Full scale dynamic testing of the Annacis bridge

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Essais dynamiques sur le pont Annacis

Schwingungsmessungen an der Annacis Schrägseilbrücke

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SUMMARY

This paper reports the results of full scale vibration tests on the cables and deck structure of Annacis cable-stayed bridge. Natural frequencies and damping values were measured for a variety of cable lengths. A parametric study was carried out to determine the nature of the damping. Large cable motions caused by wind induced motion of the supports were investigated. The complete bridge deck was excited in several lower modes and damping values were obtained.

RÉSUMÉ

L'article présente les résultats d'essais dynamiques en vraie grandeur sur les câbles et le tablier du pont à haubans d'Annacis. Les fréquences propres et les valeurs d'amortissement ont été mesurées pour différentes longueurs de câble. Une étude a été faite dans le but de déterminer la nature des amortissements. Les grands déplacements des câbles, causés par le mouvement des supports sous l'effet du vent ont été étudiés. Le tablier complet du pont a été soumis à diverses vibrations afin d'obtenir les valeurs d'amortissement.

ZUSAMMENFASSUNG

Diese Veröffentlichung berichtet über die Ergebnisse von Schwingungsuntersuchungen an den Seilen und an der Fahrbahnplatte der Annacis Schrägseilbrücke. Eigenfrequenzen und Dämpfungswerte wurden für eine Reihe von unterschiedlichen Seilen gemessen. Eine Parameterstudie wurde ausgeführt, um die Art der Dämpfung zu bestimmen. Starke Seilschwingungen, die auf winderzeugten Auflagerbewegungen zurückzuführen sind, wurden untersucht. Die gesamte Fahrbahnplatte wurde in mehreren Eigenfrequenzen erregt, um die entsprechenden Dämpfungswerte zu messen.

1. INTRODUCTION

Throughout history bridges have been some of the most challenging structures for engineers. As the desire for longer span bridges increased, suspension and more recently cable-stayed bridges came to the forefront. The latter type of bridge consists of a light deck directly suspended by stay cables and is usually constructed using a balanced cantilever technique.

In the past, the light superstructure associated with cable-supported bridges created aerodynamic problems, culminating in the collapse of the Tacoma Narrows bridge in 1940. Since that time, a great deal of work has been done in the area of deck aerodynamics, using models and wind tunnels to examine the effects of eddy shedding and phenomena such as buffeting and galloping. In addition to deck motion there may also be cable oscillations caused by wind.

Some recently constructed cable-stayed bridges have had cable motion problems. At the Brotonne Bridge in France the cable ends were encased in steel pipes and cement grouted which resulted in a very low damping value. The cable motions became so large that two of the longest cables, which were oscillating out of phase, came into contact with one another. These long cables are almost parallel to one another and are 1957 mm apart at the centre. This problem was solved by installation of discrete dampers near the lower socket on all of the cables. Although effective, these dampers (which resemble large shock absorbers) are aesthetically very unappealing.

Cable stays are taut compared to suspension cables and little experimental research has been done on the dynamic response of inclined cables with low sag. For small oscillations, these cables are assumed to behave linearly with respect to both material and geometric properties. A simple closed form solution exists for an inextensible taut string with fixed ends. Closed form theoretical solutions and numerical solutions for inclined cables with large sag have also been proposed. However, the behaviour of the stay cable lies somewhere between these two extremes of taut string and large sag cables. Although some analytical and experimental (scale model) work has been done on cable stays, very few full scale tests have been carried out to determine damping values.

With the Annacis cable-stayed bridge under construction in the Vancouver area a unique opportunity existed to undertake full scale cable tests. The bridge spans the Fraser River between New Westminster and Delta, British Columbia, Canada and its main span is currently the longest of its type in the world. The Annacis bridge consists of cables in two planes in a modified fan arrangement supporting a 465 m main span, two 182.75 m side spans and two 50 m transition spans (Figures [1] and [15]). The 192 main cables and 8 tie-down cables consist of long lay galvanized bridge strand sheathed with black polyethylene. The strands are made of 7 mm diameter galvanized steel wire with an ultimate tensile strength of 1520 MPa. Every cable has a zinc-filled, cast-steel socket at both ends. The cables terminate at tie beams in the towers where provision is made for jacking and adjustment. Cable lengths range from 49.5 m to 237.5 m and outer diameters vary between 99 mm and 149 mm. In order to increase damping each cable is fitted with a split ring neoprene damper contained in the anchorage assembly at both ends (Figure [2]).

