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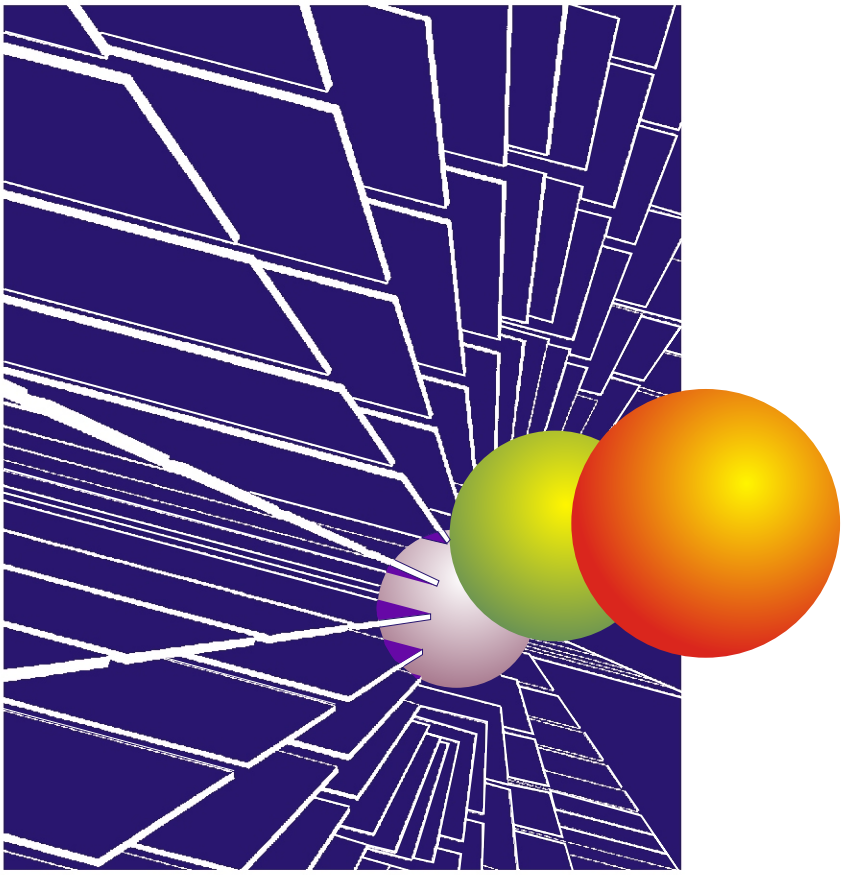
Wind Induced Vibration of Stay Cables

Interim Final Report

RI 98-034



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16. Abstract Cable-stayed bridges have become the form of choice over the past several decades for bridges in the medium to long span range. In come cases serviceability problems involving large amplitude vibrations of stay cables under certain wind and rain conditions have been observed. This study was conducted in order to develop a set of consistent design guidelines for mitigation of excess cable vibration on cable-stay bridges.

In order to accomplish this objective the Project Team started with a thorough review of existing literature to determine the state of knowledge and identify any gaps that must be filled in order 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 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 brief studies on live load induced vibration and establishing driver/pedestrian comfort criteria.

Based on the above, design guidelines for mitigation of wind-induced vibrations of stay cables were developed.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2002)

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EXECUTIVE SUMMARY

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 firmly established the 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 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
- 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. This tunnel had a test section 10-ft (3-m) wide, 20-ft (6-m) high, and 39-ft (12-m) long, and could reach 87-mph (39-m/s) maximum wind speed. With a removable roof section, this tunnel was ideal for the high speed galloping tests of inclined full-scale cable segments.

The results of the Project Team's dry inclined cable testing have significant implications for the design criteria of cable-stayed bridges. The 2001 PTI Recommendations for Stay Cable Design, Testing, and Installation indicates that the level of damping required for each cable is controlled by the inclined galloping provision, which is more stringent than the provision to suppress rain/wind vibrations. This testing, however, suggests that if even a low amount of structural damping is provided to the cable system, inclined cable galloping vibrations are not significant. This damping corresponds to a Scruton number of 3, which is less than the minimum of 10 established for suppression of rain/wind vibrations. Therefore if enough damping is provided to mitigate rain/wind vibrations, then dry cable instability should also be suppressed.

The Project Team obtained matching funds from Canada's National Engineering and Sciences Research Council (NSERC) for the testing at the University of Ottawa, effectively doubling the FHWA's 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 at no cost to the FHWA.

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, design guidelines for mitigation of wind-induced vibrations of stay cables were developed. These are presented with two worked examples that illustrate their application. 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 judgement and due consideration of special conditions surrounding each project.

1 INTRODUCTION

1.1 BACKGROUND

Cable-stayed bridges are a relatively new structural form made feasible with the combination of advances in manufacturing of materials, construction technology, and analytical capabilities that took place largely within the last few decades.

The first modern cable-stayed bridge was the Stromsund Bridge built in the 1950's in Sweden with a main span of 600-ft (183-m) and two symmetrical back spans of 245-ft (75-m) each. It had only two cables on each side of the tower, anchored to steel I edge girders.

Today, cable-stayed bridges have firmly established their unrivalled position as the most efficient and cost effective structural form in the 500-ft (150-m) to 1500-ft (460-m) span range. The cost efficiency and the general satisfaction with their aesthetic aspects has propelled this span range in either direction, with both increasingly shorter and increasingly longer spans being designed and constructed at the present time. The record span built to date is the Tatara Bridge connecting the islands of Honshu and Shikoku in Japan, with its 2920-ft (890-m) long main span. The planned Stonecutters Bridge in Hong Kong has a 3280-ft (1000-m) long main span. The early engineering approach to the stay cables was essentially derived and hybridized from the already established engineering experience with suspension cables and post-tensioning technology.

The stay cables are laterally flexible structural members with very low fundamental frequency (first natural mode). Due to the range of different cable lengths (thus the range of frequencies) the collection of stay cables on a cable stayed bridge has practically a continuum of fundamental and higher mode frequencies. Thus, any excitation mechanism with any arbitrary frequency is likely to find one or more cables with either a fundamental or higher mode frequency sympathetic to the excitation. Cables also have very little inherent damping and are therefore not able to dissipate much of the excitation energy, making them susceptible to large amplitude build-up. For this reason, the stay cables can be somewhat lively by nature and have been known to be susceptible to excitations, especially during construction, wind, and rain/wind conditions.

Recognition of this susceptibility of stay cables has led to the incorporation of some mitigation measures on several of the earlier structures. These included cable cross-ties that effectively reduce the free length of cables (increasing their frequency) and external dampers that increase cable damping. Perhaps due to the lack of widespread recognition of the stay cable issues by the engineering community and the supplier organizations, the application of these mitigation measures on early bridges appear to have been fairly sporadic. However, those incorporating cable cross-ties or external dampers have generally performed well.

Field observation programs have provided the basis for characterization of stay cable vibrations and the environmental factors that induce them (Wianecki, 1979; Hikami & Shiraishi, 1988; Matsumoto et al. 1997). Peak-to-peak amplitudes up to 6 ft (2 m) have been reported, with typical values of around 2 ft (60 cm). Vibrations have been observed primarily in the lower cable modes, with frequencies ranging approximately from 1 to 3 Hz. Early reports described the

vibrations simply as transverse in the vertical plane, but detailed observations suggest a more complicated elliptical loci.

High-amplitude vibrations have been observed over a limited range of wind speeds. At several bridges in Japan the observed vibrations were restricted to a wind velocity range from 13 mph to 38 mph (6 to 17 m/s) (Matsumoto et al., 1993). More recent field measurements revealed large-amplitude vibrations at around 90 mph (40 m/s). The wind speed did not reach values sufficiently high to determine whether these vibrations were also velocity-restricted (Matsumoto et al., 1997).

The stays of the Brotonne Bridge in France were observed to vibrate only when the wind direction was 20°-30° relative to the bridge longitudinal axis (Wianecki, 1979). On the Meiko-Nishi Bridge in Japan, vibrations were observed with wind direction greater than 45° from the deck only on cables that declined in the direction of the wind (Hikami & Shiraishi, 1988). However, instances have also been reported subsequently of simultaneous vibration of stays with opposite inclinations to the wind (Matsumoto et al., 1990).

From field observations it became evident that these large oscillation episodes occurred under moderate rain combined with moderate wind conditions, and hence were referred to as "rain/wind vibrations" (Hikami & Shiraishi, 1988). Extensive research studies at many leading institutions over the world have undoubtedly confirmed the occurrence of rain/wind induced vibrations. Totally unknown prior to its manifestation on cable-stayed bridges, the mechanisms leading to rain/wind vibrations have been identified. The formation of a so-called water rivulet along the upper side of the cable under moderate rain conditions and its interaction with wind flow have been solidly established as the cause through many recent studies and wind-tunnel tests (Hikami & Shiraishi, 1988; Matsumoto et al., 1992; Flamand, 1994; Verwiebe & Ruscheweyh, 1997).

Based on this understanding, exterior cable surface modifications that interfere with the water rivulet formation have been tried and proven to be very effective in the mitigation of the rain/wind vibrations. Particularly popular (and shown to be effective through experimental studies) are the double spiral bead formations affixed to the outer surface of the cable pipes (Flamand, 1994). Cable exterior pipes with such surface modifications are available with all major cable suppliers with test data applicable to the particular system. This type of spiral bead surface modification has been applied on many cable-stayed bridges both with and without other mitigation measures such as external dampers and cable ties. From the observations available to date, the bridges incorporating stay-cables with effective surface modifications appear to be generally free of rain/wind vibrations.

At the time of the present investigation, it was evident that the rain/wind problem had been essentially solved, at least for practical provisions for its mitigation. The Scruton number, identified later in the report, is generally accepted as the key parameter describing susceptibility of a given cable to rain/wind vibrations. Raising the Scruton number by increasing damping or, alternatively, use of cable cross-ties has been recognized as the standard solution for the mitigation of rain/wind vibrations. Generally these are applied in combination with a proven surface modification.

However, no such clarity existed with respect to other potential sources of cable vibration. High speed galloping of inclined cables (discussed later) was the single most issue that limited the designer's options. The only effective method available for satisfying the existed criterion on galloping was to raise the natural frequency of cables through the use of cable cross-ties. However, the inclined dry cable galloping criteria being used was postulated on such a limited set of data that its application was frequently brought into question.

Thus, to meet the project objective of formulating design guidelines, some further experimental and analytical work was needed to supplement the existing knowledge base on stay cable vibration issues.

1.2 PROJECT OBJECTIVES AND TASKS

The charter of the Project Team, established early in the development of the program, consisted of the following objectives with regards to "Wind Induced Vibration of Stay Cables":

- Identify gaps in current knowledge base
- Conduct analytical and experimental research in critical areas
- Study performance of existing cable-stayed bridges
- Study current mitigation methods
- Develop procedures for aerodynamic performance assessment
- Develop design and retrofit guidelines for stay cable vibration mitigation

Overall project goals were translated into a series of Tasks A through F:

Task A: Synthesize Available Information

Reference Database (Appendix A1, Section 2.1)
Descriptions of Wind-Induced Cable Vibrations (Appendix B1, Section 3.1)

Task B: Inventory of U.S. Cable-Stayed Bridges

Inventory Database (Appendix A2, Section 2.2)

Task C: Perform Analysis/Evaluation/Testing

Wind-Tunnel Testing of Dry Inclined Cables (Appendix B2, Section 3.2)
Study of Mitigation Methods (Appendix C1, Appendix C2, Section 3.4)
Study of Other Excitation Mechanisms (Appendix D2, Section 3.3)
Field Measurements of Stay Cable Damping (Section 3.5)
Study of User Comfort (Appendix D3, Section 3.6)
Calculations on Mechanics of Inclined Cables (Appendix D1)

Task D: Develop Guidelines for Design and Retrofit (Section 4.0)

Task E: Formulate Recommendations for Future Research (Section 5.0)

Task F: Project Documentation

2 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 has 198 references and includes the article titles, authors, reference information, and abstracts when attainable and has built-in search capabilities. Examples of search pages and a full listing of the references in this database are included in Appendix A1.

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. Complete descriptions, examples of data forms, and a full list of bridges in the database are given in Appendix A2.

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 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:

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

All of these mechanisms are discussed in detail in Appendix B1. Vortex excitation, rain/wind, wake galloping of groups of cables, and galloping of single dry inclined cables all require careful consideration by the designer and are described in summary form later in this section.

The following parameters are relevant to these wind-induced vibrations:

Reynolds Number

A key parameter in the description of compressible fluid flow around objects (such as wind around stay cables) is the Reynolds number. The Reynolds number is a measure of the ratio of the inertial forces of wind to the viscous forces (Simiu & Scanlan, 1996) and is given by the following:

$$\mathrm{Re} = \frac{\rho V D}{\mu} \tag{1}$$

ρ = air density (lbf/ft^3, kg/m^3)
V = wind velocity (ft/sec, m/sec)
D = cable diameter (ft, m)
μ = viscosity of air (lbf/ft-sec, g/m-sec)

Strouhal Number

The Strouhal number is a dimensionless parameter relevant to vortex excitation:

$$S = \frac{N_s D}{V} \tag{2}$$

N_s = frequency of vortex excitation

The Strouhal number remains constant over extended ranges of wind velocity. For circular cross-section cables in the Reynolds number range $1x10^4$ to $3x10^5$, S is about 0.2.

