Examples of Vibration-Based Bridge Structural-Identification for Management

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ABSTRACT: Vibration monitoring (operational modal analysis) and vibration testing (modal analysis) of constructed systems for identifying their dynamic characteristics are becoming more common, however, vibration monitoring/testing that actually helped enable critical management decisions by the owners and operators of constructed systems are rare and in many cases the results of such tests remain "academic." Successful vibration monitoring/testing requires the integration of expertise both in theoretical and experimental structural dynamics as well as in the domain of the constructed system that is being studied. In this paper, two application examples on highway bridges by Drexel University researchers are introduced. In the case of the highly deteriorated Smithers Bridge, the flexibility obtained by MIMO testing by impact was validated by static truck load testing under proof-load levels. The bridge deflections based on the modal flexibility and the deflections measured from truck load test correlated very well, indicating that MIMO testing, if properly designed, executed and processed, can serve as a bridge management tool. In the case of the long-span Burlington-Bristol movable lift-bridge, one tower span exhibited larger operational vibration amplitudes than its symmetric counterpart. After conducting ambient vibration testing of the entire bridge, followed by vibration testing of the piers, the FEM calibrated by the measured dynamic properties was utilized to conduct time-history numerical simulation used for confirming the phenomenon. Both examples reveal that there may be wider application prospects of vibration-monitoring/testing by qualified engineers, if they integrate experimental structural dynamics expertise and constructed system domain knowledge, in order to support operational and maintenance management.

KEY WORDS: Structural identification; ambient vibration monitoring; Multiple Input Multiple Output (MIMO) vibration testing; constructed systems; bridge operations; bridge maintenance management.

1 INTRODUCTION

Structural Identification (St-Id) was summarized as a six-step analysis-experiment-decision integration cycle by the ASCE St-Id of Constructed Systems Committee [1], and a state-of-the-art [2] report concludes the recent progresses and applications. Visual approaches used in condition and structural assessment and for designing interventions in the case of deterioration, damage and other performance problems such as excessive vibrations often prove unreliable. In spite of its potential, St-Id is a relatively new discipline for civil infrastructure stakeholders including owners, operators, managers, consultants and contractors. They need clear information about structural deficiencies that may affect safe and economic operations while the research community provides abundant amounts of data but often fails to provide insight and recommendations for reliable solutions. The need for significant technology expertise may discourage condition assessment based on St-Id especially when we consider the needs for controlled load testing or complex finite element modeling, and related correlation and calibration studies. A very few civil engineering consultants actually possess the expertise that is necessary for reliable St-Id, and those in mechanical engineering or mechanics may lack the domain knowledge necessary for successful applications to constructed systems. Additional drawbacks such as cost and disruption of service have ultimately discouraged widespread applications of St-Id.

Experimental modal analysis technology has gradually been incorporated by civil engineers and there is a variety of successful applications on medium and long span bridges which employ the testing technique to identify modal frequencies, evaluation of vibration serviceability, checking effectiveness of retrofits, investigation of resonance problems, and calibration of models for performance prediction. Vibration measurement can also provide information similar to a truck load test in some specific cases. For example, a rigorous MIMO impact test can replicate the results from a truck load test by providing the same load surface measured under trucks by displacement transducers.

2 CHALLENGES, MOTIVATION AND OBJECTIVES

Beginning in the late 1980s, the authors have been involved in successful testing of a wide-range of operating bridges using multi-reference impact test (MRIT) [3-7] as an experimental tool for St-Id. However, experimental modal analysis of large constructed systems still poses major challenges to researchers. For example, MRIT on reinforced concrete (RC) highway bridges face several major obstacles for successful implementation. Substantial energy is required to excite the structures, particularly in their higher modes of vibration. The success of St-Id based condition assessment depends entirely on the accuracy and completeness of the identified structural dynamic properties. If the measured modal properties are incomplete or if these are affected by significant measurement uncertainty, the St-Id for condition assessment becomes of little value.

Long-span bridges represent a special, critical class of construction in terms of their value and impact of their performance on the economic well-being of metropolitan areas. Each long-span bridge offers unique challenges in its individual design, construction, evaluation, operations and maintenance. In St-Id, the global dynamic properties of these bridges are typically identified by ambient vibration monitoring, and the structural dynamic properties extracted from excitation due to traffic and wind are used for calibrating analytical models. Ambient vibration monitoring offers insight into the current state of the structure and reduces uncertainty within analytical predictions.

In this paper, two recent and validated applications of vibration-based St-Id are discussed. The first application describes a rigorous truck load test and MIMO dynamic test on a medium-size cast-in-place reinforced concrete T beam bridge. The second example shows the application of ambient vibration-based St-Id based on frequency and time domain analyses to address a performance concern identified by the bridge owner. These successful applications demonstrate the potential of vibration-based St-Id to support constructed infrastructure systems management decision-making.