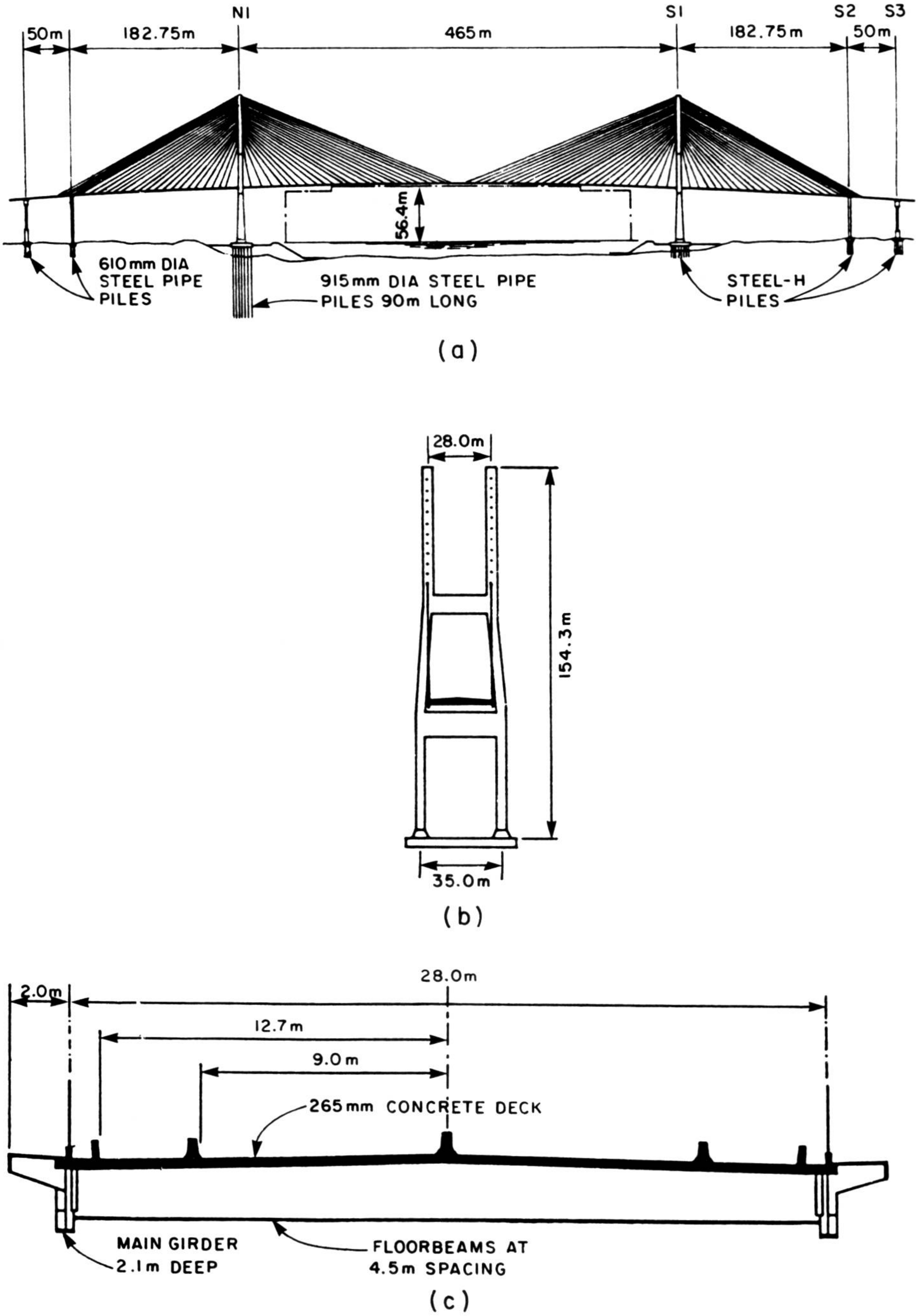

Fig. 1 Annacis Cable Stayed Bridge
(a) Elevation
(b) Tower Section
(c) Typical Cross Section (from ref. [12])

2. BACKGROUND

Previous literature contains closed-form and numerical solutions for both the static and dynamic analysis of cables with large sag. An investigation into the static and dynamic analysis of mooring lines was done by Chang and Pilkey [1]. The work involved a two-dimensional static analysis considering forces normal to the line only and a dynamic solution using incremental integration. Research into suspension cables provided several closed-form solutions. Irvine and Griffin [2] used a linear mathematical approach to solve for the response of a cable with its end points being excited sinusoidally. The solution assumes no damping and ignores second order effects and out-of-plane motion. Henghold, Russell and Morgan [3] studied the free vibrations of a cable in three dimensions. It was discovered that for a cable with large sag, crossovers occur between in-plane and sway modes. However, for low sag cables the in-plane and sway mode frequencies are the same and the cable behaves like a taut string. Wilson and Wheen [4] assumed an initial parabolic shape and derived design expressions for an inclined taut cable under self-weight, uniform dead loads and point loads.

Analytical solutions have been proposed by West, Geshwindner and Suhoski [5] who modelled the cable using a linkage of straight pin ended bars; and Henghold and Russell [6] who developed a three node cable element and used a nonlinear finite element approach to the problem.

A static analysis formulated by Hooley [7] contained a cable element defined by dimensionless parameters. Consider the catenary cable in Figure [3]. As the load on the structure changes (i.e. the deck load) the cable end thrust changes from H_0 to H. The elongation of the cable, Δ, is given as the relative displacement of points A and B in direction S. This Δ is the deflection vector

$$\Delta = \frac{S^2}{AEL}\left(H - \frac{\omega^2 L^5 AE}{24 S^3 H^2}\right) - \frac{S^2}{AEL}\left(H_0 - \frac{\omega^2 L^5 AE}{24 S^3 H_0^2}\right)$$

If we introduce two new parameters,

$$\alpha = \frac{S^2}{AEL} \qquad \beta^3 = \frac{\omega^2 L^5 AE}{24 S^3}$$

then,

$$\Delta = \alpha\left(H - \frac{\beta^3}{H^2}\right) - \alpha\left(H_0 - \frac{\beta^3}{H_0^2}\right) \qquad (1)$$

Equation (1) can be put into dimensionless form by substituting $H = \kappa\beta$ and $H_0 = \kappa_0\beta$, where κ and κ_0 are dimensionless variables which describe the thrust for different loadings.

$$\frac{\Delta}{\alpha\beta}\left(\kappa_0 - \frac{1}{\kappa_0^2}\right) = \kappa - \frac{1}{\kappa^2} \qquad (2)$$

The left side of Equation (2) is known for any load on the structure since:

- α, β are constant
- κ_0 is known from initial conditions
- Δ is known from output

These parameters were used to study cable behaviour under increasing load. An ultimate load analysis was carried out using the load case (Dead Load) + λ (Live Load) and a plot of $\Delta/\alpha\beta + (\kappa_0 + 1/\kappa_0^2)$ vs. $(\kappa - 1/\kappa^2)$ was made for the cable most affected by changes in live load (Figure[4]). Cables with κ greater than 1.5 are considered stiff. There is a linear relationship between deflection and load for a stiff cable, whereas for a slack cable this relationship becomes nonlinear.

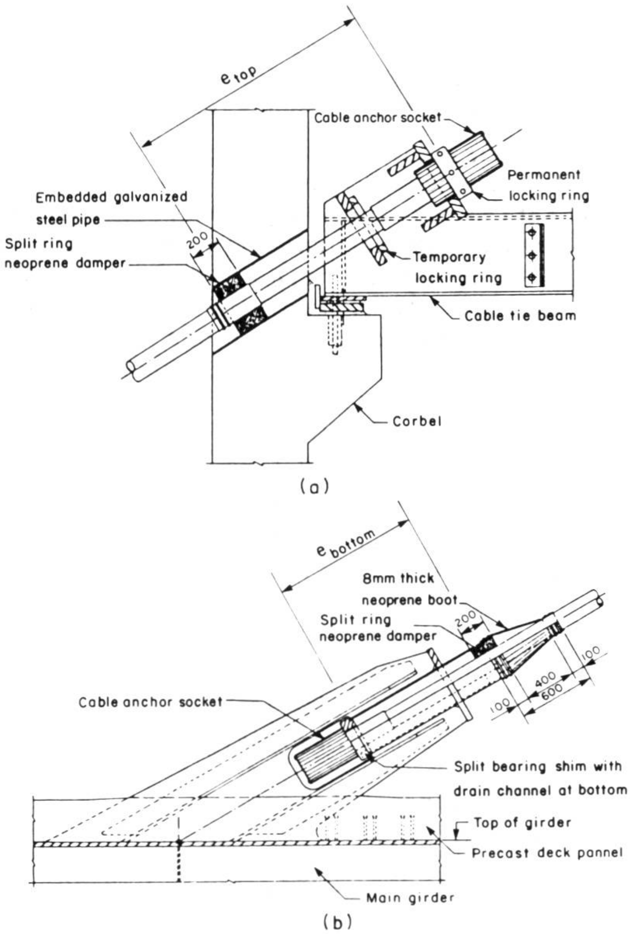

Fig. 2 Cable End Socket Detail Showing Split Neoprene Ring
(a) Top Socket
(b) Bottom Socket (from ref. [12])

