Modal Test-Based Condition Assessment

Scott J. Kangas, Arthur J. Helmicki, Victor J. Hunt, Bahram Shahrooz, and David Brown
University of Cincinnati Infrastructure Institute
ECECS Dept., ML-30, University of Cincinnati, Cincinnati, OH 45221-0030

Michael S. Lenett
URS Greiner, Cincinnati, OH 45202

ABSTRACT

Multi-reference impact testing has been utilized as a basis for as-is condition assessment of highway bridges. The major advantage of a modal (impact) test is the quick setup, execution and teardown of such a test. A modal test can be performed within a 3-4 hour period providing a minimal amount of down time to the structure being tested. Previously, it was a laborious task to transform impact data to modal flexibility. Once the data had been collected, it took several days before a reliable flexibility could be identified. However, after upgrading the software package used, this task has been reduced from several days to a matter of minutes, allowing a real-time assessment of the condition of the bridge.

One bridge tested, a continuous five-span reinforced concrete deck on steel-girder bridge in Dayton, Ohio, had its original deck replaced with an interlocking system of fiber reinforced polymer (FRP) panels. Impact tests were performed on the original bridge to establish baseline signatures such as natural frequencies, mode shapes and flexibility. Follow-up tests were performed on the FRP system to determine the influence the new deck had on the overall bridge behavior. Changes to the bridge characteristics could then be attributed to the existence of a lighter bridge deck, loss of unintended composite action with the deck, and unexpected delamination of the FRP panels.

Nomenclature

$[H(\omega)]$	FRF matrix
$[K]$	Stiffness matrix
$[\ f\ ]$	Flexibility matrix
$\{\psi\}_r$	r^{th} mode shape
$\{\psi\}_r^T$	Transpose of r^{th} mode
$\{\psi\}_r^*$	complex conjugate of r^{th} mode
$\{\psi\}_r^{*T}$	Hermitian of r^{th} mode
λ_r	r^{th} eigenvalue $(=\sigma_r + j\omega_r)$
M_{Ar}	Modal A (scaling) for r^{th} mode

INTRODUCTION

The University of Cincinnati Infrastructure Institute (UCII) was contracted by the Ohio Department of Transportation (ODOT) to investigate the utility of fiber reinforced polymers (FRP) in the rehabilitation of an existing bridge. The test specimen (Figure 1) was the MOT-49-10 bridge located in Dayton, Ohio. This bridge consists of five spans whose lengths are, respectively, 130 ft., 137 ft., 145 ft., 137 ft., and 130 ft. Each span consists of six built-up steel girders. The reinforced concrete deck was badly deteriorated and was replaced with interlocking FRP panels in the Summer of 1999. The large size of the bridge allowed the installation of FRP deck systems from four manufacturers. Reising, et. al.,[1] provides a description of each individual FRP deck system used to replace the existing the reinforced concrete (RC) deck.

FRP panels are characterized by their lighter weight, higher strength, and greater flexibility compared to conventional RC decking. This retrofit to the MOT-49-10 bridge constitutes a change in superstructure stiffness and thus a change in structural condition. Past UCII research has demonstrated that multi-reference impact test methods provide an accurate measure of in-situ flexibility for steel-stringer bridges.[2, 3, 4] Flexibility can be used as a conceptual index to assess the condition of and/or damage to such bridges. Consequently, it was proposed that a regimen of field testing be conducted before and after the retrofit to track any immediate and long-term changes in structural condition. The testing program included (a) testing of the original bridge with RC deck, (b) testing of the new bridge with FRP panels before any traffic service, and (c) repetition of these controlled tests every quarter for a two-year period.

This exploratory project discovered several unexpected problems with the retrofit procedure and the long-term behavior of the FRP panels.[5] The initial test of the new bridge was conducted in December, 1999. In the Spring of 2000, some of the panels begin to delaminate under traffic service and develop noticeable “bubbles”. An investigation revealed the location of many pockets of such deterioration, as well as a concern regarding the proper seating of the panels to the girders at all locations. Extensive repairs were required and the testing regimen was delayed. In this paper, modal test results from 2001 are included.

