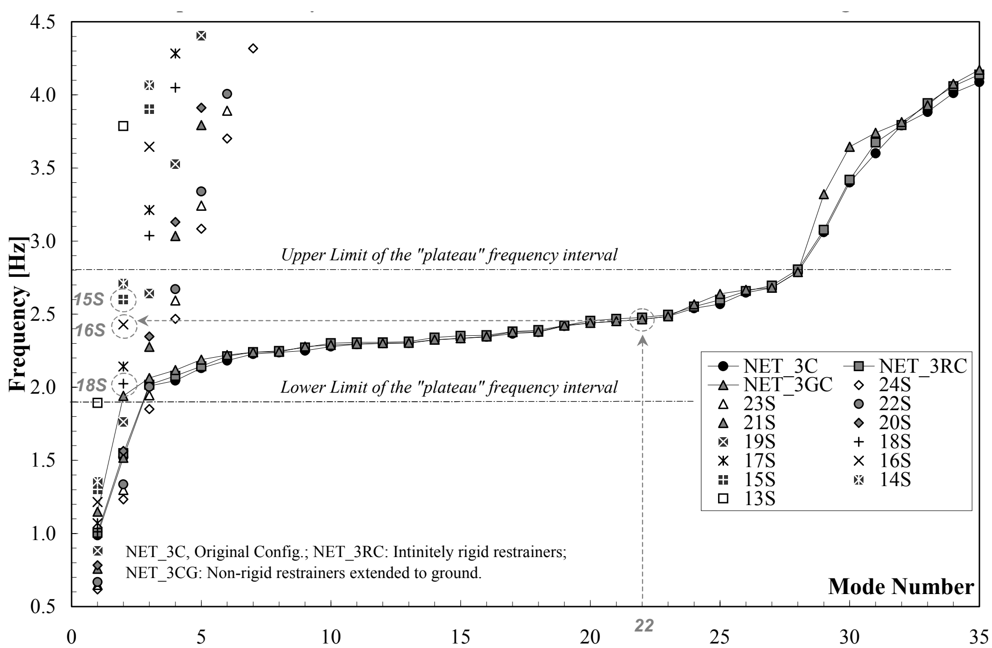

Figure 5: Comparative analysis of network vibration characteristics and individual-cable behavior; Fred Hartman Bridge; NET_3C, original configuration; NET_3RC, infinitely rigid restrainers; NET_3CG, spring connectors extended to ground (Restr. 2,3).