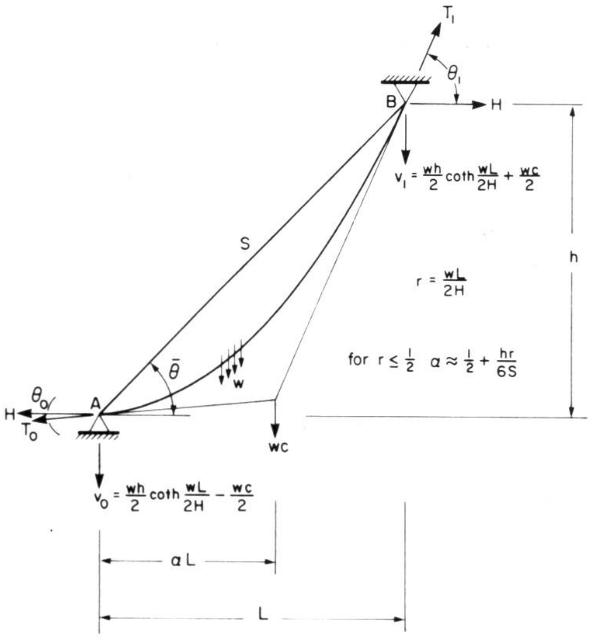

Fig. 3 Catenary Cable Notations

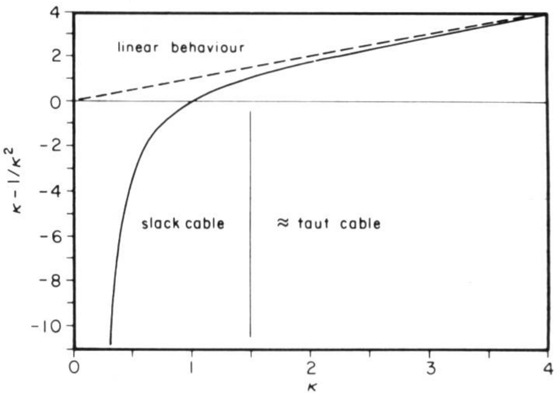

Fig. 4 Behaviour of a Suspended Cable

The value of κ_0 (for the D.L. condition) for the cables of Annacis Bridge lies between 3.38 and 5.59, which puts all of the cables well into a range where the behaviour can be linearly approximated (see Fig. [4]). Thus all cable tensions have been calculated using taut string theory.

A recent paper by Kovacs [8] discusses the effects of component interaction and introduces two possible causes of large cable vibrations. The first of these is caused by excitation of the supports perpendicular to the cable axis. For interaction to occur the natural frequency of the support, f_s, and the cable, f_c, must be almost equal (i.e., f_c: $f_s \approx 1:1$). In an undamped system this interaction can cause large cable motions; however, these motions can be reduced by increasing cable damping either by changing cable properties or installing a discrete damper. Kovacs investigated the effectiveness of a discrete damper located a distance e (see Fig. [2]) from the cable end socket. He concluded that if the stiffness of the damper is properly specified then the dynamic amplification factor (DAF) is controlled by the location of the damper, such that DAF $\approx 1/\varepsilon$, where $\varepsilon = e/S$.

The other important interaction is caused by support motion parallel to the cable axis causing harmonic changes in cable tension. Kovacs refers to this phenomenon as "parameter" excitation. Structural interaction will occur if ($\omega_s : \omega_c \approx 2:1$) and a description of this interaction is shown in Figure [5]. For a continuous system with fixed ends, such as a cable, motion perpendicular to the axis is described by:

$$y = Y_0 \sin \frac{\pi x}{l} \sin(2\pi f_c t) \tag{3}$$

This motion creates a harmonic support force:

$$T = T_0 - \Delta T_{max} \sin(4\pi f_c t) \tag{4}$$

where T_0 is the original cable tension at rest.

If the cable supports oscillate parallel to the cable axis at a frequency of $2f_{c1}$ their effect is to create an equivalent negative logarithmic damping function δ,

$$\delta = -\frac{\pi}{2} \frac{\Delta T_{max}}{T_0} \tag{5}$$

where the damping is negative because it is in phase with the velocity. Thus, for a cable with its ends moving parallel to its axis at twice the first natural frequency of the cable, we have the following expression to describe its motion.

$$y = Y_0 \sin \frac{\pi x}{l} \left[e^{\left(\frac{\pi}{2} \frac{\Delta T_{max}}{T_0} f_c t\right)} \sin(2\pi f_c t) + \ldots \right] \tag{6}$$

For the case when $f = f_c$ we find that the maximum amplitude of cable vibration A_c depends only on the amplitude of the support motion A_s and the cable length.

$$A_{c_{stable}} \approx \frac{2}{\pi} \sqrt{2 A_s l} \tag{7}$$

This maximum amplitude is attained when the "strainless" situation occurs. At this point the motion of the supports no longer forces the cable past its equilibrium deformed shape (i.e., when the cable is at maximum amplitude the motion of the end supports do not effect the tension in the cable and the entire system is in equilibrium).

The motion will continue at this stable amplitude until the support motion ceases or the frequency changes. Because the energy which creates the support motion can come from any part of the structure and the driving force is always parallel to the cable axis, this cable motion cannot be controlled with the installation of a discrete damper working perpendicular to the cable axis.

