Wind study for City Island cable-stayed bridge over Eastchester Bay in the Bronx, NY

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ABSTRACT: The existing seven-span bridge connecting the Bronx and City Island was opened to traffic in 1901. The existing bridge is planned for replacement by a new single-span asymmetrical cable-stayed bridge with a span length of 185 meters and a width of 20.85 meters. The steel tower (pylon) will be 50 meters high above the composite concrete deck and will be placed in Pelham Bay Park in the Bronx. The minimum vertical clearance above the water will be 4.50 meters. A wind study was initiated to determine: 1) the response of the bridge to vortex induced motions; 2) critical flutter wind speeds; and 3) buffeting responses of the bridge to design wind speeds, in its final configuration. The study also aimed to determine mitigating measures, if needed, to make the performance of the bridge acceptable. This paper presents the details and results of the study.

1 EXISTING CITY ISLAND BRIDGE

Figure 1 shows City Island Bridge in relation to LaGuardia Airport in New York and Newark Airport in New Jersey. Figure 2 shows the bridge location plan. The existing bridge connects Pelham Bay Park in the Bronx to City Island in the Bronx It was opened to traffic in 1901 and consists of seven spans. Spans two and three were swing spans which were made fixed in the 1960s. The bridge is 173.28 meters long and 15.10 meters wide with three substandard travel lanes. Figures 3 and 4 show the elevation and an aerial view of the bridge. The bridge has outlived its useful life. It is not possible to replace one-half of the bridge at a time due to width limitation. It was decided by the New York City Department of Transportation (NYCDOT) to replace the existing bridge with a new cable-stayed bridge at the same location.

Figure 1. Area map showing City Island Bridge in relation to LaGuardia and Newark Airport.

Figure 2. Bridge location map

Figure 3. Elevation of existing bridge looking northeast.

Figure 4. Aerial view of existing bridge looking northeast.

2 PROPOSED CITY ISLAND BRIDGE

Gandhi Engineering, Inc. (Gandhi) was selected by the NYCDOT to design the new replacement bridge. The NYCDOT retained the services of Dr. Michel Virlogeux of France as a peer-review consultant to review the work of Gandhi at various stages of completion and offer comments for improvements. Renderings of the new bridge show the side elevation and front elevation (Figures 5 and 6).

The proposed City Island Bridge is a single span asymmetrical cable-stayed bridge with its single tower located in Pelham Bay Park. The width of the steel bridge with a composite concrete deck is 20.85 meters, the height of the tower is 50 meters above the deck, and the span length is 185 meters.

Because the existing bridge was converted from a swing bridge into a fixed bridge, it has a minimum vertical clearance of about 4 meters above Eastchester Bay. The new bridge is designed with a minimum vertical clearance of 4.50 meters because only pleasure boats pass under it. Even though the vertical clearance is comparatively low, it is susceptible to action of wind because of its asymmetrical geometry and span length. Hence, it was decided to perform a wind study to evaluate the performance of the proposed bridge in strong winds.

Figure 5. Rendering of the new bridge – side elevation looking north.

Figure 6. Rendering of the new bridge - elevation looking west.

3 HISTORY OF WIND ACTION ON CABLE-SUPPORTED BRIDGES

Before we proceed to describe the wind tunnel tests, we would like to review the history of wind action on suspension bridges. John A. Roebling, the father of modern wire suspension bridges, understood the dynamic behavior of a suspension bridge subjected to high winds more than 160 years ago. In 1846 Major Charles B. Stewart, who was planning a bridge over the Niagara Gorge, sent a circular letter to prominent bridge engineers in Europe, Canada, and the U.S. asking their opinion about the feasibility of a suspension bridge over the Niagara Gorge (Stewart 1871). Only four engineers, Charles Ellet, Jr., John A. Roebling, Samuel Keefer, and Edward W. Serrell, replied by saying that it was feasible. It is interesting to know that each one of these four engineers subsequent to this initial inquiry designed and built a suspension bridge below the falls confirming their realistic optimism.

On January 7, 1847 when Roebling responded to Stewart, he already had prepared a design of a stiffened suspension bridge (Figure 7a). In 1801 James Finley was the first known builder in the western world to build a suspension bridge with a stiffening truss over Jacob's Creek between Fayette and Westmoreland counties in Pennsylvania (Kawada 19). Roebling was the first one to use a stiffening truss and stays in the design of a suspension bridge (Gandhi 2006). In May of 1847 he wrote "Specifications of the Niagara Bridge" which corroborated his drawings of January 1847, and he provided abovefloor and underfloor stays to stabilize his bridge against the wind (Roebling 1847).

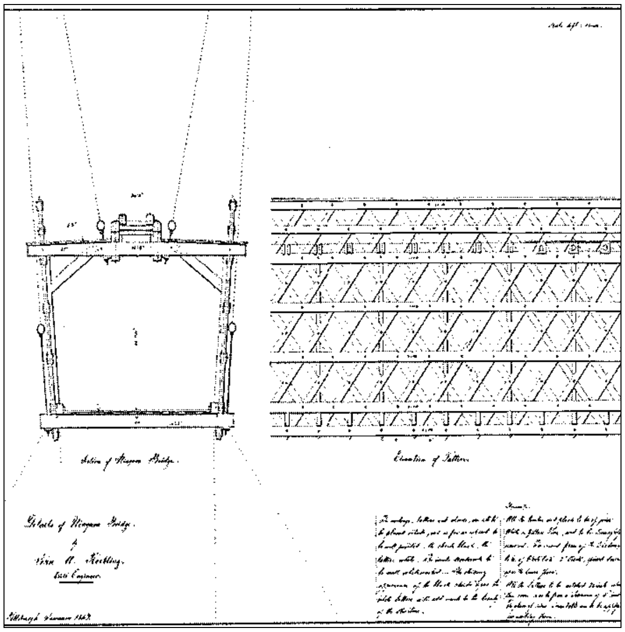

Figure 7a. Stiffened Railway Suspension Bridge by Roebling – January 1847.
(RPI Archives, Roebling Collection, Troy, NY 12180)

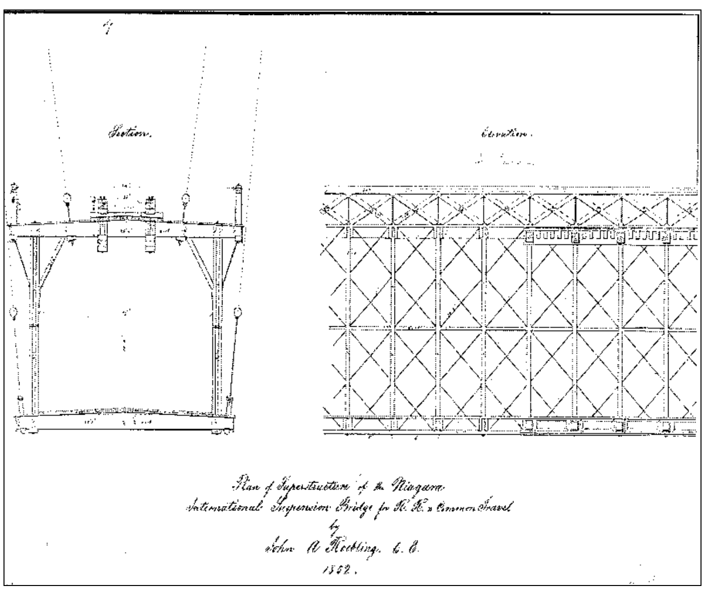

Figure 7b. Stiffened Suspension Bridge by Roebling – 1852
(RPI Archives, Roebling Collection, Troy, NY 12180).

