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Tacoma Narrows 50 years later—wind engineering investigations for parallel bridges

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Wind engineering investigation of the Parallel Tacoma Bridges was performed. Half a century after the historic collapse of the original Tacoma Narrows Bridge, aerodynamic instability was again the principal concern. Since it was proposed to build a new parallel bridge only 61 m from the existing bridge, aerodynamic interference effects between the two structures were carefully examined. Initially the local wind climate and the bridges' wind exposure were examined. Sectional models and full aeroelastic models of both bridges were constructed and tested to evaluate aerodynamic stability and wind loading. In addition, full-scale measurements were undertaken on the existing Tacoma Bridge for identification of the natural frequencies of the lowest lateral, vertical, and torsional modes. The study program established that both bridges would be aerodynamically stable and provided detailed wind load distributions for structural design.

Keywords: Parallel suspension bridges; Wind engineering study; Aerodynamic stability; Wind climate; Design criteria; Wind loads; Full-scale measurements; Sectional model test; Aeroelastic model test; Theoretical and aeroelastic model response predictions

1. Introduction

The first Tacoma Narrows Bridge was opened to traffic for only four months before its spectacular collapse. During its brief life, the bridge had been frequently observed to go into wind-induced vertical oscillations. Central diagonal stays were added, limiting relative motion of the main cables and deck at mid-span, but on 7 November 1940, after several hours of a storm with relatively low winds of up to about 20 m s^{-1}, it was speculated that one or more of these diagonal stays broke loose (Washington Toll Bridge Authority 1945). This then permitted violent torsional oscillations to build up and the bridge collapsed. The famous aerodynamicist Theodore von Karman was called in before the collapse to investigate the cause of the oscillations but there was insufficient time for his research to develop an aerodynamic solution before the bridge's final demise. The disaster triggered extraordinary efforts to research the aerodynamic causes of the failure, and this research during the 1940s established many of the essential wind tunnel techniques used today, such as sectional and aeroelastic model testing. An extensive series of wind tunnel tests and theoretical studies were initiated at the University of Washington led by Professor F.B. Farquharson (Farquharson *et al.* 1954). The aerodynamic lessons learned were incorporated into a replacement bridge constructed at the same site and opened to traffic on 14 October 1950.

Half a century later, due to substantial increases in traffic, the Washington State Department of Transportation (WSDOT) initiated plans to construct a second bridge (figure 1). The new bridge would be built in very close proximity to the existing one and traffic would flow one way on the new bridge and the other way on the existing one. The construction consortium responsible for the new

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Figure 1. Artistic rendering of the future configuration (courtesy of Parsons/HNTB/WSDOT).

bridge, Tacoma Narrows Constructors (TNC), contracted the design of this new bridge, as well as changes to the existing bridge to accommodate the changes of the traffic arrangement to a joint venture of Parsons Transportation Group (PTG) and Howard Needles Tammen and Bergendorff (HNTB). The close proximity of the two bridges introduced the possibility of aerodynamic interaction between the two structures. Also, as part of the lane changes on the existing bridge the plan was to cover over some open gratings. These gratings had been incorporated into the existing bridge for aerodynamic reasons. Therefore there was concern that without them the existing bridge may not have sufficient aerodynamic stability. To undertake the aerodynamic studies that would address these issues, the designers retained Rowan William Davies & Irwin Inc. (RWDI). The studies included the following.

- Wind climate analysis including the effects of local terrain.
- Sectional model tests, addressing the aerodynamic interactions between the two decks.
- Full-scale measurements of the frequencies and damping of the existing bridge.
- Full aeroelastic models of both bridges to study in depth the interaction effects.
- Derivation of equivalent static wind loads for structural design.
- A study of the new bridge's stability and wind loading during construction.
- Sectional model studies of future modifications to the new bridge involving the addition of a lower-level deck.
- A study of possible wind-induced vibrations for the new bridge's hangers.

This paper concentrates on items of particular interest, such as the field vibration tests and key results from the sectional model and full aeroelastic model studies. The responses measured on the aeroelastic models are compared with those predicted analytically. The background on the scaling principles applicable to wind tunnel models of bridges, techniques of aeroelastic model design and construction, the role of the different types of models, and the development of design wind load distributions from wind tunnel data have been described by Irwin (1992, 1998) and will not be reiterated here. The present studies are the first to the authors' knowledge where two full aeroelastic models of major suspension bridges have been tested side by side.

2. Wind climate and exposure

The study began with a wind climate study and examination of the local terrain. For the wind climate study, historical data were obtained from three meteorological stations close to the site:

- Tacoma Narrows Airport located only 3 km west of the bridge,
- McChord Air Force Base at about 19 km southwest, and
- SeaTac International Airport located at 27 km to northeast.

These stations had 26, 24, and 36 years of wind records, respectively. The local terrain conditions around the three anemometer sites were assessed from topographical maps and photographs and the recorded wind speed data adjusted to correspond to the standard 10 m height and open terrain. Figure 2 presents the wind speed variation with return period from these studies and also shows the adopted design curve.

From this figure it is evident that the design curve is located above the curves derived from the local meteorological data, implying that it contains some conservatism. However, by adopting the design curve shown, consistency with the US ASCE 7 standard wind map for the area was obtained, and in view of the history of this site, it was felt that it was some justification for taking a conservative approach. Based on these considerations, the design criteria in table 1 were recommended. The stability criteria given in this table were for winds at zero angle of attack of the wind relative to the bridge deck.

During the studies of Farquharson and his team in the 1940s, an important issue in the aerodynamic design of the replacement bridge was the range of angles of attack that are likely to occur. Smoke bombs were suspended under the remaining cables of the collapsed bridge and simultaneously set-off to emit coloured plumes that were

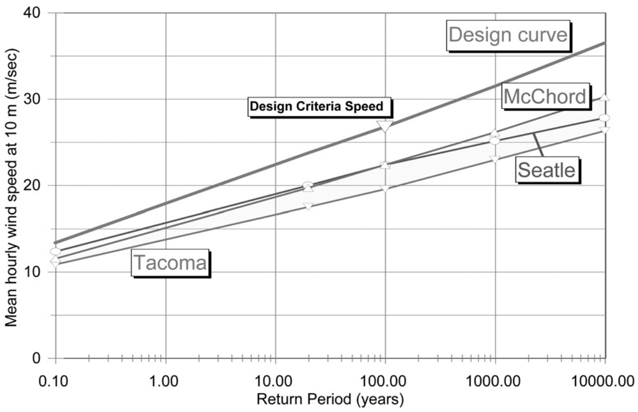


Figure 2. Hourly wind speed at 10 m elevation in open terrain versus return period.

Table 1. Tacoma Narrows Bridges—design wind speeds.

Return period (years)	Application	Wind speed (m s^{-1})	Averaging time
20	Structural design for construction	30.9	1 h
100	Structural design of completed bridge	35.3	1 h
1000	Stability during construction	44.7	10 min
10000	Stability of completed bridge	52.3	10 min

recorded with motion picture cameras (Washington Toll Bridge Authority 1945). The wind during that experiment was in the range of 7–12 m s^{-1} blowing at close to right angles to the bridge. It was found that in all cases "the smoke trail was horizontal, or very nearly so". This indicated that "the angle of wind attack was close to zero". Disregarding this evidence, however, and based on concerns that for cross-winds the flow may be deflected by the local topography, it was conservatively decided that the bridge deck should remain stable up to 53 m s^{-1} (118 mph) for all angles of attack up to 15°. It is interesting to note that the old speed criteria are almost identical with the presently adopted speed of 52 m s^{-1} for zero angle of attack which is based on an independent analysis of the historical data recorded since that time. To achieve this high speed over that wide range of angles was very difficult, however. After 76 different sections were tested, this was eventually achieved by incorporating three rows of vents in the deck, covered by open gratings.