A paper by Maeda, Maeda and Fujiwara [9] discusses cable deck interaction. In their study each cable has a different natural frequency and the authors claim that the oscillations of the deck are interrupted by the cables when $f_c \approx f_s$. When this interaction occurs there should be a transfer of energy from the deck to the cables. Therefore, the deck should show an increase in damping and the cables show a decrease in damping. This transfer of energy is said to cause an increase in the dynamic amplification factor, DAF, of the cable and a decrease in the DAF of the deck. This phenomenon is referred to as "system damping". The authors claim that by properly tuning the cables to the various natural frequencies of the bridge, these oscillations can be damped by the cables. Leonhardt and Zellner [10] report evidence of "system damping" in cable-stayed bridges with stiff cables. They postulate that this system damping arises from the nonlinear sag effect in individual cables and the dynamic interference between cables having different lengths and natural frequencies.

Godden [11] undertook a seismic model study of the proposed Ruck-a- Chucky bridge, which is a curved box-girder bridge supported by 48 cable stays. His research indicates that local cable vibrations do not have a significant influence on the overall response of the bridge and that cable forces are primarily related to the vertical oscillation of the bridge as a whole. The cables exhibited a linear response. The study concluded that cable vibration, though visible under all conditions of seismic ground motion, has little effect on the behaviour of the bridge as a whole.

3. OBJECTIVES

Dynamic tests on the Annacis bridge were to be performed on cables varying in length from 50 m to 220 m to determine natural frequency, cable tension and damping values. The objective was to investigate the parameters affecting the cable damping. Possible interaction between cables, deck and tower was of interest and the effectiveness of the discrete neoprene dampers contained in the cable anchorage assembly was to be evaluated. A final objective was to obtain values for frequency and damping of the first vertical, first lateral and first torsional modes of the completed bridge.

4. TEST SET-UP AND RESULTS OF CABLE BEHAVIOUR

The cable vibration problems at the Brotonne bridge involved first mode oscillations. Therefore, a test which induced first mode vibration was desired. The most readily available instrumentation to record the motion was strain gauge-type accelerometers. Expected accelerations were small (0.02 - 0.10 g at the mid point) and accelerometer type and mounting location were chosen accordingly.

A three-wheeled hand cart equipped with an accelerometer was designed and built for this purpose (Figure [6]). The cart could be pulled into position allowing manual excitation of the cable. To avoid any significant effect of added mass, the cart was designed to be very light. It was constructed in two parts, with spacers to accommodate different cable diameters (between 99 and 149 mm). Three nylon rollers used to track the cable were tapered to ensure a large contact surface in order to avoid any damage to the cable sheathing. In case of any small cable irregularities the bolts connecting the two halves of the cart were fitted with springs to allow a slight expansion of the system. The cart had a rope connected to the upper end, by which it was pulled into position. A downrigger was connected to the lower end to provide a bottom-end tie-back and also to measure the actual location of the cart on the cable, through a built-in counter. Accelerometers were used to measure the response of the various components. The amplified signals from the accelerometers were recorded on a multi-channel analogue tape recorder and later digitized.

Given the free vibration decay and the response spectrum, the displacement record was calculated by integrating the acceleration record twice with respect to time. First mode damping was obtained using the logarithmic decrement method. For low values of damping a simplified expression of damping over m cycles can be obtained.

$$\approx \frac{\ln \frac{x_n}{x_{n+m}}}{2\pi m} \tag{8}$$

The complete flow of data from the actual test, to the lab where it was digitized, to the first mode damping values is shown in the flow chart of Figure [7].

The frequency, mass per unit length and length of each cable were used to determine the cable tension at the time of the test. The cables under dead load are quite stiff, therefore, the following expression from inextensible string theory was used to calculate tensions.

$$T = \frac{4\rho f_n^2 l^2}{n^2} \tag{9}$$

The characteristics of first mode oscillations were investigated for cables varying in length from 50 m to 220 m. For each of the cables tested a free vibration decay and a response spectrum were evaluated. A sample of these plots for one of the cables is shown in Figure [8]. The results of these tests, along with important cable parameters are included in Table [1].

The top socket neoprene damper was installed immediately after jacking, and all test were performed with at least the top damper in place. Calculated cable tensions were generally lower than final dead load tensions since all of the tests were performed before the 50 mm concrete deck overlay was in place. However, in some cases the cable tensions would be greater than their final dead load tensions due to the location of live loads (cranes, trucks, etc.) at the time of the test.

The damping values obtained for all of the cables tested range between 0.27 and 0.52 % of critical damping. Critical damping is defined as the lowest value of damping for which no oscillation about the zero deflection position occurs in the free response of the system. All of the tests performed were for small amplitude oscillations. At larger amplitudes geometric nonlinearities may cause an increase in damping.

In order to determine the overall effectiveness of the neoprene dampers, tests without bottom dampers and with both top and bottom dampers in place were performed. Due to the construction procedure the top damper was put in place immediately after the cable was jacked to length and thus only the effectiveness of the bottom damper could be clearly examined. With the

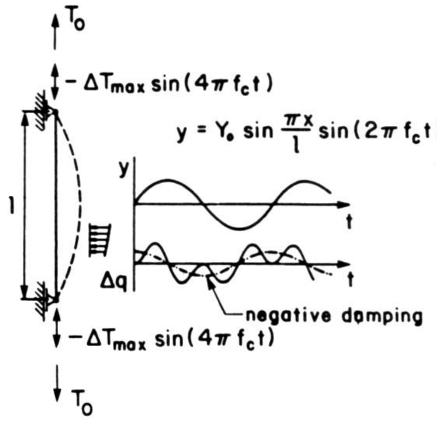

Fig. 5 Motion of a Cable, End Supports Moving Parallel to Longitudinal Axis at 2 fc (from ref. [8])