3 SMITHERS BRIDGE STUDY

3.1 Introduction

The Smithers Bridge (Figure 1) was constructed in 1930, as a three span, simply supported T-beam Bridge with a skew of approximately 18^{o} (Figure 2). Each of the spans is approximately 14.63m long, with a width along the skew of 14.63m as well. There are six girders along each span, with a transverse diaphragm along the width in the middle and on the downstream side of the span as shown in the plan in Figure 2.

Figure 1. Smithers Bridge

Two additional diaphragms increase the stiffness and the mass on one side. The bridge was posted at 37T and 38T for two and three axle trucks, respectively. The posting impeded coal transportation and negatively impacted the economy of the region. As a result, the WVDOT elected to have a load test and St-Id performed to more accurately assess the capacity and perhaps justify the removal of the posting. The Drexel team conducted a series of static truck load tests and dynamic impact tests on this bridge.

3.2 St-Id of Smithers Bridge

Following the St-Id Steps discussed in Section 1, these were used to design the experiments for the Smithers Bridge. In Step 1 on-site measurement and condition investigations were conducted since the bridge served for 80 years and there was a lack of documentation and drawings. Materials were sampled and characterized. Then, in Step 2 a priori FEM model using the Sap2000 software is constructed to predict safe load capacity, deflections and the frequency range of interest to help design experiments. After MIMO and truck load testing in Step 3, data was processed and interpreted in Step 4. Researchers extracted dynamic characteristics including modal flexibility and static deflections, tilts and strains under various truck-load configurations. In Step 5 these results were used for updating a microscopic FE model constructed by using the software Strand 7 [8]. Analytical model was used to simulate the deflections that were measured during truck load tests, these correlated with the measured static deflections. The measured static deflections were also correlated with those simulated by virtually loading the modal flexibility by the truck loads that were placed on the bridge. Correlation of deflections measured by transducers during the truck-load tests, with those simulated by virtually loading the modal flexibility and also with the deflections simulated by the calibrated FE model revealed the consistency and validated the reliability of the measured data. After a systematic analysis of the bridge by the calibrated model and estimating safe load capacity rating factors for the bridge, it was possible to recommend the removal of posting (Step 6).

3.3 Static truck load test

The static load test was performed on November 16, 2008. The loads were applied using six special military dump trucks capable of being loaded up to a total of 44.64 tons (100 kips) each. A dense array of instrumentation including various strain, displacement and tilt sensors (Figure 2) was utilized to capture the response for the 1st span of the structure.

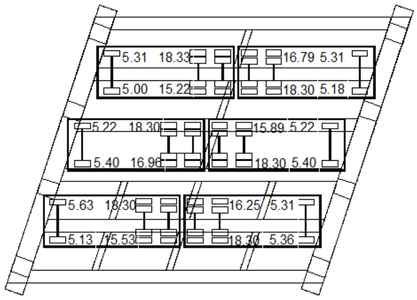


Figure 2. Full truck load configuration for the 1st span

The NCHRP manual for Bridge Rating through Load Testing indicated that a proof load of 89.29 tons per lane or 267.86 tons total load was required. The bridge was loaded incrementally from three empty trucks to six full trucks for a total of 269.64 tons without any damage or distress. On average, each of the front-wheel tire loads were approximately 4.46 tons and the back-wheel tire loads were approximately 8.93 tons. The full truck load test, in which the load configuration is shown in Figure 2, was used for parameter identification. The static instrumentation of the first span of the Smithers Bridge included nearly 40 gages to capture rebar strains, vertical displacements and beam settlements as shown in Figure 3.

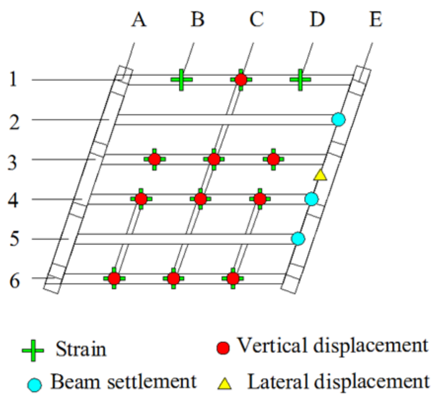


Figure 3. Static test instrumentation layout

Based on the vertical displacement measurements, it was concluded that the bridge showed very little continuity between spans. The displacement profiles at ¼, ½ and ¾ lines of the first span are presented under the full truck load test (Figure 4), and as a result of the skew, the response at ¼ is slightly larger than at ¾. Maximum displacement in the first span was 3.20mm.