Scruton Number

The Scruton number is an important parameter when considering vortex excitation, rain/wind induced vibrations, wake galloping, and dry inclined cable galloping.

$$S_c = \frac{m\zeta}{\rho D^2} \tag{3}$$

m	= mass of cable per unit length (lbf/ft, kg/m)
ζ	= damping as ratio of critical damping
ρ	= air density (lbf/ft^3, kg/m^3)
D	= cable diameter (ft, m)

This relationship shows that increasing the mass density and damping of the cables increases the Scruton number. Most types of wind-induced oscillation tend to be mitigated by increasing the Scruton number.

Vortex Excitation of an Isolated Cable and Groups of Cables

Vortex excitation is probably the most classical type of wind-induced vibrations. It is characterized by limited-amplitude vibrations at relatively low wind speeds. Vortex excitation of a single isolated cable is caused by the alternate shedding of vortices from the two sides of the cable when the wind is approximately perpendicular to the cable axis. The wind velocity at which the vortex excitation frequency matches the natural frequency (N_r) is found by using the Strouhal number S:

$$V = \frac{N_r D}{S} \tag{4}$$

The amplitude of the cable oscillations is inversely proportional to the Scruton number S_c. Increasing the mass and damping of the cables increases the Scruton number and therefore reduces oscillation amplitudes.

Inherent cable damping ratios can range anywhere from 0.0005 to 0.01, and an accurate value is difficult to predict. The lower end of this range is typical of very long cable stays without any grout infill, while the upper end of this range is more typical of shorter cable-stays with grouting and perhaps some external damping. A realistic estimate of inherent cable damping ratios on in-service bridges is in the range from 0.001 to 0.005. (See Section 3.5 for field measurements.)

For example, a cable consisting of steel strands grouted inside the cable pipe and with a damping ratio (ζ) of 0.005 has a Scruton number of about 12, and the amplitude of oscillation is only

about one half of a percent of the cable diameter. During construction and prior to grouting, the damping ratio of stay cables can be extremely low, e.g. 0.001, and the amplitude could conceivably increase to about 4% of the cable diameter, which is still small. Therefore, vortex shedding from the cables is unlikely to be a major vibration problem for cable-stayed bridges. By adding a small amount of damping, vortex excitation will be effectively suppressed.

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):

$$\frac{m\zeta}{\rho D^2} > 10 \qquad (5)$$

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).

Wake Galloping for Groups of Cables

Wake galloping is the elliptical movement caused by variations in drag and across-wind forces for cables in the wake of other elements, such as towers or other cables. This occurs at high wind speeds and leads to large amplitude oscillations. These oscillations have been found to cause fatigue of the outer strands of bridge hangers at end clamps on suspension and arch bridges. Similar fatigue problems are a theoretical possibility on cable-stayed bridges, but to date none have been documented.

The Scruton number is an important parameter with regard to wake galloping effects. An approximate equation for the minimum wind velocity U_{CRIT} above which instability can be expected due to wake galloping effects has been proposed (Irwin, 1997; Cooper, c.1985; PTI, 2001). It is given by:

$$U_{CRIT} = c\, f\, D\sqrt{S_c} \qquad (6)$$

c = constant
f = natural frequency
D = cable diameter
S_c = Scruton Number

For circular sections, the constant c has an approximate median value of 40. For cable-stayed bridges, this constant depends on the clear spacing between cables, and the following range of values based on the cable spacing are commonly used:

c = 25 for closely spaced cables (2D to 6D spacing)
c = 80 for normally spaced cables (generally 10D and higher)

Due to the level of uncertainty associated with practical applications, it is recommended that these values be applied conservatively, exercising engineering judgement.

The critical wind velocity may be low enough to occur commonly during the life of the bridge. Wake galloping therefore has the potential to cause serviceability problems. The equation for U_{CRIT} suggests several possibilities for mitigation. By increasing the Scruton number or natural frequency, the cables will be stable up to a higher wind velocity. However, increasing the frequency is far more effective in raising U_{CRIT} due to the square root manifestation of Sc in the above expression. The Scruton number increases with additional damping. The natural frequency may be increased by installing spacers or cross-ties along the cables to shorten the effective length of cable for the vibration mode of concern.

It should be noted that wake galloping is not a major design concern for normal well separated cable arrangements. For unusual cases, however, it is recommended that some attention be paid to the possibility of wake galloping.

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 (this general relation was given in a different form in Equation 6 for wake galloping, with c = 40):

$$(U/f\mathrm{D})_{\mathrm{CRIT}} = 40\sqrt{\mathrm{S_c}} \qquad (7)$$

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, as described in Section 3.2 of this report.

3.2 WIND-TUNNEL TESTING OF DRY INCLINED CABLES

Introduction

From the information reported on the various types of cable vibrations due to wind loads, it was determined that galloping of dry inclined cables was the most critical issue requiring further experimental research. The wind tunnel data of Saito et al. (1994) showed evidence of dry inclined cable oscillations with some of the characteristics of galloping, and stability criteria were suggested in their paper. However, based on their criteria, many existing cable-stayed bridges would have shown more evidence of dry cable galloping than has actually been observed. In order to clarify the dry cable galloping phenomenon and evaluate the stability criteria proposed by Saito et al. (1994), the Research Team conducted a series of wind tunnel tests of a full-size 2D sectional model of an inclined cable in the "Propulsion" wind tunnel at the Montreal Road campus of the Institute of Aerospace Research, National Research Council Canada (IAR/NRCC). A full description of the testing is included in Appendix B2.

The objectives of this study were to:

- Investigate the existence of dry inclined cable galloping
- Clarify the mechanisms of this type of vibration
- Determine the effects of the following parameters – wind speed, structural damping, surface roughness, and wind direction
- Refine the stability criterion proposed by Saito et al. (1994)

The following section summarizes the test program and its results.

Testing

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. The model was supported in the wind tunnel with the angles Φ and α being adjustable to represent different θ and β combinations. Figure 37 of Appendix B2 shows the relationship of these angles to the cable and wind direction. Similar to the Japanese studies (Saito et al., 1994), θ and β represent respectively the angle between the horizontal plane and the cable, and the yaw angle between the wind direction and the longitudinal bridge axis. The orientation of the two degree of freedom springs (perpendicular to the cable longitudinal axis) could be rotated about the cable axis through an angle α. Φ is the angle between the wind tunnel floor and the cable. The angle is only important when the vertical and horizontal frequencies of the cable are tuned to different frequencies. In the wind tunnel

this led to testing with an adjustable virtual ground plane. The relationship between the cable and the mean wind direction is represented by Equations 8 and 9:

$$\cos \Phi = \cos \beta \cos \theta \tag{8}$$

$$\tan \alpha = \tan \beta / \sin \theta \tag{9}$$

The aerodynamic behavior of the inclined cable model was investigated with different combinations of model setup, damping level and surface roughness as described in the following tables:

Table 1: Dry inclined cable testing - Model setup.

Full Scale Cable Angles Tested			Model Test Cable/Pipe Angles	
Model Setup	θ^o	β^o	Φ^o	α^o
1B	45	0	45	0
1C	30	35.3	45	54.7
2A	60	0	60	0
2C	45	45	60	54.7
3A	35	0	35	0
3B	20	29.4	35	58.7

Table 2: Dry inclined cable testing - Damping levels.

Damping Description	Dampers Used	Approximate Damping Range (% of critical) damping is amplitude dependent
Low damping	No damper added	0.03 to 0.09
Intermediate damping	16 elastic bands per sway spring	0.05 to 0.10
High damping	28 elastic bands per sway spring	0.15 to 0.25
Very high damping	Airpot damper with 1 1/4 dial turns	0.30 to 1.00

Table 3: Dry inclined cable testing - Surface condition.

Smooth Surface	Polyethylene (PE) Pipe with clean surface
Rough Surface	Polyethylene (PE) Pipe with glue sprayed on the windward side of the cable

Results Summary

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 (defined in Section 3.1) 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 $\theta \sim 45°$, the increase of surface roughness made the unstable range shift to lower wind speeds.

The results of this testing showed a deviation from the criteria described in the introduction. 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.

Figure 1 shows the results of this experiment as compared to the instability line determined by Saito et al. (1994). The graph presents Reduced Wind Velocity Ur vs. the Scruton number, where:

$$Ur = U_{CRIT} / (f D) \qquad (10)$$

U_{CRIT} = critical wind velocity at which instability occurs
f = natural frequency
D = cable diameter

The bold points indicate cable motions with amplitudes from ±0.4-in (10-mm) to ± 3.1-in (80-mm). Note the test rig would not allow for motions greater than ± 3.1-in (80-mm). However, only one test case reached this limit and is denoted by the triangular point. One point from Miyata et al. (1994) is also shown.

Conditions with oscillations less than ±0.4-in (10-mm) are denoted with an open circle. Many of these points lie in the region denoted as Unstable based on the Instability Line of Saito et al. (1994). It is suggested that this line can be redefined based on the dashed line denoted as FHWA Instability Line.

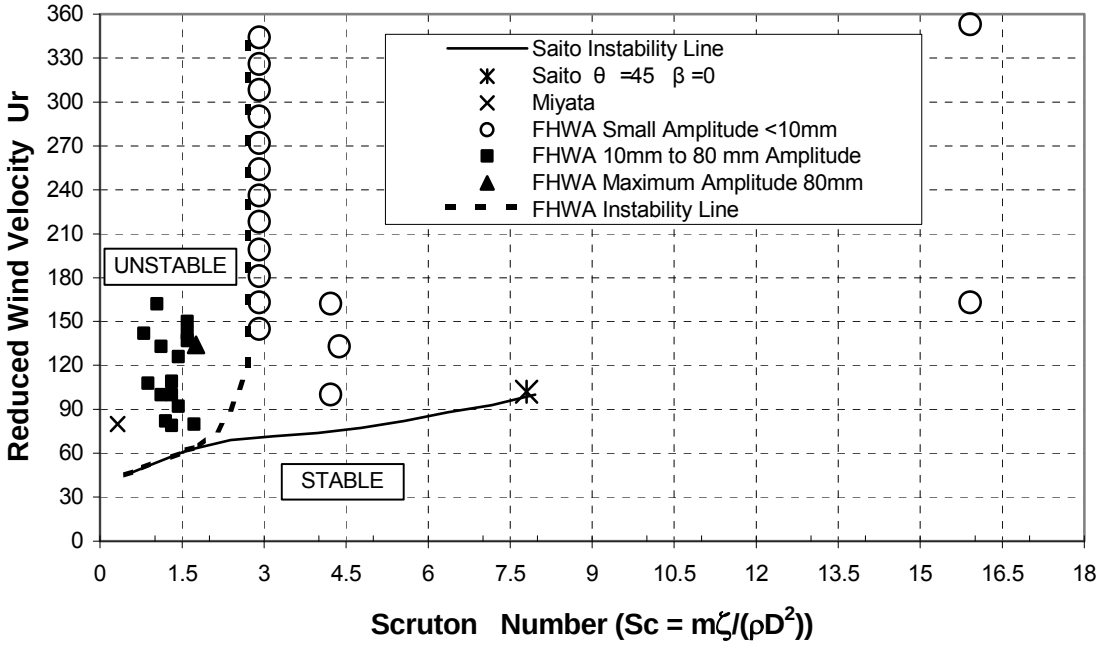

Figure 1: Comparison of wind velocity-damping relation of inclined dry cable.

This testing suggests that if even a low amount of structural damping is provided (ζ>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 (discussed in Section 3.1). Therefore dry cable instability should be suppressed by default if enough damping is provided to mitigate rain/wind vibrations. A complete report of the wind-tunnel testing by the Project Team on dry inclined cables is given in Appendix B2.

A second phase of testing was conducted on a static model to verify the findings of the initial study, using the same orientations where the large amplitude oscillations occurred. Pressure taps were added to record aerodynamic force measurements. The objectives of this phase were to clarify the mechanism of dry inclined cable galloping and investigate the differences between galloping and high-speed vortex shedding. The test report was not available as of the production of this document.

3.3 OTHER EXCITATION MECHANISMS

Effects Due to Live Load

This study was carried out by the Project Team to assess the amount of vibration caused by live loading and determine if this movement is significant as compared to wind vibration. To address this problem a computer model of a real bridge was subjected to a moving train load, and the vibrations of an individual cable were analyzed. A moving train has a greater effect on cables than passing trucks or random traffic. The cable tensions, displacements and anchorage rotations obtained from the dynamic time history analysis were compared to an analysis ignoring all dynamic effects as well as the results obtained from influence line calculations which are normally carried out during design. A summary of this work is given in this section, and the complete report is included in Appendix D2.