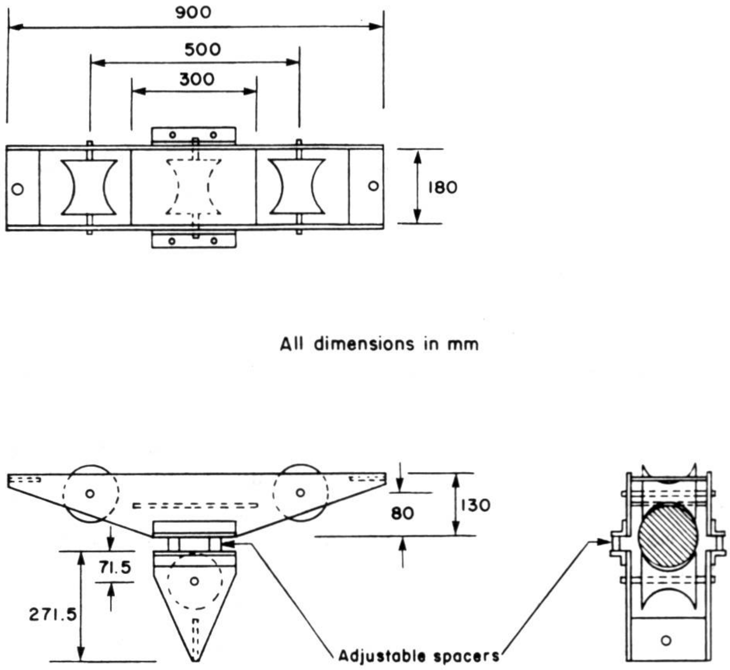

Fig. 6 Three-Wheeled Sensor Cart Used for Cable Tests

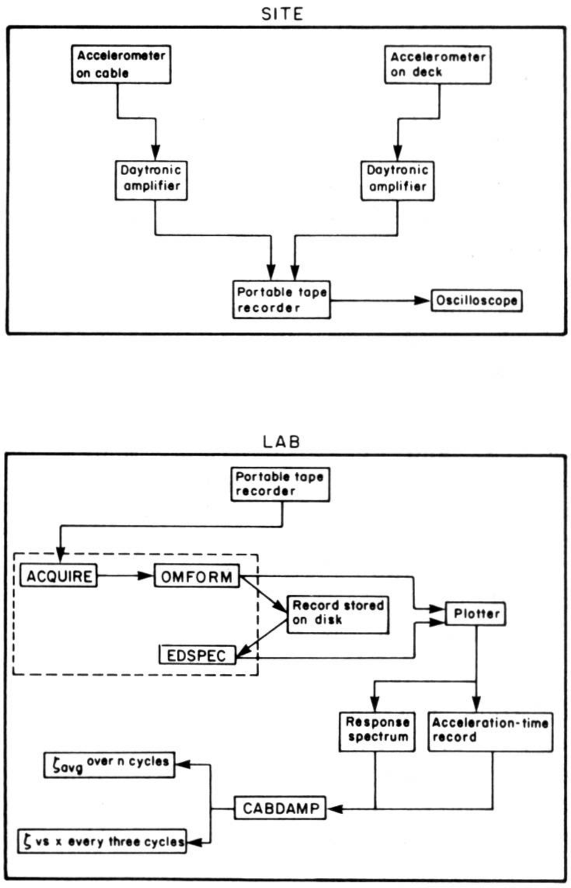

Fig. 7 Flow Charts Showing Collection and Analysis of Data

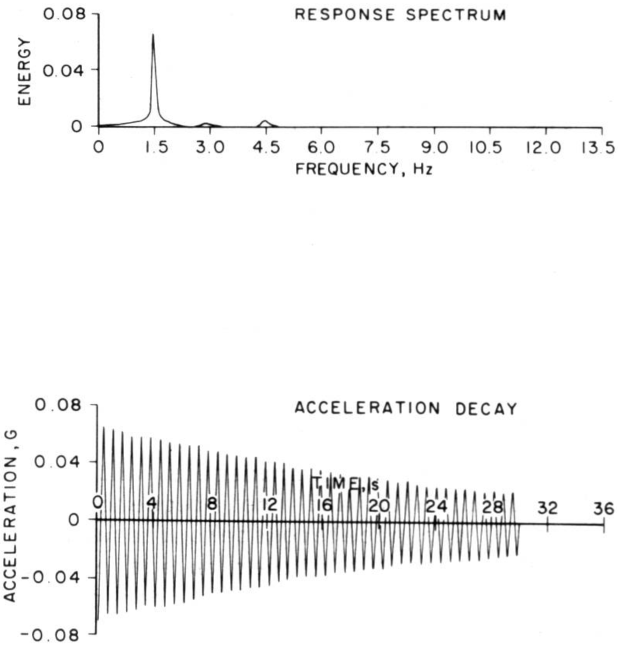

Fig. 8 Response Spectrum and Acceleration Decay for Cable #27 (1-Damper)

damper placed approximately at e = 1.5 m from the end socket, the ratios of end distance, e, to cable length, L, varied between $\varepsilon = e/L = 1/35$ for the short cables to $\varepsilon = 1/140$ for the long cables. Recalling that the effectiveness of the damper depends on its distance of application from the end point (DAF $\approx 1/\varepsilon$), large increases in damping were not expected with the installation of a second damper. It was also expected that the influence of the second damper would be more evident in the shorter cables where the ε is larger.

Five cables of various lengths were tested with one and two dampers in place. The natural frequencies (and thus tensions) varied slightly between the one- and two-damper tests for a given cable. However, these slight differences in tension should have no significant effect on damping. The damping values for the two cases are compared in Table [1] and in Figures [9] and [10].

One can easily see the direct correlation between damping and relative location of the damper, as already predicted by Kovacs [8]. The relationship shown in Figure [9] reinforces the statement that higher vibration modes will be more effectively damped than lower modes, for a damper location near the ends of the cable (since higher modes have a greater lateral deflection at the damper location). This is shown in Figure [11].

From wind tunnel studies and analysis it was found that higher mode response is dominant in the cables due to environmental influences.

5) FULL BRIDGE MEASUREMENTS AND RESPONSE

The natural frequency and damping values for the lowest modes of the completed bridge were of great interest as a check of assumptions made for the aerodynamic stability analysis.

Several attempts were made to excite the first vertical mode by impact testing, but the energy input was not enough to induce measurable values of acceleration. The frequency was later identified by spectral analysis of ambient vibration, but no reliable damping data was obtained.

The first lateral and torsional modes of the bridge were more easily excited. A pendulum with a relatively large mass swinging perpendicular to the bridge deck was used. The pendulum consisted of a rough terrain crane, positioned at centre span, with a 2000 kg concrete block at the end of its line. The frequency of the first transverse lateral mode was not exactly known, but the crane allowed tuning by varying the length of the pendulum. Ropes connected to the mass allowed two men, positioned on opposite sides of the bridge, to maintain a relatively constant-amplitude sinusoidal input force.

Analytical models indicated that the first lateral and first torsional modes were perhaps coupled. Accelerometers were used throughout the experiment at different locations on the bridge deck. The pendulum was first adjusted to the analytically determined frequency of the first lateral mode. The bridge was excited at this frequency for several minutes and then the mass was placed back on the deck. A spectral analysis of the response was performed immediately and the pendulum length was adjusted to match the natural frequency of the deck.