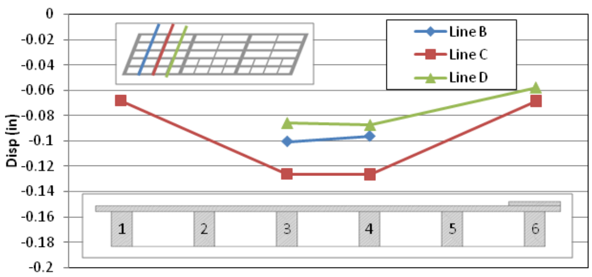


Figure 2.Measured displacements along the three lateral diaphragms of the 1st span (1in=2.54cm)

3.4 *Multi-reference impact test (MRIT) and modal analysis*

In conjunction with the truck load test, MRIT was conducted to extract modal flexibility for the measurement grid shown in Figure 5. The acceleration responses to a dynamic impact force were measured and the frequency response functions (FRF's) were computed to yield modal parameters. Multi-reference testing provided an estimate of the modal parameters by incorporating and smearing the effects of various mechanisms of nonlinearity. The unit-mass-normal modal coefficients obtained through such a test can be directly transformed to flexibility [4].

In the case of the Smithers bridge dynamic test, the input was made by an instrumented 25lb PCB sledge hammer with the second hardest tip embedded in the impact head. PCB 393C piezoelectric accelerometers were attached under the girders by magnetic mounts to steel plates firmly anchored into the concrete. An HBM 24-bit data acquisition system was used for data collection and a sampling frequency of 2400Hz was used in order to capture the impact force at a higher resolution rate in time domain.

Similar to the static instrumentation layout, the modal grid was dense on the first span (Figure 5). The hammer was used to impact the roadway surface at 24 different locations with forces ranging from 15,568 N to 22,241N.

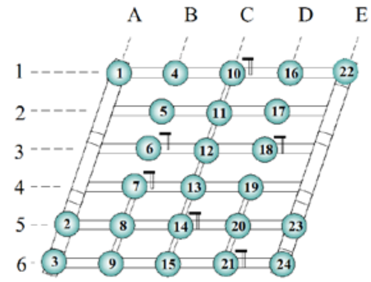


Figure 3. Dynamic instrumentation layout for the 1st span

A main advantage of MRIT is the increased accuracy and better consistency of the FRFs compared to single input test methods. In the reported test, six reference points on the locations 6, 7, 10, 14, 18 and 21 (Figure 5) were selected as the references and were impacted. The signals were pre-processed by using Rectangular-Exponential window to reduce the output leakage during signal processing with 2^15 FFT points to provide a dense frequency resolution. The H1 method [9] of estimating modal parameters was used to extract FRFs for further analysis with CMIF, SSI and PolyMAX methods along a 60Hz bandwidth.

Table 1. Identified modes by CMIF, PolyMAX and SSI.

	CMIF		PolyMAX		SSI	
	Freq (Hz)	Damp. (%)	Freq (Hz)	Damp. (%)	Freq (Hz)	Damp. (%)
1	11.19	6.30	11.22	5.20	11.14	6.36
2	12.95	13.33	13.03	7.35	12.99	7.08
3	12.99	9.90	15.38	9.16	15.56	8.37
4	18.07	7.28	17.68	2.72	17.54	3.91
5	18.93	6.42	18.97	4.79	18.99	5.74
6	29.71	2.69	29.70	2.92	29.67	3.05
7	37.76	6.48	38.19	5.15	37.86	4.79
8	40.85	8.48	39.78	11.21	/	/
9	45.48	6.44	45.40	4.90	45.29	4.69
10	47.61	2.59	47.77	5.96	47.58	2.73
11	50.35	4.29	50.35	3.45	50.72	4.41
12	57.59	2.76	57.48	2.62	57.62	2.50

The identified modes between 0-60Hz exhibited high correlation among the different post-processing algorithms, as indicated by similar frequencies (Table 1). The large damping ratios presented challenges in successfully identifying the lower modes obscured in nearly flat portions of FRF's. The FRF singular value curves produced by the CMIF method are shown in Figure 6.

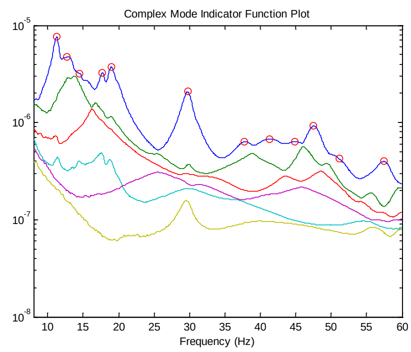


Figure 4. Mode identification in CMIF

3.5 Modal flexibility calculation for Smithers Bridge

The modes identified by the CMIF method were chosen to calculate the modal flexibility. The modal grid shown in Figure 5 had to be reconciled with the truck-loading pattern shown in Figure 3. The 36 truck tire positions did not correspond to acceleration measurement locations and this presented a challenge to rationally correlating the static deflections and those obtained through modal flexibility (Figure 7).

