Summary: Wind engineering studies of the Parallel Tacoma Bridges were undertaken by Rowan Williams Davies & Irwin (RWDI). Because the two parallel bridges will be separated by only 61 m, aerodynamic interference effects formed an important part of the studies, which examined the aerodynamic stability of both bridges. The historic collapse of the original Tacoma Narrows bridge in 1940 due to aerodynamic instability placed added emphasis on the importance of these studies. In addition to wind tunnel tests, full scale measurements were undertaken on the Existing Tacoma Bridge for identification of its essential structural properties such as modal frequencies of the lowest lateral, vertical, and especially torsional modes. In the wind tunnel tests sectional models were used initially. These examined the stability of the two bridges, and also examined the effect of closing off gratings on the existing bridge, this last measure being part of the conversion of the existing bridge to revised traffic lane layouts. Following the sectional model tests two full aeroelastic models were constructed and tested. The study program demonstrated that the bridges would be aerodynamically stable and provided detailed wind load distributions for the structural design.

Keywords: Parallel Suspension Bridges, Wind Engineering Study, Aerodynamic Stability, Wind Loads, Full Scale Measurements, Aeroelastic Model
1. Introduction

The first Tacoma Narrows Bridge was operational for only a few months before its spectacular collapse on November 7, 1940, at a wind speed of only about 19 m/s. During its brief life the bridge had been frequently observed to go into wind induced vertical oscillations and the final collapse was caused by large amplitude torsional oscillations. The famous aerodynamicist Theodore von Karman was hastily called in before the collapse to investigate the cause of the oscillations [1] but there was insufficient time for his research to develop a solution before the bridge’s final demise. The disaster triggered extraordinary efforts to understand the mechanism of failure, and this fundamental research 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 [2]. The aerodynamic lessons learned were incorporated into a replacement bridge constructed at the same site and opened to traffic on October 14, 1950.

Half a century later, substantial increases in traffic led to the decision to build a second bridge parallel to the one completed in 1950 (Fig. 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 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 original 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, a joint venture between Parsons Transportation Group and HNTB Corporation, retained RWDI. The studies included the following.

- Wind climate analysis including the effects of local terrain
- Sectional model test, including 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

![Fig. 1: Artistic rendering of the future Configuration (courtesy of Parsons/HNTB/WSDOT)](image-url)
Wind Tunnel Testing of the Parallel Tacoma Bridges

- 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 describes the results of these studies and the design, construction and testing of the aeroelastic models.

2. Wind Climate and Terrain Analysis

For this analysis historical data was obtained from three meteorological stations close to the site: i) Tacoma Narrows Airport located only 3 km west of the bridge, ii) McChord Air Force Base at about 19 km southwest, and iii) SeaTac International Airport located at 27 km to northeast. These stations had 26, 24, and 36 years of data 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 10 m height in standard open terrain. Fig. 2 shows the wind speed variation with return period from these studies and also shows the adopted design curve.

![Fig. 2: Hourly wind speed at 10 m elevation in open terrain vs. return period.](image)

It can be seen that the design curve lies above the curves implied by the local meteorological data, implying that it contained 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 better to be conservative than the other way round. Based on these considerations, the following design criteria were recommended.

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Application</th>
<th>Wind speed (m/sec)/(mph)</th>
<th>Averaging time</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>Structural Design for Construction</td>
<td>30.9 / 69</td>
<td>1 hour</td>
</tr>
<tr>
<td>100</td>
<td>Structural Design of Completed Bridge</td>
<td>35.3 / 79</td>
<td>1 hour</td>
</tr>
<tr>
<td>1000</td>
<td>Stability during Construction</td>
<td>44.7 / 100</td>
<td>10 min</td>
</tr>
<tr>
<td>10000</td>
<td>Stability of Completed Bridge</td>
<td>52.3 / 117</td>
<td>10 min</td>
</tr>
</tbody>
</table>

The stability criteria given in this table were for winds at zero angle of attack relative to the bridge deck.
During the studies of Farquharson and his team in the 1940’s, an important issue in the aerodynamic design of the replacement Tacoma Narrows 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 recorded with motion picture cameras [1]. The wind during that experiment was in the range of 7 to 12 m/sec 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 deflect due to the local topography, it was very conservatively decided that the bridge deck should remain stable for angles of attack up to ±15 degrees. To achieve stability over this range is very difficult, and the only way it was eventually achieved, after much wind tunnel testing, was by incorporating 3 rows of vents in the deck, covered by open gratings.

Since those days, however, subsequent studies [3, 4] have demonstrated that over open water and at high wind speeds, the probable wind inclinations would likely be in the order of “1 degree. 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 [5]. Therefore it has been common practice 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 “2.5 degrees, and 0.5 for “5 degrees 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 (Fig. 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 traffic, the effect of closing them on stability was also studied. Other minor modifications such as changes of the 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 flow (see Fig. 4). At the beginning the existing deck was tested alone and found to be very stable. Flutter was not observed up to 63 m/sec (225 km/hr). 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. This study showed no major adverse effects coming from aerodynamic interference effects. The proposed bridge was found to be very stable against flutter, being very stiff in torsion. 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

During the vibration measurements on the existing bridge, in addition to measuring the response to ambient excitation by traffic and wind, forced vibration tests were carried out. In the forced vibration tests, alternating forces at selected frequencies were imparted to the bridge in the absence of traffic, and the deck motions monitored. The forced vibration technique was based on the application of a large pendulum to excite the bridge. A similar approach was used on the Lions gate Bridge in Vancouver B.C. Canada [6]. This methodology provided much cleaner data for analysis and modal identification.

