Challenges in operational modal analysis of suspension bridges

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Abstract

Operational modal analysis is being increasingly used as an experimental technique to identify the dynamic properties of many types of large-scale constructed systems. The writers conducted such an experiment to identify the dynamic properties for the masonry towers of a landmark suspension bridge with the objective of providing data to validate and improve the reliability of a seismic evaluation and retrofit investigation. One of the towers was instrumented by a dense and stationary array of sensors, and the other tower and spans were instrumented with a limited number of reference sensors. The spans had been tested and characterized earlier by different researchers using ambient and forced excitation techniques. The authors did take advantage of those results in designing the instrumentation to characterize the towers. This paper describes some of the challenges encountered in trying to identify the dynamic properties of the towers using this approach, and the techniques the writers used in an attempt to overcome these challenges. Recommendations for future applications with this type of structure are also discussed.

1 Introduction

Operational modal analysis is popular experimental technique for objectively characterizing the dynamic properties of a broad range of constructed systems including buildings, bridges, offshore structures and dams. Since the technique relies on ambient sources of excitation to extract the dynamic properties from the measured structural responses, it is well-suited for characterizing a large-scale structure that may not be easily evaluated using forced-excitation dynamic testing methods. The identified dynamic properties are frequently employed in a structural identification framework to improve the reliability of analytical models of constructed systems; however, a significant amount of research has also been focused on using these properties to identify and characterize damage and deterioration for health monitoring applications. Despite the significant amount of research that has been conducted on different methods and algorithms that will permit the most reliable identification of the dynamic properties, and the relatively large number of real-world implementations on constructed systems since the 1970's, there are still many unresolved issues related to the design, execution, analysis, and interpretation of operational modal analysis experiments that require further research.

The writers encountered a number of important challenges while trying to identify the dynamic properties of the masonry towers of a landmark suspension bridge by operational modal analysis. An ambient vibration testing program was initiated for the Brooklyn Bridge towers with the objective of improving the reliability of a seismic evaluation and retrofit investigation that was being conducted for the bridge. The spans of the bridge had been tested and characterized earlier by different researchers who utilized ambient and forced-excitation techniques; however, the response of only one tower was measured with a single accelerometer. The authors did take advantage of the results of these earlier tests for designing the instrumentation to characterize the towers. Irrespective of the results available from the previous tests, a major challenge that had to be overcome related to the nature of the dynamic excitation of the towers.

The suspended spans of the bridge were subject to spatially distributed, stochastic ambient excitation primarily due to traffic. The dynamic excitation of the towers; however, was of a significantly more complex nature; consisting of both stochastic and harmonic excitations due to the oscillations of the spans and additional unknown sources. The harmonic components of the ambient excitation appeared as peaks in the frequency spectra for the towers along with the resonant frequencies of the tower structures themselves. This complicated the identification of the natural frequencies of the tower. Another more fundamental challenge was related to the dynamic behavior of a global system consisting of very stiff subcomponents, such as the towers, coupled with very flexible subcomponents, such as the suspended superstructure. The dynamic behavior and interactions of the coupled global system and of the individual subcomponent systems must be clearly conceptualized in order to conduct reliable modal identification.

One of the most relevant lessons that emerged from the research related to the design and execution of an operational modal analysis experiment for reliable structural identification of a large-scale constructed structure comprised of components with very different mass and stiffness characteristics. A related issue was the analysis and interpretation of the measurement data where the ambient excitation of one component may be governed by vibrations transmitted through another, especially when vibration is transmitted from components having significant differences in mass and stiffness from the component being monitored. Structures comprised of components with significant differences in mass and stiffness, and connected by highly flexible elements such as the cables in a suspension bridge have been described as weakly-coupled systems, and this terminology is adopted for this paper to relate the writers' observations to the general dynamic behaviors of civil and mechanical systems. The writer's efforts to overcome the challenges described above for reliable modal identification the Brooklyn Bridge towers are presented and discussed in this paper.

2 Ambient vibration testing of the Brooklyn Bridge towers

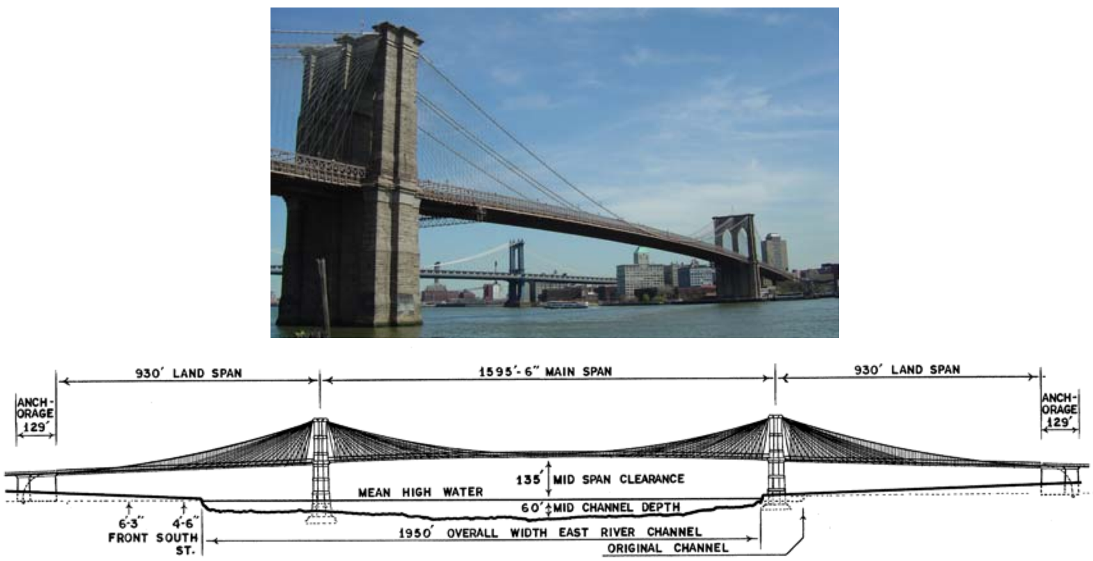
2.1 Description of the bridge

The Brooklyn Bridge (Figure 1) is a major suspension bridge that spans the East River between the Borough of Manhattan and the Borough of Brooklyn in New York City. The landmark structure is one New York City's most widely recognized icons, and is considered to be one of America's great national civil engineering treasures. The bridge was designed by John A. Roebling and its construction was completed under the supervision of his son, Col. Washington Roebling. At the time of its completion in 1883, the Brooklyn Bridge was the longest suspension bridge in the world. The Brooklyn Bridge was also the first suspension bridge to use steel wire instead of wrought-iron for its cables, and this enabled it to support much longer and heavier spans than its predecessors.

The bridge consists of a single 486m-long main span and two 283m-long land spans. The two main towers, which are the focus of this paper, are massive masonry structures constructed from large granite stones. The towers consist of three columns joined at their tops by Gothic arches above the roadway deck. Each tower is approximately 84m-tall measured from the mean high water elevation, and is supported on large concrete-filled timber caissons. The caisson supporting the Manhattan Tower is somewhat deeper than for the Brooklyn Tower. The cross sectional dimensions of the Manhattan Tower are also very slightly larger than those of the Brooklyn Tower. The base of the Brooklyn tower has nominal plan dimensions of 43m by 17m, while the Manhattan Tower has nominal plan dimensions of 40m by 14m. The Manhattan Tower also weighs more than the Brooklyn Tower, with the self-weight for each tower approximately equal to 863MN and 703MN, respectively [1].

The suspended structure consists of 4 trusses and a concrete deck carrying a total of 6 lanes of traffic. The stiffening trusses are supported by vertical suspenders and 4 main cables each with a diameter of 40cm. The stiffening trusses are also partially supported near the towers by a series of diagonal stay cables radiating from the top of each tower to the lower panel points of the stiffening trusses. The stiffening trusses are fixed at each tower, pinned at each anchorage and have expansion joints (axial and moment releases) located at the middle of the main span and each land span. The dead load of the suspended structure is approximately 144 kN/m [2]. The mass of each tower is approximately 10 times greater than the mass of the portion of the suspended spans carried by each tower.

The main cables are supported at the tops of the towers by saddle structures on rollers that were originally designed to permit the cables to move longitudinally with temperature changes and unbalanced loads on the spans [3]. These rollers have become frozen with age, and the cables may effectively be considered to be fixed to the tops of the towers. This condition also permits the bridge to act as three independent suspended spans [3]. Past field surveys of the towers indicated that the deflection profile of the towers under controlled loading was consistent with rigid body rotation about the base of the towers. This behavior has been described as due to the elastic compressibility properties of the timber portions of the caissons [2, 3].



(Drawing: Historic American Engineering Record, National Park Service, Paul Berry, 1985)

Figure 1: The Brooklyn Bridge

2.2 Objectives and scope of the test program

The focus of the ambient vibration testing described herein was to identify the modal properties for the masonry towers of the Brooklyn Bridge. The suspended spans of a symmetric half of the bridge had been tested using ambient and forced-vibration testing methods as part of a previous investigation related to pedestrian vibrations, thus the scope of the test program was limited to the towers. The design, analysis and interpretation of the ambient vibration test program designed for the towers were guided in large part by the following questions relevant to this type of suspension bridge: (1) What is the ambient vibration environment of the entire bridge and of the individual subcomponents? (2) What are the dynamic properties of the global system, and more importantly, of the subcomponent towers? and (3) Can the dynamic properties of the various subcomponents of the bridge be directly extracted by measuring their respective operating vibration responses, or is it essential to model, test, and identify the entire weakly-coupled system as an interconnected system?

Two additional issues were considered for the design of the vibration test program to identify the dynamic properties of the towers. The first issue related to the vibration environment for the bridge and whether or not the responses of the massive towers under ambient excitation would be large enough to generate the levels of signal-to-noise ratios in the measurements necessary to permit reliable identification of the towers' dynamic properties. The second issue related to the characteristics of the tower excitation due to a seismic event. The nature of seismic excitation and its transmission through the structure would be fundamentally different from the normally dominant sources of ambient excitation such as traffic. The discrepancy between the bandwidth and transmission characteristics of the ambient and the design seismic excitations would certainly introduce a considerable level of uncertainty related to the dynamic properties of the towers since these would be identified from very low-level and likely linear responses under ambient excitation. Furthermore, the material stiffness and damping characteristics of the tower and the behavior of the boundary conditions would likely be very different during a design seismic event than under ambient dynamic excitation levels.

In light of these considerations, a test program that included ambient vibration testing followed by forced-vibration testing of the towers at the identified natural frequencies was proposed to the bridge owner. A properly designed forced-excitation test could reveal behaviors such as the movement of the main cable saddles or rigid body rotation of the towers about their timber caissons. These behaviors were not expected to be observable under ambient excitation levels. A forced-excitation test would also enable more reliable identification of the natural frequencies and damping ratios of the towers. The bridge owner ultimately decided to proceed with ambient vibration testing for identifying dynamic properties of the towers due to other constraints.