The number of accelerometers and their locations varied depending on the vibrational mode and the frequency of interest. To determine the torsional response, two accelerometers measuring the vertical component of acceleration were placed on the girder on either side of the bridge, and one accelerometer measuring the horizontal component was located on the centre-line of the bridge.

Cable No.	Damping (% crit.)	2.Damper (% crit.)	Density (kg/m)	Length (m)	Freq. (Hz)	Tension (kN)	Mass (kg)	Stiffness (kN/m)	e_{top}/L (e/l)	e_{bot}/L (e/l)
12	0.3154		46.585	141.729	0.65	1557	6602	10.986	0.011	0.008
22	0.4172		36.595	61.751	1.56	1358	2260	21.992	0.029	0.019
23	0.3703		36.595	56.104	1.61	1198	2053	21.353	0.043	0.021
26	0.4238		40.469	55.267	1.74	1497	2237	27.087	0.038	0.022
27	0.3754	0.4131	40.595	61.558	1.49	1353	2491	21.979	0.025	0.019
28	0.3374	0.3611	36.595	68.165	1.47	1470	2494	21.565	0.019	0.018
29	0.3676	0.4015	36.595	75.906	1.32	1470	2778	19.366	0.023	0.016
30	0.3223	0.3476	36.595	83.346	1.16	1374	3050	16.485	0.020	0.014
39	0.3119	0.3453	50.255	159.141	0.70	2488	7998	15.634	0.010	0.008
40	0.3621		50.255	168.510	0.70	2806	8468	16.652	0.010	0.007
42	0.3067		54.330	185.960	0.58	2517	10103	13.535	0.009	0.006
44	0.3148		54.330	203.791	0.59	3085	11072	15.138	0.008	0.006
45	0.2794		54.330	212.845	0.50	2422	11564	11.379	0.008	0.006
46	0.3308		54.330	221.893	0.50	2633	12055	11.866	0.008	0.005

Table 1 Parameter Summary

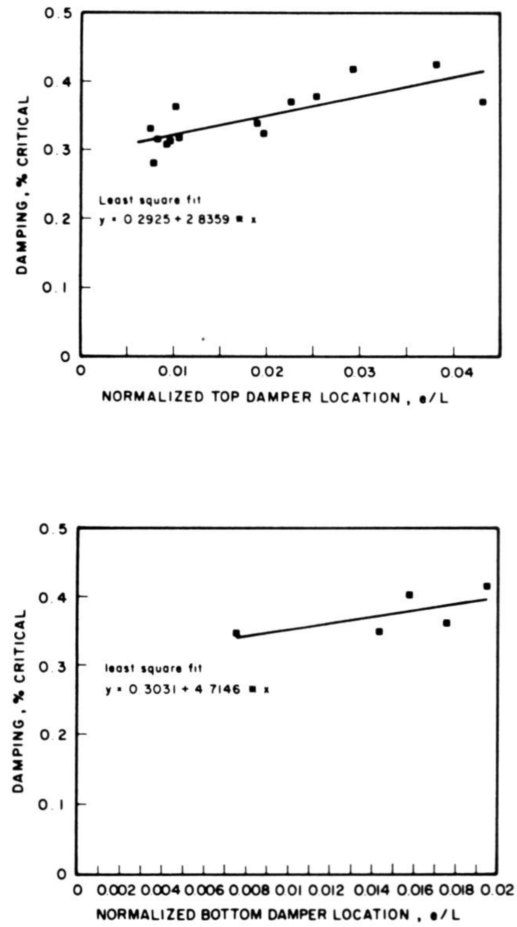

Fig. 9 Influence of Damper Location on Damping Behaviour of Cables for
(a) One Damper (Top) in Place
(b) Two Dampers (Top and Bottom) in Place

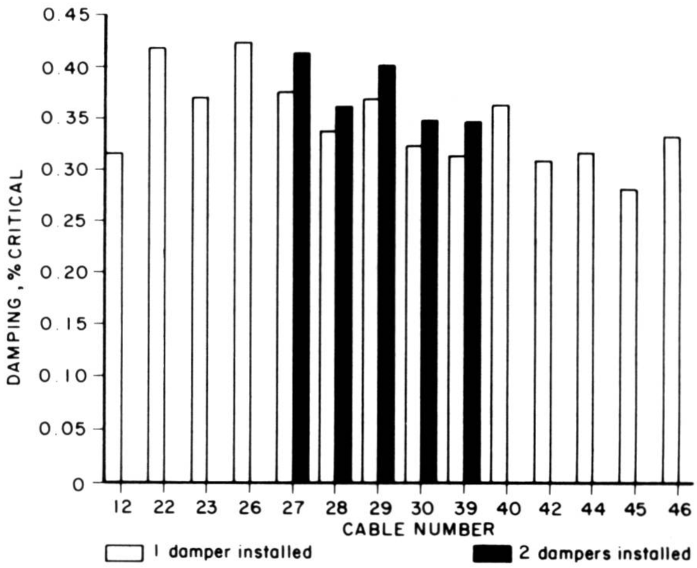

Fig. 10 Bar Graph Showing the Increase in Damping with the Addition of a Second Damper

The response spectra for the bridge deck are shown in Fig. [12]. A large peak at 0.35 Hz in the lateral spectrum compared with a small peak at 0.35 Hz in the spectrum indicates that the first transverse lateral mode is coupled. Through further tests the exact first lateral mode frequency was found to be 0.325 Hz.

After exciting the bridge a 0.325 Hz for approximately five minutes an amplitude of between 50 and 100 mm was achieved and the mass was brought to rest so that the free vibration response could be obtained. Lateral response spectra of the two girders showed each to have a strong peak @ 0.325 Hz., and the displacement-time response showed that the two oscillations were in phase. This again indicated a transverse lateral mode (providing the shear center is at deck level. The free response plots (Fig. [13]) show an average damping value of 0.46 percent of critical for the first lateral mode.

The next mode to be isolated was the first torsional mode with a frequency of 0.475 Hz. After several minutes of excitation, amplitudes of approximately 100 mm at the bridge deck edges were obtained and slight cable motions due to this excitation were observed. At this point the mass was brought to rest and the free vibration response was again recorded. The resulting spectrum) shows peaks from both vertical accelerometers at 0.475 Hz. The displacement-time traces show that the two responses were 180 degrees out of phase, indicating the first torsional mode. The average damping value obtained from these plots was 0.30 % of critical. The damping values for the first lateral and first torsional modes are taken from relatively low amplitude tests.