Since those days, subsequent studies have demonstrated that over open water and at high wind speeds, the probable inclinations of the wind would likely be in the order of $\pm 1°$. Roberts (1958) reported detailed anemometer measurements carried out for the Severn Bridge in England. In that study four anemometer stations were installed across the Severn Estuary on the Severn Railway Bridge, at 45.7 m (150 ft) elevation. Wind speeds, directions, and angle of inclination were recorded for a period of 20 months. On about 100 occasions, inclinations that were more than 5° occurred for durations of at least half a minute. Of these instances however, only four were of a duration 1 min or more. The maximum duration was 1.25 min with inclination 5° at only one station for a mean wind speed of 10 m s^{-1}. Clear indication was found that at higher speeds the inclination becomes smaller and steadier. Inclinations higher than 5° were found on only four occasions for gust speeds higher than 15 m s^{-1}, and these speeds were not maintained for the whole period of inclination. During the measurements over 20 months, only two simultaneous records occurred with the same sign inclination of 5° and those were not registered by adjacent stations. The study concluded: "It seems virtually impossible, therefore, that a wind of high velocity inclined on ± 5 degrees would act upon the whole bridge front for as long as half a minute at any wind speed over 20 knots (10 m/sec) ... this upper limit of ± 5 degrees at 20 knots will probably decrease to ± 1 degrees at 100 knots (50 m/sec)." Similar trends were reported on the Lions' Gate Bridge by Teunissen and Williams (1978).

Considering the concern that for cross-winds, the flow may incline over approaching topography, it should be pointed out that bridge decks become more stable for winds off the normal to the deck and it is the wind component normal to the span that is important for flutter (Irwin and Schuyler 1977). Consequently, it has become a common practice today to apply reduction factors on the required flutter speed for non-zero angles of attack. Factors frequently used are 0.8 for angles of attack $\pm 2.5°$, and 0.5 for $\pm 5°$ and these were adopted in the present Tacoma Narrows studies. This approach was originally used on the Severn Bridge in the UK and has been used for a large number of other bridges since.

3. Sectional model tests

The objective of the initial sectional model tests was to investigate the stability of the two deck sections (figure 3) against flutter and vortex shedding. Since it was desirable to close the vents on the road surface of the existing deck for better utilization of the road for one-way traffic, the effect of closing them on stability was also studied. Other minor modifications such as the replacement of the old traffic barriers were also investigated.

Sectional models of the existing and the proposed bridge were built at a scale 1:50 and tested in both smooth and turbulent flows (see figure 4). At the beginning the existing

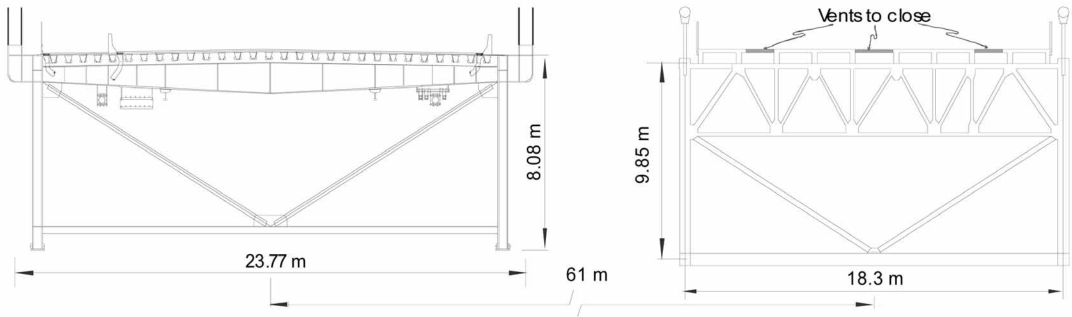


Figure 3. Truss decks of the Parallel Tacoma Bridges (courtesy of Parsons/HNTB/WSDOT).

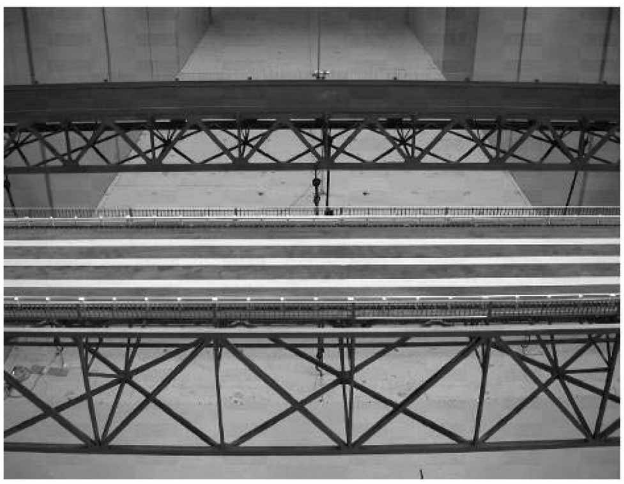


Figure 4. Sectional models of the parallel bridges in RWDI's wind tunnel (vents closed).

deck was tested alone and was found to be very stable. At zero angle of attack, flutter was not observed up to 63 m s^{-1} (225 km h^{-1}). Then the two sections were positioned at the appropriate elevation and spacing in the wind tunnel. During this test only one of the two sections was instrumented and able to move vertically and in torsion. However, by shifting and rotating the decks' positions unwind and downwind of the test rig, all required positions were investigated.

Figure 5 shows the measured static drag, D, lift, L, and torsional moment, M, per unit length, normalized into aerodynamic coefficient form for the two bridge decks. Results are shown as a function of angle of attack for each bridge deck with the other deck downwind and upwind. Angle of attack is the angle of the wind relative to the bridge's horizontal plane, with wind blowing upwards towards the underside of the deck representing a positive inclination. The static aerodynamic coefficients are defined as follows

$$\begin{aligned} &\text{Drag Coefficient} \qquad C_D = \frac{D}{1/2\rho U^2\, d}, \\ &\text{Lift Coefficient} \qquad C_L = \frac{L}{1/2\rho U^2\, b}, \\ &\text{Moment Coefficient} \quad C_M = \frac{M}{1/2\rho U^2\, b^2}, \end{aligned} \tag{1}$$

where ρ = air density, U = wind velocity, d = depth of deck (10.06 m for the existing bridge and 7.16 m for the proposed bridge) and b = width of deck (18.29 m for the existing bridge and 21.64 m for the proposed bridge).

It can be seen that there was a substantial drop in the drag coefficient in both cases when the other bridge was upwind as compared with downwind, as might be expected. The other two coefficients were less affected, although there is a noticeable tendency for the slopes of the curves to be reduced with the adjacent deck upwind. Figure 5a shows also data for the isolated existing deck, and it can be seen that the aerodynamic coefficients were virtually identical to those with the proposed bridge in place on the downwind side.