2.3 Summary results from the earlier characterization of the spans

The dynamic properties of the spans were identified by others during a previous vibration test of the bridge [4]. A total of 35 modes were identified for the bridge spans from acceleration measurements recorded on the main span and Brooklyn side span. The frequency band of the identified modes ranged from 0.180 Hz to 2.397 Hz. The experimentally identified modes were exclusive to either the main or the side span, and with the exception of two side span modes, they were also exclusive to the lateral, vertical, and torsional directions of each span. The 5th lateral and 3rd torsional modes of the side span were identified at the same frequency (1.100 Hz), and the 6th lateral and 4th torsional modes of the side span were identified at the same frequency (1.508 Hz). The former set of lateral and torsional modes were identified as being very closely spaced in a 3D FE model of the bridge, while the later set of lateral and torsional modes were identified at the same frequency (coupled mode) in the analytical model. The experimentally identified modes were correlated with various modes ranging from the 1st to the 137th mode in the 3D FE model.

2.4 Experimental setup

The instrumentation scheme developed to characterize the towers included a fixed array of 43 uniaxial accelerometers. As shown in Figure 2(a), a total of 28 accelerometers were installed at 7 different elevations on the Brooklyn Tower. Each of the three independent tower columns was instrumented with two accelerometers to measure the longitudinal and transverse (lateral) vibrations at the two levels between the top of the tower and the roadway level. A total of three accelerometers were installed at each of the remaining levels on the tower to measure the longitudinal/torsional and transverse vibrations. In addition, two vertically oriented accelerometers were installed at the top of the tower and at the base of the tower. The Manhattan Tower was instrumented with three accelerometers at a level located just below the roadway deck. Although the principal focus of the testing was the towers, a limited number of accelerometers were also placed on the stiffening trusses in the main span and side span near the Brooklyn Tower to aid in the interpretation of the tower measurements and to correlate the findings with the results of the previous investigation. The tower accelerometers were installed using fixtures that were temporarily anchored into the masonry stones. Access to the sensor locations on the tower was accomplished by rappelling down to each measurement level from the top of the tower as shown in Figure 2(b).

A total of 12 accelerometers were distributed between two measurement stations along the side span, and two stations along the main span. There were two vertical accelerometers installed on the bottom chords of the outermost stiffening trusses to measure vertical and torsional vibration responses, and one transversely oriented accelerometer was installed to measure lateral vibration responses. The side span measurement stations were located at distances of 73m and 138m from the tower, and the main span stations were located at distances of 56m and 120m from the tower.

The accelerometers used for the vibration test included capacitive accelerometers and seismic accelerometers both from PCB, Piezoelectronics. The sensitivity for both types of accelerometers used was 1 V/g. The data acquisition was performed using a VXI mainframe, and all accelerometer channels were measured simultaneously during the testing. The acceleration measurements were sampled at either 20 samples/second (20 Hz) or 40 samples/second (40 Hz), and were recorded in time periods ranging from 30 minutes to about 3 days in length. The vibration testing was conducted over a period of about one month. A wind sensor was installed on top of the Brooklyn Tower to measure the wind speed and direction during the ambient vibration testing. The ambient temperature at the bridge was also periodically measured during the vibration testing.

3 Data processing methodology and results

3.1 Data pre-processing

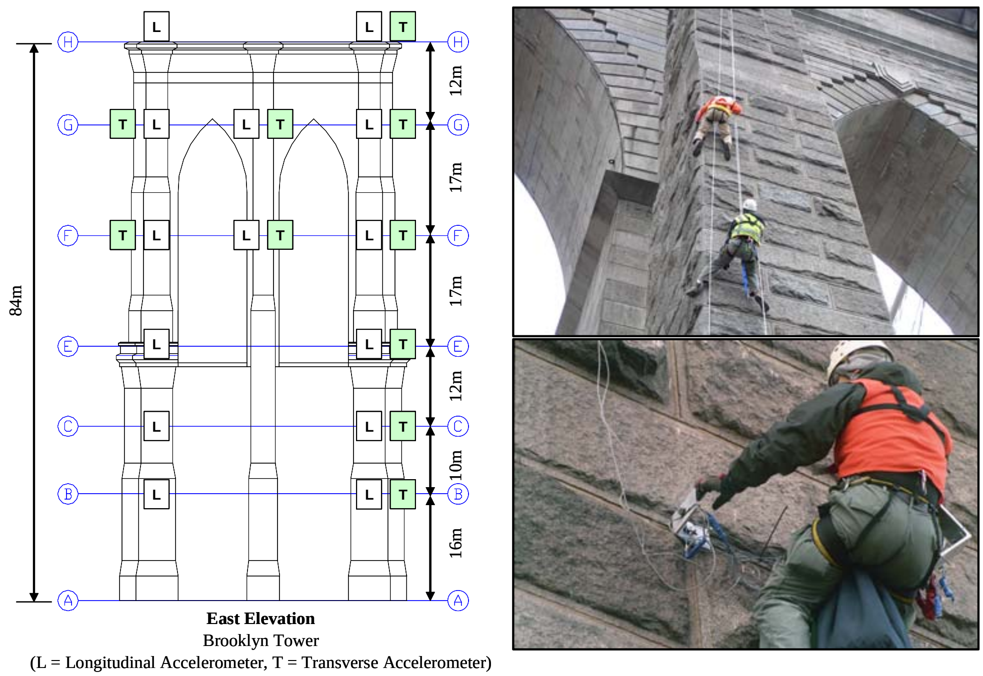
The raw acceleration time records were pre-processed in the time domain to ensure the data were of sufficient quality and to prepare them for subsequent use in identifying the dynamic properties of the towers. The steps in this pre-processing were as follows: (1) the record for each channel was inspected to verify that the accelerometer had functioned properly and to note the presence of any obvious data errors in the record, (2) the data records were high pass filtered at 0.3 Hz to remove any DC offset in the records, (3) the individual sample points that formed any spurious spikes were removed from the records, and finally (4) the responses from the two longitudinal/vertical channels at a each tower level/span station were added to extract the pure longitudinal/vertical response from the coupled longitudinal/vertical responses contained in each channel, and one channel was subtracted from the other at each level/station to extract the pure torsional response from the coupled responses. It should be noted that when the sample points that formed a spurious spike were removed from one channel's record, the corresponding sample points were also removed from the records of all other accelerometer channels so that the phase information between the simultaneously sampled channels would be preserved. These data pre-processing steps were performed using the program MATLAB.

Examples of the raw acceleration records from a longitudinal accelerometer at the top of the tower and a vertical accelerometer on the main span stiffening truss are shown in Figure 3. An example of a typical noise spike that was removed from the acceleration records can be seen in the record for the longitudinal tower record. Similar examples of the lateral acceleration records for the same two locations are shown in Figure 4. These figures also illustrate the relative differences between the acceleration amplitudes recorded in two different directions and between those recorded from different components of the structure. The amplitudes of the span accelerations were generally nearly an order of magnitude larger in the vertical direction than in the lateral direction. Similarly, the amplitudes of the measured accelerations in the tower's longitudinal direction were generally an order of magnitude larger than in the lateral direction for the same sensor location. The amplitudes of the span accelerations in the vertical and lateral directions were on the order of milli-g's whereas the amplitudes of the tower accelerations in the longitudinal and lateral directions were on the order of 100 micro-g's.

3.2 Frequency domain processing

The results presented and discussed in this paper are from a data set in which all of the channels were simultaneously sampled for approximately 4 hours at 40 Hz. The frequency domain data processing included three steps. First, autopower spectra were computed using Welch's method for the cleaned acceleration records in the longitudinal, vertical, and torsional responses. The autopower spectra were computed for 8192 point long segments of the time records yielding a frequency resolution of 0.0049 Hz.

A Hanning window was also applied to each data segment to minimize spectral leakage, and the segments were overlapped by 50%. The autopower spectra were used to locate the natural frequencies in each response direction. The second step was to compute cross power spectra between each measurement level on the towers or measurement station on the spans and a selected reference location. The same parameters used for computing the autopower spectra were used to compute the cross power spectra. The magnitude and phase information contained in the cross power spectra were used to construct mode shapes from the measurement data. The last step was to calculate the coherence between each response at the different measurement locations on the towers and spans and the selected reference locations.



(a) (b)

Figure 2: (a) Accelerometer layout for the Brooklyn Tower, and (b) Accelerometer installation

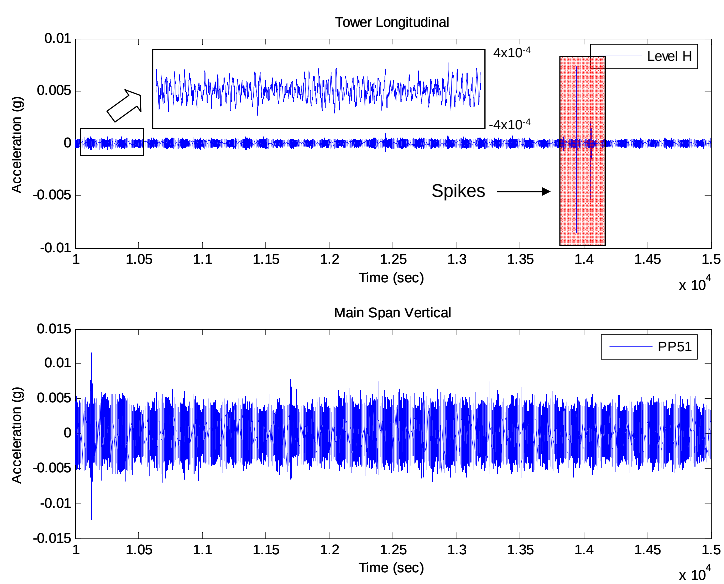


Figure 3: Brooklyn Tower longitudinal and main span vertical accelerations

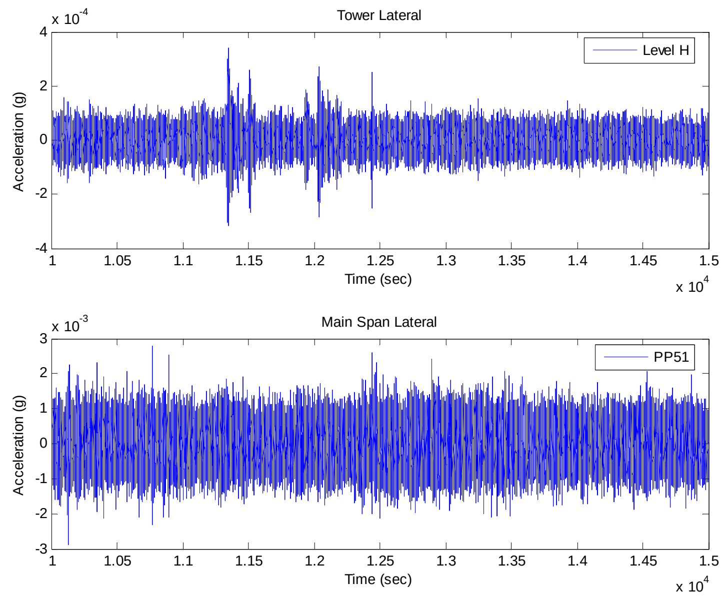


Figure 4: Brooklyn Tower and main span lateral accelerations

3.3 Results

3.3.1 Observations on the vibration environment and measured acceleration responses

The time variation of the frequency domain spectral content for the longitudinal, torisonal, and lateral vibrations at different levels on the towers are shown in Figures 5 – 7, respectively. The majority of the significant spectral content for the tower vibrations was located in the 0 to 8 Hz frequency band. The response at Level F is excluded from these figures since one of the longitudinal accelerometers at this level ceased to function during the test and the logistics of the sensor installation did not permit replacing this sensor. The intensity of the vibration at a given frequency line is indicated by the color shade in the graphs with the darkest regions corresponding to largest vibration amplitudes. It can also be seen from the figures that the measured vibration amplitudes were generally largest at the top of the tower.