One strategy is to use the measured mode shapes (each mode includes 24 coordinates) and the estimated modal mass to calculate the modal flexibility, so a 24×24 modal flexibility matrix for each accelerometer instrumentation point is obtained. The truck tire positions were then moved as equivalent static loads corresponding with the accelerometer instrumentation points. Then a 1×24 load vector can be developed to calculate a deflected surface for the bridge under the truck loads (Truck Load Surface or TLS). The truck load re-distribution for this strategy is shown in Figure 7 where Figure 7(a) shows the original tire-load pattern and (b) shows the loads transformed to modal grid points as static equivalent loads. This has been a common approach in order to correlate dynamic and truck-load test results, but it does not permit a rigorous understanding of the impacts of known measurement errors, modal truncation and the inevitable epistemic or bias uncertainty in modal flexibility and the subsequent TLS calculation that may come from errors due to the idealizations in the truck load redistribution.

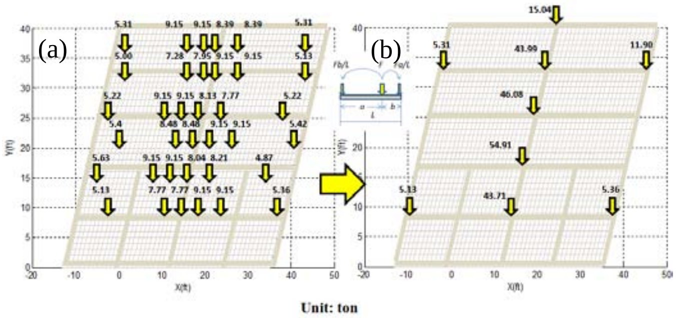


Figure 5. Redistribution of the truck load for method 1.

In extracting modal flexibility from MIMO test results, the first 12 modes were used. A modal flexibility to static flexibility convergence analysis for the instrumentation points on girder 3 was performed as shown in Figure 8. This study indicated that the first 5 modes essentially contribute the same modal flexibility as 12 modes. This implies that the differences between the deflections measured under trucks and those deflections calculated by using modal flexibility (ranging between 6.53%-19.26%) may be due to errors and uncertainty. The question was how much of the deflection discrepancy may be attributed to the transformation of truck tire locations to the modal measurement grid locations.

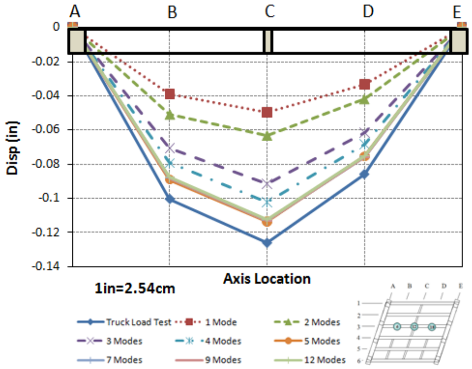
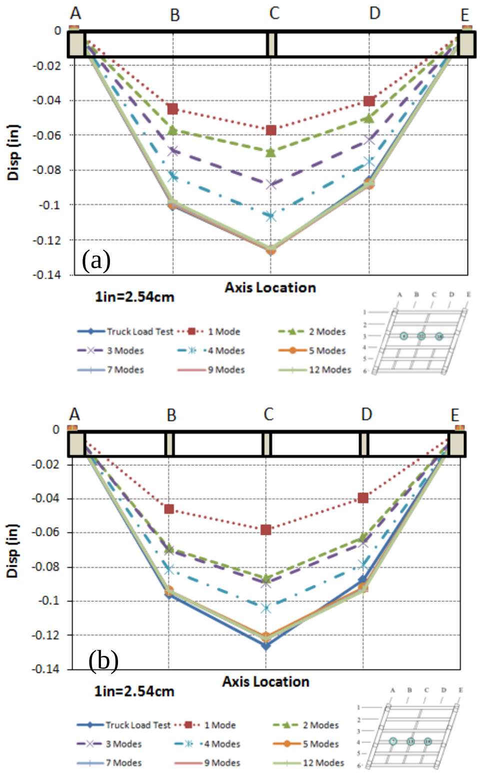


Figure 6. Mode contribution to TLS in method 1 for girder 3

To understand the impacts of transforming static load positions for correlating the deflections from static and dynamic tests, a second strategy was formulated based on modal expansion. In this approach, the mode shapes that were discretized into the 24 coordinates of the modal analysis grid were expanded into refined mode shapes with nodes that matched the actual truck tire load positions as close as possible. The TLS calculated from the refined modal flexibility based on the denser discretization could be directly correlated with the TLS measured under the truck loads. In order to match the tire positions of the six trucks with the expanded modal coordinates, the FE model in Strand7 was used to develop 41*49 interpolation functions numerically which were used to fit cubic polynomials. These cubic interpolation functions were used to expand the experimental mode shapes and also utilized for FE model calibration.