An important objective of the sectional model tests was to examine the two bridges' susceptibility to vortex excitation and flutter instability. Neither bridge deck exhibited any significant vortex-induced oscillations. The torsional aerodynamic damping is an important indicator of flutter and is shown in figure 6 for zero angle of attack. When the torsional aerodynamic damping goes negative by an amount equal to the structural damping, it is an indication of the beginning of flutter instability. It can be seen that both bridges maintain positive aerodynamic damping up to very high wind speeds, indicating good stability. The presence of the proposed bridge on the upwind side generally improved the stability of the existing bridge as compared with the case where it was on the

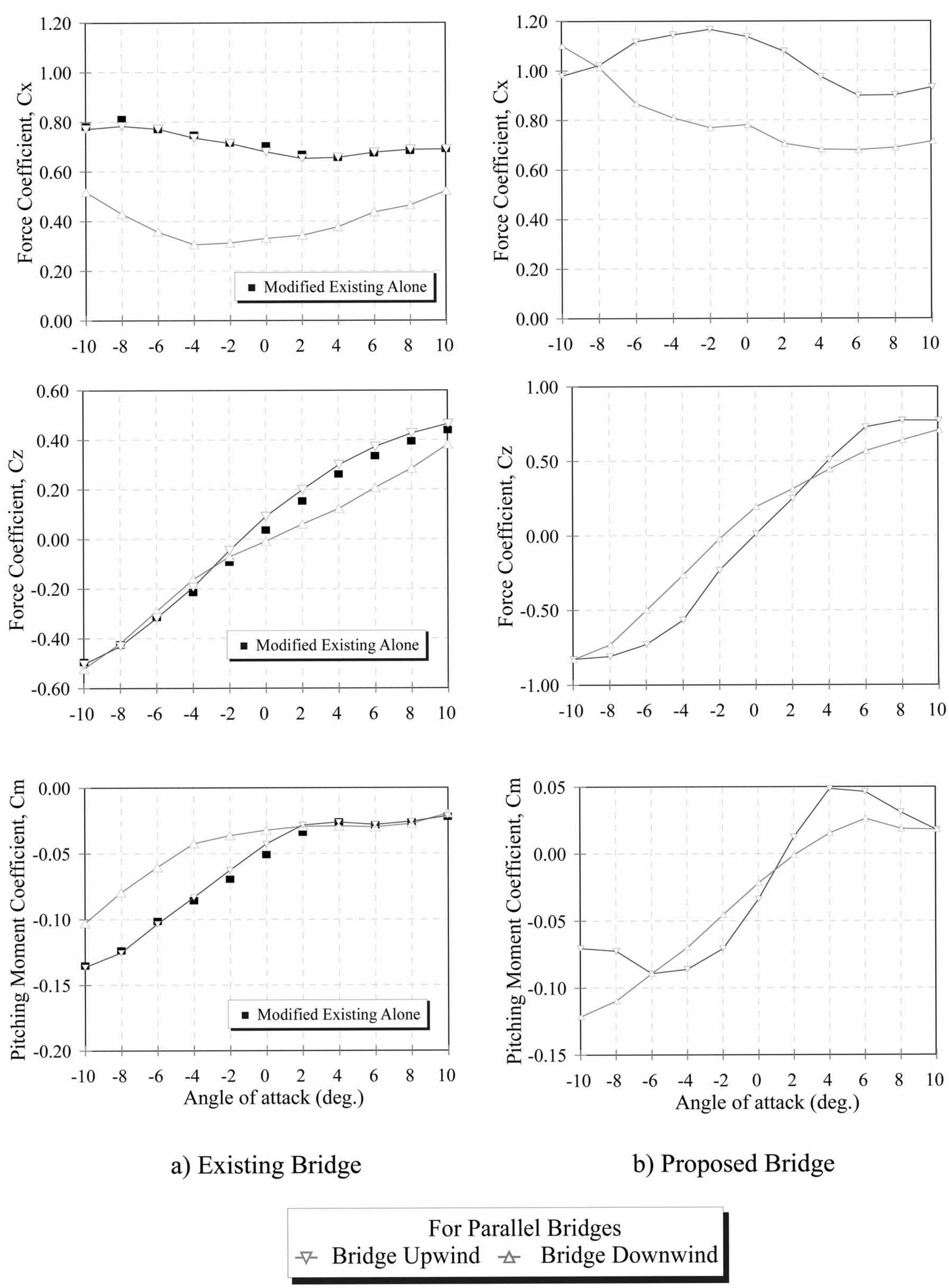
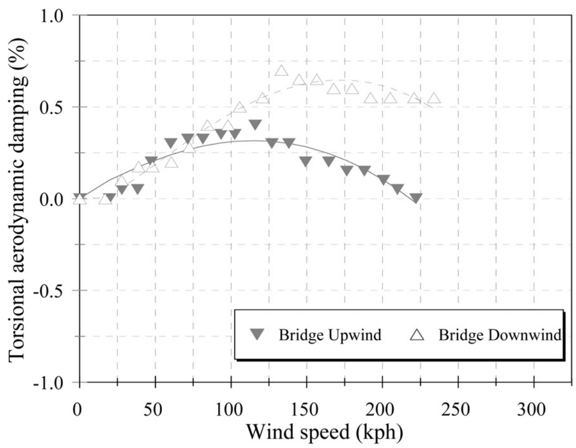
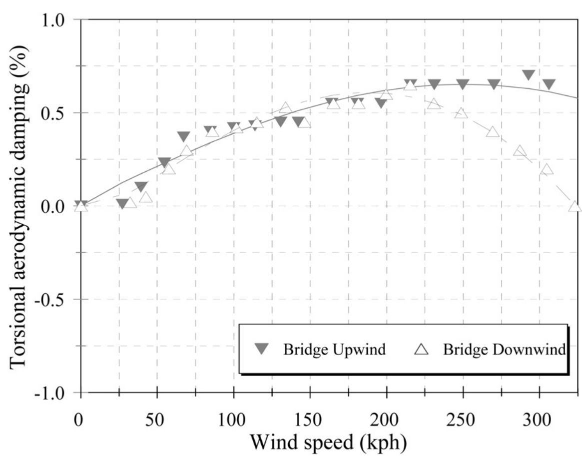


Figure 5. Static force and moment coefficients of the Parallel Tacoma Bridges.



a) Existing bridge



b) Proposed bridge

Figure 6. Aerodynamic damping of the deck sections.

downwind side. At non-zero angles of attack ($\pm 2.5^{\circ}$ and $\pm 5^{\circ}$) the stability tended to be further improved at some angles and degraded at others, but in all cases satisfied the applicable stability criteria. It should be noted that, apart from the effects of topography, non-zero angles of attack are caused by wind turbulence. On a sectional model it is not possible to fully simulate wind turbulence effects, due to its large size relative to the wind tunnel and due to its limited length. However, a partial simulation of high-frequency turbulence is possible on a sectional model, as described by Irwin (1998), and some tests of this nature were undertaken on the Tacoma Narrows sectional models to evaluate the sensitivity of the stability to turbulence levels. They indicated that the effects of turbulence on stability were beneficial. A more comprehensive simulation of turbulence was possible in the full aeroelastic model tests which are described in section 5.

As expected, it was found that closing the deck vents on the existing bridge reduced the speed for the onset of flutter but, provided the frequency of the lowest torsional mode was high enough, the bridge still met the established flutter criteria. Therefore it became important to be sure of the frequency of the lowest torsional mode of vibration, and this led to the decision to undertake vibration measurements at full-scale on the existing bridge.

4. Full-scale measurements

The full-scale measurements on the existing bridge included recording the response to ambient excitation by traffic and wind, and forced vibration tests. In the forced vibration tests, alternating forces at selected frequencies were imparted to the bridge in the absence of traffic, and the deck motions were monitored. The forced vibration technique was based on the use of a large pendulum to excite the bridge. A similar approach was used on the Lions' Gate Bridge in Vancouver B.C. Canada (Buckland 1981). This methodology provided much cleaner data for analysis and modal identification than the ambient vibration tests.