The lateral response at the top of the Brooklyn Tower (Level H) is markedly different from the response of the lower measurement levels for frequencies close to 3 Hz. There is no discernable lateral response for the top of the tower near this frequency in Figure 7, while lateral response is clearly visible near this frequency for the lower tower levels. The lack of response at the top of the tower may indicate that this is a nodal point in the lateral deflection profile for the tower at this frequency, or that top is restrained from lateral motion at this frequency by the main cables and the diagonal stays. The latter explanation would be somewhat unexpected since the main cables are not very stiff in bending.

Figures 5 – 7 show that the spectral content for each response and at each level on the tower was nearly constant during the approximately 4 hour period when the measurements were recorded. Therefore, it may be concluded that the ambient excitation of the towers was practically stationary. The only exception to this observation is highlighted by the fenced portions on each graph between 4 Hz and 5 Hz. It is clear from the graphs that some unusual and rather significant excitation occurred between 8,000 and 13,000 seconds affecting the Brooklyn Tower responses. The vibration amplitudes resulting from this unusual excitation were largest in the lateral direction, and at the lower measurement levels of the tower (roadway deck level and below). The additional spectral content was not observed in the Manhattan Tower responses, but could be identified in the some of the responses for the spans.

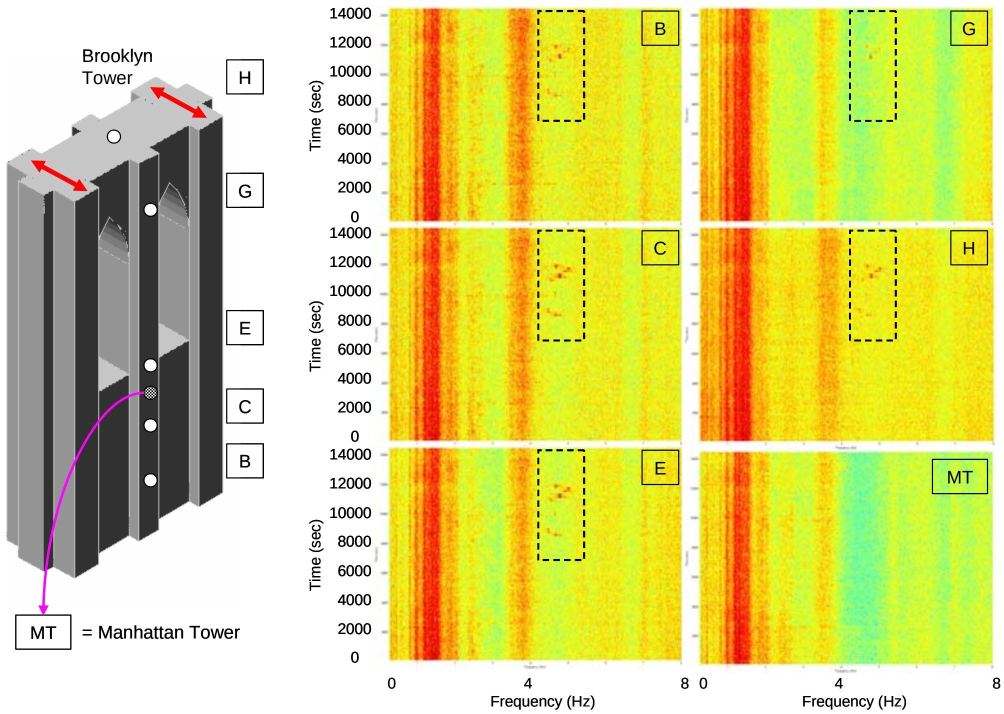


Figure 5: Time variation of longitudinal spectral content

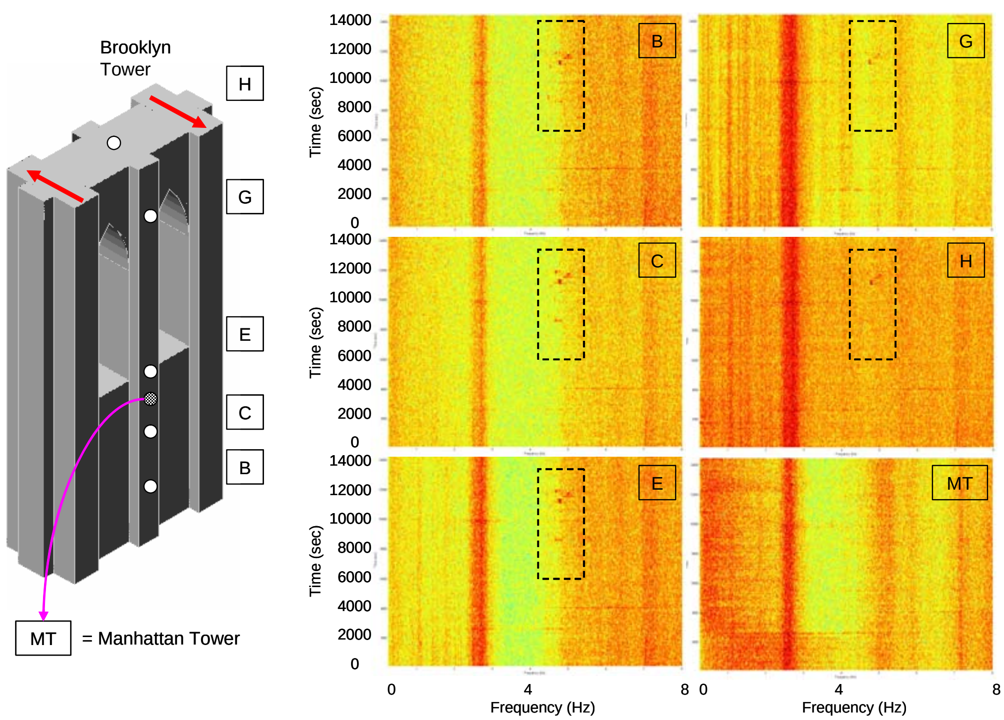


Figure 6: Time variation of torsional spectral content

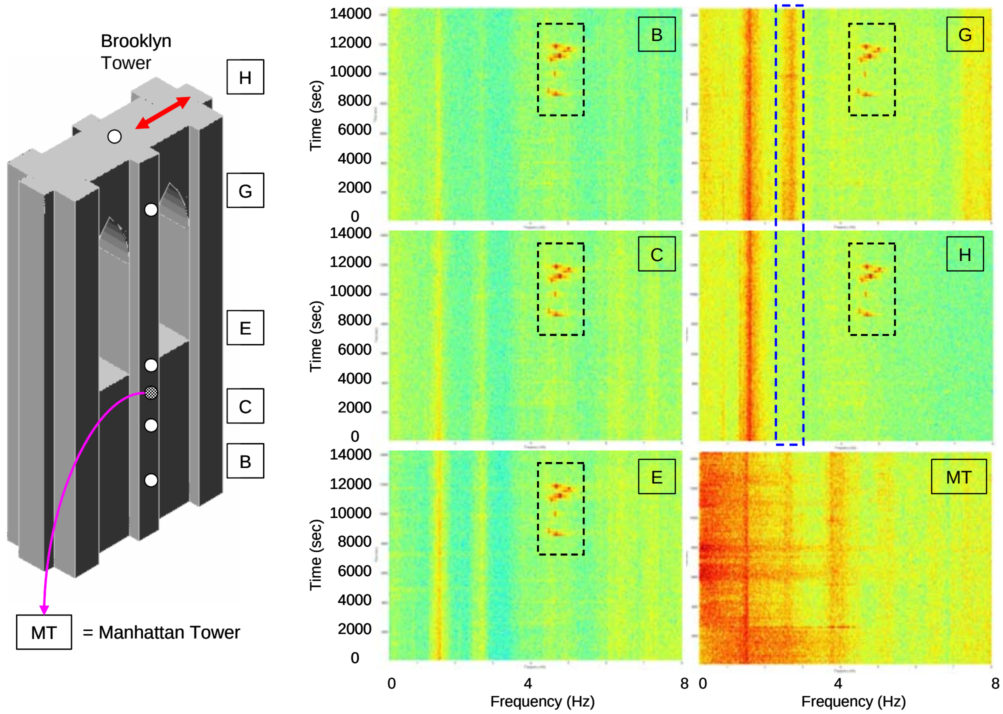


Figure 7: Time variation of lateral spectral content

Several segments of the lateral acceleration time records containing the additional spectral content described above for the Brooklyn Tower are shown in Figure 8. The accelerations shown are representative of real responses and are obviously not a manifestation of measurement noise. These responses are also distinctly periodic. The exact cause of the excitation that led to these responses could not be identified; however, wind excitation was ruled out as the cause since the wind speed and direction records did not contain any significant measurements. Wave action was also excluded as a possible cause of the excitation since the Brooklyn Tower is located on dry land.

3.3.2 Factors adding complexity to the identification of the dynamic properties

The primary objective of this investigation was to identify the dynamic properties of the Brooklyn Tower through operational modal analysis. This objective was especially difficult due to two distinct but related factors that may be observed for many suspension bridges. The first was related to the fundamental dynamic behavior of a 3D system that consists of very stiff and massive components (such as the towers) that are connected to relatively lighter and far more flexible components (such as the spans and the cables). The motions of the individual components will be coupled with each other as a result of the physical connections between them; however, if the mass and stiffness characteristics of the various components are sufficiently different, the dynamic characteristics of each component may be essentially distinct. This type of behavior has been associated with weakly-coupled systems. In structures with dynamic behaviors such as this, global structural modes may exist; however, these modes should correspond only to the frequencies at which all of the principal components are experiencing simultaneous resonant motions.