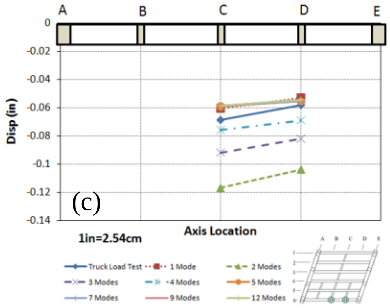


Figure 7. Mode contribution to TLS in method 1 for different girders (a) Girder 3 (b) Girder 4 (c) Girder 6

The correlations between TLS obtained from static load test and the modal flexibility that was based on modal expansion are shown in Figure 9. The TLS from modal flexibility is closer to the displacement profiles measured from the static truck load test and the maximum percent error is reduced to 7.08%.

The modal flexibility is only an approximation of the actual structural flexibility because of truncation given the limited number of modes obtained in dynamic tests. In order to investigate these truncation effects on the accuracy of the flexibility matrix, an index called Load-dependent Modal Flexibility Convergence, *LMC*, is introduced in Eq.(1),

$$LMC = \sqrt{\frac{\sum_{i=1}^{N}(u^m - u\prime)^2}{N}} \tag{1}$$

Where u_m is the virtual displacements based on the loading of a modal flexibility matrix, which is derived from N number of measured frequencies and mode shapes, and u' denotes the displacements obtained from independent load tests. The *LMC* for different girders is shown in Figure 10, and clearly shows that all points practically converge to the measured deflection after the first 5 modes are included. It follows that the accuracy in the identification of the first few modes is crucial to obtain a reliable modal flexibility. Since the bridge deck has asymmetric mass and asymmetric stiffness, the convergence trend is different for each girder as observed from the points on girder 3 and girder 4 that reveal a different convergence trend between mode 2 and mode 3. It is interesting to note that the points on girder 6 diverge at the beginning and then begin to converge after mode 2 is added. This behavior is caused by the asymmetric mode shape and it is a unique characteristic for this type of structure. It has to be emphasized that the flexibility coefficients for the points on girder 6 continue increasing as additional modes are added.

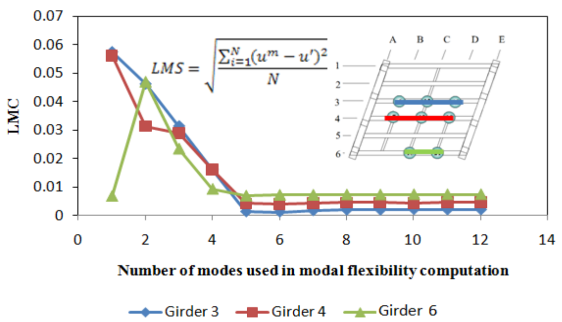


Figure 8. Modal flexibility convergence study

The measured frequencies and mode shapes were used for FE model calibration and for simulating the rating factors under the state legal load of 35.71 tons. New rating factors were significantly greater than 1, indicating that with minor cosmetic repair, the posting of the bridge may be safely removed to permit the bridge to serve the coal trucks.

The Smithers Bridge case study is characterized by challenges associated with high damping, difficulty in identifying and selecting the modes, FE model updating techniques, mode shape interpolation, and truck load redistribution. It is important to note that the MIMO dynamic test based St-Id method can be used on a complicated and locally deteriorated constructed system such as the Smithers Bridge. It also revealed that extensive domain-specific expertise is needed for the application of MIMO testing and modal parameter extraction for a reliable condition assessment. Writers believe that with additional demonstrations of the reliability of MIMO test based modal flexibility by showing good correlations with static test measurements, eventually a new standard for bridge condition assessment based on impact testing may be developed and a new industry to complement visual inspections may be born.

4 THE BURLINGTON-BRISTOL BRIDGE STUDY

4.1 Introduction

The Burlington Bristol Bridge (BBB) serves the communities of Burlington, NJ and Bristol, PA by spanning the Delaware River fifteen miles north of Philadelphia, PA. The bridge consists of a main, 164.59m (540ft) steel through-truss vertical lift span with adjacent 60.96m (200ft) steel through-truss tower spans and steel deck-truss approach spans (Figure 11). The bridge was built in 1931 and has been owned and operated by the Burlington County Bridge Commission (BCBC) since 1948.

Figure 9. Burlington Bristol Bridge

After the completion of the project, the authors and the BCBC realized the potential of creating a partnership which would allow for both short term and long term monitoring projects aimed at maintaining a safe and undisrupted operation of the structure. The authors began building the framework for a Structural Health Monitoring and Management (SHM) plan through interviews with the bridge maintenance staff, operators and bridge engineers with experience in movable bridges, as well as capturing dynamic responses of the structure. One question that was posed by the maintenance engineers related to explaining why one tower span had considerably higher perceived vibrations than its apparently symmetric counterpart. Initially, this question was slated for further review since an in-depth vibration survey of the entire structure was needed to verify the claims that one span vibrated more than the other.