A 3D computer model of the Rama 8 Bridge in Bangkok, Thailand, was created for this analysis. The bridge has a single tower and a 984-ft (300-m) main span. The third longest cable (M26), with an unstressed length of 981-ft (299.1-m), was studied to determine the effects of live loading.

A static live load analysis was first conducted as a baseline using a 5-car transit train, neglecting dynamic properties of the train. Influence line analyses were performed to determine the maximum and minimum effects due to live load. For dynamic analysis, the transit train was modeled as a mass on damped springs and moved across the bridge at a speed of 50 mph (80 km/hr), taking into account the dynamic interaction between the train and the structure. The tension in Cable M26 is plotted in Figure 2 to compare the dynamic effects to the static effect. Note that the increase in maximum cable tension due to dynamic effects is less than 10%.

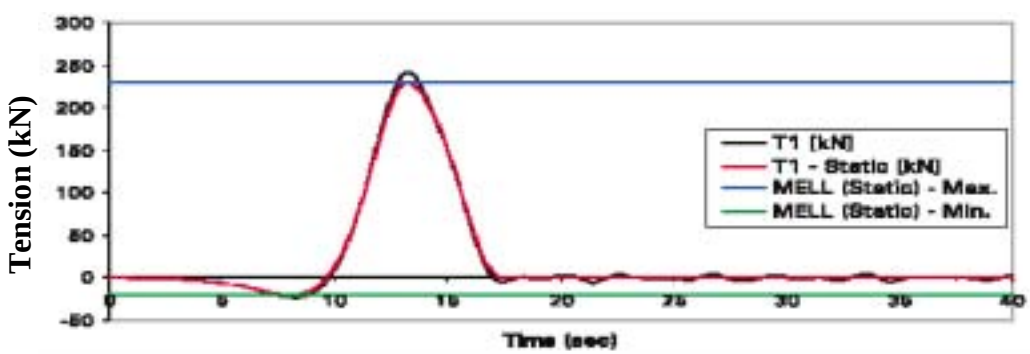

Figure 2: Cable M26, Tension vs. Time. Transit train speed 50 mph (80km/hr).

The results of this study indicated the following:

- A stay cable which is discretized with twenty elements accurately predicts the free vibration characteristics of a stay cable.
- Once the cable is modeled as part of the real structure with the tower and the deck providing realistic end conditions, the cable frequencies only change slightly but the mode shapes become spatial rather than being purely in-plane or out of plane.

- The cable tensions, displacements and end rotations are dominated by the 'static' deformation response associated with the passing of the moving load. Subsequent dynamic oscillations are typically an order of magnitude smaller than the static maximum.
- It appears that the dynamic response of the cable, during the train passage and in the subsequent free vibration phase, is driven by the vibration of the bridge deck.

Deck-Stay Interaction Due to Wind

Measurements of both deck and stay movements were taken at the Fred Hartman Bridge during the passage of a storm. For this specific record, Figures 3 and 4 show the time-histories (first 5 minutes) and power spectral densities (PSDs) of vertical deck acceleration at midspan and of the adjacent stay cable AS24, respectively. Cable AS24 has a length of 650-ft (198-m) and a natural frequency of approximately 0.59Hz. Figure 5 shows the wind speed at deck level.

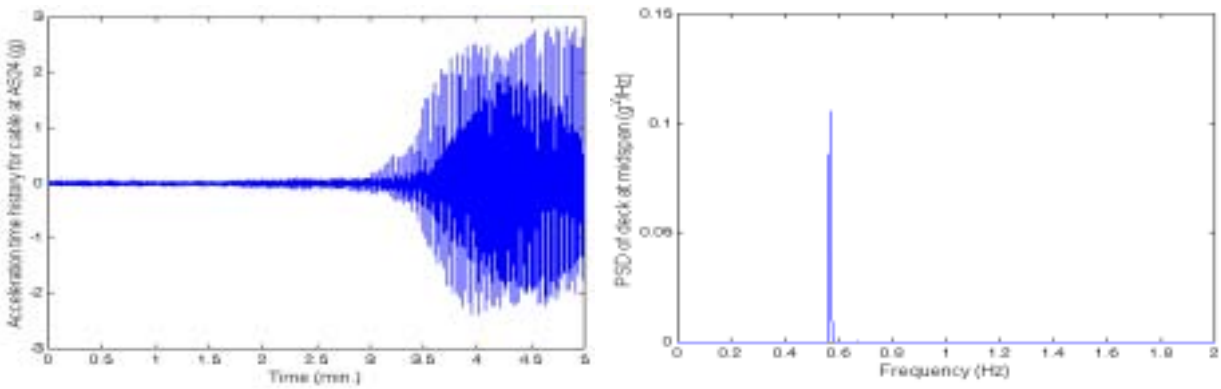

Figure 3: Time history and power spectral density (PSD) of the first 2-Hz for deck at mid-span (vertical direction).

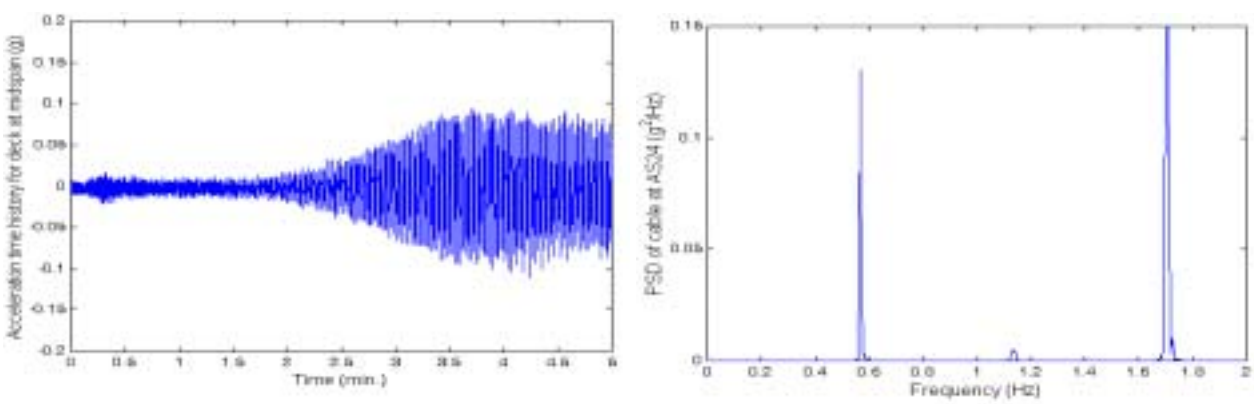

Figure 4: Time history and power spectral density (PSD) of the first 2-Hz for cable at AS24 (in-plane direction) deck level wind speed.

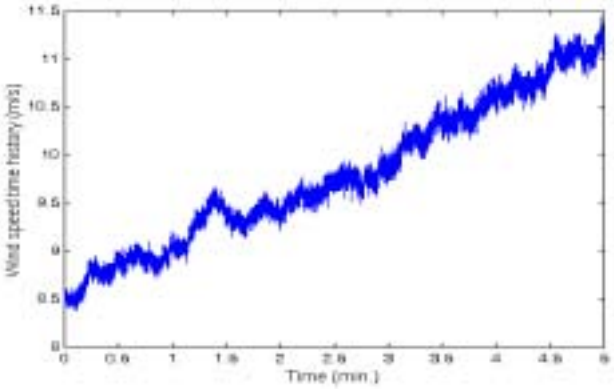

Figure 5: Deck level wind speed.

Figures 3 and 4 show a dominant frequency of vibration at approximately 0.58 Hz. It is important to note that this frequency corresponds quite closely to the third symmetric vertical mode of the superstructure, and is also close to the first mode of the stay cable AS24. This is an interesting and important observation since the first-mode vibrations of a cable at this level of acceleration are generally associated with large displacements. In fact, by integrating the acceleration time-history, the displacement amplitude (peak to peak) was estimated to be approximately 3-ft (1-m).

Furthermore, by observing the time-histories it can be seen that the significant vibrations are initially observed at the deck instead of the cable. This observation, as well as the similarity of modal frequencies, suggests that 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. These findings are not complete at the time of production of this report.

It is emphasized that this appears to be a rare event; very few occurrences of this nature have been identified.

3.4 STUDY OF MITIGATION METHODS

Development of recommended design approaches was based on previous and current research focusing on cable aerodynamics, dampers, and cross-ties. Theories on the behavior of linear and nonlinear dampers and cross-tie systems were developed and compared to field measurements on the Fred Hartman Bridge, Leonard P. Zakim Bunker Hill Bridge, Sunshine Skyway Bridge, and Veterans Memorial Bridge. Basic findings are discussed below, and more detailed discussions are found in Appendix C2 and in the technical papers listed in Appendix C1.

Linear and non-Linear Dampers

To suppress the problematic vibrations of stay cables, dampers are often added to the stays near the anchorages (due to practical limitations of installation). Although the mechanisms that induce the observed vibrations may still not be completely understood, dampers have had relatively widespread use, and their effectiveness has been demonstrated. However, criteria for damper design are not well established. Current recommendations for required damping levels to suppress rain/wind vibrations were developed using relatively simplified wind-tunnel models (PTI, 2001), and it is not clear whether these guidelines are adequate or appropriate for vibration suppression in the field. In addition, it is important to note that vibrations can occur in more than one mode of the cable, and little has been done to address the question of required damping levels for each mode. The anticipated widespread application of dampers for cable vibration suppression justifies further research aimed at better understanding the resulting dynamic system and refinement of design guidelines. An example of a damper provided to a cable anchorage is shown in Figure 6.

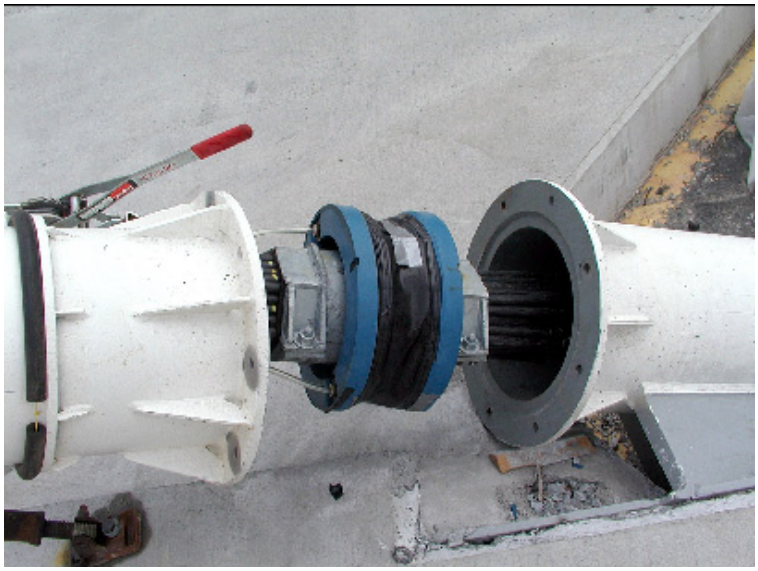

Figure 6: Damper at cable anchorage.

Linear dampers

Free vibrations of a taut cable with an attached linear viscous damper were investigated in detail. In designing a damper for cable vibration suppression, it is necessary to determine the levels of supplemental damping provided in the first several modes of vibration for different values of the damper coefficient and different damper locations. Previous investigations of linear dampers have focused on vibrations in the first few modes for damper locations near the end of the cable. However, damper performance in the higher modes is of particular interest, as full-scale measurements indicate that vibrations of moderate amplitude can occur over a wide range of cable modes. This study investigates the dynamics of a taut cable-damper system in higher modes and without restriction on the damper location (see Figure 7).

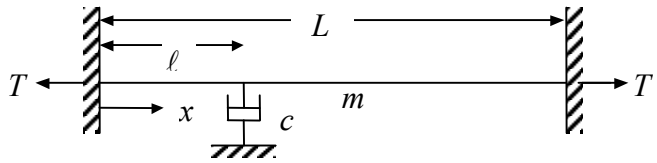

Figure 7: Taut cable with linear damper.

An analytical formulation of the complex eigenvalue problem for free vibration 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/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. For damper locations, such as near the end of the stay, resulting in small frequency shifts, the following relationship can be derived:

$$\frac{\zeta_i}{(\ell/L)} \cong \frac{\pi^2\kappa}{(\pi^2\kappa)^2+1} \quad ; \quad \kappa \equiv \frac{c}{mL\omega_{o1}} i \frac{\ell}{L} \qquad \text{(11a, b)}$$

i	=	mode of vibration
c	=	damping coefficient
m	=	mass per unit length
ω_{o1}	=	fundamental circular frequency
ℓ/L	=	normalized damper location

In Figure 8, the normalized damping ratio $\zeta/(\ell/L)$ has been plotted against the nondimensional damping parameter, κ, for the first five modes for a damper location of ℓ/L=0.02, and it is evident that the five curves collapse very nearly onto a single curve in good agreement with the theoretical approximation.

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. This is a potential limitation for linear dampers, 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.