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 for all units. Each package was secured from displacing with a 22 kg sand bag placed on the 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 Fig. 5. 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 measurements were carried out over a one week period. The response of the bridge to normal excitations, i.e., wind and traffic, was recorded over the week. The force vibration tests took place during the night of the last day.

![Field measurement set up on the Existing Tacoma Bridge](image)

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

*Fig. 5. Field measurement set up on the Existing Tacoma Bridge*

Fig. 6 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.

![Cross-spectrum of middle & quarter span](image)

*Fig. 6 identification of 1st lateral and vertical modes from ambient data*

The identified first lateral and vertical frequency were almost identical with the predictions of numerical models and coincided with those previously measured by Prof. N.P. Jones of Johns Hopkins University [7]. However identification of torsional modes 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 test, the wind was rather strong, peak speeds of up to 16 m/sec were registered, and the traffic was heavy to very heavy for most of the time. During the night of the force vibration test however, wind speeds of at most 5 m/sec were registered, dropping down to about 2.5 m/sec when the force vibration test was completed. For the time of the 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 (see Fig. 7). By adjusting the length of the chain, the frequency of the pendulum was varied in the range of 0.24 Hz to 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 ±30° arch, creating an oscillating peak force of about 5560 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 the decay of deck motions was recorded. Fig. 8 shows identified torsional modes at 0.33, 0.38 and 0.4 Hz.

![Fig. 7 Forced vibration set up, active mass 1315 kg.](image)

![Fig. 8 Cross-spectrum and phase of torsion from force vibration test.](image)
The close to zero phase found between the mid-main span stations 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 1st 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 was used for full scale 3D predictions. Due to the complexity of the problem, simplifications were necessary that needed verifications. For example only one of the two deck models at a time was moving dynamically. There were also concerns for instabilities and high loads on the closely spaced towers due to wake interactions. For these reasons full aeroelastic models of both bridges were built and tested side by side in the wind tunnel (see Fig. 9).

Following an established dynamic similarity rules [8], both aeroelastic models were designed and constructed at the RWDI's model shop facilities. Considering the length of the two bridges (existing - 1524 m, proposed - 1646 m) a large wind tunnel facility was required. The 9 x 9 m wind tunnel of the National Research Council (NRC) in Ottawa was selected that 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 (see Fig. 10).

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 2.5x2.5 mm grown hollow which allowed small brass rods to be inserted later to achieve the proper mass distributions. The most intricate element to model was the connection of the deck to the towers of the existing bridge. 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 (see Fig 11).

Fig. 10. Model details – end pier supports of the Tacoma Bridges’ aeroelastic models.

Fig. 11. Model details - connection of the existing Tacoma Bridge truss to the tower
The horizontal force 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 (Fig. 12).

Fig. 12 Model details – main cables connected to towers

The models successfully reproduced the bridges’ geometry and structural dynamics. The model natural frequencies matched the computed full scale frequencies, appropriately scaled, to with 5 %. The two models were shipped about 600 km away to Ottawa and installed in the NRC’s 9 m x 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. Figs. 13 and 14 show the instrumentation of the existing bridge model. Both models were instrumented following a similar scheme.

Fig. 13 Instrumentation schema (21 points=6 lasers+5 accelerometers+10 strain gauges)
A turntable was present on the tunnel floor allowing the models to be rotated relative to wind direction and accommodate all instrumentation cables. In total 45 channels of instrumentation, including 3 speed sensors, were installed.

The study began with the existing bridge upwind tested in smooth flow. Both bridges were found to be stable up to at least 55 m/sec (200 km/hr). Tests at higher wind speeds were not carried on 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 (Fig. 15). The mean velocity profile was characterized by a power law exponent $\alpha = 0.14$ and turbulence intensities 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 $L_u = 175$ m, $L_w = 25.3$ m, $y_u = 25.5$ m, and $y_w = 19.4$ m. These parameters were considered to represent well the expected turbulence at the site.

In Fig. 16 are shown the mid-span responses of the existing bridge in torsion, and in the vertical and lateral directions are shown. Results are shown for the existing bridge both
upwind and downwind of its partner. When the existing bridge was upwind, at the design speed of 35.3 m/sec (127 km/hr), the peak lateral deflection reached 5.2 m. This deflection was reduced to 2.7 m for the downwind position.

Fig. 16 Existing Tacoma Bridge – Peak and mean responses at the middle main span

The vertical and torsional responses also became smaller in the downwind position. Flutter or any other instability was not observed up to at least 55 m/sec (200 km/hr).

For the proposed bridge (see Fig. 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. A gradual increase of the buffeting type response with the wind speed was observed without any signs of a switch to flutter or galloping.
Wind directions 15, 20, 22.5, 33, and 45 degrees 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. Wind Loads for Structural Design

The wind loads required for structural design were derived from theoretical predictions and also directly from the aeroelastic tests. To estimate the overall loading effects on the two bridges, simplified wind loads were provided based on linear combinations of the dynamic loads in the various modes of vibration. Theoretical buffeting analysis was carried out to estimate the responses to wind buffeting [9, 10]. The input parameters were the static aerodynamic force coefficients from the sectional tests, mass and polar moment of inertia, cross-sections' dimensions, modal frequencies and shapes, structural damping, and wind turbulence properties. Fig. 18 shows an example of a quasi static load combination developed for the existing bridge.
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, 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 modal combinations.

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

7. Conclusions

For bridge designers and wind engineers the site of the Tacoma Narrows Bridge is imbued with tremendous historical significance. The investigations that followed the collapse of the original bridge 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 present authors 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 shown 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.

8. Acknowledgements

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References