TEST PROCEDURE

Due to the overall length of the bridge, the impact test grid for MOT-49-10 was divided into two sub-grids, North and South (Figure 2). Each sub-grid utilized 36 accelerometers mounted on the upper side of the bridge deck. These sensors were spaced at intervals that positioned them above girder-crossframe intersections and girder-bearing positions associated with the two innermost girders. Impact force was applied, through the use of a specialized, mobile impact hammer (Figure 3a), near six sensor positions in each respective sub-grid to ensure excitation of pertinent superstructure modal characteristics, i.e. mode shapes and natural frequencies. Impact force and acceleration response signals were acquired using a Hewlett-Packard (HP) VXI data acquisition system (Figure 3b) and processed with MATLAB-based software. The North and South sub-grids in Figure 2 were utilized for both the original bridge and the retrofitted bridge. Some latter tests also incorporated additional accelerometers that were installed upon the girders themselves, directly beneath their deck sensor counterparts.

IMPACT TEST RESULTS

Impact test data from both the North and South grids of the original and retrofitted bridge tests was used to identify mode shapes at natural frequencies ranging from approximately 2.5 Hz to 40 Hz. Several of these modes are depicted in Figure 4. Comparisons involving the first 8 modes of the original and retrofitted bridge tests are presented in Tables 1 and 2. These tables not only provide a comparison of natural frequencies, but also display how well the respective modes correlate. The modal assurance correlation (MAC) coefficients presented within these tables indicate the similarity between a pair of respective modes – a value of zero, or N/A, indicates there does not exist a pair of similar modes whereas a value of 1 indicates perfect similarity between the pair of identified mode shapes.

As can be seen in Tables 1 and 2, certain mode shapes identified with retrofitted bridge data were not identified from original bridge data and vice versa. Furthermore, the North and South retrofit results presented in these tables reveal that the southernmost spans – the spans that use interlocking FRP deck panels – have slightly different modal characteristics than the northernmost spans which utilize cast-in-place concrete with stay-in-place fiberglass forms and fiberglass rebar. In other words, the type of FRP system installed on the bridge has a local effect on the dynamic behavior of the superstructure – different FRP systems have different local effects. Consequently, the results presented within Tables 1 and 2 essentially reveal that the dynamic behavior of the superstructure changes when the reinforced concrete deck is replaced with FRP systems. The identified variations in dynamic behavior may be attributed to changes in superstructure stiffness induced through the FRP retrofit operations. However, changes in superstructure stiffness should also be reflected in the acquired/identified measurements of in-situ structural flexibility, or modal flexibility.

The identified modes were used to compute modal flexibility (Equations 1 and 2), which previous research has shown to be sensitive to damage and condition.[2, 3, 4]

$$\left[H(\omega = 0)\right] = \frac{1}{\left[K\right]} = \left[f\right] = \mathit{flexibility} \qquad \textbf{(1)}$$

$$\left[f\right] = \sum_{r=1}^{N} \left[\frac{\{\psi\}_r \{\psi\}_r^T}{M_{A_r}\left(-\lambda_r\right)} + \frac{\{\psi\}_r^* \{\psi\}_r^{*T}}{M_{A_r}^{\;*}\left(-\lambda_r^*\right)} \right] \qquad \textbf{(2)}$$

To obtain a simple yet direct comparison of the bridge condition for various tests, superstructure displacements along a particular girder line were used as the basis for comparisons between the original and retrofitted bridge. The particular displacement profiles were obtained by multiplying the respective modal flexibility matrices with a load vector composed of virtual downward loads of unit magnitude positioned at each measurement location along the girder line of interest. Figure 5a and 5b shows the displacement, for unit load, along girder three on the north and south side for several tests.

The first test, conducted in June of 1999 with the original reinforced concrete deck, was used as a baseline signature of flexibility for the structure. The test performed in December of 1999 was the first test conducted on the new FRP deck. It resulted in a displacement profile similar to the concrete deck, especially on the south side. However, it must be noted that this test data was initially corrupted by an impact node located coincident with a delamination "bubble" (which was only subsequently obvious after repeated traffic service).