Six accelerometers, capable of measuring as low as 10^{-6} *g*, were installed at two stations, one at the mid-span of the bridge on the main span and a second, at 170 m west from the mid-span location. At each station there were two vertical accelerometers, each placed on the opposing sidewalks 8.08 m away from the deck centre line. In addition on the south sidewalk, at both stations horizontal accelerometers were installed. Wood shims and fast-settling plaster was used to level the packages. Independent power supply was provided to all units. Each package was secured from displacing with a 22 kg sand bag placed on its cover. In addition, the local wind conditions were continuously monitored during the field program with a RM Young anemometer installed at mid-span 3 m above the deck in the plane of the south main cable. Photographs of the instrumentation are shown in figure 7. Acceleration data were collected at a rate of 40 samples per second and wind speeds and directions at a rate of 1 sample per second. The analog signals of the accelerometers were initially low-pass filtered with a 5 Hz cut-off to remove any high-frequency noise from the signals. Through appropriate summing and differencing of the accelerometer records, various vertical and torsional data were extracted from the pairs of vertical accelerometers. The response of the bridge to normal excitations, i.e. wind and traffic, was recorded over one week. The force vibration tests took place during the night of the weekend.

a) accelerometer package b) mid-span accelerometers and anemometer

Figure 7. Field measurement set up on the Existing Tacoma Bridge.

Figure 8 shows the cross-power spectrum of the vertical and lateral ambient responses taken at the mid-span and nominal quarter span locations on the main span.

The first lateral and vertical frequencies identified in the ambient vibration measurements were almost identical with the predictions of numerical models and coincided with those previously measured by Dix and Jones (1994). However, identification of torsional modes from the ambient measurements was problematic from the ambient vibration data. It was made more complex by heavy logging trucks passing over expansion joints, imparting strong random signals in the frequency range of interest from about 0.2 to 0.4 Hz.

Throughout the ambient tests, the wind was rather strong, peak speeds of up to 16 $\mathrm{m\,s^{-1}}$ being registered, and the traffic was heavy to very heavy for most of the time. During the night of the forced vibration tests however, wind speeds of at most 5 $\mathrm{m\,s^{-1}}$ were registered, dropping down to about 2.5 $\mathrm{m\,s^{-1}}$ when the force vibration test was completed. For the time of the forced vibration test runs, the bridge was closed to traffic. The forcing mechanism consisted of a pendulum of approximately 1315 kg mass suspended over the middle of the road from a Super-8 Forklift. This mass was attached with a chain to the extended boom. By adjusting the length of this chain, the frequency of the pendulum was varied in the range of 0.24 – 0.4 Hz.

The pendulum was positioned at about 1/3 span distance from the west tower. This location permitted excitation of both symmetric and asymmetric torsional modes. Using ropes, the mass was swung across the bridge deck through approximately $\pm 30^{\circ}$ arch, creating an oscillating peak force of about 5600 N applied 6 m above the road surface. After several minutes of excitation, when a steady state response was attained, the pendulum motions were suppressed within one or two cycles and a decay of the deck motions was recorded. Figure 9 shows the identified torsional modes at 0.33, 0.38 and 0.4 Hz.

The close to zero phase found between the mid-main span station and the station close to the span quarter showed that the identified torsional modes were symmetric. No other torsional mode was recognized. It was therefore concluded that the first torsional symmetric mode is at 0.33 Hz. With this frequency it was concluded that the existing bridge was stable against flutter even with the air vents closed off.

5. Aeroelastic model tests

The sectional model tests gave good indications that the two parallel decks would be free of vibration problems and have sufficiently high flutter speeds. These tests supplied most of the fundamental information required for stability evaluations and wind loads. However, the sectional model experiments provided essentially 2D data that were used for full-scale 3D predictions. Due to the complexity of the problem, simplifications were necessary that needed verification. For example, only one of the two deck models at a time was moving dynamically. The wind directionality effects on stability and wind loads were also to be investigated. For these reasons full aeroelastic models of both bridges were built and tested side by side in the wind tunnel (see figure 10).

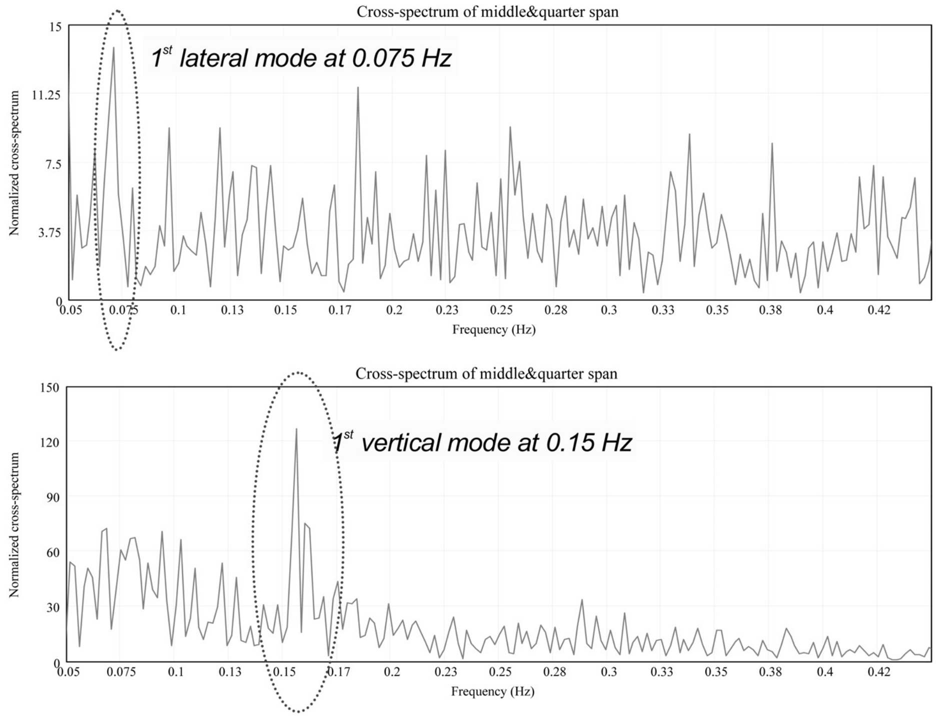


Figure 8. Identification of first lateral and vertical modes from ambient data.