4.2 *Structural Identification of BBB*

Following the established Structural Identification (St-Id) Steps (as outlined in Section 1), these Six Steps were used as a framework for designing the health monitoring plan for the BBB. Upon completing the document review and gathering of existing data (Step 1) as well as preliminary measurements taken on the structure, an a priori model of each of the Three main spans was individually built by leveraging SAP2000 (Step 2). These models were used in conjunction with the preliminary data collected (Step 3) to identify key uncertainties as well as to identify the nodal points of mode shapes, vibration amplitudes and bandwidth. This information was then used for designing an efficient instrumentation plan for an in-depth vibration survey (Step 4). Then a priori models were constructed by using frame and shell elements. Each of the Tower spans and the Lift span were initially modeled and analyzed separately, to minimize computation time.

The in-depth vibration survey was carried out in the months of July and August, 2009. A series of five instrumentation set-ups were required to capture the vibration responses of the main structure: (1) NJ deck-truss span and substructure, (2) NJ tower span and substructure, (3) lift span and substructure, (4) PA tower span and substructure and (5) PA deck-truss spans and substructure. Each instrumentation set-up utilized 45 accelerometers, 30 of which consisted of PCB 393C piezoelectric accelerometers and 15 of which consisted of PCB 3701 capacitive accelerometers. Each sensor was hard-wired into an HBM Mega-DAQ data acquisition system and was sampled at a rate of 200Hz. The decision to sample at this rate was based on the preliminary vibration surveys and natural frequencies estimated from a priori modeling, which indicated that the highest anticipated natural frequency of interest was within 20Hz, well below the Nyquist frequency of 100Hz associated with a sampling rate of 200Hz.

The data collected over the six weeks of field work was then processed using Stochastic Subspace Identification (SSI) to estimate natural frequencies and corresponding mode shapes. The frequencies and mode shapes were used to globally calibrate a finite element (FE) model of the major span systems of the bridge (Table 2).

Initially, FE models of each main span system were constructed and analyzed separately. However, due to the uncertainties associated with stiffness contributions of adjacent spans, and more specifically the loss of information in modeling adjacent spans whose boundary conditions were simulated by assigning linear springs, as well as the inability to study how various load inputs affect the vibration response of the entire bridge, a comprehensive model of the entire bridge including approaches was constructed in Strand7 (Figure 12). This model of the entire structure was rigorously error-screened by comparing analytical frequencies and mode shapes to experimental measurements in addition to using built in error-screening features within Strand7.

The model was then calibrated to globally match the data, since a more refined calibration process by also leveraging static truck-load tests was being planned for the future. Parameters that were modified to match the experimental data to analytical predictions included compatibility conditions between the lift span and the tower spans, boundary conditions at the lift span supports, tower span expansion bearings, and truss span expansion bearings, as well as compatibility conditions between the counterweights and the tower spans. The compatibility conditions at the counterweight/tower span location and lift span/tower span location were adjusted by including connection elements which allow for manual perturbation of global stiffness values. In this manner, lateral or longitudinal stiffness between these structural systems could be modified until reasonable correlation with experimental data was achieved.

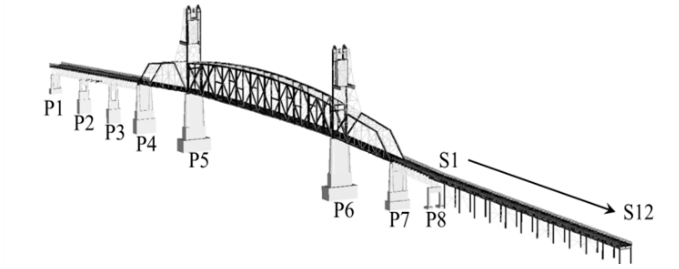


Figure 10. Strand7 FE Model of BBB

4.3 *Time History Analysis*

The main goal of this study was to identify the root cause of the asymmetrical vibration characteristics of apparently symmetric Tower spans. The two Tower spans shared the exact same contract drawings which detailed their construction and have similar adjacent approach spans.

Table 2. Measured and analytical frequencies for BBB

Mode	Span	Shape	f_{exp}	f_{ana}	% Δ
1	Lift	1st Lat.	0.55	0.54	1.46
2	Lift	1st Vert.	1.29	1.31	1.55
3	Tower	1st Lat.	1.56	1.48	5.13
4	Tower	1st Vert.	2.31	2.30	0.43
5	Tower	2nd Lat.	2.57	2.70	5.06
6	Lift	2nd Vert.	3.30	3.60	9.09
7	PA Truss	1st Vert.	3.71	3.71	0.00
8	Tower	3rd Lat.	3.53	3.74	5.95
9	NJ Truss	1st Vert.	4.00	4.10	2.50
10	Tower	2nd Vert.	4.03	4.15	2.98
11	Lift	3rd Vert.	4.73	4.44	6.13
12	NJ Truss	1st Tors.	5.98	6.10	2.01

A second iteration of the St-Id process was required to reassess the problem given the insight from the studies carried out so far. In analyzing the measured time histories of each of the spans under ambient vibrations, it was noted that the two tower spans had similar mode shapes and frequencies, but the magnitude of their acceleration time histories were different. The peak lateral vibration of the top chord on the NJ Tower Span was roughly four times the magnitude of corresponding lateral vibration on the top chord of the PA Tower Span (Figure 13a), while each of the bottom chord lateral (Figure 13b) and vertical accelerations were of equal magnitude (Figure 13c-d). It is important to note here that the time histories of both spans were not taken synchronously, but that each span was individually recorded. However, care was taken to select datasets that occurred at the same time of day to ensure similar excitation levels.