Subsequent tests performed on the FRP deck show an increase in flexibility, or decrease in stiffness. One would expect the flexibility plots, for the tests on the FRP deck, to be in agreement with one another, but this can be explained. This difference can be attributed to repairs that were performed between December of 1999 and October of 2000. Initially, the panels were delaminating and had poor bearing on the steel superstructure.[5] Follow-up tests reveal an increase in flexibility that is consistent for the three tests, two on the north side, since the repairs were performed. When comparing the original reinforced concrete deck to the FRP deck, it is apparent the flexibility of the structure has increased. This can be attributed to the loss of unintended composite action between the deck and girders.

Instrumentation was installed for latter tests in order to confirm the integrity of the repaired seating of the FRP panels to the steel girders. Considering the many problems with the retrofitted panels, there was some concern raised with this bridge specimen as to whether the sensors mounted on the deck were measuring the response of the entire superstructure or simply the decking alone. Additional accelerometers were placed upon the girders themselves, directly beneath their sensor counterparts on the deck above (see Figure 2). A comparison of their frequency response functions (FRF) illustrates that the deck and girder are indeed acting as one in response to the controlled hammer impact (Figure 6). Differences are only evident at points were electronic noise dominates. Hence, the validity of the modal tests results were confirmed for the superstructure as a whole.

CONCLUSIONS

The results from modal tests performed over a two-year time frame indicate a change in flexibility that can be attributed to the new decking system. The increase in flexibility occurred after the new deck had been installed and repairs were made to the FRP system and can be attributed to loss of composite action between the panels and the girders. Although modal testing cannot directly measure composite action, it was able to indicate a change in condition that could be explained by loss of composite action.

ACKNOWLEDGEMENTS

The authors wish to acknowledge ODOT, and in particular, Messrs. Vikram Dalal, Brad Fagrell and Lloyd Welker, for their support and guidance of this project.

Figure 1: Overview of MOT-49-10.

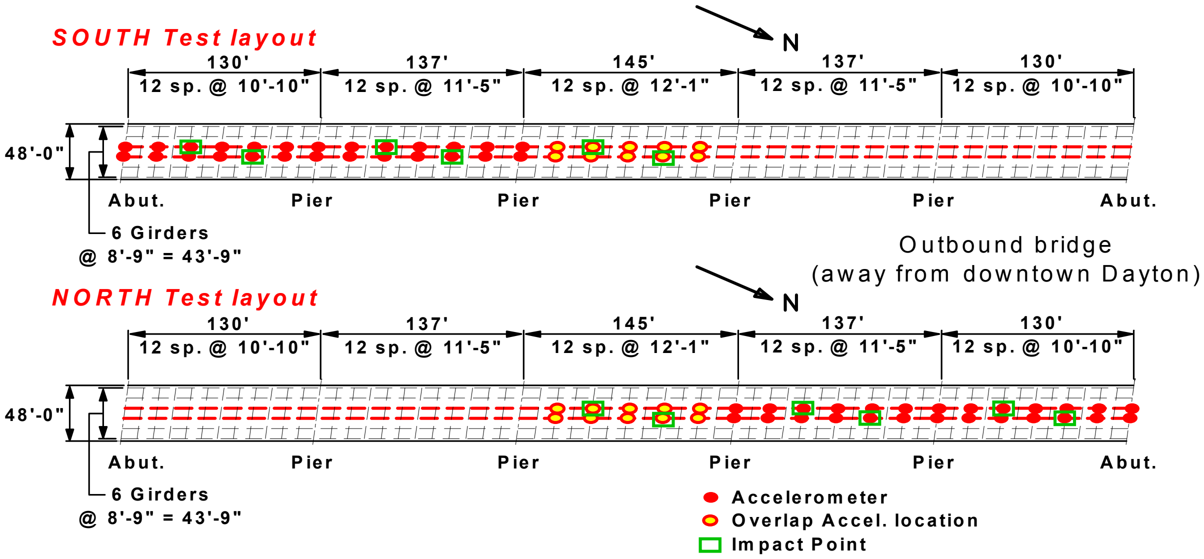

Figure 2: MOT-49 Impact Test Grids

Figure 3a: Impact Hammer.