A vertical impact test was carried out with the 2000 kg mass being dropped onto the deck every 30 seconds. These cyclic drops allowed for continual updating of the response spectrum. The large peak at 0.275 Hz indicates the first vertical mode of the bridge. A damping value for the first vertical mode could not be obtained from this impact test because the free vibration response contained many higher modes and the first mode could not be isolated from the higher modes.

6. COMPONENT INTERACTION

Tests to investigate interaction between the main bridge components were carried out with accelerometers on the cables, deck and tower. Tests showed that there was no tower motion over the ambient level during excitation of the cable.

The deck is much more flexible than the tower, therefore, it was concluded that cable motions may excite certain deck modes. A series of tests was carried out with accelerometers mounted on both the cable and deck (at the lower socket of the cable being tested). The cable was excited in its first mode and the free vibration response of the cable and deck were monitored simultaneously. The level of local deck motion increased due to the harmonic input of the cable, but this induced deck motion did not appear to be harmonic itself. No evidence of either a 1:1 or a 2:1 ratio of deck frequency to cable frequency was noted.

Evidence of component interaction occurred twice during bridge construction, however, the first occurence was just before the main span was completed. In a moderate 28 km/h SSW wind cable 4C, on the north end of the bridge, began to oscillate in its first mode, in the direction of the wind. A SSW wind acts in the direction of the bridge span. An analytical check of the frequency of eddy shedding determined that the wind was not directly exciting the cable. Cable 4C is one of the longer cables on the north side and its

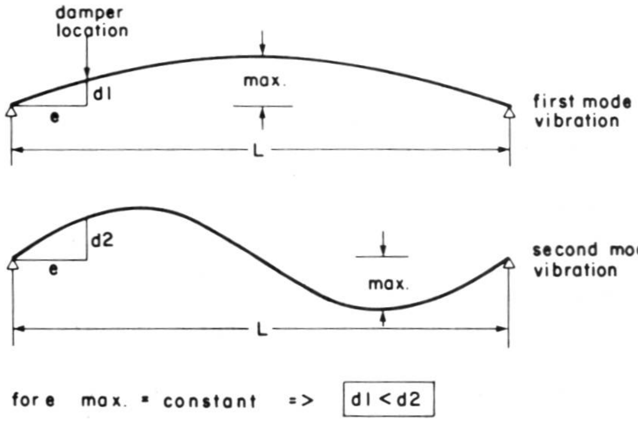

Fig. 11 Increase of Damper Efficiency
Due to Larger Displacement of Higher Modes.

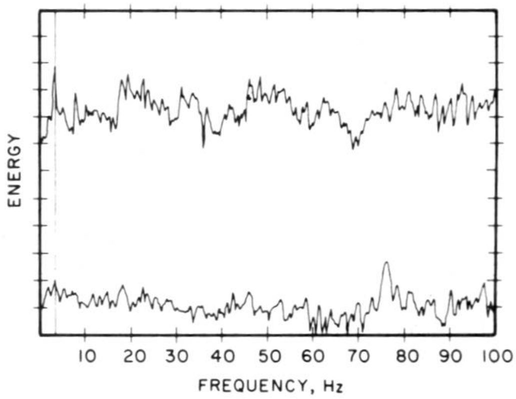

Fig. 12 Response Spectra for Lateral Mode
Upper Trace - Horizontal Response
Lower Trace - Vertical Response