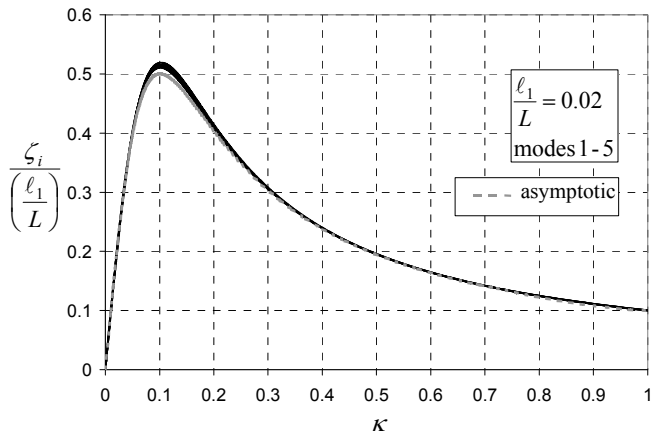

Figure 8: Normalized Damping Ratio versus Normalized Damper Coefficient: Linear Damper.

Non-linear Dampers

The dynamic behavior of a taut cable with a passive, nonlinear, power-law damper attached at an intermediate point was investigated. Recent investigations indicated that a nonlinear damper may potentially overcome the limitation in performance of a linear damper whereby optimal damping performance can only be achieved in one mode of vibration.

The exact formulation of the complex eigenvalue problem for a taut cable with a linear damper was extended to develop a single-mode approximation for the amplitude-dependent effective damping ratios for the power-law damper. An asymptotic approximate solution revealed a nondimensional grouping of parameters κ, that was used to extend the universal estimation curve for the linear damper to the case of a nonlinear damper. For a damping exponent of β=1 (linear damper), the expression for κ is the same as the previous equation derived for a linear damper. The shape of the curve is slightly different for each value of damping exponent, β, but for a given damping exponent the curve is nearly invariant with damper location and mode number over the same range of parameters as the universal estimation curve for the linear case (Figure 9).

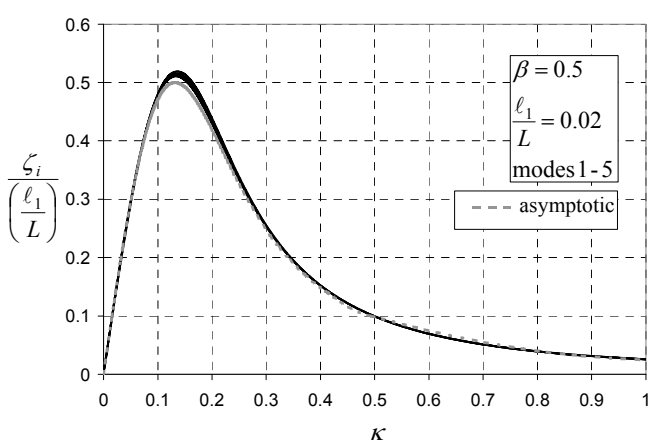

Figure 9: Normalized Damping Ratio versus Normalized Damper Coefficient: β=0.5

Because the nondimensional damping parameter κ depends on both the amplitude and mode number of oscillation, the optimal damping performance will be achieved, in general, at different amplitudes of vibration in each mode. An "optimal" value for the damper coefficient can be determined by specifying a design amplitude of oscillation in a given mode at which the optimal performance is desired. Therefore a non-linear damper has the potential to allow optimal damping performance over a wider range of modes than for a linear damper.

In the special case of β=0.5 (a square-root damper), it was observed that the damping performance is independent of mode number and depends only on the amplitude of vibration. In designing a square-root damper it is sufficient to specify only the amplitude, A_{opt}, and the optimal damping performance is achieved at the same amplitude in each mode. These features suggest that nonlinear dampers may offer some advantages over linear dampers for cable vibration suppression, while retaining the advantages of economy and reliability offered by a passive mitigation strategy.

Field Performance of Dampers

This investigation seeks to evaluate the effectiveness of passive linear dampers installed on two stays on a cable-stayed bridge by comparing response statistics before and after the damper installation and by investigating in detail the damper performance in a few selected records corresponding to different types of excitation.

Viscous dampers (dash-pot type) were installed on two stays (A16 and A23) on the main-span of the Fred Hartman Bridge (Figure 10), a twin-deck cable-stayed bridge over the Houston Ship channel, with central span of 1250 ft (380m) and side spans of 482 ft (147m). The deck is composed of precast concrete slabs on steel girders with four lanes of traffic, carried by a total of 192 cables in four inclined planes, spaced at 50 ft (15m) intervals.

Figure 10: Fred Hartman Bridge.

Properties of the two stays and the dampers that have been installed on these stays are given in Table 4. Commercially available dampers were selected for the application.

Table 4: Stay and damper properties.

	Stay Properties			Damper Properties		
	Length	Outside Diameter	Mass	Natural Frequency	Location	Damping Constant
Stay name	L ft (m)	D in (mm)	m lbf/ft (kg/m)	f_{o1} Hz	ℓ/L	c lbf-s/ft (kN-s/m)
AS16	285 (87.0)	5.5 (140)	32.2 (47.9)	1.24	.045	4800 (70.0)
AS23	599 (182)	6.3 (160)	50.0 (75.9)	0.64	.037	12000 (175.1)

Data have been collected for almost three years after the damper installation. The dampers were designed for optimal performance in the fundamental mode of vibration, which should provide adequate damping in the first several modes to suppress rain/wind vibration. Wind speed is reported at deck level. Wind direction is measured in degrees clockwise from the bridge axis, with zero degrees corresponding to wind approximately from the north, directly along the bridge axis. Acceleration data are reported from transducers installed on the stays usually about 20-ft (6-m) vertically above deck level.

Results before damper installation show patterns that are consistent with other field and wind tunnel observations (see Appendix C2 for additional details):

- A high density of points is seen near the abscissa, which generally corresponds to vortex-induced vibration in a variety of modes and low-level buffeting response of the stay to random excitations.
- Multiple points over a wide range of wind speeds are of high amplitude (RMS acceleration > 0.5g), indicating the characteristic signature of rain/wind oscillation.
- One-minute mean wind speeds at deck level reached 34 mph (15 m/s) before the dampers were installed, and almost 40 mph (18 m/s) in the period after installation. This latter value corresponds to a one-minute average wind speed of 60 mph (27 m/s) at the top of the tower, recorded during a thunderstorm.
- The dependence on wind direction is also clear, with A16 showing its peak responses between 90° and 160°, and A23 over a narrower range between 90° and 135°.

Results after installation of the dampers suggest the following:

- Amplitudes are significantly reduced across all recorded wind speeds [up to 40 mph (18 m/s) at deck level] with maximum RMS acceleration amplitudes of around 0.5g.
- The dependence on wind direction has been altered significantly, with the largest responses now nearer to a 90° angle of incidence.
- Based on the measured forces in the dampers, they are functioning and providing dissipative force to the stays as intended.

In conclusion, the benefits of adding dampers are evident. Dampers can potentially be attached unobtrusively near the stay anchorage at the deck or tower, and thus detract minimally from the aesthetics of the structure.

Cross-Tie Systems

One possible method to counteract undesired oscillations is to increase the in-plane stiffness of stays by connecting them together with a set of transverse secondary cables, defined as cross-ties. From a dynamic perspective, the properties of the single cables are modified by the presence of the lateral constraints that influence their oscillation characteristics. Similarly, a connection of simple suspended elements is transformed into a more complex cable network. Figure 11 shows an example of cable cross-tie systems applied to a bridge.

Figure 11: Cable cross-tie system.

It is evident that detailed studies of cross-ties and interconnected cable systems are not well reported and most of the recommendations that are currently followed seem to be linked to practice or previous experience. A fundamental study to better understand the behavior was therefore necessary.

This has shown that cross-ties potentially work by increasing the generalized mass and therefore Scruton number in the lower modes, and by raising frequencies and localizing the vibration in higher modes. They are also likely to increase the effective damping of a cable through the friction at cross-tie connections and also by providing a mechanism to transfer energy from one cable to another. However, these are difficult to quantify reliably using analytical methods.

The Dames Point Bridge (Figure 12) in Jacksonville, Florida, is an example of the effective use of cross-ties on an older bridge designed prior to the discovery of rain/wind vibrations. Although some wind-induced oscillations were observed during construction prior to grouting and cross-tie installation, no problematic cable vibrations have been reported on this bridge since it was completed in 1989.

Figure 12: Dames Point Bridge.

There is also the potential to combine cable cross-ties and dampers into a single device (Kumarasena, 1999). This could provide an efficient system where a damper at one location provides damping to a system of cables through the cross-ties. This method could be highly effective, as the damping would be provided at a point significantly into the cable length. The issues of aesthetics and serviceability need to be further addressed.

Analysis

An alternative analytical method was developed to examine cable cross-tie networks, and was used to study the Fred Hartman Bridge cable system.

Limited study of in-plane vibrations of complex cable networks has usually been performed by means of finite element methods due to the number of cable elements that are involved and the variability of the global characteristics of the system. An analytical method and efficient numerical procedure was developed that models the behavior of a set of interconnected cables. The analytical method is an alternative procedure for the derivation of the equation of motion (free-vibration problem) of a network, based on the taut-cable theory. This approach offers a number of advantages over the finite element method and provides a tool that is useful in the design and optimization of such systems for practical application.

The initial problem formulation is depicted in Figure 13. The example shows a simplified network, defined by a set of two taut cables, connected by means of a vertical rigid rod (to be relaxed later). See Appendix C2 for the formulation of the analytical method for this system.

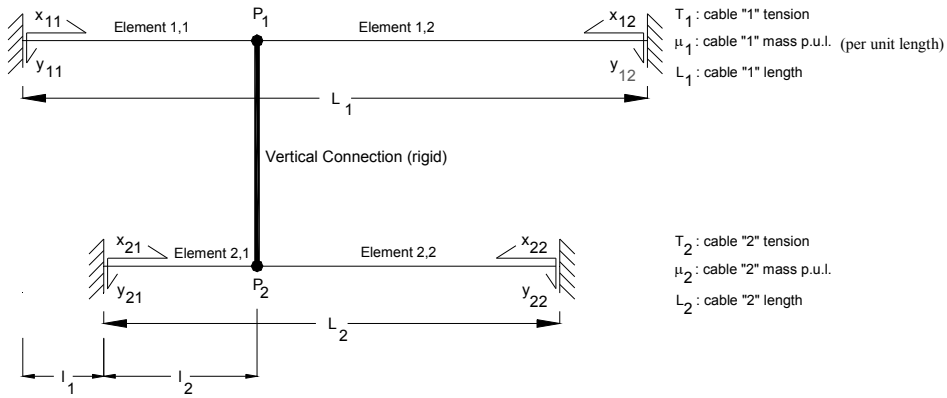

Figure 13: General problem formulation.

The free-vibration analysis method was applied to the study of a cable network that was modeled after the Fred Hartman Bridge. The investigated system corresponds to the south-tower central-span portion, a set of twelve stays with a three-dimensional arrangement (Figure 14). Stay "24S" is assumed as a reference element; all other stay quantities are normalized with respect to this element.

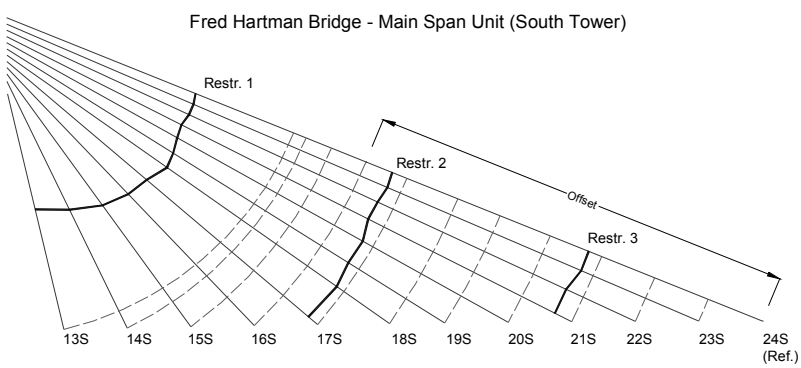

Figure 14: General problem formulation (original configuration).

The transverse connectors, an "eight-loop" steel wire rope system, are located in accordance with the existing system. The three-dimensional cable network was reduced to an equivalent two-dimensional problem. The definition of the modal frequencies and mode shapes of the network (free-vibration analysis) was performed through the study of the roots of the determinant of the system matrix as a function of the reduced frequency of the structure.