The spans of the Brooklyn Bridge are physically connected to the towers by the main cables, stay cables, and the stiffening trusses that are fixed at the towers. As a result, the motions of the towers and the spans will always remain coupled to some degree. The measurement data from the bridge reflects this as there were no frequencies at which one of these components was oscillating without some participation from the other components. The mass and stiffness characteristics of the towers and spans are also sufficiently different from each other that it is possible to have one component experiencing resonant oscillations at a particular frequency while other components physically connected to it vibrate in a non-resonant manner. This behavior is important when an operational modal analysis experiment is conducted for calibrating or validating analytical models of a structure, since to achieve the greatest reliability, the validation process should include those frequencies and mode shapes corresponding to the resonant responses of the global structure and each of its principal structural subcomponents.

By definition, the maximum amplification of response for a particular structural subcomponent will occur at that component's distinct natural frequencies, and not at frequencies for which the component is simply oscillating as a result of its connections to another component that is undergoing its own resonant oscillations. A failure to properly conceptualize and consider this possible behavior can lead to classifying all of the identified frequencies as global modes of the structure. At the same time, if the modal parameter identification process is not sufficiently rigorous, the frequencies associated with non-resonant oscillations of a component could be mistakenly classified as natural frequencies for that component even though by definition, a natural frequency implies resonant motion. A strict characterization and assignment of global and local natural frequencies could be considered as simply a matter of semantics; however, the definition of what constitutes a natural frequency for the global structure or for any of one of its components should not be loosely defined and applied to the interpretation of the experimental results since this may add uncertainty and further complexity to the FE model validation process. Given the inherently difficult and non-unique nature of structural identification, this additional complexity may lead to a model that incorrectly or incompletely represents the structure.

A second factor that added significant difficulty for identifying the true modes for the towers was the complex nature of the ambient excitation for the bridge. The ambient excitation for a suspension bridge tower may be considerably more complex than for the suspended portions of the structure. In the case of the Brooklyn Bridge, the suspended spans were subject to spatially well-distributed, random excitation within a certain frequency band that was mainly due to traffic. The wind measurements recorded during the ambient vibration testing of the Brooklyn Bridge did not indicate that wind was a significant input. In contrast, the ambient excitation of the towers may be assumed to have consisted of harmonic excitations due to the oscillations of the spans (and possibly the cables) in their natural modes that were superimposed on any other random excitation sources acting directly on the towers. The resulting ambient excitation is spatially localized to points where the stiffening trusses, cables and stays are connected to the towers. These points are located at the roadway deck level and just below the tops of the towers on the Brooklyn Bridge. Furthermore, because the spans are more flexible and experience vibrations that are an order of magnitude larger than the tower vibrations, the amplitude of the harmonic components of the tower ambient excitation are substantially larger than the random excitation input. A similar observation was made by other researchers in the past relative to the nature of the ambient excitation acting on the Golden Gate Bridge towers [5].

The presence of significant harmonic components embedded in the ambient excitation violates the commonly assumed broad-band Gaussian white noise assumption that is leveraged to extract the dynamic properties for a test structure in an operational modal analysis experiment. The harmonic components lead to additional peaks in the frequency spectra for the tower sensors that may obscure or at a minimum will complicate the identification of the true natural frequencies. The frequency spectra can also be contaminated by measurement noise and errors, spectral leakage and shifting of the natural frequencies due to non-stationary structural behavior, and the harmonic components would compound the uncertainty from such contamination sources.

3.3.3 Data processing procedure for identification of the dynamic properties

The following discussion of the identified dynamic properties for the Brooklyn Bridge Towers will be limited to the presentation natural frequencies and mode shapes in the longitudinal direction only for the sake of brevity. The initial identification of the frequencies and mode shapes for the towers was conducted using a peak-picking approach. The autopower spectra computed for the longitudinal vibrations of the towers are shown in Figure 9. In order to facilitate automated identification of the spectral peaks for the longitudinal vibrations of the towers, the responses from each level of the tower were first combined into a single indicator function.

The Average Normalized Power Spectral Density (ANPSD) function [6] was one of several different modal indicator functions that were used in this study to automatically identify the locations of any peaks in the frequency spectra for the different measurement stations at various levels on the towers. The ANPSD function has been used in numerous operational modal analysis experiments for large-scale constructed systems to identify the locations of the natural frequencies. A simple routine was written in Matlab to automatically identify the peaks in the ANPSD function by comparing the amplitude at a given frequency with the amplitudes at the frequency lines immediately before and after the frequency line being evaluated. A minimum amplitude threshold was also specified to limit the identified peaks to those with the largest signal-to-noise ratios. This procedure led to the identification of a total of 36 peaks with the best signal-to-noise ratios in the 0 - 5 Hz frequency range.

Operating deflection shapes were computed for each frequency corresponding to an identified peak using the magnitude and phase information from the cross power spectra for output pairs from responses at each measurement level and the selected reference level on the towers. The phase information between each output pair was whitewashed [7] to force normal mode characteristics on the operating deflection shapes. In other words, the actual phase angle between the output pairs was normalized to either 0 or 180 degrees to guarantee only in-phase or out-of-phase characteristics, respectively. Normal mode characteristics were imposed for the computed operating deflection shapes to facilitate the analytical model validation performed by the engineering consultant responsible for the seismic evaluation and retrofit investigation. It should be noted that the phase angles for the output pairs at the identified peak locations were usually already very close to 0 or 180 degrees before they were whitewashed. The amplitude of each operating deflection shape was also normalized to a maximum value of one in order to provide a consist basis for comparing the shapes associated with the identified peaks. The operating deflection shapes for the Brooklyn Tower that are associated with the 36 peaks identified above are illustrated in Figure 10. These shapes were computed using the roadway deck level (Level E) on the Brooklyn Tower as the reference level. Operating deflection shapes were only computed for the Brooklyn Tower since the instrumentation scheme was constrained to just two locations for the main and side spans, and to just one level for the Manhattan Tower.

It is apparent from Figure 10 that the 36 frequencies identified in longitudinal spectra for the Brooklyn Tower in the 0 to 5 Hz frequency band may be grouped according to 7 distinct types of deflection shapes. The figure shows that there are slight differences in the amplitudes along the tower height at the different frequencies contained within each shape type grouping; however; the overall character of the deflection profiles contained within each distinct shape group are generally very similar. There is also a noticeable trend in the deflection shapes from that of a cantilever beam in bending at the lowest frequencies to one more indicative of a rigid body rotation about the base of the tower in the 1.2 to 1.5 Hz frequency band.

It was highly unlikely that every one of the 36 identified spectral peaks and their associated operating deflection shapes for the longitudinal vibrations of the Brooklyn Tower corresponded to real natural frequencies of the tower or even a combination of pure tower modes and global structural modes. The challenge became exploring different methods for extracting the “true” longitudinal natural frequencies and mode shapes of the tower as a subcomponent of the weakly coupled global structure from the pool of 36 possible candidates. A highly idealized analytical model of the bridge was therefore created and analyzed to clarify and bound the fundamental dynamics of this weakly-coupled system, and to guide the identification of the natural frequencies and mode shapes for the towers.

3.3.4 Dynamics of an idealized analytical model of the bridge

A significant number of spectral peaks were identified from the measurement data for the longitudinal, torsional, and lateral vibrations of the Brooklyn Tower; however, only a subset the spectral peaks in each direction was expected to correspond to actual natural frequencies of the towers. Idealized analytical models of the bridge were constructed and analyzed to help guide the analysis and interpretation of the measurement results. In particular, the model was used to clarify the nature of the global dynamic behavior of the bridge and of the towers, and to identify the quantity and order of the tower modes that should exist within the frequency range considered. Since the purpose of this model was only to help conceptualize the dynamic behavior of the global structure and of the towers for interpreting the measurements, the model could be very approximate and idealized. The engineering consultant responsible for the seismic evaluation and retrofit investigation developed and utilized very detailed, resolute and complete 3D finite element models of the bridge that were validated using different types of measurement data, including the final results of the vibration testing discussed here. The finite element models developed and used by the engineering consultant are not presented or discussed in this paper.

The idealized model was developed using SAP2000 and utilized several different representations for the primary components of the bridge. The representation of each component in the model was designed such that the dynamic responses of the towers would be unobstructed as much as possible by the dynamic responses of the other components. Both towers were represented with identical dimensions in the model although they have slightly different cross sectional dimensions and different foundation depths (the Manhattan Tower is approximately 10 m deeper than the Brooklyn Tower). The elastic modulus for the granite material in the Manhattan Tower was reduced in proportion to the difference in the foundation depths to account for this effect on the stiffness of the Manhattan Tower. The towers were modeled using 3D non-prismatic beam elements with cross sectional properties coinciding with the tower at the various levels and material properties corresponding to granite. The portion of each tower located below the roadway level was represented by a single beam element, while the portion above the roadway level included three separate beam elements to represent each tower column. Rigid links were used to connect the three beam elements together at the top of the tower and at the roadway level. The foundation conditions at each tower were not explicitly considered in this model, and the beam elements representing the lower portions of the towers were simply fixed in all directions at the ground level. The stiffening trusses were modeled using a single line of 3D beam elements along the longitudinal centerline of the spans that was fixed at the towers and pinned at the anchorage locations. Axial force and moment releases were provided at the midpoint of the beam elements at the middle of each span to simulate the expansion joints at those locations.

The four main cables were represented by horizontal, linear spring elements connected to the tops of the towers, and pinned at the anchorage locations. The main cables were modeled using springs so that their contributions to the overall dynamic response of the structure would be included, but their dynamic responses and interactions with the other components in the model would be minimized. The stiffness of each spring was computed from the axial stiffness (AE/L) of the cable for the horizontal projection of the main cable length in each span. This approximation yields a spring stiffness that may be expected to be larger than the actual cable stiffness but this is considered suitable given the objectives of this model. Only one pair of diagonal stay cables at each tower was included in the model. The pair of diagonal stays extending the farthest distance from the towers in the main span and side span from each tower was modeled using axial spring elements. The masses assigned to the axial springs representing the main cables and stays were based on the reported weights or dimensions of these components. The vertical suspender cables were not included in the model since they are not physically connected to the towers and their dynamic responses would add unnecessary complexity to the analysis.

The stiffness of the beam elements representing the stiffening trusses was adjusted in the vertical and lateral bending directions until the fundamental frequencies of the main span matched the frequencies identified from an earlier vibration test of the spans [4]. Multiple versions of the model in which different primary components (the cables, stays and stiffening trusses) were excluded from the model were analyzed to study the effects of these components on the dynamic behavior of the towers. The towers were also analyzed as free-standing structures that excluded all of the other bridge components to serve as a basis for the comparisons with the other idealized model analysis cases. Figure 11 shows isometric views of the two model cases discussed in this paper.