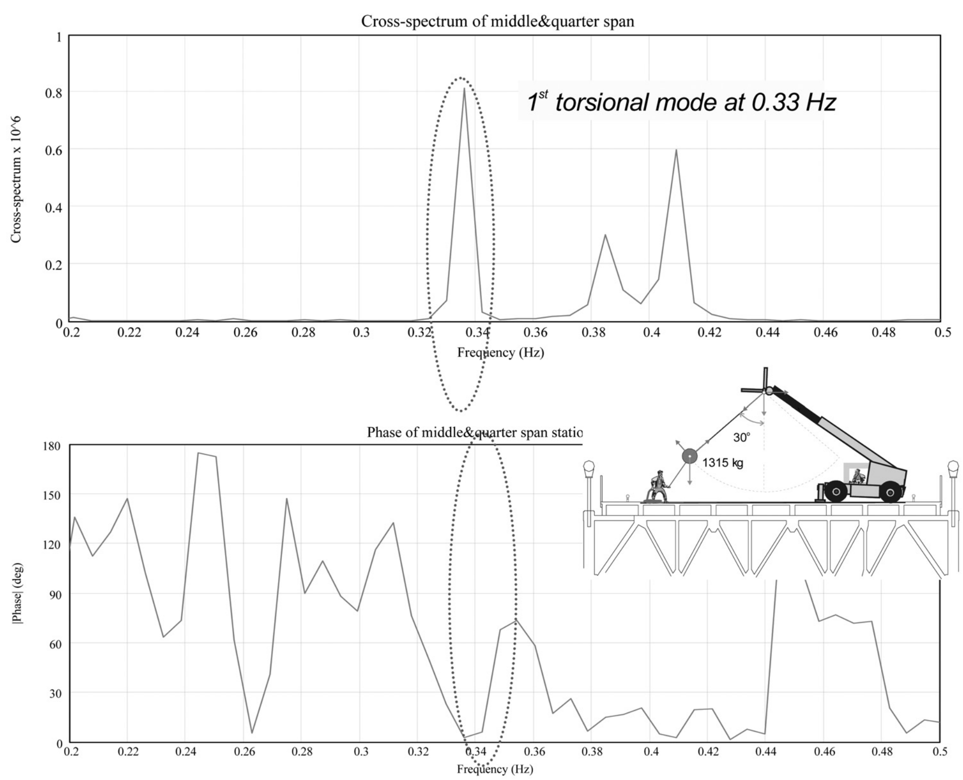
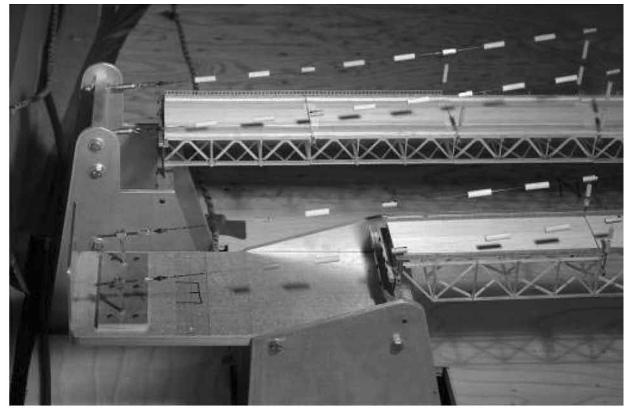
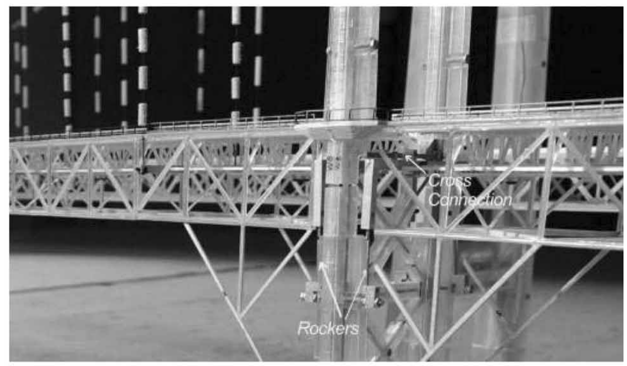


Figure 9. Cross-spectrum and phase of torsion from force vibration test.

Figure 10. Parallel Tacoma Narrows Bridges—the full aeroelastic models in the wind tunnel.



a) end pier supports of the two bridges



b) connections of the existing Tacoma Bridge truss to the tower

Figure 11. Tacoma Bridges' aeroelastic models—support details.

Following established dynamic similarity rules (Irwin 1992) both aeroelastic models were designed and constructed at the RWDI facilities. Considering the length of the two bridges (existing: 1524 m, proposed: 1646 m) a large wind tunnel facility was required. The 9 $\times$ 9 m wind tunnel of the National Research Council (NRC) in Ottawa was selected since it could accommodate full aeroelastic models at a scale of about 1:200. The working section of this tunnel was also long enough to achieve an acceptable simulation of the planetary boundary layer at this scale.

The essential stiffness properties of the main structural components, truss decks, towers, supports and cables were scaled down and incorporated into simplified structural elements such as spines, wires, and flexures of the model. The correct mass and geometry was represented by segments rigidly attached to the spines. In order to match exactly the commercially available cross-sections of piano wires to the required cable stiffness of the main cables, the model scale ratio was refined to 1:211. The aluminum spines of the decks and towers were water cut with high precision, and small elements such as the railings were laser-cut out of thin plywood sheets. The end piers and tower supports were manufactured out of solid aluminum and brass. The most intricate element to model was the connection of the deck to the towers of the existing bridge (see figure 11). The deck of the existing bridge is discontinuous at the towers and under nominal loads each deck can slide along the bridge. By special double pendulum supports called 'rockers' only vertical forces can be transmitted from the deck which is suspended on the hangers. This connection was modeled with a tiny piano wire inserted in a small brass rod. The horizontal force from the deck was transmitted by a piano wire connecting the tower spines to the deck spine. These connections allowed free motions along the bridge and sway rotations of each deck while restricting lateral and vertical motions. Mimicking the real connection, the main cables were connected to the towers with 'saddle' joints.

Based on computer 3D solid models, the towers' and decks' segments were formed by a stereo lithography apparatus, which allowed precise and consistent overall geometry and mass to be achieved. For example, the truss chords of the existing bridge had a cross section of 2.5 x 2.5 mm formed hollow, which allowed small brass rods to be inserted later to achieve the proper mass distributions.

The models successfully reproduced the bridges' geometry and structural dynamics. Their natural frequencies matched the computed full-scale frequencies, appropriately scaled, to within 5%. The two models were shipped about 600 km, from RWDI's Guelph facilities to Ottawa, and installed in the NRC's 9 m $\times$ 9 m wind tunnel for testing.

In the tests, selected important locations, such as the base of the towers, were instrumented with strain gauges. Accelerometers were installed at the top of the towers. Laser displacement transducers were used to measure the deflections of the deck. Figure 12 shows the instrumentation of the existing bridge model. The proposed bridge model was instrumented following the same scheme.

A turntable was installed on the tunnel floor allowing the models to be rotated relative to wind direction and accommodating all instrumentation cables. Wind speeds were recorded at the middle of each side spans at deck elevation, 1.2 m upwind and as well in the free stream at elevation 2.1 m above the tunnel floor. In total 45 channels of instrumentation, including the three speed sensors, were installed.

The study began with the existing bridge upwind tested in smooth flow. The proposed bridge in the upwind position was also tested. Both bridges were found to be stable in either position up to at least 55 m s^{-1} (200 km h^{-1}). Tests at higher wind speeds were not carried out since there was a concern about damaging the models. All further tests were carried out in a turbulent flow.

Turbulent flow was generated with spires, roughness blocks on the tunnel floor, and a flow trip at the spires' base (figure 13). A rough carpet was laid on the floor over about half of the test section to enhance the surface roughness. The mean velocity profile was characterized by a power law exponent $\alpha = 0.14$ and turbulence intensities at deck level of $I_u = 12.7\%$ for the longitudinal component, and $I_w = 8.5\%$ for the vertical component. The length scales were estimated by von Kármán spectra fitting. Converted to full-scale they were ${}^{x}L_u = 175$ m, ${}^{x}L_w = 25.3$ m, ${}^{y}L_u = 25.5$ m, and ${}^{y}L_w = 19.4$ m. These parameters were considered to represent reasonably well the expected turbulence at the site.

Test were conducted for various wind directions starting with either bridge positioned upwind and covering several directions up to $\pm 45^\circ$ off the normal to the bridges' span.

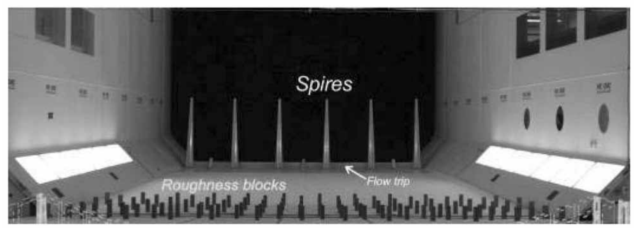


Figure 13. The turbulence boundary layer set up: roughness blocks, spires and flow trip.