Figure 15 depicts the eigenfunctions for Modes 1 and 5 of the model, normalized so that the resulting modal mass of the network is unitary. A careful study of the solution patterns showed two categories of roots:

> Global modes, where the whole set of cables is involved in the oscillation. (ex., Mode 1)
>
> Local modes, where the maximum amplitudes are located in the intermediate segments of specific cables. The overall characteristics of these modal forms 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. (ex., Mode 5)

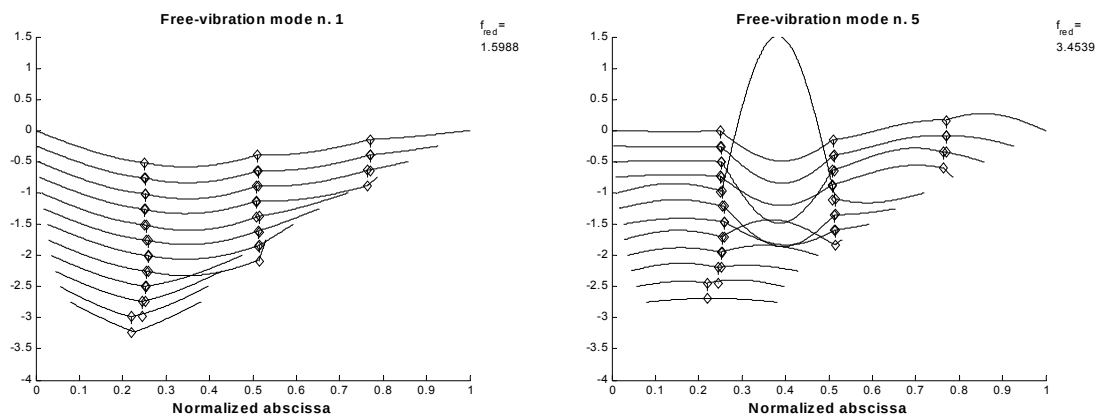

Figure 15: Eigenfunctions of the network equivalent to Fred Hartman Bridge; modes 1 and 5.

In Figure 16, the natural frequencies (Hz) are plotted as a function of the mode number and compared to the individual cable behavior. Solutions are shown for three comparative cases (NET_3C, original configuration; NET_3RC perfectly rigid transverse links; NET_3CG modified non-rigid configuration with ground restrainers). For practical purposes, the connectors in the original configuration can be considered as effectively rigid, since the two graphics (NET_3C, NET_3RC) essentially overlap.

From Figure 16, it can be seen that the sequence of fundamental global modes is followed by a high-density solution pattern, corresponding to the localized modes. An upper and lower limit frequency can be detected, in this case 1.9 and 2.7 Hz, respectively. Both limits are connected by the frequency of anti-symmetric second modes in the individual stays. The upper value is directly related to the frequency of shorter cables, while the lower value is influenced by a combination of the stays in the central part of the structure (presence of pseudo-symmetric components). The high density of frequencies suggests a potential sensitivity to forced oscillations, exciting the network within this range, in which the dynamics can be influenced by a combination of these modal forms. Beyond this upper limit the situation reverts to a set of higher network modes and thereafter, a second plateau appears in the frequency range coincident with high-order antisymmetric individual segment modes. This pattern of consecutive “steps” defines a typical pattern for the behavior.

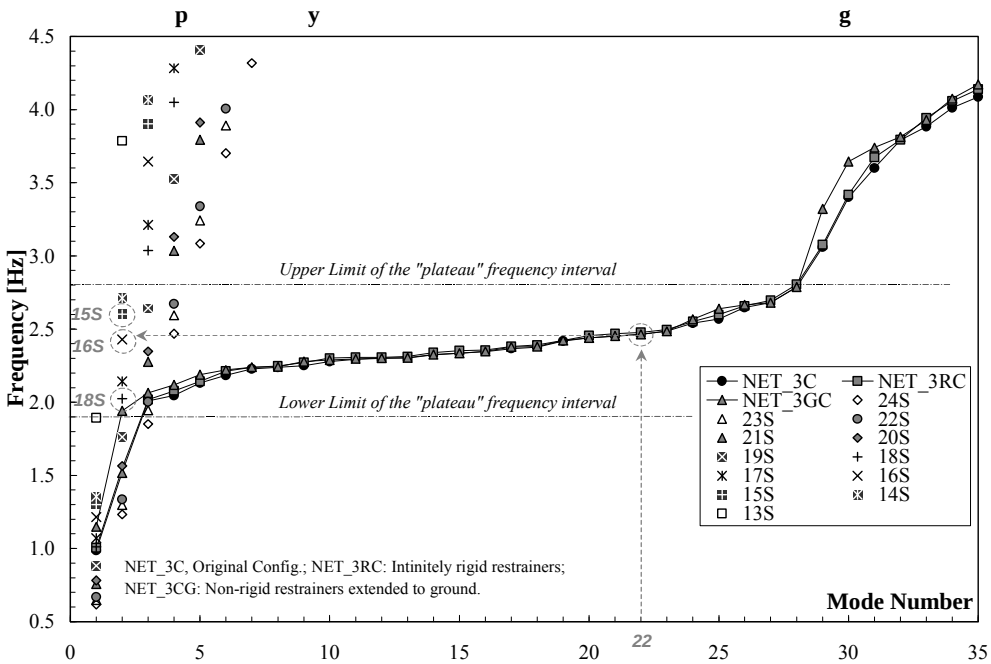

Figure 16: Comparative analysis of network vibration characteristics and individual-cable behavior; Fred Hartman Bridge.

Field Performance

Field measurements of the Fred Hartman Bridge were compared to analysis results to validate the analysis approach for cross-tied systems. A long-term ambient vibration survey was conducted to monitor stay-cable vibration and to better understand the overall performance of the structure and its modal characteristics. More than ten thousand trigger files were recorded during a period of three years and continued thereafter. An algorithm was designed for the automatic processing of the records.

The methodology was applied to the study of the side-span unit of the south tower of the bridge (see Figure 17). The cable network is configured by means of three transverse restrainers. The data set was extracted from the records of four in-plane and out-of-plane accelerometers, placed along stays AS1, AS3, AS5 and AS9 at a height of approximately 22 ft (7m) from the deck level. The correspondence between the predicted modal characteristics and the real behavior was carried out by simultaneous spectral analysis of the in-plane acceleration record database of the four locations. The analysis was founded on the simultaneous identification of the same dominant in-plane frequency on all the four investigated cables.

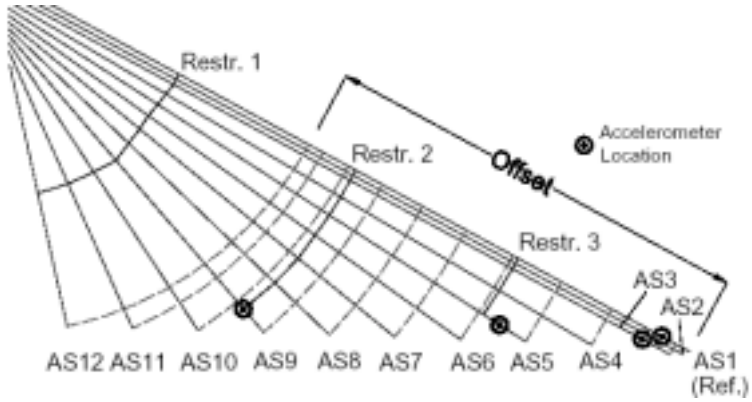

Figure 17: Fred Hartman Bridge, field performance testing arrangement.

The investigation considered the presence of cable network behavior along with individual cable and, eventually, global (deck) structure modes. The fundamental frequencies of each stay were identified through field measurements, and were adopted as input data in the numerical procedure to allow for a consistent comparison of the results with the real situation. The results of the simulation are summarized in Table 5, in which the frequencies of the modes between 0 and 4 Hz are indicated. The subdivision into global and local network modes is indicated along with information about the general characteristics of the modal shape.

The computed frequency of NM1 (0.926 Hz) is close to the 10th bending mode of the bridge (0.924 Hz), which suggests a potential susceptibility to interaction of the deck-stay system in this range. Low-density values of frequency can be seen up to 2.4 Hz (NM5) and beyond 3 Hz (NM30). The plateau behavior was detected between 2.4 and 3.0 Hz, associated the local behavior and a high density of solutions (NM5-NM29).

Table 5: Cable network modes (0-4 Hz) predicted by the model.

Mode N.	NM1	NM2	NM3	NM4	NM5	NM6	NM7	NM8	NM9	NM10	NM11	NM12	NM13	NM14	NM15	NM16	NM17	NM18
Frequency [Hz]	0.926	1.458	2.043	2.332	2.368	2.406	2.430	2.448	2.467	2.472	2.500	2.556	2.582	2.600	2.631	2.644	2.662	2.677
Mode type	G-S	G-AS	G-S	L	L	L	L	L	L	L	L	L	L	L	L	L	L	L

Mode N.	NM19	NM20	NM21	NM22	NM23	NM24	NM25	NM26	NM27	NM28	NM29	NM30	NM31	NM32	NM33	NM34	NM35	NM36
Frequency [Hz]	2.689	2.697	2.720	2.747	2.765	2.797	2.819	2.862	2.956	2.994	3.018	3.217	3.347	3.417	3.456	3.689	3.948	4.077
Mode type	L	L	L	L	L	L	L	L	L	L	L	G-S	G-S	G-S	L	G-AS	L	L

G = Global Network mode; L= Local Network mode; S = symmetric; AS = antisymmetic

The results of the field data analysis showed consistent similarities with the predicted values. They also indicated the potential presence of some of the "new" modal forms in frequency ranges in which modal characteristics from other sources (individual stays, global structure, etc.) were often excluded. NM1 was clearly identified in one occasion only for a continuous time interval of about fifteen minutes, corresponding to an extremely rare occurrence in terms of ambient-induced vibration. This event was mainly driven by the high-amplitude motion of the deck due to vortex shedding and related to a strong wind with direction almost perpendicular to the bridge axis.

Considerations for Cross-Tie Systems

This analysis shows that careful consideration must be given for the design of cross-ties. The general frequency increment in the fundamental modes that is usually attained by the introduction of transverse connectors must be balanced with the potential undesirable behavior of the local modes. A set of considerations for the improvement of the response of a cable network is proposed:

1. Keep the location of the first plateau as high as possible, distant from the fundamental modes, in order that the antisymmetric individual-cable modes (2nd mode) that mostly contribute to the network modal shape are those of the shorter cables. 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.

2. Relative stiffness of the transverse restrainers does not seem to play a significant role in the definition of the modal characteristics. Rather, the frequency ratio among different stays (related to the reference cable frequency) seems to be responsible for the network mode sequence. A good compromise between small frequency ratios (shorter stays) and high values (longer stays) is preferred for the optimization of the system. Unfortunately this aspect is not connected to the cross-tied configuration but to the existing setup of the bridge.

3. A cable network cannot be designed to withstand all possible excitations; the best thing that can be done is to select some sources of vibrations and "tune" the network not to respond to them, since the behavior becomes complex due to the presence of the plateau. Localized modes are practically unmanageable since they mainly concern internal elements of the system, although in these cases the vibration is confined to a selected portion of the structure and the potential implications for long-term structural sensitivity (fatigue damage) are less relevant.

4. A network combined with additional mechanical dampers, connected to the deck at specific locations, might be seen as an efficient way of vibration reduction. The presence of pseudo-symmetric behavior should be carefully assessed: it generates "non-dampable" modes (half of the structure is at rest). This behavior is enhanced by the presence of ground connectors. It might be possible to use a ground-restrainer in the proximity of the shortest cable, close to the tower in combination with dampers on the segments on the right side of the longest cables closest to the deck. In fact the first pseudo-symmetric modes, in this model, are those related to the unconstrained regions of the network and the presence of a damper in this position might become efficient. This opportunity must be balanced with the fact that, when the addition of these devices is necessary, the high inclination of the short cables might make them difficult to install.

5. It should be noted that cross-ties may be less effective at controlling out-of-plane oscillations. This further suggests the importance of using dampers in coordination with cross-tie systems.

6. Finally the optimal design of a network should be based not only on structural performance considerations but also on cost provisions (installation, maintenance). In this way the preferred solution should be suggested perhaps through a more extended optimization technique.

Cable Surface Treatment

The effectiveness of different surface modifications are determined from wind tunnel tests. No accepted methodology exists for the design of these elements. All major cable suppliers provide cable pipes that include surface modifications to mitigate rain/wind vibrations. Several types of cable surface treatments are shown in Figure 18. Extensive research has been done in the past and these surface treatments have been shown to be effective for mitigation of rain/wind vibration.

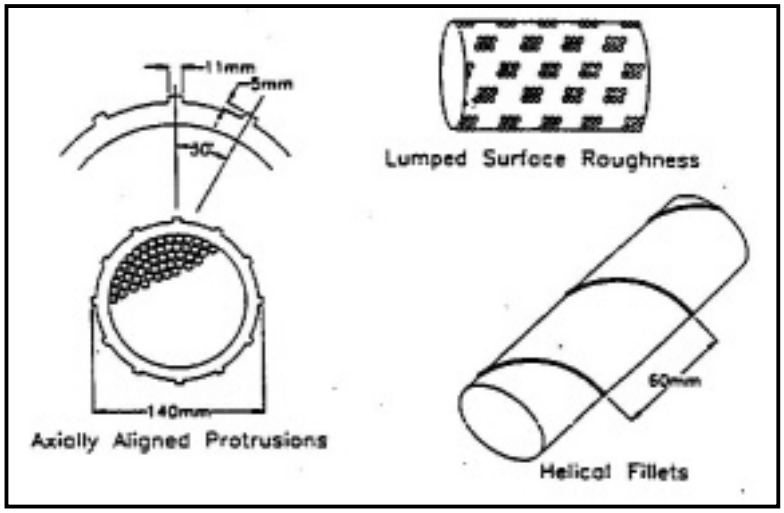

Figure 18: Types of cable surface treatments.