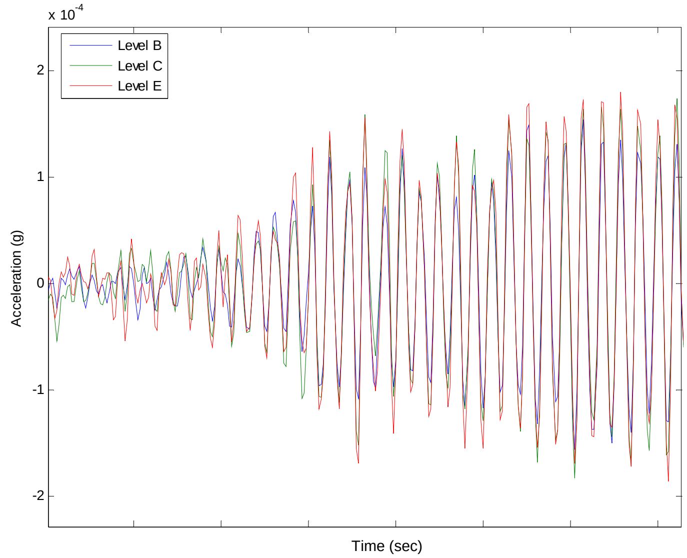


Figure 8: Unusual lateral vibration of the Brooklyn Tower

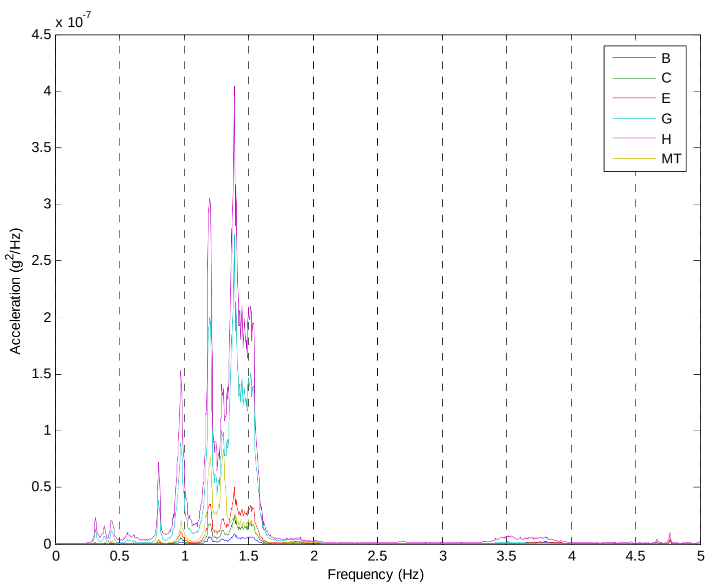


Figure 9: Autopower spectra for tower longitudinal vibrations

The vibration modes of the Brooklyn Tower from the free-standing towers model case and the model case that included representations of all primary bridge components are summarized in Table 1. There were 6 modes identified for the Brooklyn Tower observed in the 0 to 10 Hz frequency range for both model cases. These modes included the first and second longitudinal, lateral, and torsional modes. The same 6 modes are also observed for the Manhattan Tower, although they occur at lower frequencies since the tower is more flexible than the Brooklyn Tower in the idealized model. The frequency of the first longitudinal mode of the Brooklyn Tower shifts to a slightly higher value in the model case that included representation of the primary bridge components, which would be expected due to an increase in stiffness in this direction from the main cables. The quantity and order of the tower modes remains the same in both model cases.

There is very little interaction observed between the two towers in the second model case as each tower vibrates in its natural modes. This would indicate that the tower modes may be classified as distinct for each tower. The spans are observed to vibrate at the tower natural frequencies; however, these oscillations appear to be consistent with local and higher modes of the spans. The towers are also observed to vibrate in their fundamental mode shapes at some of the natural frequencies associated with the spans, but the modal displacements in these cases are significantly less than those which occur at the frequencies associated with the modes of each tower. The modal displacements from the analytical model shown in Figure 12 reflect this observation. The unit normalized longitudinal modal displacements of the towers at frequencies corresponding to vertical span modes are shown in the leftmost graph in the figure. These mode shapes are very similar to the first unit normalized mode shape for the tower modes shown in the center graph in the figure. The rightmost graph in this figure compares the longitudinal modal displacement amplitudes in an absolute sense (un-normalized) for the modes associated with the vertical span modes and the tower longitudinal modes. The absolute modal amplitudes are clearly largest at the frequencies associated with the tower modes that were observed in both idealized model cases. It should be noted that the towers of modern suspension bridges are usually significantly more flexible than the Brooklyn Bridge towers, and the dynamic behavior observed from the idealized models for this bridge would be expected to differ for such structures.

The observations from the analyses of the idealized models that are most relevant to the interpretation of the experimental measurements may be summarized as follows: (1) the critical tower modes are distinct from modes associated with the other bridge components, (2) the absolute modal displacements of the towers at the distinct tower modes are only significant for one direction; therefore, the tower modes may be characterized as pure longitudinal, pure torsional, or pure lateral modes, (3) the towers do participate in varying degrees in all directions with modes associated with other bridge components, but the absolute modal displacements of the towers in any direction at such frequencies are insignificant when compared with the modal displacements associated with the pure tower modes, and (4) there may be 6 distinct natural frequencies identified for the Brooklyn Tower in the frequency range of 0 to 10 Hz. The natural frequencies identified for the Brooklyn Tower may be quite different from those in the idealized model as this represents a very approximate representation of the bridge; however, the order of the modes should follow the trend observed in this model.

Table 1: Frequencies and mode shapes from the idealized analytical model

	Frequency (Hz)		
Mode	Free-Standing Towers	Towers, Cables, Stays, Trusses	Mode Shape
1	1.799	1.976	1st Longitudinal
2	2.896	2.790	1st Lateral
3	4.046	4.032	1st Torsional
4	5.096	4.983	2nd Longidutinal
5	7.196	7.088	2nd Lateral
6	8.047	7.943	2nd Torsional

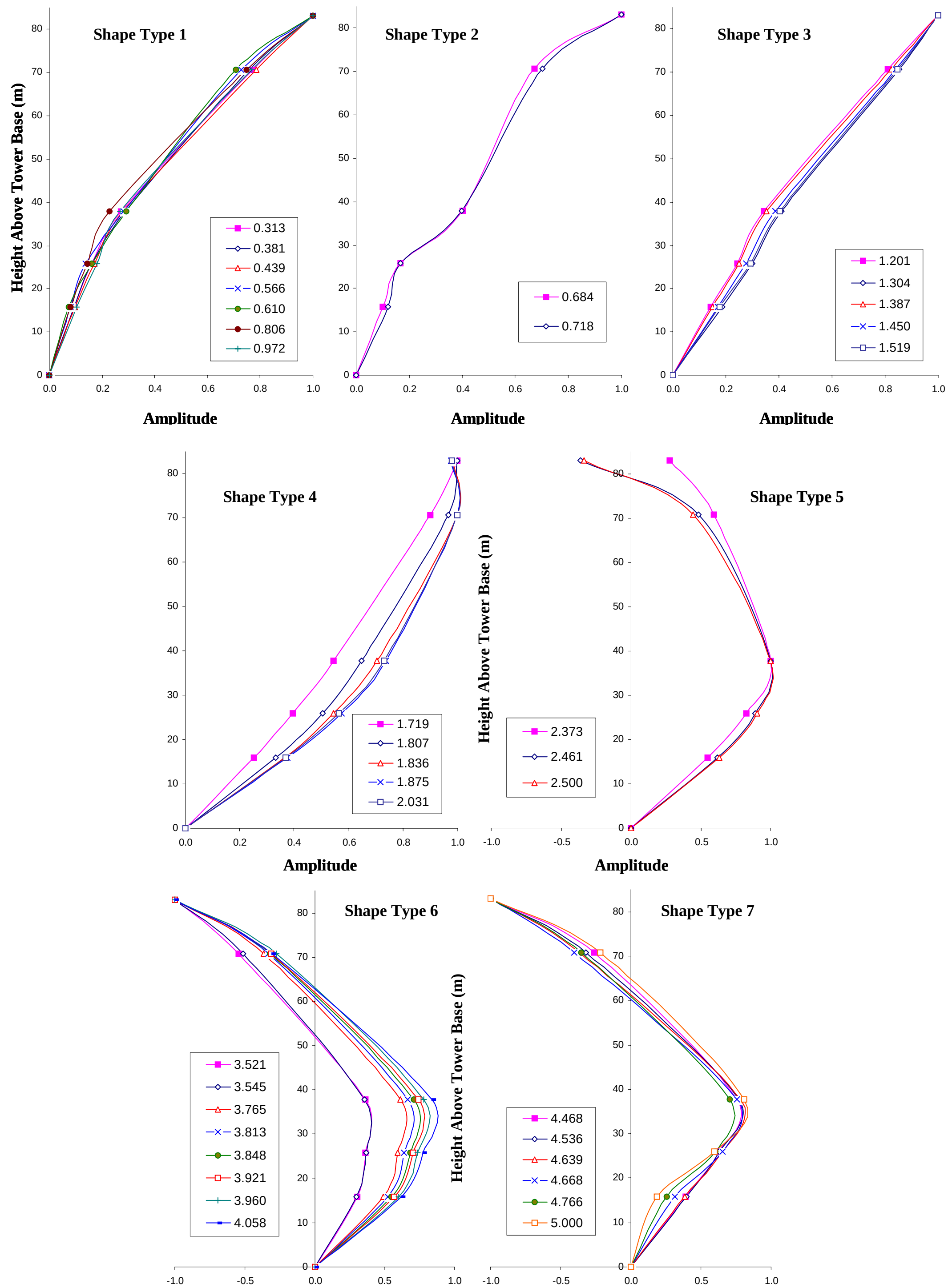


Figure 10: Operating deflection shapes for tower longitudinal vibrations

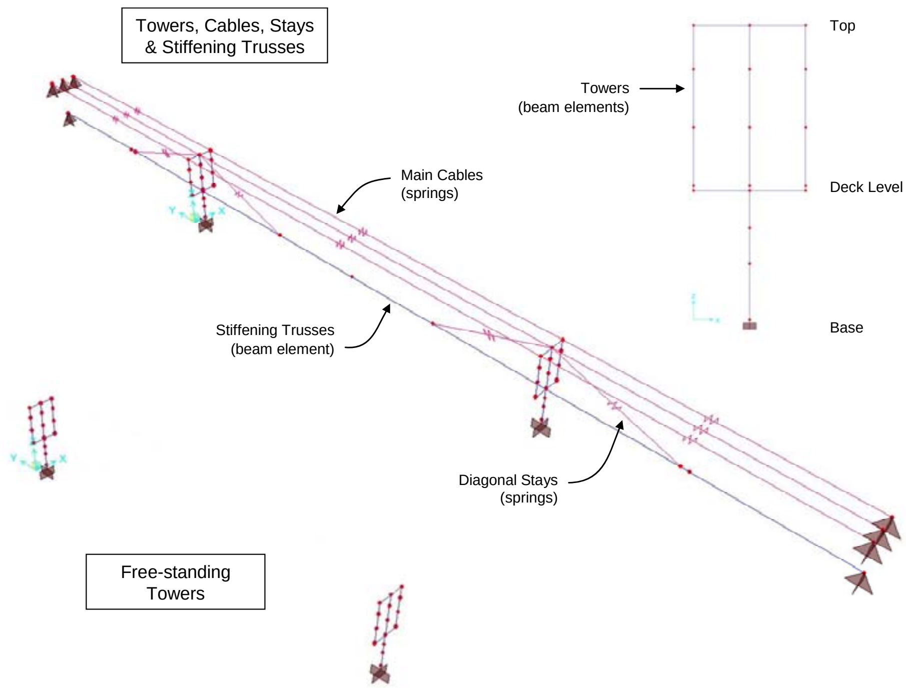


Figure 11: Idealized analytical models

3.3.5 Identification of the tower modes

The idealized model analyses described in the previous section indicated that there may be 2 longitudinal natural frequencies associated with the towers in the 0 to 5 Hz frequency range. While the idealized model incorporated many simplifications and approximations, the results nonetheless serve as a useful guide for identifying the tower natural frequencies from the numerous spectral peaks found in the experimental data.