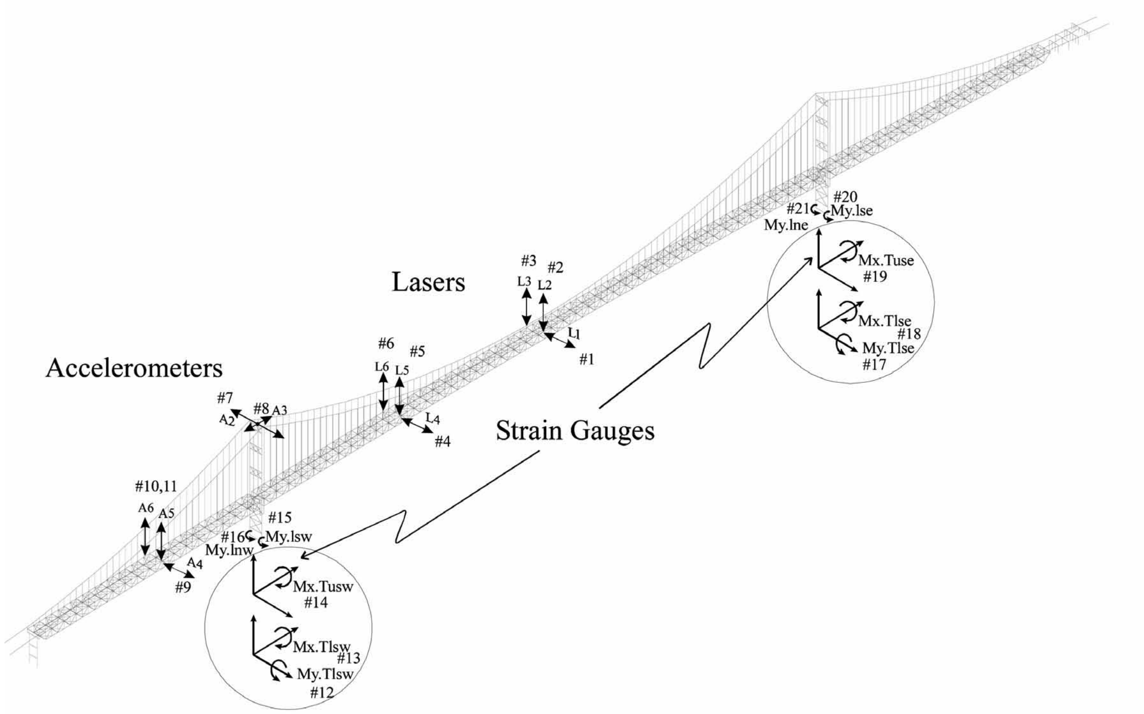


Figure 12. Instrumentation schema (21 points = 6 lasers + 5 accelerometers + 10 strain gauges).

The response of the bridges to turbulent winds was also simulated numerically (Stoyanoff 2001). Figure 14 shows an example of lateral and vertical deflections measured and numerically simulated for the middle of the main span of the existing bridge upwind. In the numerical predictions, static coefficients measured during the sectional model test were applied and as well the dynamic properties of the bridges as predicted by the designers. The responses

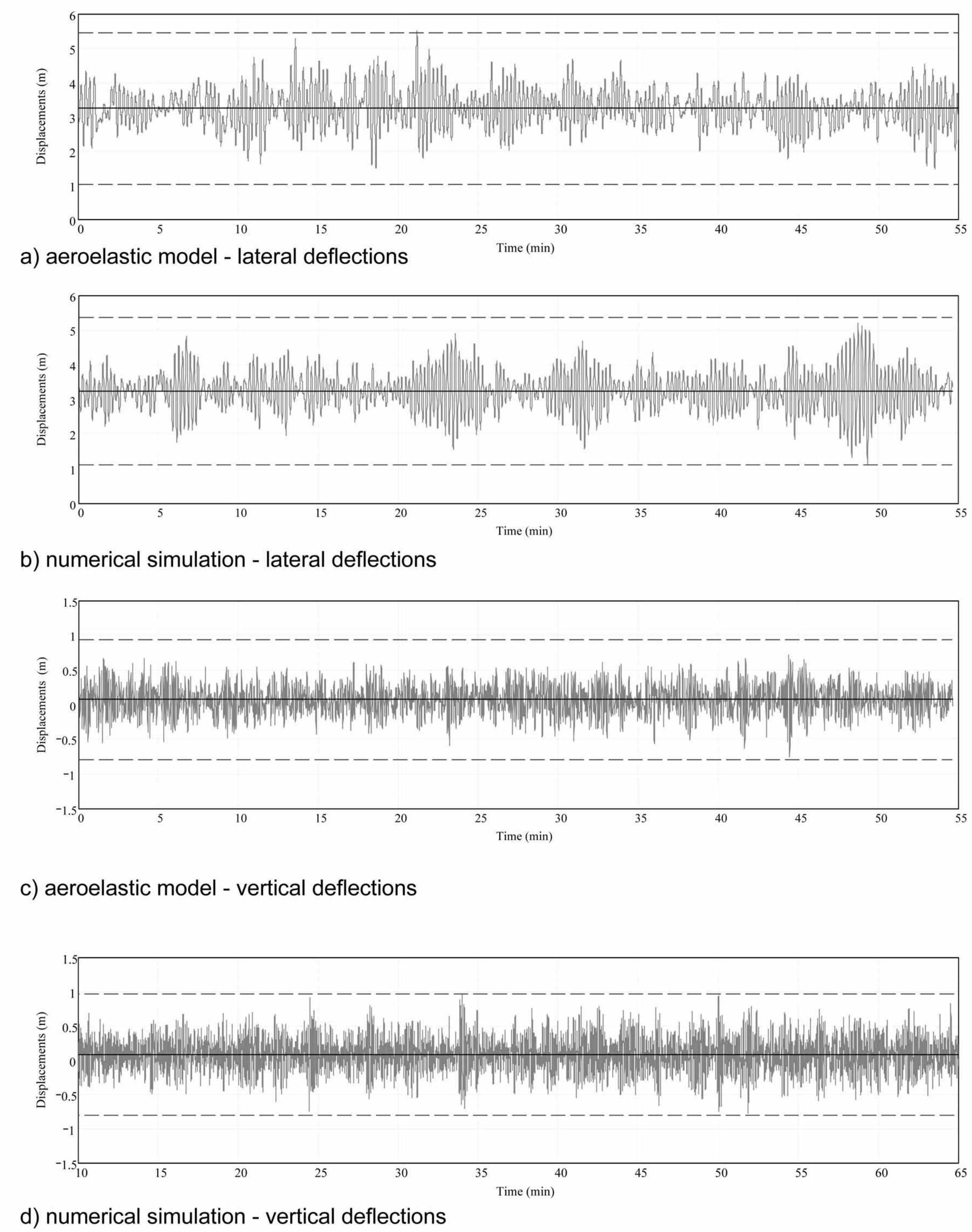


Figure 14. Existing bridge upwind: time histories of responses at the middle of the main span (turbulent flow test/simulation, wind normal to the bridges, wind speed 127 kph, time 55 min).

measured on the aeroelastic models were converted to full-scale using normal scaling methods.

It can be seen that the numerically predicted mean and dynamic deflections were quite similar in magnitude and response pattern to those measured on the aeroelastic model. Normalized power spectra of these responses are shown in figure 15.

Again, the comparison is satisfactory in terms of modal responses and as well the overall shapes of the spectra.

In figure 16 are shown mid-span responses of the existing bridge in torsion, and in the vertical and lateral directions for various wind speeds. These results are for the existing bridge both upwind and downwind of its partner and wind normal to the bridge's span. When the existing bridge was upwind, at the design speed of 35.3 m s^{-1} (127 km h^{-1}), the peak lateral deflection reached 5.2 m. This deflection was reduced to 2.7 m for the downwind position. The vertical and torsional responses also became smaller in the downwind position.