Given the conclusion that a difference in vibration characteristics existed between the two tower spans, in that the peak lateral top chord accelerations were four times the response in the NJ tower span as the PA tower span, a more refined ambient vibration monitoring was needed to target the source of this vibration amplitude difference. An ambient vibration monitoring study was planned to characterize the vibration properties of the substructure in a more densely instrumented manner than previously in the overall vibration study.

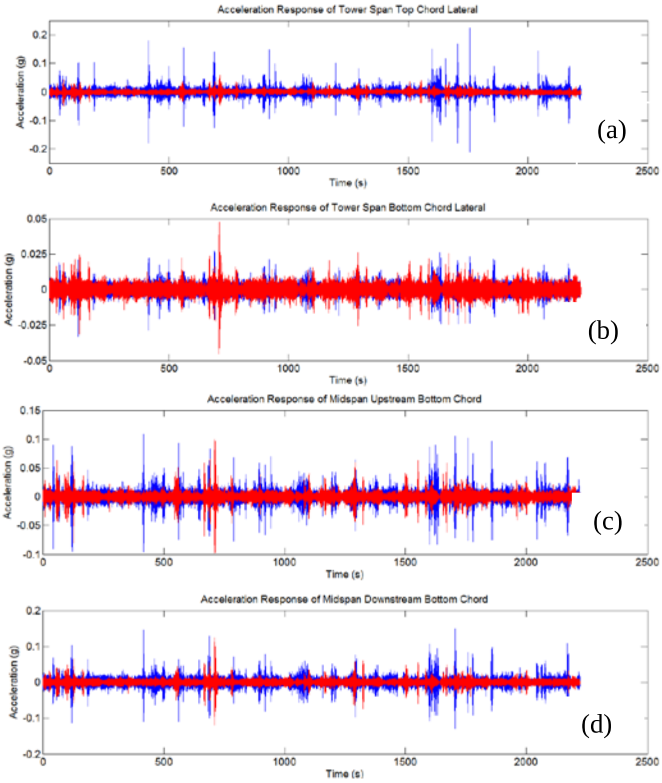


Figure 11. Measured vibration response of (a) top chord lateral, (b) bottom chord lateral, (c) upstream bottom chord vertical and (d) downstream bottom chord vertical for NJ Tower Span (blue) and PA Tower Span (red)

The results from the pier vibration study concluded that Piers 1 through 7 (Figure 12) exhibited very low ambient vibration response in lateral and longitudinal directions; however Pier 8 exhibited a much higher lateral vibration response. This can be attributed to the foundation types for each of these piers. Piers 1-7 are founded on a series of wood piles, while Pier 8 is founded on a concrete spread footing on rock with no piles. The next step in the second round of St-Id analysis was to reanalyze the model by a linear time-history analysis to compute span accelerations in response to a defined input.

4.4 Linear Time-History Analysis

The Strand7 model was once again utilized in carrying out the linear time-history analysis. Two cases were analyzed for this investigation consisting of different live loading patterns: (1) one legal truck on each adjacent approach span, and (2) one legal truck on each tower span (Figure 14).

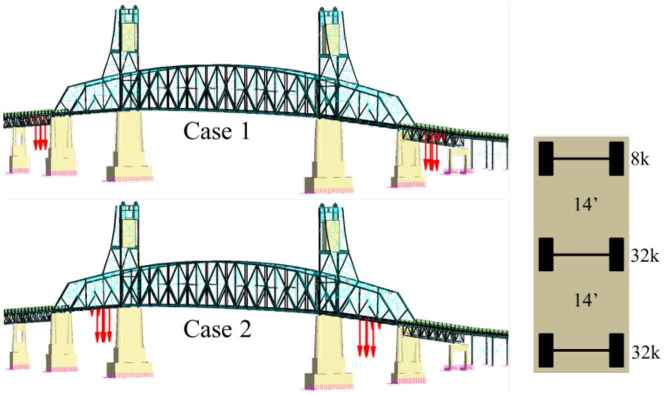


Figure 12. Load cases and truck configuration for linear time step analysis

The truck loads were characterized by an input with random noise between the bands of 6 and 14 Hz (Figure 15), which was based on typical input frequency of truck inputs to long-span structures. The force time-history was generated automatically in Matlab by creating a series of random data and then passing the data through an appropriate filter to obtain the desired banded response. A time step of 0.001s was used in the analysis over a total analysis time of 100s.