The double helix spiral bead formations are the most common on new bridges, such as 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 Cape Girardeau Bridge (MO). As a manufacturer proprietary item, test data demonstrating their effectiveness is generally available from the cable suppliers (Figure 19).

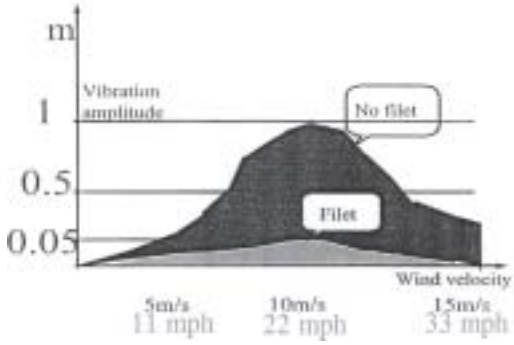

Figure 19: Example of test data for spiral bead cable surface treatment.

3.5 FIELD MEASUREMENTS OF STAY CABLE DAMPING

Leonard P. Zakim Bunker Hill Bridge (over Charles River in Boston, MA)

FHWA performed measurements during construction of the Leonard P. Zakim Bunker Hill Bridge (Figure 20), a cable-stayed bridge with inverted-Y shaped towers and a main span of 745-ft (227-m). The bridge is 183-ft (56-m) wide with 10 lanes (2 cantilevered lanes). The cables are ungrouted and arranged in a single plane for the back spans and two inclined planes for the main span. External visco-elastic dampers at the roadway level, cable cross-ties, and a double helical fillet cable surface treatment were applied to mitigate cable vibrations. Measurements were taken before and after installation of dampers and cross-ties.

Figure 20: Leonard P. Zakim Bunker Hill Bridge.

Preliminary results available as of this writing (private communication – reported data has not been independently verified by the study team) demonstrate the effectiveness of these mitigation methods. Table 6 shows the fundamental frequencies and damping ratios for the longest cables of the bridge before and after damper installation. In general, results show that the addition of dampers increases the frequency (by about 10%) and significantly increases the damping of the cables. Further evaluation of the measurements may be necessary, particularly since the data indicate that a few cables have smaller damping ratios after addition of dampers. In general, the damping ratio of ungrouted cables without external dampers appear to vary from 0.10% to 0.36% with an average of 0.20%. The external dampers raise the cable damping to an average of 0.37%.

Figures 21, 22, and 23 show the time histories of decay of manually excited cables before installation of dampers and cross-ties, after damper installation, and after cross-tie installation, respectively. The preliminary results demonstrate the effectiveness of not only the dampers, but also the cable cross-ties in raising the level of effective damping of the cable system. As the test method consisted of exciting one cable and recording the decay of its oscillations, some energy transferred from the excited cable to the adjoining cables through the cross-ties. Thus the reported damping with cross-ties may reflect values higher than the actual damping in the global system.

Table 6: Preliminary cable damping measurements - Leonard P. Zakim Bunker Hill Bridge.

Cable	Length (ft)	No Dampers Freq. (Hz)	No Dampers Damping Ratio	After Damper Installation Freq. (Hz)	After Damper Installation Damping Ratio
C1N	336	1.11	0.23%	1.21	0.46%
C2N	321	1.16	0.24%	1.27	0.37%
C3N	306	1.29	0.20%	1.42	0.44%
C32NE	397	0.96	0.20%	1.09	0.11%
C32NW	397	0.93	0.34%	1.14	0.34%
C33NE	417	0.89	0.14%	1.04	0.33%
C33NW	417	0.90	0.36%	1.04	0.44%
C34NE	438	0.86	0.10%	1.04	NA
C34NW	438	0.86	0.17%	0.97	0.38%
C34SW	422	0.89	0.14%	1.01	0.35%
C34SE	422	0.85	0.20%	1.05	0.14%
C33SW	402	0.91	0.28%	1.04	0.38%
C33SE	402	0.94	0.11%	1.10	0.34%
C32SW	383	1.01	0.12%	1.14	0.60%
C32SE	383	0.99	0.16%	1.14	0.40%
C3S	315	1.21	0.21%	1.33	0.36%
C2S	331	1.11	0.18%	1.22	0.58%
C1S	346	1.08	0.14%	1.16	0.27%
AVERAGE	382	1.00	0.20%	1.13	0.37%

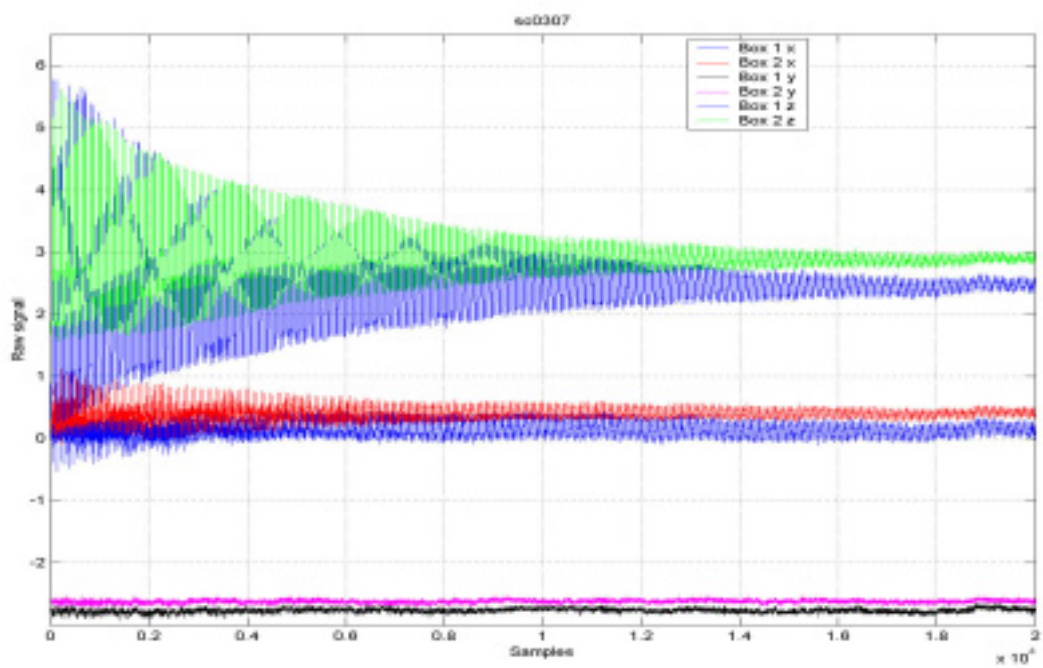

Figure 21: Sample decay: No damping and no cross-ties.

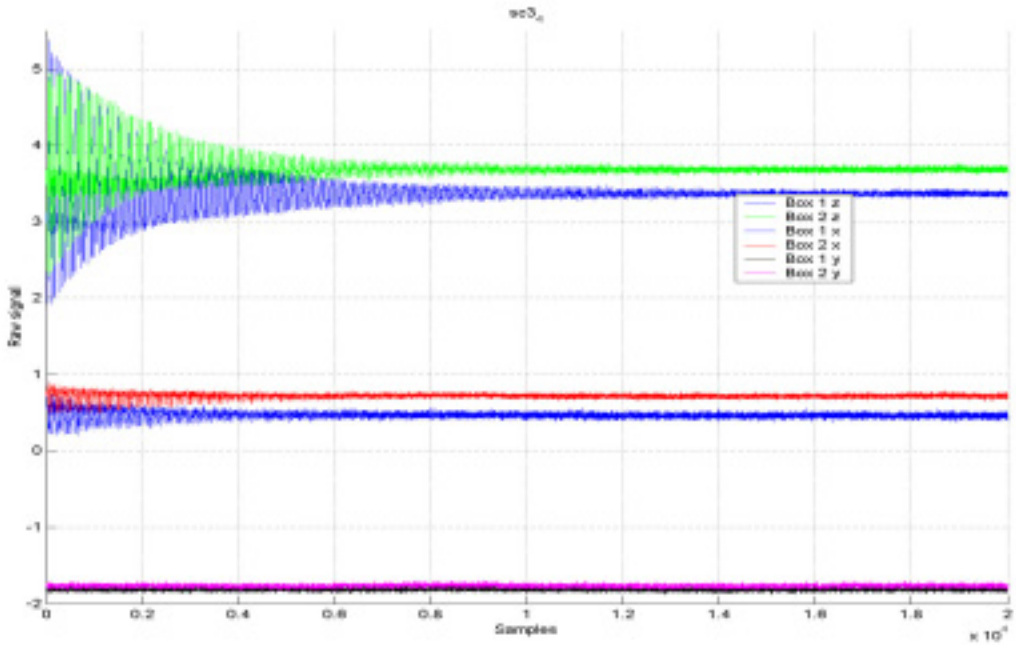

Figure 22: Sample decay: With damping and no cross-ties.

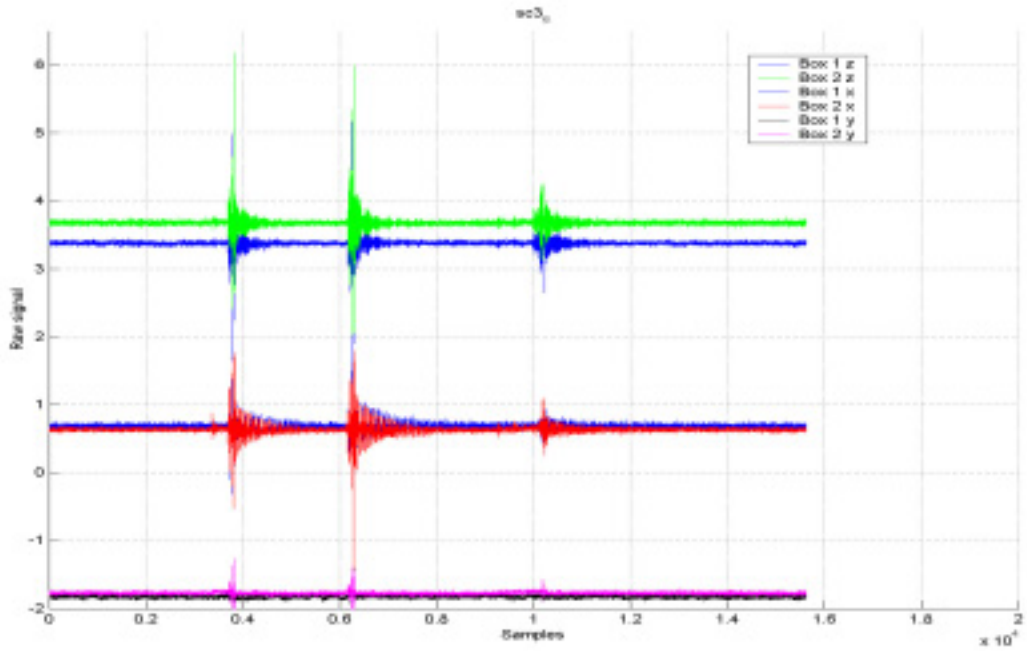

Figure 23: Sample decay: With damping and cross-ties.

Sunshine Skyway Bridge (St. Petersburg, FL)

The Sunshine Skyway Bridge (Figure 24) has been in service since 1982 and has not had reported problems with cable vibrations. The bridge has a main span of 1200-ft (366-m) between two single-mast towers and 84 cables in a single cable plane. The cables are provided with external viscous dampers (Figure 25), but no cross-ties or surface treatment.

Figure 24: Sunshine Skyway Bridge.

Figure 25: Stay and damper brace configuration.

Field measurements on the cables of the Sunshine Skyway Bridge were taken to study damping in multiple modes of vibration. This data provides a baseline for comparison of future measurements to evaluate cable damping performance of other cable-stayed bridges. Table 7 contains preliminary frequency and damping estimates from the ambient data (private communication).

What also makes this set of data interesting is that the cables on the Sunshine Skyway Bridge are all installed with two inclined struts that are each connected to three viscous dampers. This type of configuration enables the dampers to provide supplemental damping in two directions, unlike the case of the dampers on the Fred Hartman Bridge, which are oriented in the in-plane direction only. In fact, no excessive cable vibrations have been reported for the Sunshine Skyway Bridge, even though it is located in a region full of frequent wind and rain, while for the cables of the Fred Hartman Bridge there were still cases of some reported large-amplitude vibrations in the lateral direction that could not be suppressed by the damper.

Table 7: Preliminary cable damping measurements from the Sunshine Skyway Bridge.