Applying buffeting theory (Irwin 1977, Stoyanoff 2001), the same responses were calculated at the design speed. For the downwind position, the corresponding static force and moment coefficients measured in the wind tunnel were applied (see figure 5) with the same wind speed and turbulence properties as used for the bridge in upwind position. It can be seen that the predicted lateral responses

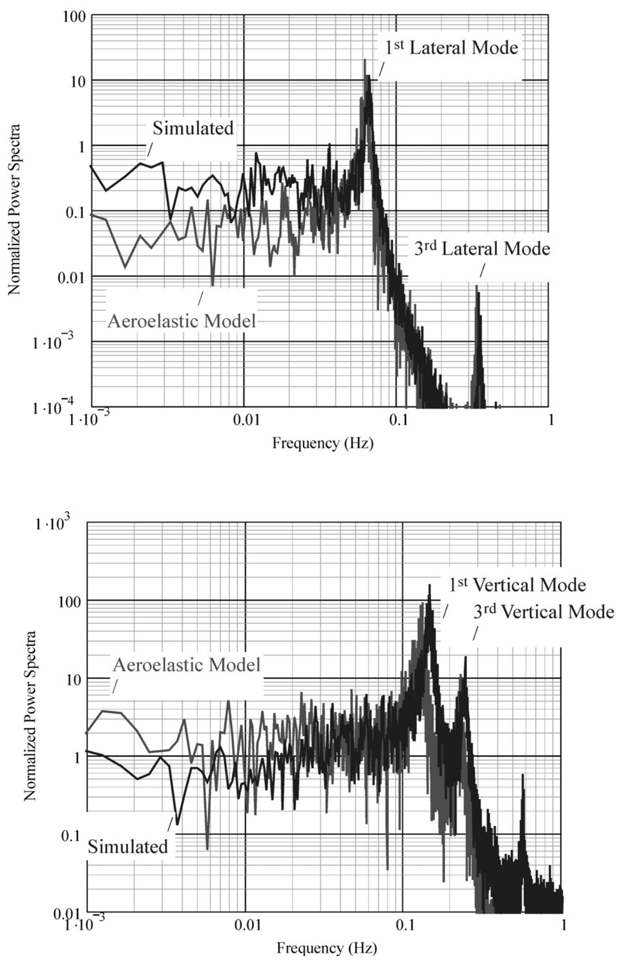


Figure 15. Existing bridge upwind: power spectra of responses at the middle of the main span (turbulent flow test/simulation, wind normal to the bridges, wind speed 127 kph, time 55 min).

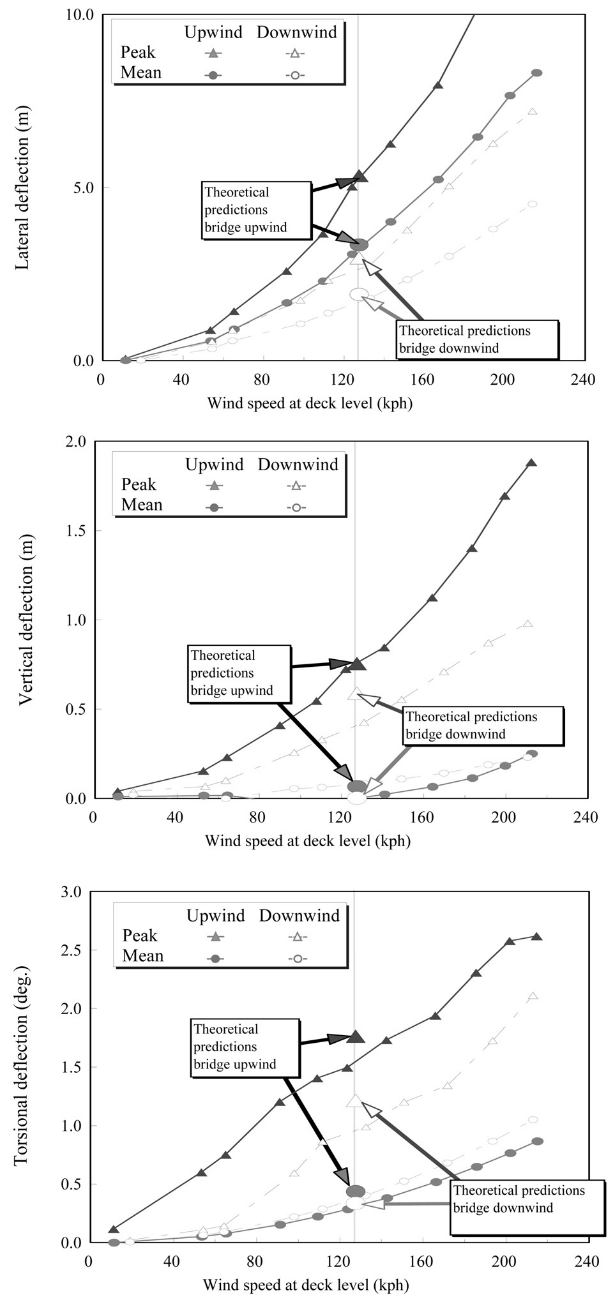


Figure 16. Existing bridge: measured and predicted responses at the middle of the main span.

were quite close for both upwind and downwind position. The vertical responses predicted in the downwind position were higher than the measured. Higher torsional responses were predicted in both positions.

For the proposed bridge (see figure 17) because of the much higher lateral stiffness, the peak lateral deflection was only 1.4 m. When the proposed bridge was positioned downwind, however, higher vertical and torsional responses were measured.

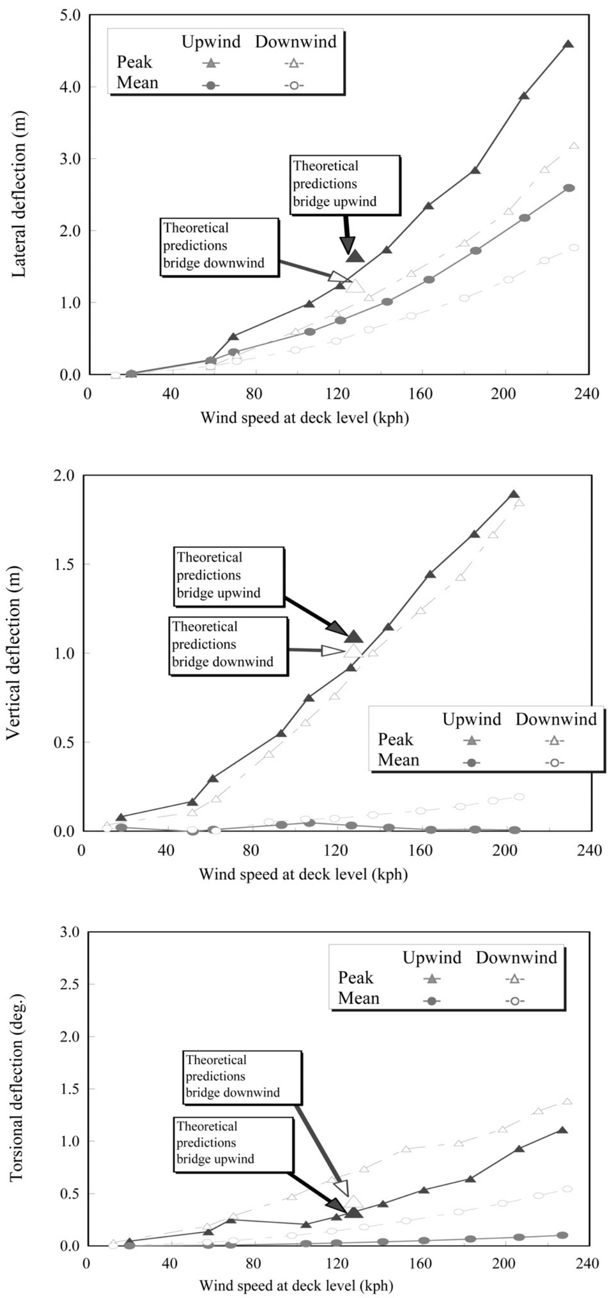


Figure 17. Proposed bridge: measured and predicted responses at the middle of the main span.