Roebling was approached by Stewart in 1852 to build the Niagara Bridge. Roebling's refined design is showed in Figure 7b. He clearly understood the principle of "torsional rigidity" and expressed it in his report as follows (1852):

Since you have decided in favor of a double floor, the upper one to be used for the rail road, the lower one for common travel, a very good opportunity has offered for doubling the trusses, and of adding another valuable feature, that of the box or tube. Your bridge will form a hollow straight beam, of 20 feet wide and 18 feet deep, composed of top, bottom, and sides.

Roebling's bridge was opened to traffic on March 18, 1855. On May 17, 1854 the Wheeling Suspension Bridge designed by his competitor Ellet was destroyed due to high winds. After watching the performance of his bridge for more than a month, Roebling submitted his final report on May 1, 1855 in which he summarized the four basic means employed by him to design the Niagara Bridge (Roebling 1855):

The means employed are: Weight, Girders, Trusses, and Stays. With these any degree of stiffness can be insured, to resist either the action of trains, or the violence of storms, or even hurricanes; and in any locality, no matter whether there is a chance of applying stays from below or not. And I will here observe, that no suspension bridge is safe without some of these appliances.

After the collapse of the Tacoma Narrows Bridge in Washington State on November 7, 1940, there was a world-wide interest in analyzing motions of suspension bridges subjected to high winds using analytical and experimental methods. One of the findings, using Roebling's words, was "there is no bridge in the world, neither of stone cast or wrought iron, which is free from all vibrations." These long-span bridges are subject to aerodynamic forces generated by structural motions. These motions, referred to as "self-excited", are in turn affected by the aerodynamic forces they generate. Roebling, although not aware about the aerodynamic forces, had the uncanny ability to visualize the failure mechanism of the Wheeling Bridge and addressed it in the report as follows:

Weight is a most essential condition, where stiffness is a great object, provided it is properly used in connection with other means. If relied upon alone, as was the case in the plan of the Wheeling Bridge, it may become the very means of its destruction. That Bridge was destroyed by the momentum acquired by its own dead weight, when swayed up and down by the force of the wind.

4 DETERMINATION TO PERFORM WIND STUDY

At the 60% design completion stage it was agreed by Gandhi and Michel Virlogeux, the Peer Review Consultant to the New York City Department of Transportation (NYCDOT), the need for a wind study. West Wind Laboratory, Inc. (WWL) of Marina, California was selected by the NYCDOT to perform the wind study.

WWL was assisted for its analytical study by 1) Applied Research Associates, Inc. of Raleigh, North Carolina to perform the wind risk study for City Island, and 2) Weidlinger Associates, Inc. of Marina del Rey to assess cable vibrations and to recommend mitigation measures, and which in turn subcontracted with Partha P. Sarkar of Iowa State University for this assignment.

Gandhi provided WWL a set of 60% design drawings, a detailed finite element model of the bridge, and the first 22 dominant mode shapes and frequencies of the bridge. The plan and elevation, as well as the typical cross section of the bridge, are shown in Figures 8 and 9 respectively.

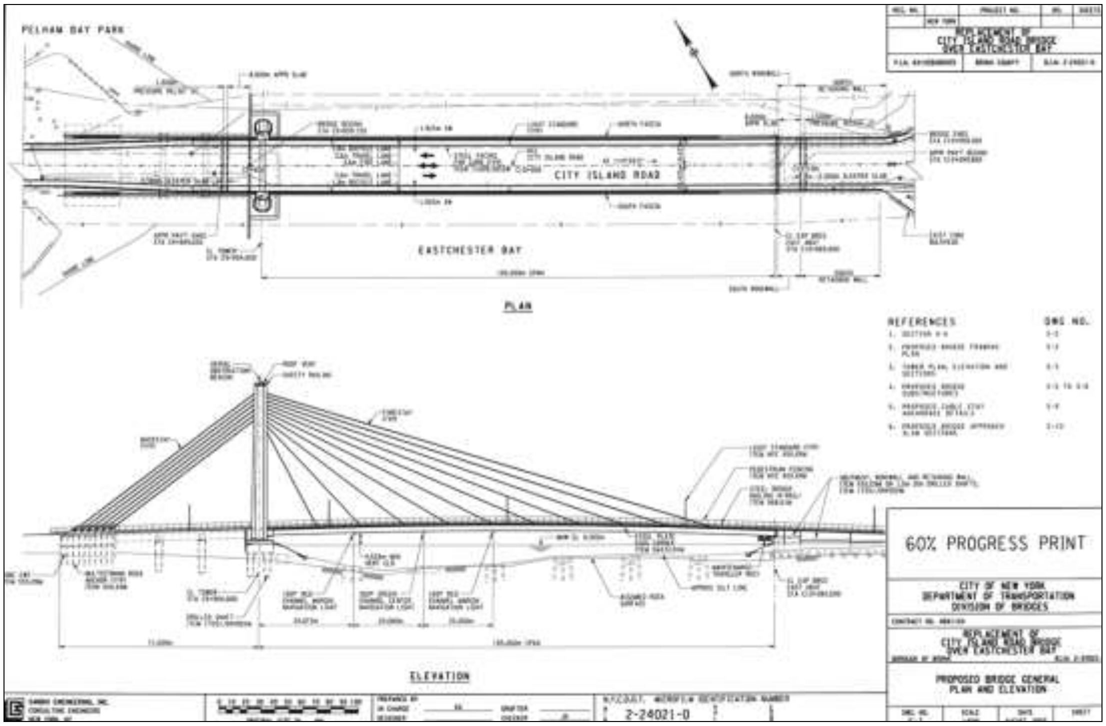

Figure 8. Plan and elevation of the bridge.

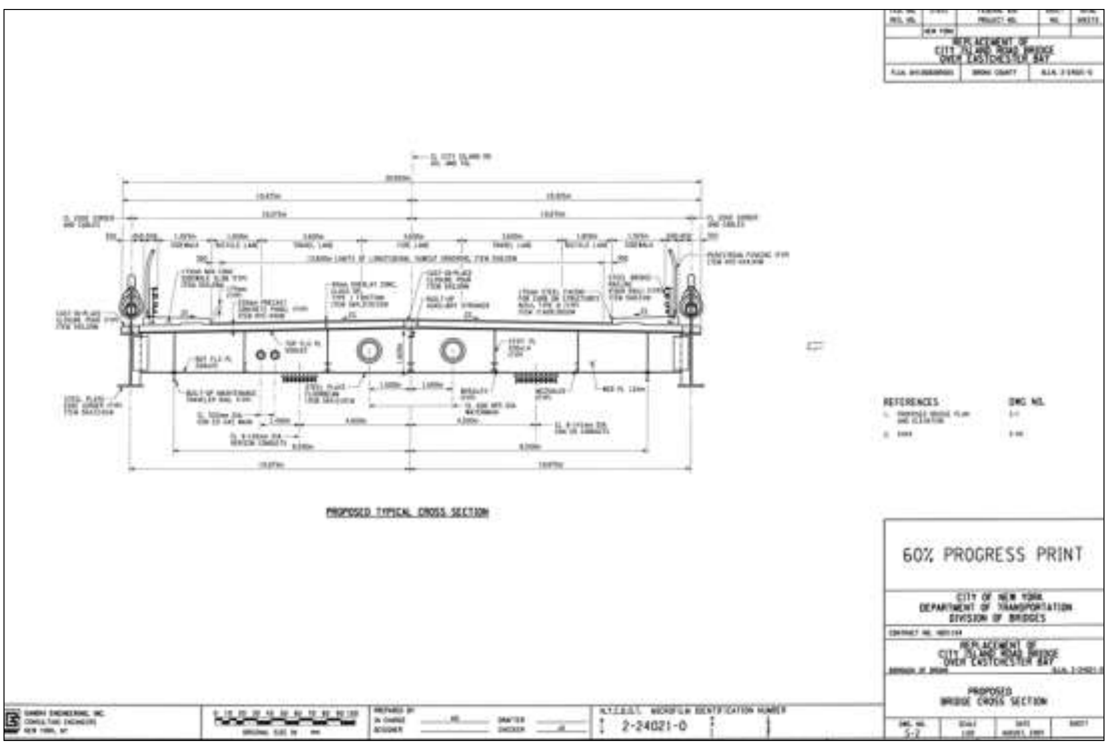

Figure 9. A typical cross-section of the bridge.