Cable	Mode 1		Mode 2		Mode 3	
	Frequency (Hz.)	Damping (%)	Frequency (Hz.)	Damping (%)	Frequency (Hz.)	Damping (%)
N01	2.92	0.30%	5.87	0.10%	8.88	0.13%
N03	2.24	0.47%	4.52	0.24%	6.80	0.11%
N05	1.66	0.48%	3.32	0.19%	N/A	N/A
N07	1.41	0.36%	2.82	0.19%	4.24	0.13%
N09	1.19	0.48%	2.37	0.36%	3.57	0.22%
N10	1.08	1.05%	2.21	0.38%	3.33	0.30%
N11	1.02	0.69%	2.00	N/A	3.07	0.24%
N12	0.97	1.12%	1.92	0.48%	2.89	0.30%
N13	0.92	0.70%	N/A	N/A	N/A	N/A
N15	0.83	0.40%	1.64	0.53%	2.46	0.27%
N18	0.73	0.50%	1.44	0.35%	N/A	N/A
N20	0.62	N/A	1.25	0.37%	1.89	0.56%
N21	0.61	N/A	1.20	0.37%	N/A	N/A
N22	0.61	N/A	1.21	0.70%	1.82	0.36%
N23	0.65	N/A	1.29	N/A	1.93	0.24%
N24	0.68	N/A	1.34	N/A	N/A	N/A
N25	0.72	0.37%	1.43	0.29%	2.15	0.28%
N26	0.77	1.53%	1.52	0.55%	2.28	0.42%
N28	0.81	0.61%	1.60	0.43%	N/A	N/A
N30	0.89	0.65%	1.77	0.37%	2.67	0.28%
N32	1.00	0.79%	2.00	0.25%	3.01	0.20%
N34	1.12	N/A	2.24	N/A	3.40	0.16%
N36	1.34	0.69%	2.66	0.33%	3.99	0.21%
MIN	0.61	0.30%	1.2	0.10%	1.82	0.11%
MAX	2.92	1.53%	5.87	0.70%	8.88	0.56%
AVERAGE	1.08	0.66%	2.16	0.36%	3.43	0.26%

3.6 BRIDGE USER TOLERANCE LIMITS ON STAY CABLE VIBRATION

Field records from the large amplitude cable vibrations typical of rain/wind vibration episodes have noted that such vibrations can have the effect of alarming bridge users and general observers on the safety of the structure. In reality however, even for extreme cases, such vibrations have only produced limited damage to non-structural cable anchorage components. In closer examination, even these failures of the secondary elements can generally be traced to fatigue prone or other unsuitable details. There is little field evidence to indicate permanent structural damage to the cables or other primary load carrying members as a result of rain/wind vibrations on the affected bridges. A detailed examination of the level of stress generated in the primary elements due to these oscillations may generally indicate that the amplitude of dynamic stresses are not high enough to be a serious fatigue issue for cable strands or other bridge components either.

While the design displacements based on fatigue (or other design criteria) can be readily obtained through proper engineering calculations, there is little information in the literature to determine what level of cable oscillations can be permitted based on user tolerance to such displacements. In some cases, it can be expected that the displacement limits based on user acceptance may govern over those based on direct stress evaluation. The user acceptability threshold for a rural high level crossing where the observer is the motorist could be much less critical than that for an urban bridge where the public may be able to observe the bridge in close proximity. Detailed determinations of such limits are both complex and somewhat subjective. Thus, at least a preliminary identification of these human comfort thresholds was deemed important in developing design guidelines.

A design criteria set forth by these factors would be independent of those required by the structural effects involved, similar in nature to existing codes limiting deflection in bridges or drift in tall buildings. The human mechanism of perception of stay cable vibration is quite different from, say, the perception of building or floor movement by an occupant. The former is purely visual where as the latter is mostly physical feel. If a building occupant is near a window, some visual aspects of building movement with respect to fixed features of the landscape may become apparent. However, buildings rarely may exhibit such levels of movement and it is likely that other design criteria may prohibit a building design this flexible. Building occupant comfort is typically ensured by limiting acceleration and jerk (rate of change of acceleration). For torsional movements of tall buildings however, there are some limits based on visual perception that may have some level of similarity to the case under study. The parameters may be:

- User Proximity (Rural setting vs. urban)
- Cable diameter
- Cable displacement
- Mode shape
- Cable length
- Velocity

Focusing on typical cable sizes and typical vibration frequencies applicable to practical situations can further condense these parameters.

The study of user tolerance limits described in Appendix D3 was an attempt to establish some preliminary criteria on user perception of stay cable vibrations. In this study, user perception was determined with vibration mode-shape, velocity, and vibration amplitude as variables.

The two most important factors affecting user comfort were found to be the amplitude of the vibration and the velocity. As the frequency range is somewhat limited, it would stand to reason that the comfort criteria could be based on the amplitude. The study indicated that a reasonable recommendation of a limit on vibration amplitude (single) would be 1 cable diameter. Ideally, further reducing this to 0.5 diameters or below has the effect of making the vibrations virtually unnoticeable.

This study is preliminary and based on a relatively small sample size. Therefore, further investigation should be performed to refine any design criteria based on user comfort.

4 DESIGN GUIDELINES

4.1 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 (S_c).

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 (S_c):

$m\zeta / \rho D^2 > 10$	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 (see Figure 8). 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 (β=0.5) 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 visco-elastic 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 (see Section 3.4) 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 (see Appendix D3) on sensitivity of bridge users to stay cable vibrations has indicated that the comfort criteria for cable displacement can be described using the following maximums (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.

4.2 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.

4.3 WORKED EXAMPLES

EXAMPLE 1

The following application shows how the information presented in this report can be used to assess the vulnerability of a given cable and provide mitigation measures.

Properties:

Assume the following has been established based on the site meteorological data:

Wind velocity for structural design : 90 mph (100 year return)
Wind velocity for stability design : 130 mph (10,000 year return)

Cable C1: Total length = 350 ft.
Exterior Diameter = 11 in.
Mass = 127 lb/ft
Tension = 1485 kips (under dead load)

As the live load cable tension under normal service is only a small fraction of the dead load, the cable vibration evaluations typically ignore the live load.

Under dead load, the modal frequencies of the cable are computed to be:

f_1 = 0.875 Hz
f_2 = 1.750 Hz
f_3 = 2.625 Hz

Rain/Wind:

Design for $S_c = m\zeta / \rho D^2 \geq 10$, as established in the design guidelines
Note that typical damping inherent in cables are in the range of 0.1 to 0.2%.

For $S_c = m\zeta / \rho D^2 \geq 10$, using $\rho = 0.0765$ lb/ft^3,
the minimum damping required for $S_c \geq 10$ is $\zeta \geq 0.005$ (0.5%).

Galloping:

(Note that the present investigation has shown that galloping is not a major issue for normally spaced cable stays. However this is included in this example to demonstrate its application.)

The critical wind velocity for onset of galloping was given in Equation 6: $U_{CRIT} = c\, f\, D\sqrt{S_c}$

The numerical value of the constant c is typically taken as:

for wake galloping, c = 25 for closely spaced cables (2 to 6D)
c = 80 for normally spaced cables (10 to 20D)

for inclined dry cable galloping, c = 35*

As galloping leads to divergent oscillations, it is considered a stability issue and the critical wind speed for onset of galloping is desired to be over 130 mph.

For $U_{CRIT} = c\, f\, D\sqrt{S_c} \geq 130$ mph (190.7 ft/sec),

$$f \geq \frac{U_{CRIT}}{cD\sqrt{S_c}} = \frac{65.8}{c}$$

Assuming $S_c \geq 10$, $f \geq$ 2.63 Hz for c = 25 (closely spaced cables)
0.82 Hz for c = 80 (normally spaced cables)
1.88 Hz for c = 35*

Design Options:

A) If the cable geometry falls within normal cable spacing and ignoring dry inclined galloping*:

Galloping: The requirement for mitigation of wake galloping is automatically met (f_1 = 0.875 Hz $> f$ = 0.82 Hz). Thus cable cross-ties are not required to raise the natural frequency of the cable.

Rain/Wind: For rain wind, two options exist – (i) damping and (ii) cross-ties

i) provide damping such that $\zeta \geq 0.005$ (0.5%, corresponding to $S_c = 10$)

Assuming $\zeta \geq 0.005$ is to be achieved in the 1st mode and the damper is to be located 11'-8" from the lower anchorage of the cable,

$$\frac{\zeta_i}{l/L} = \frac{0.005}{11.67\text{ft} / 350\text{ft}} = 0.15$$

*Note that the wind tunnel tests conducted in the current study indicate that this can be ignored for normally spaced cables with damping ratios exceeding about 0.3%.

From Figure 8 and Equation 11a, $\kappa = \frac{c}{mL\omega_{ol}} i \frac{l}{L} = 0.0125$

Therefore $c = 0.0125 \frac{mL\omega_{ol}}{i} \frac{L}{l} = 0.0125 \frac{(127\text{lb/ft})(350\text{ft})(2\pi * 0.875/\text{sec})}{i * 0.0333} * \frac{\text{slugs}}{32.2\text{lb}}$

$$c = \frac{2850}{i} \frac{\text{slugs}}{\text{sec}} = \frac{2850}{i} \frac{\text{lbf sec}}{\text{ft}}$$

Rain/wind vibrations generally involve the first 3 modes of cable vibration, mode 2 being the most predominant.

Provide $\zeta_1 = 0.005$ (1st mode damping ratio 0.5%)

This requires a damper with a damping constant

$c = \frac{2850}{i}$ lbf-sec/ft = 2850 lbf-sec/ft

Compute damping ratio in 2nd and 3rd modes:

2nd mode (i=2): $\kappa = \frac{c}{mL\omega_{ol}} 2 \frac{l}{L} = 0.0250$

From Figure 8, $\frac{\zeta_2}{l/L} = 0.23$, therefore $\zeta_2 = 0.0077$ (>0.005 OK)

3rd mode (i=3): $\kappa = \frac{c}{mL\omega_{ol}} 3 \frac{l}{L} = 0.0375$

From Figure 8, $\frac{\zeta_3}{l/L} = 0.36$, therefore $\zeta_3 = 0.0120$ (>0.005 OK)

Therefore, a damper with a damping coefficient of c = 2850 lbf-sec/ft at 11'8" from the lower end anchorage provides the following damping ratios for the first 3 modes.

Mode 1	$\zeta_1 = 0.0050$ (0.50%)	and	$S_c = 10.0$
Mode 2	$\zeta_2 = 0.0077$ (0.73%)	and	$S_c = 15.2$
Mode 3	$\zeta_3 = 0.0120$ (1.17%)	and	$S_c = 23.7$

These are considered sufficient to mitigate rain/wind vibrations based on the Scruton number criterion. Note that the above computation ignores the natural damping present in the cable. Hence the actual Scruton numbers, including the inherent damping, will be larger than those computed, making the design somewhat conservative.

ii) provide cable cross-ties

Note that by providing two cable cross-ties at the 1/3 points cable modes 1 and 2 are eliminated, raising the natural frequency of the cable net to about 2.63 Hz. The cable cross-ties also raise damping of the cable system considerably, as demonstrated by the preliminary vibration measurements taken from the Leonard P. Zakim Bunker Hill Bridge (Section 3.4).

B) If the cables are spaced closer together, using

$$U_{CRIT} = 25\, f\, D\sqrt{S_c} \geq 130 \text{ mph}, \quad f = 2.63 \text{ Hz}$$

It is noted that providing two cable cross-ties at the 1/3 points would raise the natural frequency to 2.63 Hz. This is out of range for rain/wind vibrations. It is generally believed that additional dampers can be neglected as the cable cross-ties are more than likely to raise the damping ratio beyond the 0.5% needed for mitigating rain/wind vibrations

Summary:

Thus the cable vibration mitigation following recommendations of the current study would consist of:

a) For normal cable arrangements : Either a damper with c = 2850 lbf-sec/ft mounted 11'8" from the end anchorage **or** cable cross-ties at 1/3 points

b)For closely spaced cable arrangements : Cable cross-ties at 1/3 points. The designer can consider providing additional damping depending on the details of the project, careful analysis of the specifics, and engineering judgement and expertise.

In all cases an effective surface treatment such as the double helix spiral beads provided by leading cable manufacturers is recommended for normal cable spacings. Limited tests show that the damper size may be reduced by as much as 50% with an effective surface treatment. Note that the elimination of dry inclined cable galloping as a design consideration for normal cable arrangements makes it possible to provide a mitigation design using only dampers, without the use of cross-ties.

EXAMPLE 2

Example 1 demonstrated the use of the design guide in sizing the dampers so that the minimum Scruton number criteria is met for the 1st and higher modes. Example 2 is based on data provided in Table 4, and is an illustration of providing maximum possible damping in mode 1.