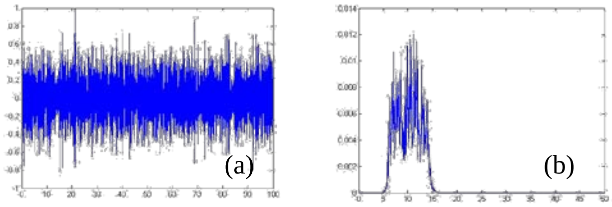


Figure 13. Forcing function (a) time history (b) Spectrum

The results from the analysis were analyzed in similar manners as the experimental data. By visually inspecting the level of peak amplitude response in the lateral direction of each tower span, Case 2 (where the load was applied on the deck of the tower spans) was determined to be inconclusive since all vertical and lateral responses were identical between the spans. The Case 1 results told a much different story, however. When the time histories computed by the analysis were compared between the spans, an almost exact replication of the experimental data was achieved. It was seen that the lateral top chord accelerations were roughly four times the amplitude in the NJ Tower Span than its PA counterpart, while the bottom chord lateral and vertical accelerations were similar. The lateral time histories are shown from the finite element analysis (Figure 16).

The finding that the analytical acceleration response computations to a legal truck load configuration applied at a specified frequency band matched the experimentally measured amplitudes from the acceleration time-history records provided meaningful conclusions. Most pertinent to the bridge owner was that the perceived vibration difference between the two spans was a real phenomenon, however was not due to any structural or sub-structural damage. The iterative St-Id process forced the researchers to conclude that the vibration difference was due to the fact that the spans were not exactly symmetrical, due to differences in the manner the super-structures were supported by the sub-structures and in the lateral stiffness provided by the adjacent spans.

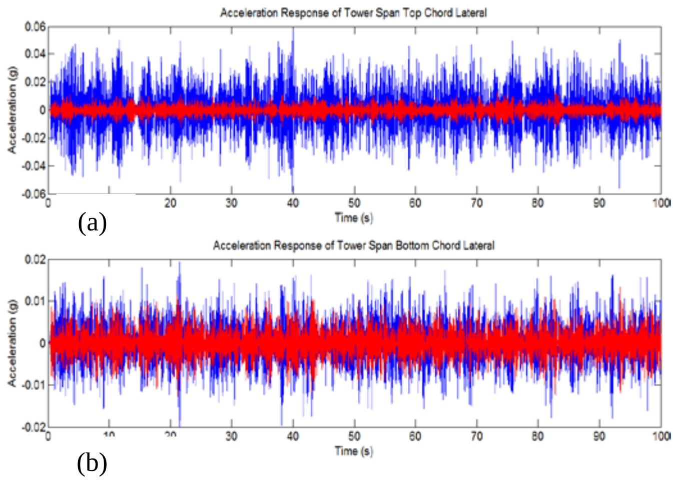


Figure 14. Analytical vibration response of the tower spans to loading on the adjacent approach spans in (a) the top chord lateral and (b) bottom chord lateral locations.

The main reason for the higher lateral vibrations on the top chord of the NJ Tower span was due to a much more flexible approach span along the NJ corridor. Figure 12 shows that in addition to the fact that Pier 8 is founded on shallow footing, the remaining approach system S1-S12 are supported on steel columns. When the approach spans were excited with the legal truck load, the lateral vibrations induced by coupled vertical-torsional frequencies of the NJ Truss Span were transmitted into the NJ Tower Span. The fact that one tower span vibrated more than the other could therefore be attributed to the difference in the geometry of the entire structure, including the approach spans, and not to any specific damage or lack of stiffness in the identical spans. This experience revealed the importance of considering an entire superstructure-substructure-foundation-soil system with all the intricacies in their interfaces in designing and evaluating vibration measurements on a portion.

5 CONCLUSIONS

Vibration-based Structural Identification is not a new application within the civil engineering discipline; however it has found only limited success with practical applications due to the costs associated with deployment, equipment and the level of expertise that is needed. This paper provides overviews of two different studies to demonstrate the potential of practical applications of vibration-based techniques applied within a St-Id framework to operating constructed systems.

The first example demonstrated a challenging St-Id problem corresponding to the load rating of the Smithers Bridge. Barriers included very high damping due to material deterioration at various local areas on the deck of the bridge, identification of the critical poles, FE model updating techniques, mode shape interpolation and truck load simulation. It is noted that MIMO modal analysis can be used on such a complicated structure but requires integration with domain-specific expertise for reliable condition assessment. The second example demonstrated how an in-depth vibration characterization of a long-span structure required not only modal parameter estimation to calibrate FE models of individual spans, but also provided time domain measurements that allowed for direct comparison with an analytical linear time-history analysis. A perceived risk was mitigated, as the difference in vibration of two seemingly symmetric spans was attributed to transmitted vibrations from the much different foundation conditions and interface details of the approach spans.

These two cases demonstrate that vibration-based St-Id can be used to support constructed infrastructures management decision-making. More powerful and practical prospective condition evaluation technology based on vibration testing should be expected to become available in the near future.

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