The bridge with its 2.44 meter (8 feet) high safety fence with a very fine mesh offered a very bluff body to wind acting normally to the bridge. Hence properties of the fence and railing were very important in developing a section model of the bridge. Figures 10 and 11 provide the fencing and railing details, respectively.

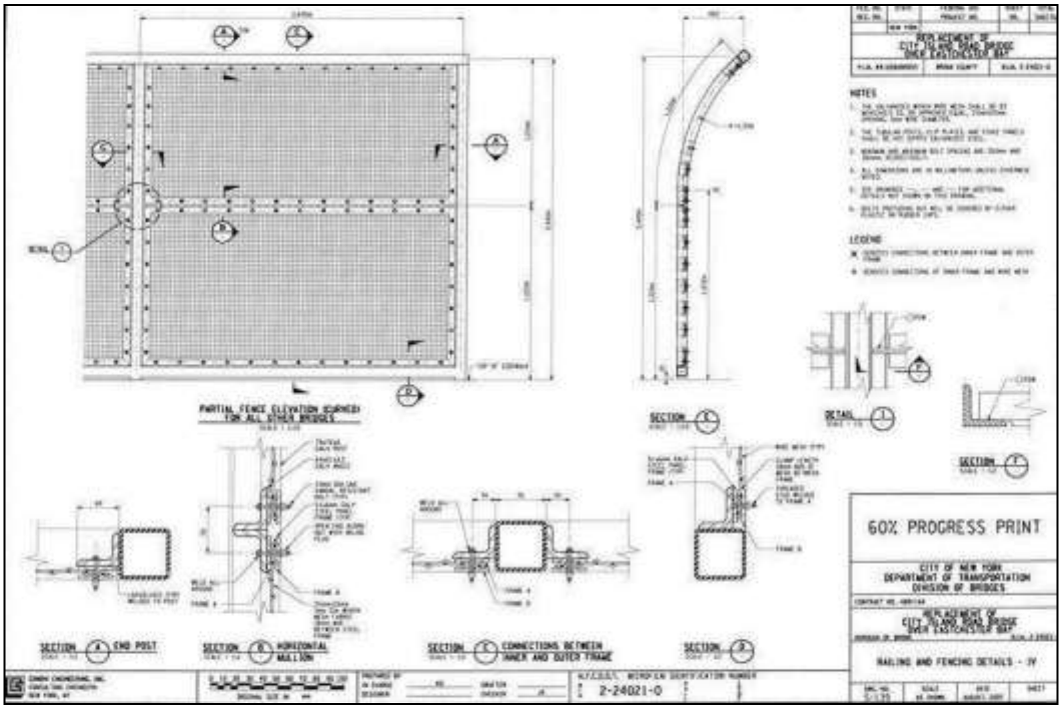

Figure 10. Fencing details.

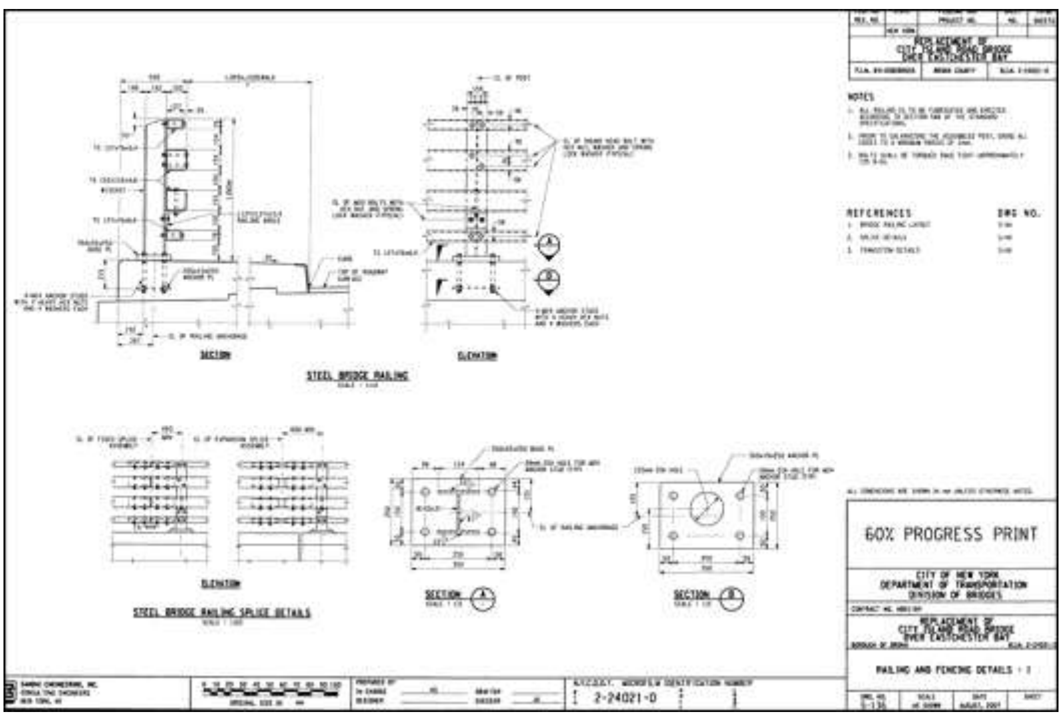

Figure 11. Railing details.

5 TYPICAL WIND INDUCED PROBLEMS

Cable supported, long-span bridges typically are flexible and often experience large wind induced vibrations. There are four typical types of vibrations: 1) large amplitude vibrations resulting from periodic vortices shed into the wake; 2) random buffeting vibrations from the turbulence in the wind (and in the wake); 3) diverging, unstable oscillations that can lead to the catastrophic failure of the bridges; and 4) excessive cable vibrations. Vortex induced motions typically are not a structural problem but can be discomforting to pedestrians and drivers of vehicles. If allowed to persist, they can lead to fatigue failures. All winds are turbulent, and all winds will generate a buffeting response. That buffeting response was used by the design engineers, Gandhi, for the design of the bridge structure. An aeroelastic instability (like that which destroyed the original Tacoma Narrows Bridge) is to be avoided at all cost. Cable vibrations can be large and discomforting to observe. If allowed to persist, large cable vibrations can also lead to fatigue failures. All four typical wind induced motions were investigated for the replacement City Island Bridge. Although the procedures used for this study were standard, there were some potential "un-standard" wind induced behaviors that needed to be investigated.

First, the clearance below the bridge is only 4.5 m. The behavior, in a strong wind, of a flexible bridge that close to the water is unknown. And second, the projection of the bridge to the wind is quite bluff. There are 2.2 m deep edge girders that are bluff, and on top is a 2.4 m high security screen. The porosity of the security screen is 0.58 making the screen quite opaque to the wind. The behavior of the bridge projecting this bluff front was unknown.

6 WIND SPEED CRITERIA

Wind speed criteria were developed specifically for this project from an analysis of historical wind speeds, and from simulations of expected hurricane winds (Raggett, 2008, Appendix 11).

It is usual to use the wind speeds shown in Table 1 for the design of the bridge in its final and construction stage configurations.

Table 1. Wind speed criteria.

Case	Return period (yrs)	Averaging time (sec)
Construction stage service load design	10	3600
Final stage service load design	100	3600
Construction stage stability	1000	600
Final stage stability	10000	600

The predicted omnidirectional, one hour-averaged (3600 seconds), wind speeds at an elevation of 10 m, in an "open" environment (Exposure C), for which a surface roughness of z_0 = 0.03 m is reasonable, are shown in Figure 12 (Raggett, 2008, Appendix 11). For design the "combined" curve was used.