Data from Table 4

	Stay Properties			Damper Properties		
	Length	Outside Diameter	Mass	Natural Frequency	Location	Damping Constant
Stay name	L ft (m)	D in (mm)	m lbf/ft (kg/m)	f_{o1} Hz	ℓ/L	c lbf-s/ft (kN-s/m)
AS16	285 (87.0)	5.5 (140)	32.2 (47.9)	1.24	.045	4800 (70.0)
AS23	599 (182)	6.3 (160)	50.0 (75.9)	0.64	.037	12000 (175.1)

Cable AS16:

Equation 11a: $\kappa = \frac{c}{mL\omega_{ol}} i \frac{l}{L} = \frac{(4800\text{lbf sec/ ft})(0.045)}{(32.2\text{lb/ ft})(285\text{ft})(2\pi * 1.24/\text{sec})} * (32.2 \frac{\text{lb}}{\text{slug}}) * i = 0.0973 * i$

Equation 11b: $\frac{\zeta_i}{l/L} = \frac{\pi^2 \kappa}{(\pi^2 \kappa)^2 + 1}$

Mode 1 $\kappa = 0.0973$, $\frac{\zeta_1}{l/L} = 0.500$, $\zeta_1 = 0.045*0.500 = 0.0225$ (2.3%), Sc = 45.1

Mode 2 $\kappa = 0.1946$, $\frac{\zeta_2}{l/L} = 0.410$, $\zeta_2 = 0.045*0.410 = 0.0184$ (1.8%), Sc = 36.9

Mode 3 $\kappa = 0.2919$, $\frac{\zeta_3}{l/L} = 0.310$, $\zeta_3 = 0.045*0.310 = 0.0140$ (1.4%), Sc = 28.1

Cable AS23:

Equation 11a: $\kappa = \frac{c}{mL\omega_{ol}} i \frac{l}{L} = \frac{(12000\text{lbf sec/ ft})(0.037)}{(50.0\text{lb/ ft})(599\text{ft})(2\pi * 0.64/\text{sec})} * (32.2 \frac{\text{lb}}{\text{slug}}) * i = 0.1187 * i$

Mode 1 $\kappa = 0.1187$, $\frac{\zeta_1}{l/L} = 0.494$, $\zeta_1 = 0.037*0.494 = 0.0183$ (1.8%), Sc = 43.4

Mode 2 $\kappa = 0.2374$, $\frac{\zeta_2}{l/L} = 0.361$, $\zeta_2 = 0.037*0.361 = 0.0134$ (1.3%), Sc = 31.8

Mode 3 $\kappa = 0.3561$, $\frac{\zeta_3}{l/L} = 0.263$, $\zeta_3 = 0.037*0.263 = 0.0097$ (0.97%), Sc = 23.0

Note that the optimum value of $\zeta/(l/L)$ for linear dampers near the end of the stay is approximately 0.50, as is evident from Figure 8. For cables AS16 and AS23, the damping provided in the 1st mode is very close to this optimum, with damping in higher modes falling below the optimal level. The damper coefficients for the dampers used here are very large compared to the value obtained in Example 1, and the Scruton Numbers for modes 1-3 are much greater than the minimum of 10 recommended for suppression of rain/wind vibrations.

5 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. Wind-tunnel testing of dry inclined cables.

Galloping of dry inclined cables was investigated using wind-tunnel testing in the current study and was found to be less critical than previously believed. However, further research would be valuable to confirm the findings and to study the effects additional parameters. Future wind-tunnel testing could include :

- More data points to validate the first 2 phases
- Study the effect of cable frequencies, which are potentially important for the aerodynamic behavior of the inclined dry cable
- Further study of the Reynolds number effect, resulting from the model surface condition and the orientation angle

2. Deck-induced vibration of stay cables.

Deck-induced vibration of stay cables has been observed in a few instances in the field-measured data, occurring in the fundamental mode of a stay with quite large amplitudes. Further investigation of this potential source of excitation would be beneficial, and may include comparison of analytical predictions and field measurements and analytical modeling of the influence of attached dampers. Effort needs to be made to identify more records in which interaction is evident between vibrations of the bridge deck and stays. As more and more newer bridge superstructure sections are aerodynamically refined to mitigate vortex-induced oscillations of the roadway as a user comfort criteria, the deck-stay interaction may become primarily an issue for older bridges.

3. Mechanics of rain/wind induced vibrations.

Rain/wind vibrations appear to be the most problematic of the measured vibrations, with their large amplitudes and relatively frequent occurrence, and are among the most significant considerations in the design of mitigation measures for stay cables. One of the primary components of future research could be to develop a more in-depth understanding of the underlying mechanics of rain/wind induced stay-cable vibration. This effort would have a focus somewhat different from that adopted in this study, where the objective was simply to develop a

sufficient understanding to be able to arrest the practical occurrence of objectionable levels of stay cable vibrations. It is believed that such an in-depth study could build upon the findings from this and other recent projects, and include continued detailed analysis of the wealth of full-scale data collected (data on U.S. bridges as well as large-scale data from Japan as available) and a critical review of existing proposed mechanisms.

This effort may include the following two components: identification of key observed characteristics and evaluation of proposed mechanisms. Further analysis of the measured vibration records both before and after the damper installation could provide additional insight into the nature of the rain/wind excitation mechanism and clarify which types of proposed mechanisms are more appropriate. A good start on this task has already been made, but there are more data that need to be carefully analyzed that were outside the present project scope. Some of the more interesting observations need further detailed investigation using the diverse data sets.

Many of the proposed mechanisms were postulated based on wind tunnel investigations. While useful, some caution must be taken in interpretation of these data as field conditions may contain three-dimensionality of the flows that the wind tunnel testing cannot replicate.

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

In designing effective and economical dampers for rain/wind vibration suppression, it would be of great assistance to have a model with the capability to predict, for an arbitrary stay cable, the following characteristics of rain/wind vibration:

- Preferred Mode: The full-scale measurements have indicated that rain/wind vibrations tend to occur in a preferred mode over a fairly wide range of wind speeds; for a given stay, which mode(s) will be preferred?
- Wind Speed and Direction: Over what range of wind speeds and wind directions will the problematic vibrations occur? (This question is actually not of significant concern in the design of dampers.)
- Damping Levels: How much damping is necessary to adequately suppress vibrations?
- Amplitudes, Forces, and Power: What will be the steady-state amplitudes as a function of damping ratio? What levels of force may be expected in the damper, and what will be the power dissipation demands?

It is likely that rain/wind vibrations can be modeled in a manner similar to vortex-induced vibrations: as a nonlinear oscillator characterized by a negative-damping type instability at small amplitudes, and a limit cycle at large amplitudes. Such a model could be developed using the following steps:

- Obtain reliable estimates of inherent mechanical damping in the undamped stays, as inherent damping is expected to be a critically important parameter in estimating oscillation amplitudes.
- Identify records from various stays corresponding to the onset of rain/wind vibration, and characterize the energy input as a function of amplitude.

- For each stay, from the records collected previously, identify an election of "worst-case" time histories corresponding to the maximum energy input under the ideal wind conditions for rain/wind vibration.
- Using parameters such as inherent mechanical damping; stay length, diameter, tension, and mass/length; wind speed and air density, obtain the best possible normalization of the "worst-case" energy-input versus amplitude curves so that the curves from various stays can collapse (almost) onto a single curve.
- Identify a nonlinear oscillator model that can capture the measured energy input versus amplitude curves, and use the field-measured data to estimate the model parameters.
- Investigate the dependence of the model parameters on wind speed and wind direction for the various stays, and identify appropriate normalizations of these parameters to allow comparison among stays.

There are strong indications that a nonlinear negative damping model may be a promising approach. This is based upon observation of what appears to be a negative-damping type of instability for small amplitudes, causing the amplitude to increase until a limit cycle is reached. Vibrations occur over a fairly narrow range of wind directions, and occur in one or two preferred modes over a fairly wide range of wind speeds. These observations and potential model can be used to carefully evaluate proposed mechanisms for consistency and reasonableness.

While ultimately the model resulting from the proposed work may take the form of an analytical formulation, in which equations of motion for cable vibration are expressed mathematically, it may also be an empirical model which seeks to obtain a good fit to the field measurements from different stays, it may simply take the form of general conclusions achieved from a statistical analysis of the measured vibrations, or it may represent a combination of the three approaches.

Such a model will enable a much more comprehensive treatment of the problem both in terms of when such vibrations are likely to occur, and what (quantitatively) will be the effect of various mitigation approaches: dampers, cross-ties, and aerodynamic treatments. The ability to do such analysis presently does not exist.

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

The most practical application of such a model would be to be able to predict the level of stay oscillations after the application of mitigation methods. This would also enable the combination of important observations from the modeling of the field-measured vibrations and from the analysis of the restrained system in order to predict performance after vibration mitigation efforts (specifically dampers and cross-ties). Special attention again could be devoted to a discussion of the mitigation of rain/wind induced vibration. Mitigation of other types of vibration, such as vortex-induced vibrations, particularly in the higher modes and deck-stay interaction could also be evaluated. More in-depth analysis of field measurements of damper performance can also be performed in order to compare measured and predicted oscillation amplitudes after mitigation.

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

As has been demonstrated in this report, dampers are not the only method available for mitigation of stay-cable vibration. Other potentially suitable solutions include the provision of additional damping by other means, such as redesigned anchorages, cross-ties, and aerodynamic treatments. It is considered important to assess their relative performance using the model. It is noted that we have data from stays with cross-ties from the Fred Hartman Bridge, and stays with aerodynamic treatments (rings) from the Veterans Memorial Bridge. In addition, efficient analytical/numerical tools for the prediction of the dynamic response of two-dimensional cable "networks" (representing cross-tied stay systems) have been developed using this project.

Using the results of the predictions (Item 5), the performance of these approaches can be compared and this information used to evaluate the suitability of the various methods. Important economic assessments must also be considered.

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

The inherent damping in a cable, generally accepted to be fairly low, is not readily quantifiable, and there are very few reported measurements. A better understanding of damping in cables (both with and without external dampers) could have some benefit in practical applications from extensive field measurements, so as to facilitate more effective and rational mitigation of stay cable vibrations. Substantial progress has been made towards this goal by analyzing the full-scale measurement data collected at the Fred Hartman Bridge and by preliminarily estimating modal frequencies and damping ratios for two cables on the bridge as described in this report. By comparing the amplitudes of vibrations before and after damper installation, dampers were found to be generally successful in mitigating the vibrations. In a few cases, when the vibrations were primarily in the lateral direction and the dampers were oriented in the in-plane direction, there is evidence that the dampers were not effective at suppressing them.

Frequency shifts caused by the dampers were also clearly identified and the discovered relationship between modal frequency and vibration amplitude seems to be in agreement with the actual configuration of the cable-damper system in the field. Compared to frequency estimation, results from damping estimation are more complicated: modal damping ratios estimated from different records are usually scattered and the relationship between these estimated damping ratios and the characteristics of the vibrations is still not very clear. Nonetheless, the results do suggest that the level of inherent damping in the cables is indeed very low and that the rain/wind mechanism introduces some kind of negative damping into the system, consequently making the vibration amplitude large.

Relationships between the modal damping ratios and such quantities as modal displacement and modal damper force need to be investigated. Testing of an actual damper is also deemed as beneficial. The damper should be driven by either force or displacement with different periods. Numerical simulation can also be used for this purpose and force-displacement curves thus obtained can be compared to field measurement data. It will be helpful to separate the part of the vibration when the damper is evidently engaged from the rest of the vibration when it is not

engaged e.g., by using wavelet transforms. Energy methods can also be applied to the part of the vibration when the damper is engaged to estimate the effective damping during that period of time.

Research so far has been concentrated on full-scale measurement data from two cables of the Fred Hartman Bridge. Data from other instrumented cables of this bridge and some cables of the Veterans Memorials Bridge, which were instrumented during the same period of time, are also available for analysis. In addition, forced vibration tests and short-term ambient tests have been conducted on some cables of the Sunshine Skyway Bridge in Tampa, Florida. Data collected from these tests are of special interest because cable damping can be estimated from the forced vibration tests using the logarithmic decrement method and compared with the values estimated from the ambient tests. No excessive cable vibration has been reported for the Sunshine Skyway Bridge so far, even though it is located in a region full of frequent wind and rain. More detailed analysis of the full-scale data from this bridge will help understand the reasons for these observations.

8. Improved understanding of cross-tie solutions.

As pointed out in the report, cross-ties combined with dampers seem to offer the possibility of enhanced performance over their component counterparts. Application of such a combined system has been previously discussed (Kumarasena, 1999). It is considered important to extend the modeling developed herein to better study and understand these characteristics, and to provide a tool for designers. The ability to optimize cross-tie configurations also seems to be an important area for future research.

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

While the design recommendations proposed are believed to be sufficient as minimum criteria for practical mitigation of unacceptable levels of stay vibrations, these efforts could help by addressing quantitatively the following issues:

- How much supplemental damping should be provided in each mode for vibration suppression?
- Which type of nonlinear damper (including the case of linear) is most appropriate for cable vibration mitigation?
- For which mode of vibration should the damper performance be optimized?
- For what force levels and energy dissipation capacity should the damper be designed?

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.