There are a number of parameters that may be utilized in conjunction with the identified spectral peaks to identify the most likely longitudinal modes of the tower. The parameters that were considered for identifying the modes of the Brooklyn Tower from the pool of 36 identified spectral peaks were: (1) the character of the operating deflection shape, (2) the relative magnitude of the spectral peaks corresponding to the same type of deflection shape, (3) the coherence between the measurement level on the tower selected as a reference and each of the other measurement levels on the tower, the coherence between the reference measurement level on the tower and the other bridge responses such as the span vibrations, and (4) the modal assurance criterion (MAC) values between the operating deflection shapes constructed using each tower measurement level as a reference for a given spectral peak. The availability and effectiveness of these parameters for identifying the modes will depend on the design of the operational modal analysis experiment.

The character of the operating deflection shape may be used to rule out some of the operating deflection shapes if they are outwardly unrealistic, but the effectiveness of this approach is heavily dependent on the heuristics of the data analyst. This type of visual evaluation approach is observed to be more effective if all of the tower measurement levels are utilized as references for generating the operating deflection shapes at a particular spectral peak since the signal-to-noise ratios at all tower levels will tend to reach their maximums at the natural frequencies of the tower. This approach is possible if the vibrations are recorded at multiple tower levels simultaneously. The magnitudes of the spectral peaks for which there are repeated operating deflection shapes may be compared to identify the most likely natural frequency since the maximum response should occur at the natural frequency. The coherence between different measurement levels on the tower should also be maximum (equal to 1.0) between the reference level and all of the other measurement levels on the tower at a resonant frequency [8]. The evaluation of coherence between the different measurement locations on the tower may also be made more rigorous by utilizing all of the measurement levels on the tower as reference levels in a multiple reference approach, but this is limited to experiments that employ a fixed sensor array instead of roving instrumentation scheme. Similarly, the coherence between the tower measurement levels and the measurement stations on the spans can be computed at various spectral peaks to help characterize some of the spectral peaks identified for the towers. If the coherence is high between the measurement stations on the span and the measurement levels on the tower, it may indicate that the mode is a global mode of the structure in which both components are resonating, or possibly that the motions of one component are being driven by resonant oscillations of another component. The MAC values for the operating deflection shapes constructed using different reference levels on the tower may be used to evaluate the overall consistency of these shapes at a given spectral peak.

If the parameters described above are considered for the pool of 36 spectral peaks identified from the measurement data, the following observations may be made. First, it is very difficult to rule out any of the operating deflection shapes shown in Figure 10 that were generated using only Level E on the tower as a reference location as possible tower modes based on their visual character alone. There are no absolutely unreasonable deflection shapes contained in the figures. Even if these shapes are visually compared with expected tower mode shapes from the idealized model, it is difficult to confidently exclude most of the deflection shapes on the basis of their appearance alone. However, if the operating deflection shapes are generated using each tower level as a reference station for a particular spectral peak, it is possible to rule out many of the resulting shapes. Figure 13 and Figure 14 show the operating deflection shapes generated using multiple reference levels for a subset of the identified spectral peaks. Based on the visual consistency or lack thereof for the deflection shapes generated using this multiple reference approach, it is possible to rule out 0.313 Hz, 0.381 Hz, 0.439 Hz, 0.566 Hz, 0.610 Hz, 0.684 Hz, 0.718 Hz 1.719 Hz and 1.807 Hz as likely natural frequencies of the tower. The effectiveness of utilizing a multiple reference approach for ruling out spurious spectral peaks is clearly demonstrated by these figures. When the operating deflection shapes are generated and evaluated for the remaining spectral peaks, only 14 of the total 36 spectral peaks are found to have reasonable consistency for all possible references. The 14 remaining candidate natural frequencies are summarized in Table 2.

The 14 remain spectral peaks can now be evaluated according to other parameters in order to try to identify the longitudinal natural frequencies of the Brooklyn Tower. The first two peaks were identified as the fourth and fifth main span vertical modes in a previous ambient vibration test of the bridge spans [4]. Given the results of the idealized model which showed that the natural modes of the towers are distinct from the spans, it is likely that these two spectral peaks are the result of the spans undergoing resonant oscillations in these modes. The character of the operating deflection shapes for the tower at these frequencies also suggests a trend approaching the third shape type shown in Figure 10. The operating mode shapes associated with the spectral peaks in the range of 1.201 Hz to 1.519 Hz (spectral peaks 3 – 7 in Table 2) are therefore most reasonable for the first longitudinal mode shape of the tower. None of these frequencies were identified as span modes in the previous study, although the 6th vertical mode of the main span was identified at 1.311 Hz, and a corresponding spectral peak is identified from the main span measurements in the present study at 1.309 Hz. As shown in Table 2, the spectral peaks at 1.201 Hz and 1.387 Hz have the largest magnitudes in this deflection shape type group, and are the most likely candidates for the first longitudinal mode of the Brooklyn Tower. Although the magnitude of the spectral peak at 1.387 Hz is slightly larger than at 1.201 Hz, the operating deflection shapes for both are nearly identical. The spectral peaks from the ANPSD function at 3.765 Hz and 3.813 Hz have the largest magnitudes in the sixth shape type group, with the magnitude at the latter frequency being slightly larger than the former. The magnitude of the spectral peak at 3.765 Hz is slightly larger than the peak at 3.813 Hz if the PSD magnitudes for each channel are simply summed with out any normalization at these frequencies, which represents a slight contradiction to the result obtained from the ANPSD function. Therefore, both of these spectral peaks should be considered as possible candidates for the second longitudinal mode of the Brooklyn Tower. The spectral peak at 4.766 Hz has the largest magnitude in the seventh deflection shape type group. The operating deflection shape associated with this spectral peak is also consistent with what might be expected for the 2nd longitudinal mode shape of the tower.