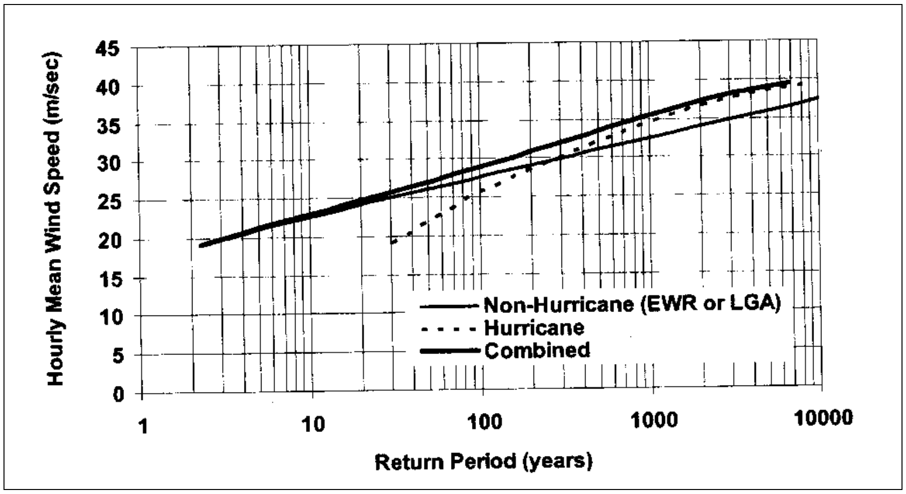

Figure 12. Wind speed v. return period.

The one-hour averaged wind speeds from the "combined" analysis as a function of return period are given in Table 2.

Table 2. Average wind speed as a function of the return period

Return period	U (m/s)	(mph)
10	23.2	51.9
100	29.0	64.9
1000	35.8	80.1
10000	40.0	89.5

The average elevation of the bridge deck above the water surface is about 7 m. It has not been the custom of West Wind Laboratory to reduce wind speeds for elevations less than 10 m, so wind speeds at an elevation of 10 m were used for the design wind speeds. The reference wind speeds presented are for winds in an "open" (Exposure C) environment. In both upwind directions to the bridge, the exposures are over a smooth water surface (with some beneficial effects from the park lands and from City Island itself). The upwind exposures can be assumed to be "open" (an "open" exposure - Exposure C - is recommended in ASCE/SEI 7-05 to be the proper exposure for all water surfaces in hurricane prone areas). Therefore, the predicted one-hour averaged wind speeds can be used without corrections as the appropriate one-hour averaged design wind speeds for the City Island Bridge (Raggett, 2008, Appendix 11). It can readily be shown that the corresponding 10-minute averaged wind speed (600 second), at 10 m in this environment, is 1.061 times the one-hour averaged wind speed (Simiu 1996). The final recommended design wind speeds at the bridge deck elevation, at the bridge site, are shown in Table 3.

Table 3. Recommended wind speed criteria.

Case	Return period (yrs)	Averaging time (sec)	U(m/s)	(mph)
Construction design	10	3600	23.2	51.9
Final design	100	3600	29.0	64.9
Construction stability	1000	600	38.0	85.0
Final stability	10000	600	42.4	94.8

For buildings, ASCE/SEI 7-05 recommends that the reference wind speed not be less than the wind speed with a 500 year return period divided by the square root of 1.5. This 3-second averaged wind speed is 39.8 m/s (89 mph) (Raggett, 2008, Appendix 11). The reference wind speed is assumed to have a return period of 50 years. A wind speed with a return period of 100 years is 1.07 times the 50 year wind speed. Therefore, the low end, 100 year, 3-second design wind speed should not be less than 42.6 m/s (95.23 mph) (Simiu 1996). The corresponding one-hour averaged wind speed (at an elevation of 10m in an "open" environment) is 28.8 m/s (64.3 mph). The 100 year, one-hour averaged wind speed presented in Table 3 of 29.0 m/s is critical.

It should also be noted that the reference 3-second averaged wind speed in ASCE/SEI 7-05 for City Island is approximately 46.9 m/s (105 mph). This again is a 50 year wind speed. The corresponding 100 year wind speed is again 50.2 m/s (112.4 mph). The 100 year one-hour averaged wind speed of 29.0 m/s (64.9 mph) was predicted, with a corresponding 3-second averaged gust wind speed of 43.0 m/s (96.1 mph) (Raggett, 2008, Appendix 11). Note that the site specific produced design wind speeds are .86 of the values recommended, for buildings, in ASCE/SEI 7-05 (Raggett, 2008, Appendix 11).

7 PROCEDURES USED

The procedures used to predict the behavior of the bridge in strong winds are mostly analytical. An analytical model of the wind speed flow field described by Yinghong (2000) was used. Alongwind and vertical wind speed time histories were generated at 22 locations along the bridge deck. Wind speed time histories for a duration of 5 minutes each (15,000 data points at a 0.02 second interval) were generated. Five wind speed flow fields were generated.

The mechanical character of the bridge was described with the mass distributions and dynamic response characteristics (mode shapes and frequencies) provided by Gandhi.

Aerodynamic loads on the bridge (primarily the bridge deck including contributions from the cables) were determined at the 22 locations along the bridge deck from the wind speeds at those locations, the static aerodynamic coefficients (drag, lift, and moment coefficients), and the dynamic aerodynamic coefficients (aeroelastic flutter derivatives). At any instant in time, the total aerodynamic actions at each node were generated. Generalized actions were then computed for each mode of vibration. The response of each mode was then computed 0.02 seconds later. The total bridge response at that time was then computed as the linear combination of the modal responses. From the total bridge response (displacements and velocities) at that time, new aerodynamic loads were computed at each of the nodes and the process was then repeated. This process was repeated 15,000 times generating a complete response time history of the bridge.

Numerical simulations were generated at the design wind speed in a turbulent wind speed flow field. These time histories simulated the buffeting response of the bridge to a turbulent design wind. Statistics of the response were then used to generate an expected static equivalent response for the strength design of the bridge.

Numerical simulations were also generated in smooth flow for a range of wind speeds up to, and exceeding the critical flutter wind speed threshold. Each mode of vibration was given an initial unit displacement. If the subsequent motions decayed with time, the bridge would be stable. If, at that wind speed, the motions of any one mode grew without bound, the bridge would be unstable.

All buffeting responses and stability responses included all modal vibrations simultaneously. Any aerodynamic coupling that may exist would be identified in the simulations. The aeroelastic flutter derivatives obtained determine the amount of aerodynamic coupling that may exist.

The aeroelastic flutter derivatives cannot be generated readily analytically. They were measured experimentally in the wind tunnel using a large-scale (1:40) model of a section of the deck. At this large scale, small details in the bridge deck can be modeled with accuracy. Very small details have been shown to have a significant influence on the behavior of a complete bridge in strong winds (Raggett 2004). It should be emphasized that the section model in the wind tunnel was not used to simulate the bridge behavior, but it was used only to obtain the necessary static and dynamic aerodynamic coefficients

8 FACILITIES AND MODEL

The West Wind Laboratory, Inc. owns and operates its own wind tunnel. Wind studies are performed in the 1x4 m open return type atmospheric boundary layer wind tunnel designed specifically for bridge section model and full-bridge model testing. Drawings of the wind tunnel are shown in Figure 13. Wind speeds are continuously variable from 0 to 6.1 m/s.