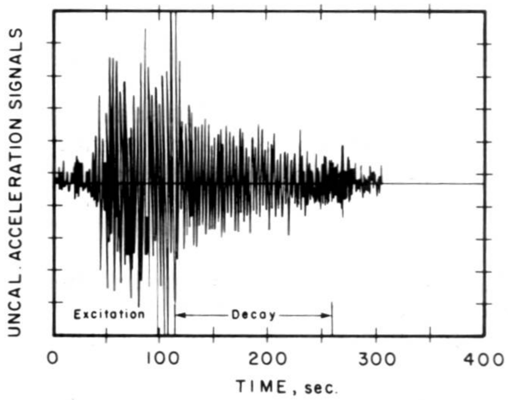

Fig. 13 Free Response Plot from
the First Lateral Mode Test

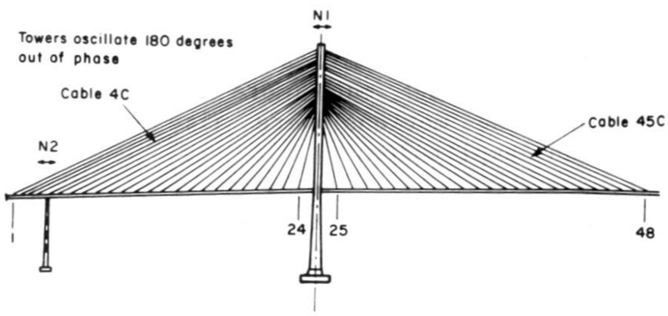

Fig. 14 North Half of Annacis Bridge

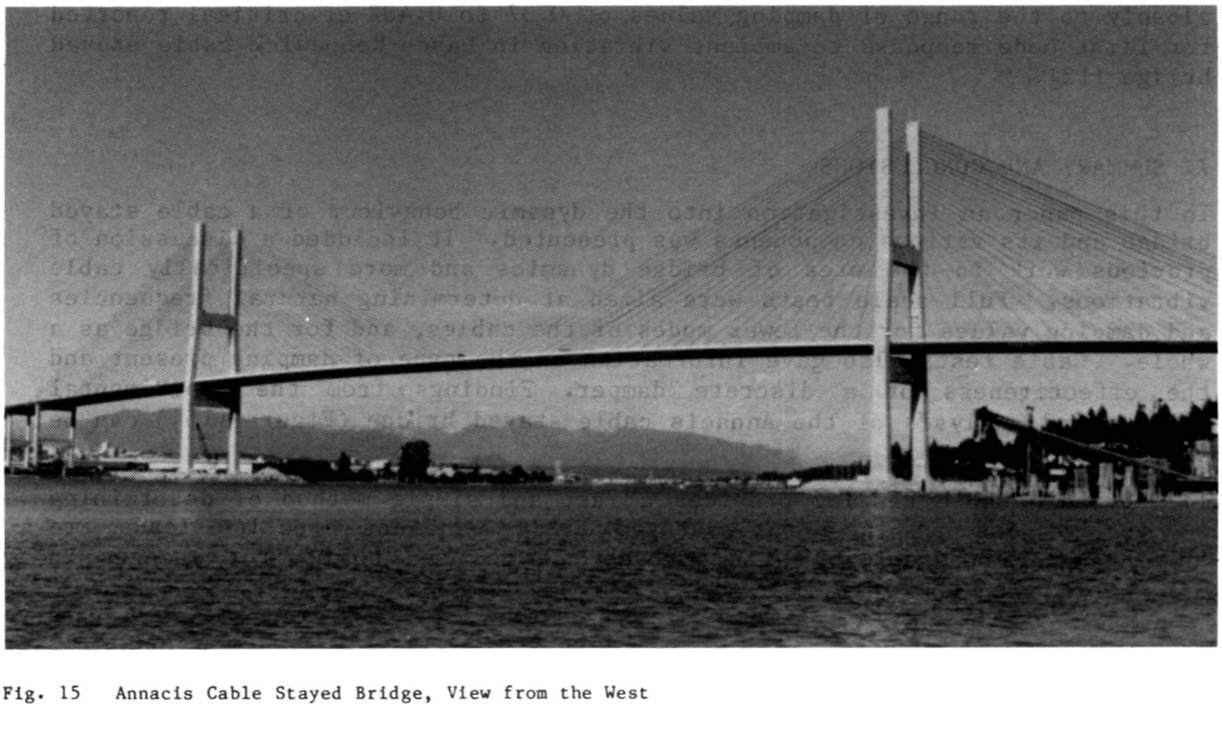

Fig. 15 Annacis Cable Stayed Bridge, View from the West

sockets are near the top of tower N1 and directly above the bent (N2) as shown in Figure [14].

Cable 4C oscillated a steady state amplitude of approximately 300 mm. This condition lasted for several hours, during which time smaller motions were also evident in main span cable 45C which shares a tie beam with cable 4C. The first mode period of cable 4C was 3.1 seconds. This motion was created by support excitation and its steady state nature indicates that was the "parameter effect" discussed by Kovacs [8]. However, it is a slight deviation from the original concept. In the case of the Annacis Bridge the side towers are relatively stiff and all driving energy is coming directly from the main span. This type of motion cannot be reduced by discrete dampers acting on the cable. A discrete damper could not dissipate the amount of energy necessary to excite the tower or deck. This parameter excitation will eventually reach a steady state level above which amplitudes will no longer increase. The steady state amplitude of the cable depends on the amplitude of the support motion (in this case the towers) according to Equation (7).

$$A_{c_{stable}} \approx \frac{2}{\pi} \sqrt{2A_s l} \tag{7}$$

This phenomenon was witnessed twice after the main span had been completed. Under a similar but slightly stronger wind, cables 3C and 4C began to oscillate parallel to the bridge. However, amplitudes were considerably smaller (approximately 75 mm) and the resulting motion seemed to contain higher modes. Although much more difficult to verify (e.g. mode shape, period, etc.) these motions were also evident for several hours at relatively constant amplitudes.

Evidence of "system damping" as reported by [9] and [10] was sought in the dynamic response of Annacis Bridge superstructure to forced vibration by tuned pendulum and the subsequent free vibration decay. Measured total damping values were 0.46% of critical in the first lateral mode and 0.30% of critical in the first torsional mode. No evidence of system damping was noted.

The measured damping values indicate that at small amplitudes (up to 100 mm) the bridge superstructure is lightly damped. The observations correspond closely to the range of damping values of 0.37 to 0.48% of critical reported for first mode response to ambient vibration in Pasco-Kennewick cable stayed bridge [12].

7. SUMMARY AND CONCLUSIONS

In this paper an investigation into the dynamic behaviour of a cable stayed bridge and its various components was presented. It included a discussion of previous work in the area of bridge dynamics and more specifically cable vibrations. Full scale tests were aimed at determining natural frequencies and damping values for the lower modes of the cables, and for the bridge as a whole. Cable tests also gave information on the type of damping present and the effectiveness of a discrete damper. Findings from the experimental results and analysis of the Annacis cable-stayed bridge (Figure [15]) can be summarized as follows:

1) Dynamic cable testing provides a fast and simple method of determining cable tensions in a cable-stayed bridge. Since erection loads are changing constantly, this approach can be used to monitor changes in cable tensions.

2) The harmonic input force from an oscillating pendulum can be used to excite the lower modes of vibration of a cable-stayed bridge. The modal frequencies can be isolated and a properly tuned pendulum (even if the input force is quite small) produces measureable motions in a short period of time.

3) With one neoprene damper in place, damping values for all of the cables tested were found to be between 0.27 and 0.52 percent of critical damping. For the existing DAF $\approx 1/2\zeta$ these low damping values help to explain why large local cable vibrations have occurred in the past.

4) Cable damping depends largely on the damper location (for constant damper characteristics). Higher vibration modes are likely to be damped more effectively than lower modes. The influence of the neoprene damper characteristics, such as volume, hysteresis, hardness, etc., was not investigated since similar dampers were used throughout the entire bridge.

5) The addition of the second neoprene damper near the lower cable end socket increased damping (an average of 10 percent above the damping value with one damper in place) for the five cables tested. The tests conducted were for small amplitude vibrations. The dampers were near the end sockets and resisted only very small cable motions. For higher mode oscillations the cable displacement at the location of the damper would be larger and it should be more effective.

6) The cable tests provided no evidence of component interaction. The small amount of energy required to produce cable motion was not enough to induce motion in the stiffer and more massive deck and tower. In order to properly investigate component interaction, a test which inputs energy into either the deck or tower would be required. Although not evident at the time of testing, wind induced cable motions did occur during erection of the bridge. The so-called parameter effect was evident on two occasions during moderate steady winds. The resulting cable oscillations with amplitudes of approximately 150-300 mm were not produced by eddy shedding. Instead, these steady-state motions were induced by harmonic support motion produced by wind energy being transferred to the bridge at some other location. The energy being input to the system through the supports far exceeds that which could be dissipated by a discrete damper.

7) Overall damping values for the first lateral and first torsional modes of the bridge were 0.46 and 0.30 percent of critical damping respectively. The damping tests were performed with amplitudes of approximately 50-100 mm at centre span.

8) No evidence of "system damping" in a cable-stayed bridge was noted in our full scale tests. This observation agrees with model studies by Godden [11].

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