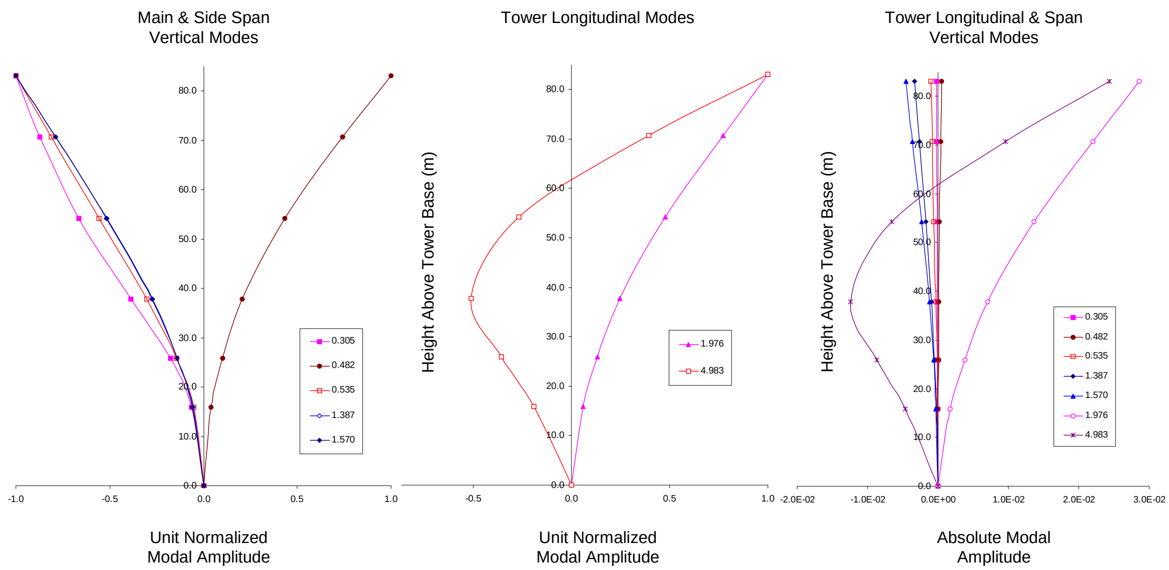


Figure 12: Analytical modal amplitudes of tower in longitudinal direction

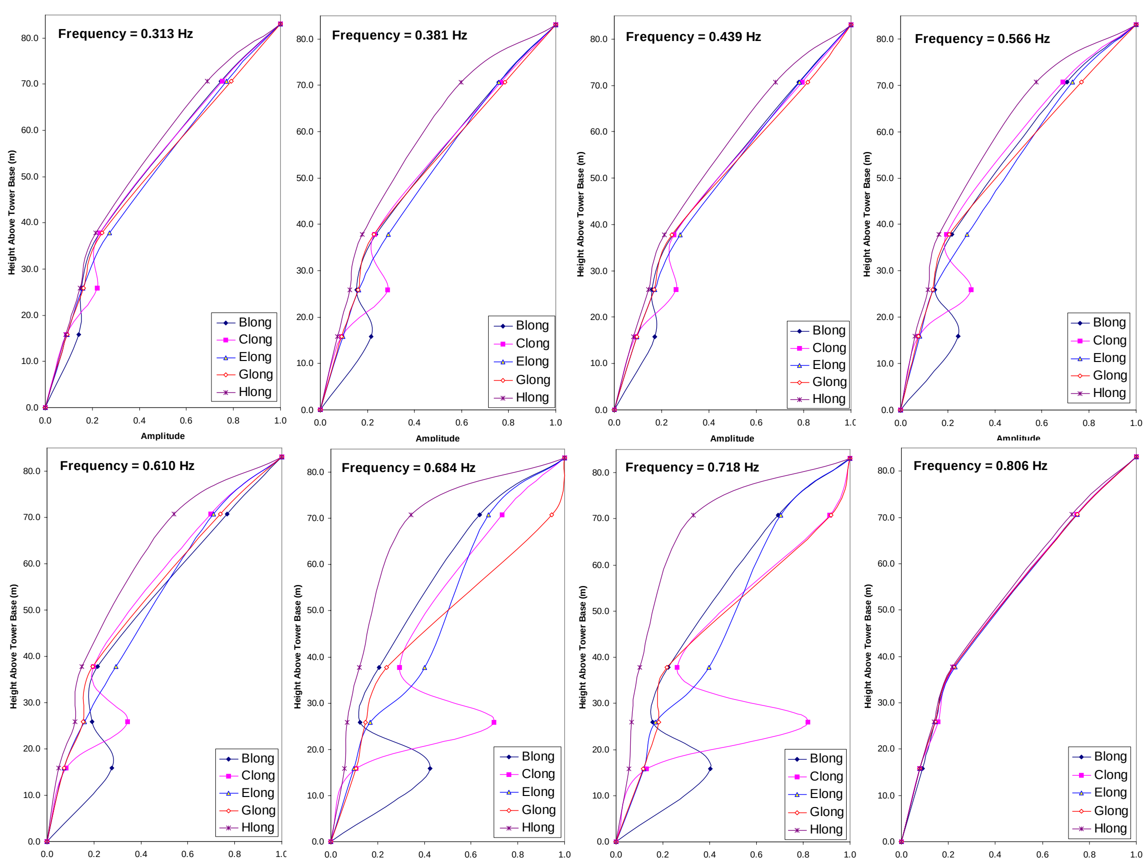


Figure 13: Longitudinal operating deflection shapes for multiple references

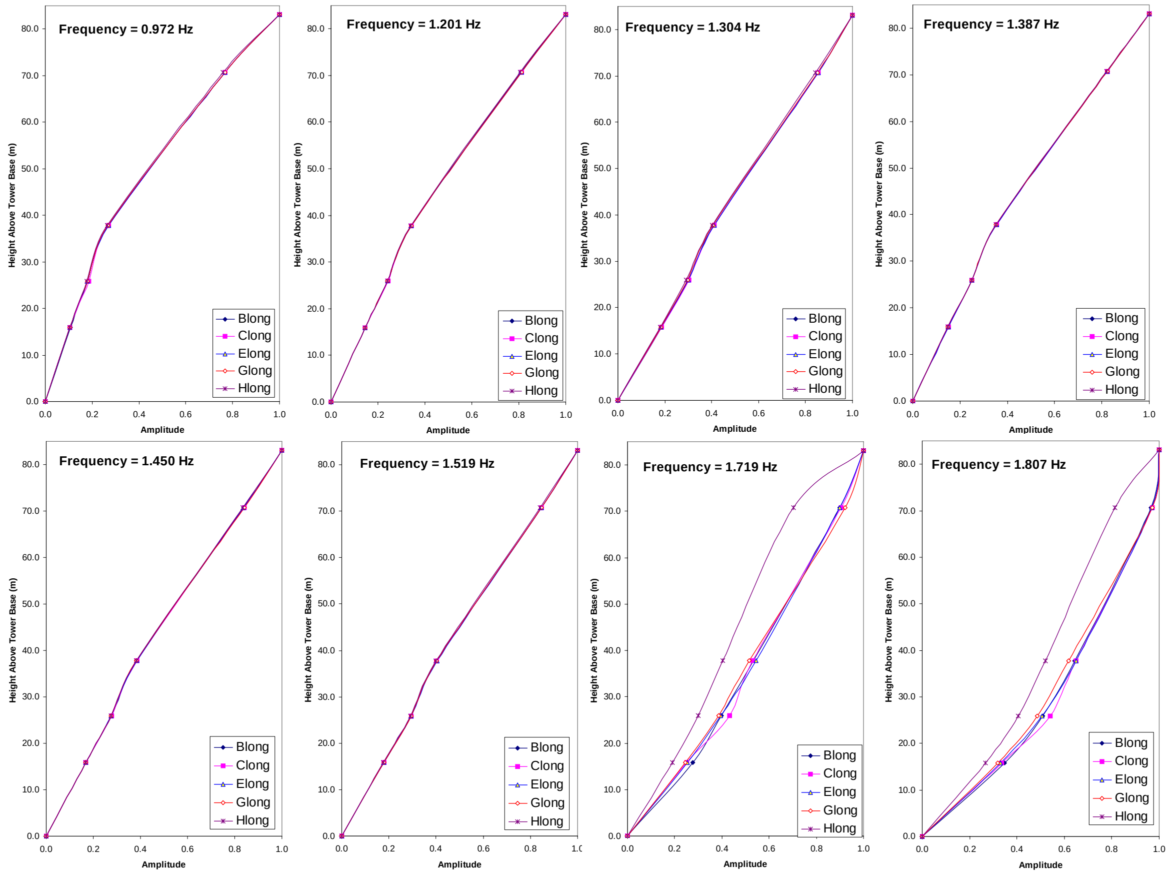


Figure 14: Longitudinal operating deflection shapes for multiple references

If the coherence between the reference level and the other measurement levels on the tower is computed and the results are summed for each frequency line, it is possible to represent the coherence as a percentage of its maximum possible value for a given spectral peak. This may also be done for the multiple reference case in which each measurement level on the tower is used as the reference location to obtain a more rigorous measure of the coherence at a given spectral peak. The resulting percentage of maximum coherence is a convenient parameter for comparing the different spectral peaks. As shown in Table 2, the coherence at 1.387 Hz is slightly better than at 1.201 Hz for both the single and multiple reference approaches, but the difference is not large enough to consider one frequency more reliable than the other as a tower mode. Similarly, the coherence between the reference level and the other tower levels is also slightly better at 3.765 Hz than at 3.813 Hz for both the single and multiple reference approaches.

Table 3 summarizes the coherences computed between the longitudinal response at Level E of the Brooklyn Tower and the two measurement stations on the Brooklyn side span, the two measurement stations on the main span, and the one measurement level on the Manhattan Tower for the 14 spectral peaks shown in Table 2. The values given in Table 3 represent the sum of the coherences between the reference location on the Brooklyn Tower and the other measurement stations on the bridge as a percentage of their maximum possible values. The coherence parameters that are at least 50% of their maximum possible sum are shown in bold text in the table. If the Brooklyn Tower modes are distinct as was observed from the analyses with the idealized models of the bridge, the coherence values should be low between the tower and the other structural responses. The spectral peaks at 1.201 Hz and 1.387 Hz have been identified as the most plausible candidates for the first longitudinal mode of the Brooklyn Tower. A comparison of the coherences between the Brooklyn Tower and the other responses reveals that the spectral peak at 1.387 Hz is more unique to the longitudinal response of the Brooklyn Tower than the peak at 1.201 Hz. The spectral peaks at 3.765 Hz and 3.813 Hz were identified possible candidates for the second longitudinal mode of the Brooklyn Tower. It is clear from Table 3 that neither of these frequencies is closely associated with the vibration responses of other parts of the structure. Since the magnitude of the spectral peak and the coherence between the different levels on the tower are slightly better at 3.765 Hz, this is most likely the second longitudinal mode of the Brooklyn Tower. The coherence parameters for the spectral peak at 4.766 Hz shown in Table 3 indicate that this peak is most likely associated with lateral response of the Brooklyn Tower.

The vibration data was also processed using more sophisticated modal identification algorithms including the Stochastic Subspace Identification (SSI) method from the LMS Test Lab software package. The tower data was processed using all of the measurement levels as references. The use of SSI resulted in the identification of considerably fewer frequencies than were identified from the initial peak picking approach for the longitudinal vibration of the Brooklyn Tower; however, as was the case with the peak picking approach, the identified frequencies had to be considered and evaluated separately as candidates for the longitudinal modes. The resulting longitudinal operating deflection shapes corresponding to the 11 frequencies identified in the LMS software are shown in Figure 15. It is clear from this figure that the deflection shapes are very similar to those identified by the peak picking approach. In addition, the frequencies identified by SSI are also similar to those obtained by peak picking. While the use of the SSI algorithm and LMS software made for a more expedient identification of possible modes, the results were subject to the same uncertainty that was encountered using the peak-picking approach. In other words, the pool of possible frequencies to evaluate was significantly reduced using SSI when compared to the initial peak-picking approach that used a single reference level on the tower to construct the operating deflection shapes; however, the identified frequencies still required additional evaluation due to the uncertainty associated with the dynamic behavior of this structure. In this application, the use of a more sophisticated algorithm did not provide any inherent advantages over the multiple reference peak-picking approach for interpreting the final modes of the tower. In addition, since each step of the multiple reference peak-picking approach was explicit and transparent, the final evaluation of the tower modes could be performed in a more conceptual manner.

The modes identified for the Brooklyn Tower from the data considered in this paper are summarized in Figure 16. The first two modes in the torsional and lateral directions were identified using the same procedure described for the longitudinal modes. It should be noted that the identified modes represent the most likely modes of the tower; however, considerable uncertainty remains associated with them. The uncertainty is inherent to the use of operational modal analysis to characterize one component of a large scale and weakly-coupled dynamic system.

4 Conclusions

This paper described the writers' efforts to identify the modal properties of a masonry tower in a landmark suspension bridge by an operational modal analysis experiment. There were many challenges encountered in the design and execution of the experiment, some of which were due to socio-technical constraints. The experimental challenges were followed by challenges related to the analysis of the experimental results contributing to a significant amount of uncertainty. Much of the uncertainty was related to the weakly-coupled dynamic characteristics of the towers and spans, and the complexity of the ambient excitation. Such a high level of uncertainty that cannot be effectively mitigated due to its epistemic nature distinguishes the experimental determination of the characteristics of constructed systems from mechanical systems that may be evaluated under controlled conditions in a laboratory, and sometimes by separating various subcomponents from each other.