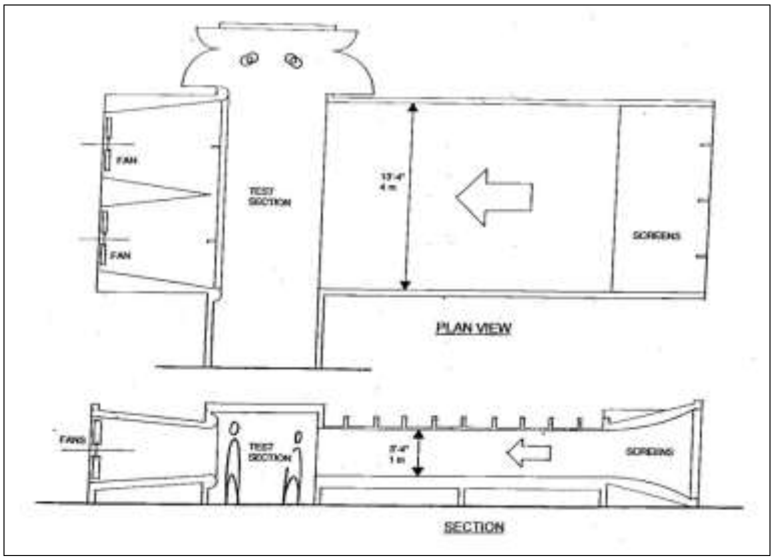

Figure 13. 1x4 m Atmospheric boundary layer wind tunnel

The test section is open without walls or a ceiling. Ambient pressures within the test chamber therefore are essentially constant. Furthermore, winds can flow around and over the models without constriction (as in the full-scale environment). Therefore, blockage effects are minimal, i.e., wind speed will not be artificially accelerated around the model because there are no walls to constrict and accelerate the flow.

The wind tunnel extends 6.1 m upstream from the test section without flair or constriction. Atmospheric boundary layers can be generated in this space with the use of spires and blocks on the wind tunnel floor.

Model displacements, and force transducer displacements are measured with Macro Sensors PRH-812-050 LVDT Transducers and Macro Sensors LPC-2000 Signal Conditioners. Mean wind speeds are measured with a Sierra Instruments Model 618 Air Velocity Meter. Mean and fluctuating wind speeds are measured with a total head tube and Setra System, Inc. 239 Pressure Transducer.

Analog signals from the transducers are digitized on a ComputerBoards PCM-DAS08 Analog to Digital Converter.

A 1:40 rigid section model of a typical portion of the deck, including a full complement of utilities hung underneath, was constructed of wood. The safety fence has a structure, and a mesh with 3 mm round members in both directions, with a spacing in both directions of 25 mm. The solid ratio of this fence, including its structure (ratio of projected solid area to gross area) is a high 0.42. One cannot model such a fence to be geometrically similar because the openings in the grid (0.55 mm) would be so small, Reynolds Number effects in the flow through those openings would be significant (the relative viscosity of air flowing through the small openings would not match the relative viscosity of the air flow through the actual openings). Therefore, the fence was modeled with a grid of round members of 4.76 mm diameter, at a near full-scale spacing of 20 mm (both directions) yielding a total solid ratio of 0.419. This fence is not geometrically similarly to the actual fence, but will be aerodynamically similar.

Because the water surface is so close to the bottom of the bridge, a water plane was also modeled.

For the collection of aeroelastic flutter derivatives, the model was purposely made to be light to maximize the ratio of aerodynamic to inertial loads (to maximize the resolution in the results). For the vortex induced motion studies the model was weighted to its proper weight (and frequency) ratios according to the laws of similitude. For the vortex induced motion studies, the model properties are presented in Raggett, 2008, Chapter D. For the collection of the aeroelastic flutter derivatives, the model properties are those shown below:

Vertical Motion	Torsional Motion
$n(Hz) = 1.3943$	$n(Hz) = 1.3943$
$\xi = 0.003610$	$\xi = 0.002891$
$m\left(\frac{kg}{m}\right) = 5.8665$	$mmi\left(\frac{kg \cdot m^2}{m}\right) = 0.19848$

The model in the wind tunnel, and water plan, are shown in Figures 14 and 15, respectively.

Figure 14

Figure 15

9 VORTEX INDUCED MOTIONS

Long, slender, flexible, prismatic bridges immersed in, and perpendicular to, a steady wind potentially may experience discomforting vortex induced motions. Typically, such motions are discomforting to pedestrians and drivers of vehicles, but rarely cause structural damage unless allowed to persist over a long period of time (and may cause fatigue failures). The proposed City Island Replacement bridge has deep, prismatic edge girders that could generate a periodic vortex in its wake, that could generate large discomforting motions (if the edge girders existed by themselves). However, there are large cable anchor plates above the deck, there is an open rail pedestrian railing, and there is a fairly opaque security fence, all of which tend to break-up the formation of uniform vortices shed into the wake. The security fence has a very high solid ratio of 0.42 (porosity ratio of 0.58). A wind tunnel test using the large-scale section model was performed to evaluate the degree of the help from the cable anchors, pedestrian railings, and security fence.

For this test, the section model was used to simulate full-bridge behavior. It was weighted so its scaled mass and mass moment of inertia matched target values, and so the ratio of fundamental to torsional fundamental frequencies matched target values. Modeling damping (0.0036 for vertical motion and 0.0029 for torsional motion) was less than target full bridge value of 0.0048. Results were adjusted accordingly.

Peak vortex induced modal motions are typically about 1.5 times the peak section model motion (typical fundamental mode shape factor), when the section model is used to simulate full-bridge behavior. Typically, vortices shed from a bridge deck are not fully correlated over the length of the bridge (as they might be over a section model). Therefore, section model results typically were not scaled by the mode shape factor, 1.5.

Vertical accelerations greater than 0.05 g are considered to be uncomfortable for mean wind speeds less than 13.4 m/s (30 mph); and 0.10 g, for wind speeds between 13.4 m/s and 22.4 m/s (50 mph). For mean wind speeds greater than 22.4 m/s, the wind itself is considered to be more discomforting than the motion. These criteria are assumed to be appropriate for evaluating vortex induced motions for the City Island Replacement Bridge.

Turbulence generally is beneficial, i.e., it tends to reduce vortex induced motions. However, it is not the policy of the West Wind Laboratory to rely upon turbulence to mask a vortex induced problem. Therefore, testing was performed both in smooth and turbulent flow.

Shown in Figure 16 are the vertical (H) and torsional (T) vertical accelerations due to vortex induced motion in smooth flow. For torsional motions, the vertical acceleration is that which would occur at the outboard edge, i.e., the walkway. There are distinct vortex induced motion peaks at a mean wind speed of 17.6 m/s (39.4 mph), but they are both well below the criterion of 0.10 g at this wind speed. Shown in Figure 17 are expected vertical and torsional motions in turbulent flow. The motions are in general greater than they are in smooth flow, but there are no distinct vortex induced motion peaks. All motions are well below the motion criteria for all wind speeds.