In both upwind and downwind positions the theoretically predicted lateral and vertical responses were somewhat higher than those measured of the model. The theoretical prediction of the increased torsional response measured in a downwind position was less successful. Perhaps in this case the self-induced turbulence from the upwind existing bridge was significantly modifying the torsional responses of the downwind proposed bridge.

Generally the comparison of the theoretical responses with those measured on the aeroelastic model was satisfactory for both bridges in upwind and in downwind positions. A gradual increase of the buffeting type response with the wind speed was observed without any signs of a switch to flutter or galloping. Flutter or any other instability was not observed up to at least 55 m s^{-1} (200 km h^{-1}). Wind directions 15, 20, 22.5, 33, and 45° off the normal to the bridge spans were tested for winds from both sides. There was no sign of flutter instability or any other unusual response up to the highest speed tested for all these directions.

6. Design wind loads

To evaluate the design wind loads, so-called "buffeting theory" (Davenport 1962, Irwin 1977) was employed, in which the excitation of each mode of vibration was estimated using spectral methods. Buffeting analysis in the time domain was also carried out (Stoyanoff 2001) in which, as shown above, the time histories of responses were numerically simulated. Static force and moment coefficients measured from the sectional model were used for the derivation of the design wind loads. In addition to the aerodynamic coefficients, the buffeting theory required knowledge of the natural modes of vibration of the bridge as well as information on the wind turbulence, such as turbulence intensities and integral length scales. Assumptions are also required on the aerodynamic admittance functions. Information on the modes of vibration was provided by the design team, while turbulence properties were deduced from ESDU (1985, 1986) and aerodynamic admittance functions were based on those described by Irwin (1977).

Figure 18 shows an example of an equivalent static load combination developed for the existing bridge. These loads consist of mean loads, direct gust or background loading, and inertial loading coming from excitation of the bridge's various modes of vibration.

For the existing bridge, 15 load combinations were developed, and for the proposed bridge 16 combinations. In these load combinations, the load patterns on the bridge were given as distributed vertical, lateral, longitudinal, and torsional pressures which had to be applied to the decks,

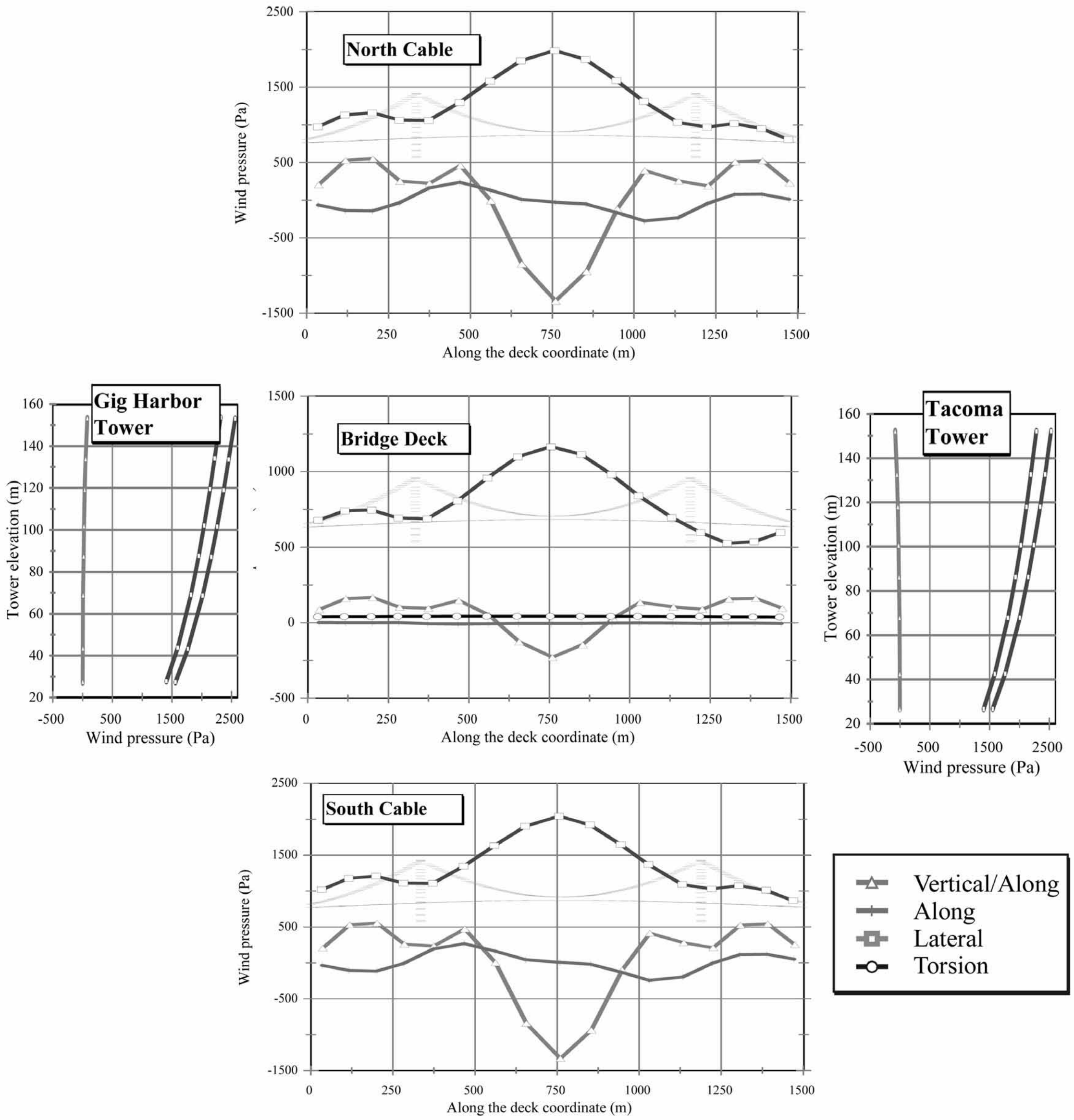


Figure 18. Wind load example: maximum symmetric vertical and lateral main span, existing bridge.

the cables, the towers, and the suspenders. Each load case represented an individual worst case in terms of symmetric and asymmetric vertical or lateral loading on the deck, on the towers, or torsion, with various combinations of vibration modes.

For both bridges in the upwind position, good agreement was found between the peak responses predicted theoretically and measured on the models. Since the responses of the existing bridge in the downwind position were lower, loads were not developed for this case. To account for the increased torsional and vertical responses measured on the proposed bridge in the downwind position, additional loading cases were developed using directly the deflections measured on the aeroelastic model.

7. Conclusions

The Tacoma Narrows Bridge site is imbued with tremendous historical significance from a wind engineering point of view. The investigations that followed the collapse of the original bridge more than 50 years ago can in many ways be said to mark the beginning of wind engineering as a specialized discipline and made bridge designers fully aware of what the dynamic interaction of wind and structure can do. Therefore the authors of this article felt it was a special privilege to work on this project. With the increased knowledge that we now have on the properties of the wind and structural response, the present studies have shown that despite their close proximity, the two bridges will not experience any major adverse aerodynamic interference effects. They have also demonstrated that on the existing bridge it will be possible to close off the aerodynamic vents, originally put there to enhance aerodynamic stability, and still satisfy the applicable aerodynamic stability criteria. Detailed wind load distributions were developed in these studies to assist the designers in developing safe and economical structural solutions.

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