Table 2: Possible candidates for longitudinal natural frequencies of the Brooklyn Tower

Spectral Peak	Frequency (Hz)	Magnitude of ANPSD Spectral Peak	Deflection Shape Type Group (see Figure 10)	% of Max Coherence with Level E	% of Max Coherence All Levels
1	0.806	1.7E-03	1	93.5	91.8
2	0.972	4.2E-03	1	98.0	97.4
3	1.201	1.1E-02	3	99.6	99.4
4	1.304	6.4E-03	3	99.0	98.5
5	1.387	1.5E-02	3	99.7	99.5
6	1.450	9.0E-03	3	99.4	99.3
7	1.519	9.6e-03	3	99.6	99.4
8	3.521	3.5E-04	6	90.6	89.5
9	3.545	3.6E-04	6	91.8	90.1
10	3.765	7.3E-04	6	92.6	90.3
11	3.813	7.6E-04	6	92.4	89.3
12	3.848	5.8E-04	6	90.2	86.4
13	4.668	4.6E-04	7	92.3	89.2
14	4.766	9.2E-04	7	94.8	93.1

Table 3: Coherence between Level E on Brooklyn Tower and other locations on bridge

Spectral Peak (Hz)	% of Maximum Possible Sum of Coherences										
	MT Long	MT Tor	MT Lat	BT Tor	BT Lat	MS Vert	MS Tor	MS Lat	SS Vert	SS Tor	SS Lat
0.806	**81.6**	0.6	0.4	3.3	5.1	**94.3**	18.1	8.5	72.5	0	2.7
0.972	34.3	0.6	0.7	6.5	12.2	18.9	4.6	3.4	91.7	21.9	30.3
1.201	**96.0**	6.7	2.6	12.2	29.2	**70.9**	17.4	9.0	37.8	10.8	6.4
1.304	**73.7**	6.6	1.3	17.7	24.0	**60.5**	32.4	28.4	9.2	2.1	6.5
1.387	**60.3**	0.2	0.5	17.8	23.1	39.1	2.0	2.7	38.8	21.0	30.0
1.450	30.2	0.1	2.4	16.7	14.9	19.6	3.6	3.5	39.0	25.7	30.9
1.519	**82.2**	0.6	0.3	17.6	13.3	30.0	11.1	8.3	32.2	12.3	14.8
3.521	**69.7**	7.9	1.2	1.2	3.5	7.1	0.4	0.5	1.3	1.3	1.7
3.545	**67.3**	4.9	1.5	1.1	3.0	1.1	1.2	0.3	0.3	0.6	0.7
3.765	29.1	0.3	0.1	2.0	3.0	3.2	2.8	2.7	0.6	0.5	0.5
3.813	20.0	2.2	1.7	3.8	4.4	0.8	1.3	0.5	1.7	0.4	0.4
3.848	9.2	2.2	0.1	3.6	5.3	0.4	0.8	2.8	4.0	0.8	0.3
4.668	11.2	2.8	18.7	**51.6**	**92.1**	3.4	3.9	13.9	7.7	3.4	19.4
4.766	14.5	9.9	11.1	**81.3**	**93.4**	0.7	12.5	16.4	3.4	13.2	**51.2**

MT = Manhattan Tower, BT = Brooklyn Tower, MS = Main Span, SS = Brooklyn Side Span,
Long = Longitudinal, Vert = Vertical, Tor = Torsional, Lat = Lateral

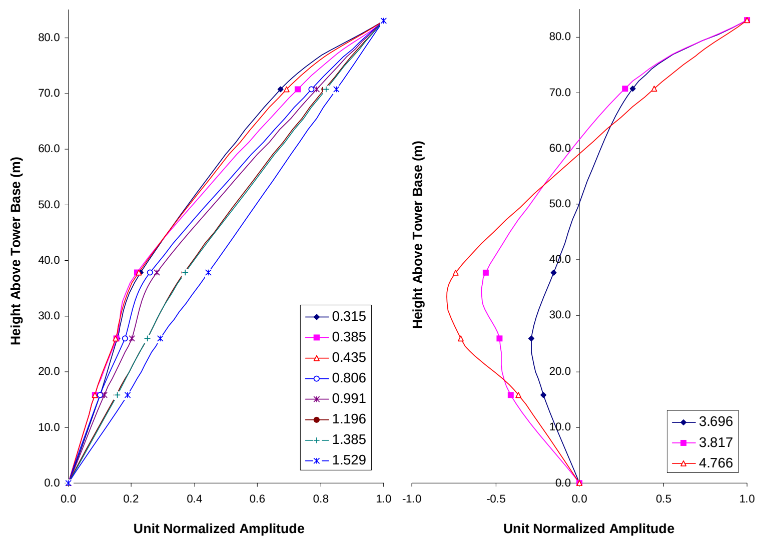


Figure 15: Longitudinal frequencies and deflection shapes from SSI

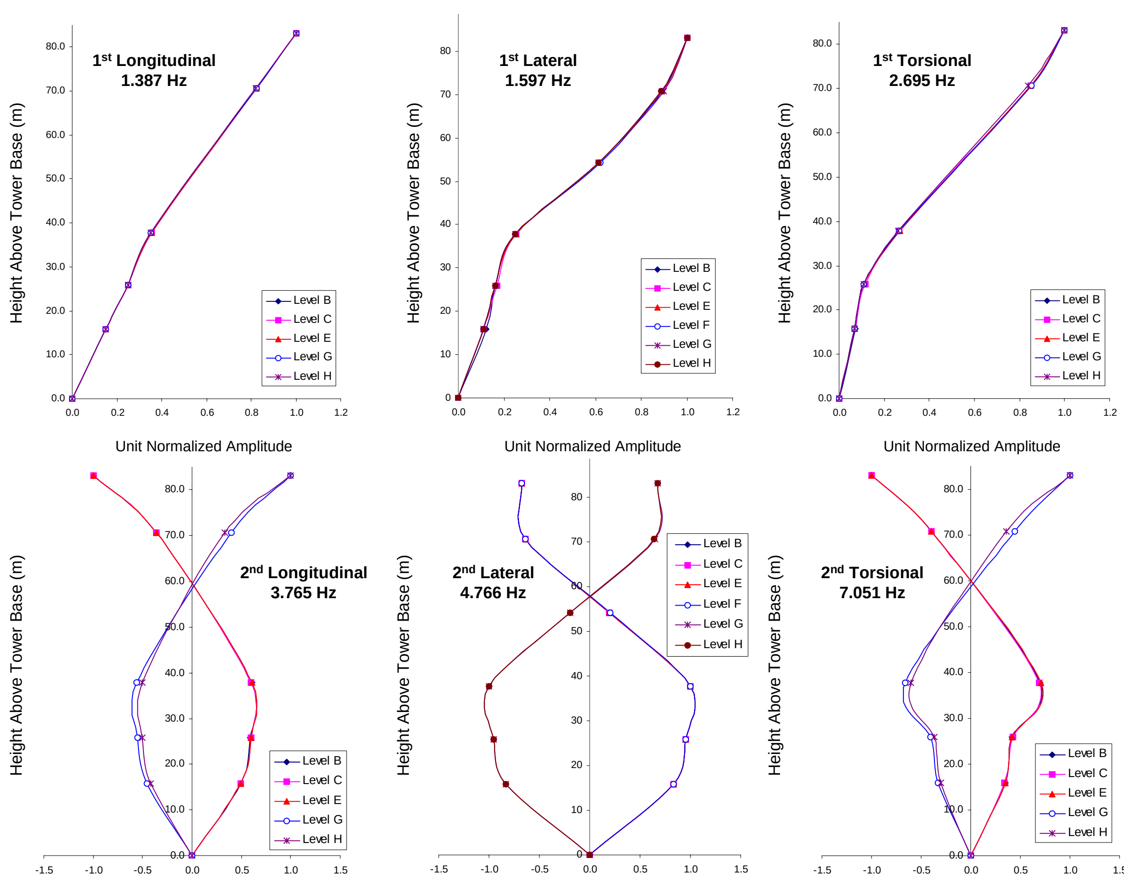


Figure 16: Modes identified for the Brooklyn Tower

In a process-based as opposed to performance-based approach to modal analysis, it would have been quite simple to extract and report a considerable number of operating shapes and corresponding frequencies. In fact, there are many reported applications were only the spans of cable-supported bridges are tested and identified without regard to the interactions or behaviors of the towers or pylons. Often, such tests are conducted by roving a limited number of sensors and justified by assuming the structure and excitation are stationary, and that the excitation is Gaussian white noise. The writers discovered that a process-based approach did not lead to any fundamental knowledge about the structure, and that there were many challenges to extracting relevant and reliable information from the test data. Only by leveraging an integrated analytical and experimental framework, termed as structural identification, was it possible to conceptualize the systems, the different vibration environments of the bridges principal subcomponents, and to differentiate between the distinct natural modes of the tower that was tested from those of the adjoining components and of the global structure.

There are several specific conclusions that may be made as a result of this study:

1. Several approaches were used to evaluate the vibration data. A basic peak-picking approach yielded numerous frequencies that required further evaluation in order to identify the modes associated with the Brooklyn Tower. In general, there would be far more peaks in a frequency spectrum than what may be expected to correspond to a meaningful number of system poles within the frequency band and response direction considered. Many of the reasons for such an overdeterminate nature of operational modal analysis, such as errors associated with a lack of periodicity, discrete numerical processing, modal coupling, and coupling between orthogonal directions are known; however, many others that are specific to a specific experiment, structure, excitation, and data processing methods are unknown. This gives rise to espistemic uncertainty in the post-processing and analysis of the results.

2. The preliminary results from a full 3D FE model of the bridge were available for interpreting the measurement results; however, they were not well-suited for conceptualizing and understanding the dynamic behavior of the principal bridge components. An idealized and approximate analytical model was required to first conceptualize the dynamic behavior of the tower components of the weakly-coupled global system as well as of the entire system itself. The construction of an idealized model that will permit the efficient identification of behavior patterns requires considerable heuristics.

3. The utilization of a multiple reference approach (mimicking the post-processing of a MIMO type impact test) for constructing the operating deflection shapes associated with the identified spectral peaks proved to be very effective graphical approach for ruling out many of the spurious peaks and identifying possible modes. This implies that a roving instrumentation scheme may not be represent the most reliable approach for characterizing the towers of a suspension bridge by operational modal analysis since an overdeterminate set of results all appearing as seemingly meaningful mode shapes may be obtained when only one reference location is used. When a more stringent multiple reference approach is used, many of the same mode shapes may be clearly identified as spurious results.

4. The uncertainty in identifying the modal properties was not related to the algorithm used to conduct the modal identification. The results obtained from more sophisticated algorithms were subject to the same degree of uncertainty as the most basic identification approach. In other words the uncertainty in this case was more a function of the structural system and its vibrations than the measurement data.

5. The coherence information between different measurement locations on the spans and towers was critical for identifying the most likely tower modes. This implies that in cases where a structure consists of very stiff components that are coupled with very flexible components, both should be monitored simultaneously even if only the modal properties of one component are to be identified. The resulting identification will be more reliable and rigorous if more measurement locations can be included on the different components.

6. The writers were able to identify the most likely first 6 natural frequencies and mode shapes of the Brooklyn Tower. There is still some uncertainty associated with the results, and this uncertainty is

greater with the higher frequencies. A feasible forced-vibration test was designed for the characterization of the towers, and this may be the most reliable way to identify the tower modes and their damping characteristics. Furthermore, a forced-vibration test may be carried out at sufficiently high force levels to activate and reveal various tower behaviors that may not be apparent under ambient vibrations.

This particular application showed the type of uncertainty that can be encountered in operational modal analysis of weakly-coupled systems. The uncertainty has important ramifications for reliable system identification and health monitoring of large constructed structural systems. Given that operating modal analysis may, in many cases, emerge as the only feasible method of testing and characterizing large constructed systems such as many landmark or "signature" cable-supported bridges, considerable research on optimal experiment design, execution, data processing is required for a product-based as opposed to a process-based approach. Such research needs to fully leverage a conceptual understanding of the dynamic behavior of a large constructed system, and success requires the use of a structural identification framework.

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