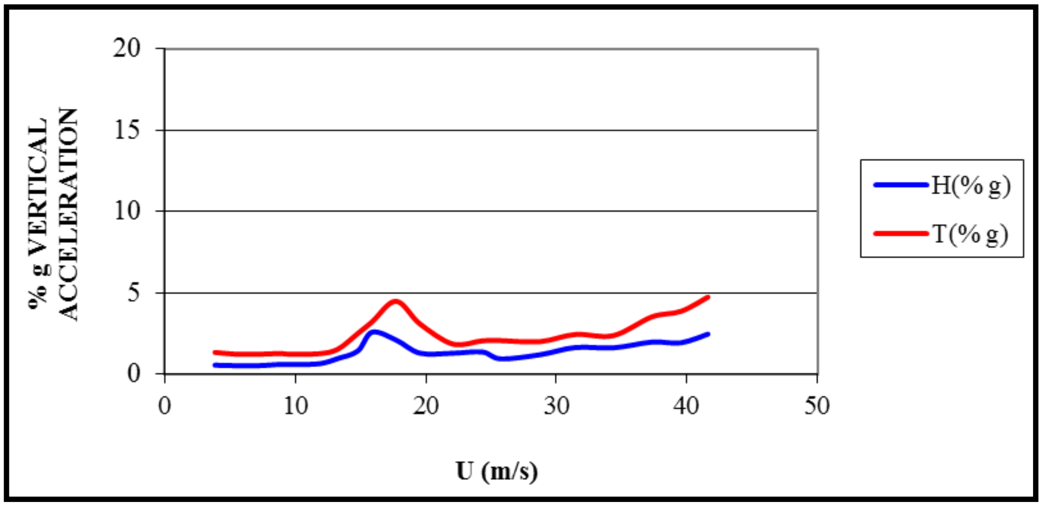

Figure 16: Smooth Flow

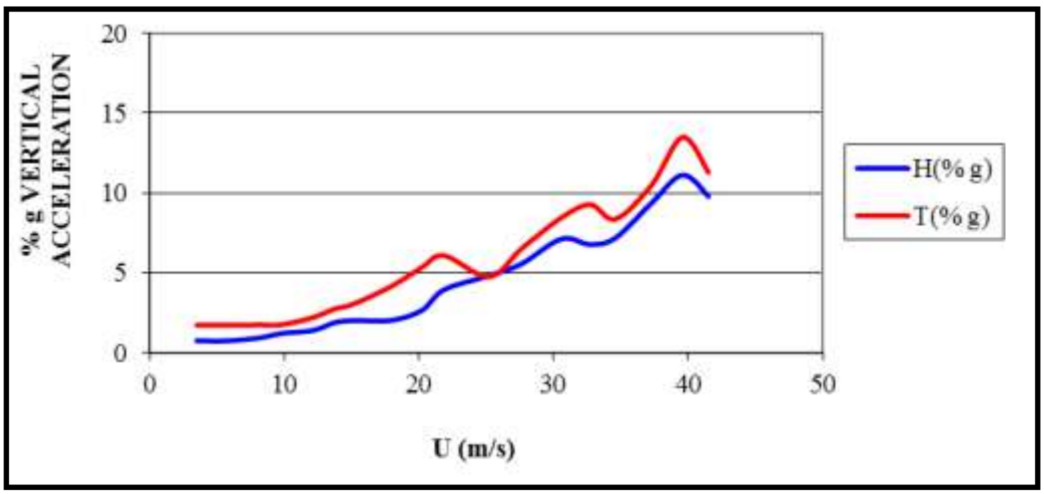

Figure 17: Turbulent Flow

10 BUFFETING RESPONSE

Five response time histories (to design level, turbulent winds) were numerically simulated using the wind speed time histories generated as described in Figure 18. Each time history had the same mean wind speed, the same turbulence intensity, the same power spectral density, but each had a different set of phase angles used in the generation of the wind speed time histories. They were different samples of a random process that were otherwise similar. Each simulation was for a full-scale equivalent duration of 300 seconds (5 minutes) with 15000 points and a digitizing interval of 0.02 seconds.

Each wind speed time history represented the fastest 5-minute averaged wind in a single 100-year event that had a one-hour averaged wind speed of 29 m/s (64.85 mph). This is shown schematically in Figure 18. The corresponding, fastest, 5-minute averaged wind speed in the simulation was 31.68 m/s (70.85 mph). The five response time histories were responses to the fastest 5-minute wind speeds in 5, 100-year events. Because the bridge periods are short, in a 5-minute time history the bridge is expected to achieve a steady-state response as though that wind speed had occurred from minus infinity to plus infinity, i.e., a stationary random process.

For each mode of vibration, the average model response was determined (forced to be identical for all five time histories). Also for each mode of vibration, the average of the five peaks was determined (PAVE), the standard deviation of those five peaks away from the average peak was determined (PSIG), and the maximum peak of the five peaks was determined (PMAX). These are presented in Table 4.

For design, it is reasonable to use a value that is PAVE + (n*PSIG) where n is some number (2, 2.5, 3, and 3.5). Those combinations are shown respectively to be PK20, PK25, PK30, and PK35.

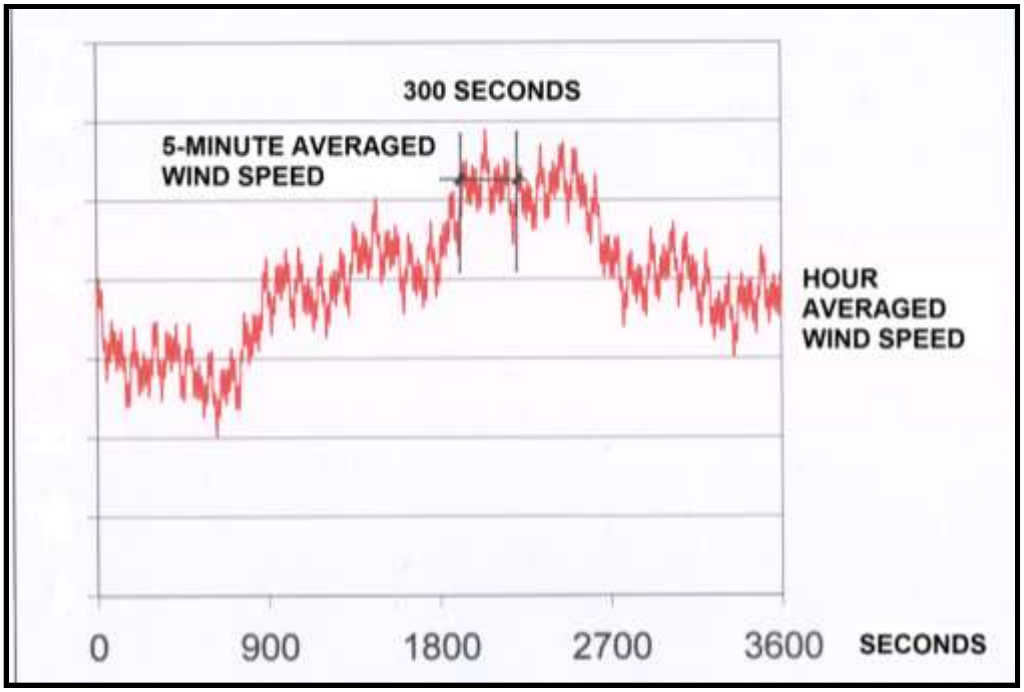

Figure 18. Wind speed time history.

Table 4. Buffeting Analysis

CITY ISLAND BRIDGE 3/31/8

NUMBER OF TIME HISTORIES = 5
LENGTH OF EACH TIME HISTORY(SEC) = 300
MEAN SIMULATION WIND SPEED (M/S) = 31.68
MEAN HOURLY WIND SPEED (M/S) = 29.04

WWL MODE	GE MODE	MT	AVE	PAVE	PSIG	PMAX	PK20	PK25	PK30	PK35
1	1	2	0.021	0.180	0.045	0.234	0.270	0.292	0.314	0.336
2	2	3	0.000	0.103	0.008	0.113	0.119	0.123	0.127	0.130
3	3	2	0.002	0.057	0.004	0.063	0.064	0.066	0.068	0.070
4	4	3	0.002	0.024	0.003	0.027	0.030	0.031	0.032	0.034
5	5	3	0.001	0.013	0.001	0.014	0.015	0.015	0.016	0.016
6	6	2	0.001	0.022	0.002	0.025	0.026	0.027	0.028	0.029
7	7	1	0.001	0.005	0.001	0.006	0.006	0.007	0.007	0.007
8	13	3	0.000	0.007	0.001	0.008	0.009	0.009	0.010	0.010
9	16	2	0.000	0.009	0.001	0.010	0.011	0.011	0.011	0.012
10	17	3	0.000	0.009	0.001	0.011	0.011	0.012	0.012	0.013
11	19	3	0.000	0.003	0.000	0.004	0.004	0.004	0.005	0.005
12	20	2	0.000	0.005	0.000	0.005	0.005	0.006	0.006	0.006
13	21	1	0.000	0.000	0.000	0.000	0.0000	0.000	0.000	0.000
14	22	2	0.000	0.002	0.000	0.003	0.003	0.003	0.003	0.003

MT MODE TYPE: 1 - SWAY; 2 - VERTICAL; 3 - TORSIONAL
PAVE MEAN MODAL PEAK RESPONSE FOR ALL TIME HISTORIES
PSIG STANDARD DEVIATION OF THE PEAK RESPONSES ABOUT THE MEAN PEAK RESPONSE
PMAX MAXIMUM MODAL PEAK RESPONSE FROM ALL TIME HISTORIES
PK20 MODAL PEAK RESPONSE EQUAL TO (PAVE + (2 * PSIG))
PK25 MODAL PEAK RESPONSE EQUAL TO (PAVE + (2.5 * PSIG)), ETC.

IF MT=1, THE MODAL RESPONSE EQUALS THE LARGEST SWAY DECK DISPLACEMENT (M)
IF MT=2, THE MODAL RESPONSE EQUALS THE LARGEST VERTICAL DECK DISPLACEMENT (M)
IF MT=3, THE MODAL RESPONSE EQUALS THE LARGEST VERTICAL DECK EDGE DISPLACEMENT DUE TO TORSION (M)

11 STABILITY ANALYSES

Stability analyses were performed for the ten cases shown in Table 5.

Table 5. Cases studied

Case	Angle of incidence (degrees)	Flow type
1	0	Smooth
2	0	Turbulent
5	-5.0	Smooth
6	-2.5	Smooth
7	2.5	Smooth
8	5.0	Smooth
9	5.0	Turbulent
10	2.5	Turbulent
11	-2.5	Turbulent
12	-5.0	Turbulent

Table 6. Critical flutter wind speeds (m/s)

Case	Critical flutter wind speed (m/s)
1	49.5
2	>54.0
5	48.0
6	48.5
7	47.0
8	48.0
9	>54.0
10	>54.0
11	>54.0
12	>54.0

Stability analyses for non-zero angles of incidence are included to evaluate the performance of the bridge should the mean angle of incidence, be non-zero, and account for a static bridge deck rotation at the strong, extreme wind events considered. Since the clearance below the bridge is so small, mean winds are never expected to deviate much from the horizontal.

Flutter wind speeds, should they be found, are lower bound estimates for a number of reasons: The stability of the bridge is evaluated primarily from the smooth flow results. Typically, turbulence will disrupt the flow around the prismatic deck section, creating a lack of coherence in the aerodynamic loading along the span, which in general forces any aerodynamic instability up to a higher wind speed. It is not the policy of the West Wind Laboratory to rely upon beneficial aerodynamic effects due to turbulence (consistent with the recommendation of British Standard BS 5400 Part 2: Design Manual for Roads and Bridges, Loads for Highway Bridges). Second, winds are assumed to be exactly perpendicular to the axis of the bridge. Destabilizing aerodynamic effects on the deck are generally strongest when they can occur exactly at the same time along the bride axis, when winds are perpendicular to the deck. And third, the mean wind speed is assumed to be absolutely uniform along the length of the bridge. If these lower bound estimates are greater than the design criteria, the bridge will be stable for realistically varying and turbulent winds.

At the beginning of each simulation, all modes of vibration began with a modal displacement of unity. Each mode was released in the specified wind, and all were allowed to vibrate freely and simultaneously (allowing for any cross coupling should there be an aerodynamic tendency to do so). Dynamic flutter instabilities (single-degree-of-freedom and coupled multi-degree-of-freedom flutter instabilities) were then identified by the ratios of the modal standard deviations at the end of the simulations to the corresponding initial modal standard deviations. If a modal ratio was greater than unity, then that mode was diverging and the bridge was dynamically unstable.

Critical threshold wind speeds used typically are 10-minute averaged wind speeds (to allow a sufficient time for an instability to develop). Since an instability can lead to a catastrophic failure of the bridge, such instabilities should be avoided at all cost. Should an instability be identified, it should not occur for a wind less than a very extreme event. A wind speed with a return period of at least 10,000 years is a typical critical flutter threshold wind speed.

The maximum 10-minute averaged wind speed, at the site, at the bridge deck elevation of 7 m, was determined to be 42.4 m/s. No dynamic instabilities were identified, for any of the ten cases, below the criterion. For all of the cases in turbulent flow, note that the critical flutter wind speed was greater than 54 m/s, and in each case, was greater than the corresponding critical flutter wind speed in smooth flows. (Table 6)

12 CABLE DAMPER REQUIREMENTS

Evaluation of the need for external dampers on the cables is given in Raggett, 2008, Appendix 12. Reduced velocity (RV) is a non-dimensional velocity that is defined as U/nD where n is the natural frequency of vibration of the cable, and D is the cable diameter.

The Scruton number, $S_c = m\xi / \rho D^2$ where m is mass per unit length, ξ is the linear viscous damping ratio, and ρ is the density of air (1.23 kg / m3).

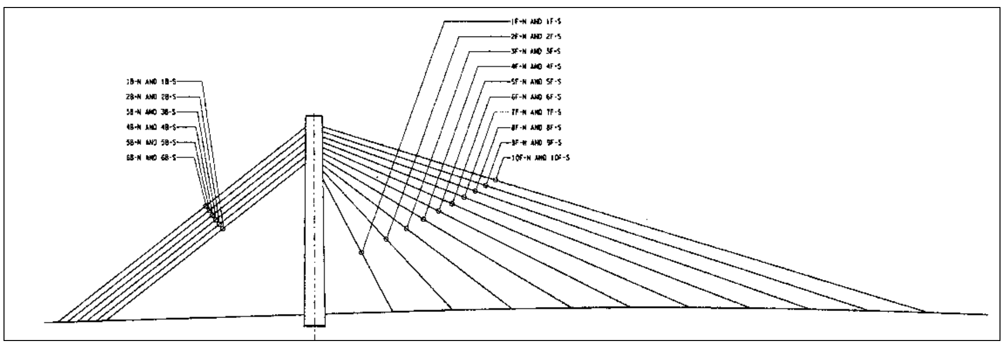

Figure 19. The cable identification numbers

Large amplitude stay cable vibrations have been observed in the past on a number of cable-stayed bridges. These wind-induced vibrations have been observed in the field with and without the presence of stay cable cross-section modified by rain rivulets and are called "rain-wind induced" and "dry cable" vibrations, respectively. Typically, design of a solution which enhances the inherent damping present in the stay cables will mitigate these large amplitude vibrations.

The assumption of an inherent damping ratio of 0.0025 in the first mode of vibration for the longest cable results in Scruton numbers for all cables which fail to satisfy the Irwin (1997) criterion of Sc $\geq$ 10 to prevent rain-wind induced vibrations (RWIV).

The minimum damping ratio that is required in the first mode of vibration for each cable based on criterion of Scruton number, Sc $\geq$ 10, was calculated to vary from 0.0063 to 0.0101 (Irwin 1997). The level of damping required per this criterion can be achieved through (a) viscous mechanical damper, (b) aerodynamic surface modification, and (c) a combination thereof. Calculations presented in this report indicate that if an optimal damper is designed, it can achieve the required level of damping.

Aerodynamic damping estimates for the cables with a double helical fillet that will be used for these cables show that the additional damping requirements have to be met for Cables 2F to 10F (18 cables) . These additional damping values vary from 0.0006 to 0.0043 This criterion is governed by both wet cable vibration or RWIV and the dry cable vibration that can occur at higher wind speeds below the design wind speed of the bridge. A probabilistic assessment of the site-specific wind climate and directionality will assist in a more precise determination of susceptibility to dry cable vibrations. The presence of double helical fillet will eliminate the RWIV in most cases but with these additional mechanical dampers it will eliminate all RWIV and may also help to eliminate other known forms of vibration.

Cables 1F and 1B to 6B (14 cables) do not need any mechanical dampers based on the Reduced Velocity (RV) criterion of 20 - 80 for the RWIV and 150 or greater for dry cable vibration. However, dampers might be required to reduce other forms of vibration if their inherent mechanical damping is below those estimated.

The additional mechanical damping requirements can be easily achieved with a less than an optimum damper. A more accurate estimate of the aerodynamic damping and hence the additional mechanical damping requirement of the cables can be made with a wind tunnel study on stay cables of given diameters and helical fillet of given size for the full-range of wind speeds up to the design wind speed and for various yaw angles. Moreover, detailed wind tunnel studies on cables with double helical fillet of larger sizes than proposed here for this bridge and other aerodynamic appendages such as a circular ring can possibly eliminate the need for any additional mechanical dampers.

13 ANALYSIS OF WIND STUDY RESULTS

The NYCDOT entrusted Michel Virlogeux with the analysis of the Wind Study Report. His report is in two parts. The first part includes the wind analysis of the bridge superstructure (wind model and reference velocity; aerodynamics coefficients of the profile; torsional divergence; aerodynamics stability, flutter, vortex shedding; and dynamic response to turbulent winds) which was directly performed by the West Wind Laboratory (WWL), Marina, California. The second part includes the analysis of cable vibrations by Weidlinger Associates and Parth P. Sarkar of Iowa State University as a subconsultant to WWL.

Our review included structural data submitted by Gandhi to WWL to perform the wind study. Gandhi’s original submission included the stay cables represented in the model with their mass distributed along their length which made it very difficult to clearly identify and separate different modes. He also requested Gandhi to submit the definition of the first thirty modes. There was absolutely no deformation in the tower in the first 30 vibration modes. The tower was too rigid to have any part in aerodynamic stability. It is expected to resist statically all imposed turbulent wind forces.

He also reviewed the methodology adopted by the WWL for its analysis as follows:

- The aerodynamics coefficients and the flutter derivatives (Scanlan or Küssner coefficients) were obtained from a section model test at a scale of 1:40.
- The wind reference velocity and other data were obtained to build a wind model.
- The bridge aerodynamic stability (critical flutter wind speeds) was analyzed numerically in smooth and turbulent flows, by analyzing the evolution of the turbulent flows, by analyzing the evolution of the selected modal vibrations with increasing wind speeds.
- From the same analysis, it was possible to check that there was no risk of torsional divergence (a second order static effect).
- The response to turbulent wind was analyzed numerically, with wind loads being simulated by their time history at the different model nodes.

He agreed with the methodology used by the WWL. During the review of the WWL Report, he asked many questions and raised many issues that seemed different from the European practice; and Dr. Raggett answered all of them to our full satisfaction. Although there were differences of opinions between WWL and us about some of the assumptions, he approved the report without any reservations.

The second part of his review was cable vibrations, and countermeasures suggested by Weidlinger Associates with Professor Partha P. Sarkar of Iowa State University. His report described the most frequent type of cable vibrations:

1. Rain-wind induced vibrations (RWIV), and
2. Dry cable vibration (DCV), generally for oblique winds

It covered the most frequently used mitigation measures such as:

1. Mechanical dampers, and
2. Aerodynamic measures, meaning some specific shaping of the stay cable surface

The Wind Study Report indicated that the average hourly wind velocity for a 10,000-year return periods, was equal to 40 m/s at 10 m above ground, corresponding to an average hourly wind velocity of 46.8 m/s at 30 m above the water level. This meant that only the three longest cables in the main span (8F, 9F, and 10F) perhaps needed a bit more damping than the global damping of 0.5%. There was no practical significance to performing fatigue analysis (or fatigue rupture of some strands) if these vibrations could occur only for winds with a return period of 10,000 years.

A global damping of 0.5% seems enough to eliminate the rain-wind induced vibrations (Virlogeux, Caetano). The backstays were out of the RWIV because their critical velocities were much higher than the 46.8 m/s velocity.

As for dry cable vibrations in the main span, they could develop only for very high winds which was not very likely.

Although the WWL and its consultants believed that some additional damping was required for cables 2F, 3F, and 4F, they disagreed because the critical velocity was higher than 46.8 m/s.

It is possible to excite some of the long forestays by deck vibrations, and these should be monitored in the initial years. Installation of the dampers as recommended by WWL would be useful and efficient.

The distance between the parallel backstays is larger than the distance which usually creates some interaction between parallel stay cables. We recommend to monitor their behavior during the first few years, and provide links between them if vibrations do develop.

We recommend Gandhi Engineering to analyze the practical conditions for the installation of dampers, either external or internal (preferably internal for bridge elegance) in the final design stage.

Table 7 gives the results of analyses, noting that all computations were not made with exactly the same loads. The evaluations made by the WWL Report corresponds to Group II tensions (with live loads) whereas we consider it more representative to work with permanent loads only.

Table 7. Comparison of wind study and analysis results

				Wind Study Report		Gandhi Engineering analysis June 17, 2008			Classical formula from F^2	Gandhi Engineering final analysis June 25, 2008-07-12 from F^2	
Stay cable	Number of strands	Length (m)	Lineic mass (kg/m)	Tension F^1 (kN)	Frequency of he first mode (Hertz)	Tension F^2 (kN)	Frequency of the first mode (Hertz) Out of plane	Frequency of the first mode (Hertz) Vertical		Out of plane	Vertical
1F	22	39.33	31.08	1 263	2.563	1 384			2.683	2.682	2.684
2F	31	49.78	43.80	2 140	2.220	2 377			2.340	2.340	2.342
3F	31	62.21	43.80	2 390	1.878	2 653			1.978	1.978	1.980
4F	37	75.85	52.28	3 168	1.623	3 337	2.225	2.218	1.665	1.666	1.668
5F	43	90.14	60.75	3 152	1.263	3 784	2.225	2.218	1.384	1.384	1.388
6F	55	104.82	77.71	4 116	1.098	4 306	1.792	1.805	1.123	1.123	1.128
7F	55	119.75	77.71	4 716	1.029	5 181	1.792	1.805	1.078	1.078	1.082
8F	61	134.84	86.18	5 571	0.943	6 124	1.579	1.588	0.988	0.987	0.991
9F	73	150.01	103.14	5 939	0.800	7 016	1.339	1.346	0.869	0.869	0.874
10F	91	165.32	128.57	8 982	0.799	9 280	1.326	1.333	0.813	0.813	0.817
1B	91	65.27	128.57	5 865	1.636	6 399		2.356	1.709	1.709	1.713
2B	91	68.57	128.57	8 213	1.843	9 006			1.930	1.930	1.932
3B	91	71.85	128.57	8 221	1.760	9 013			1.843	1.842	1.844
4B	91	75.14	128.57	8 231	1.684	9 023			1.763	1.763	1.765
5B	91	78.43	128.57	8 240	1.614	9 031	2.347	2.356	1.690	1.690	1.692
6B	91	81.72	128.57	8 292	1.554	9 088	2.347	2.538	1.627	1.627	1.629

[1] Under Group II loads.
[2] Under Dead Loads, in the middle of the stay cable

14 CONCLUSIONS

The wind study indicated that the bridge was stable for all wind speeds and angles anticipated at the City Island Bridge site for a return period of 10,000 years. All predicted motions of the bridge, for the comfort of the people on the bridge, were well below the assumed motion criteria.

Some of the forestays with longer lengths (e.g. 8F, 9F, and 10F) may require damping, and provisions should be made in the design to accommodate the damper at a later date, if necessary.

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