Figure 3b: VXI system and South test setup.

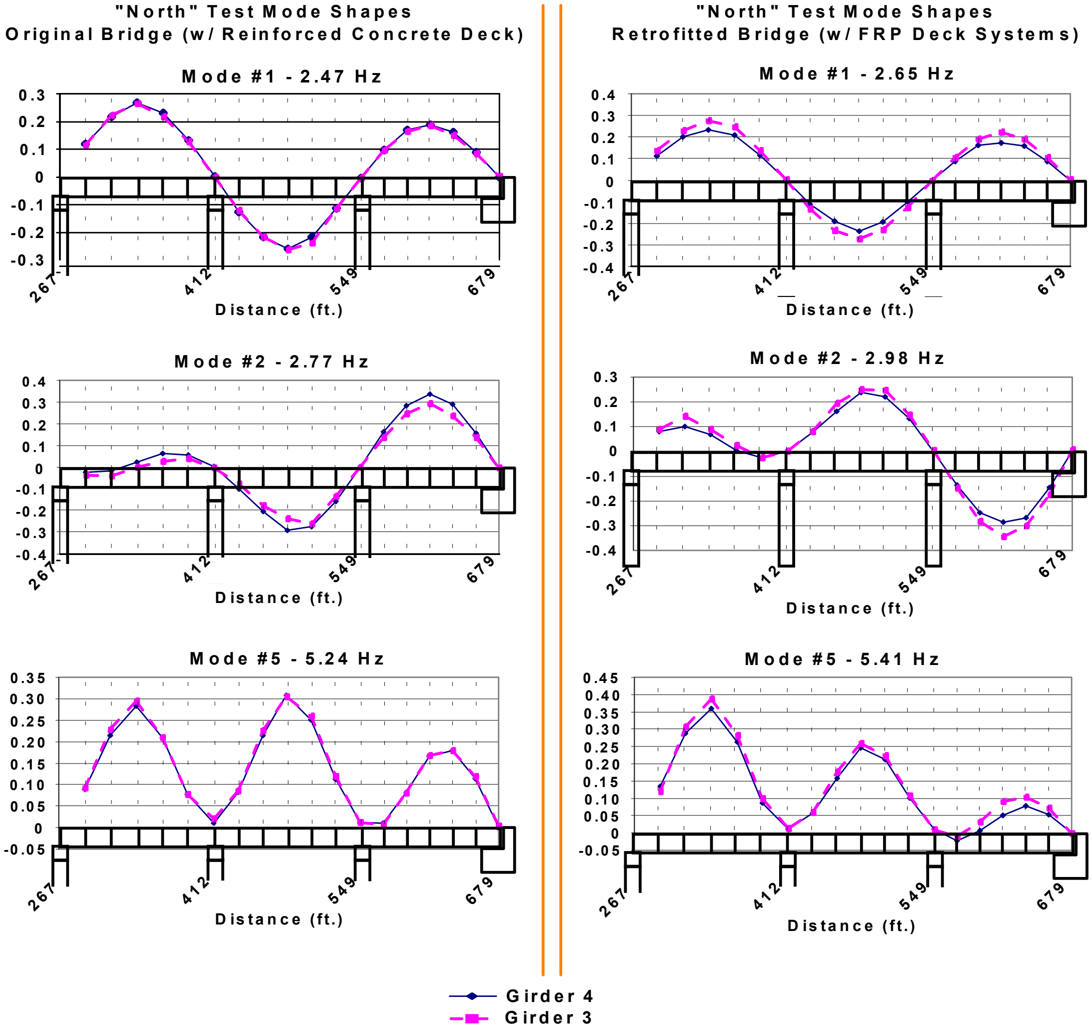

Figure 4: North test mode shapes identified for original and retrofitted bridge.

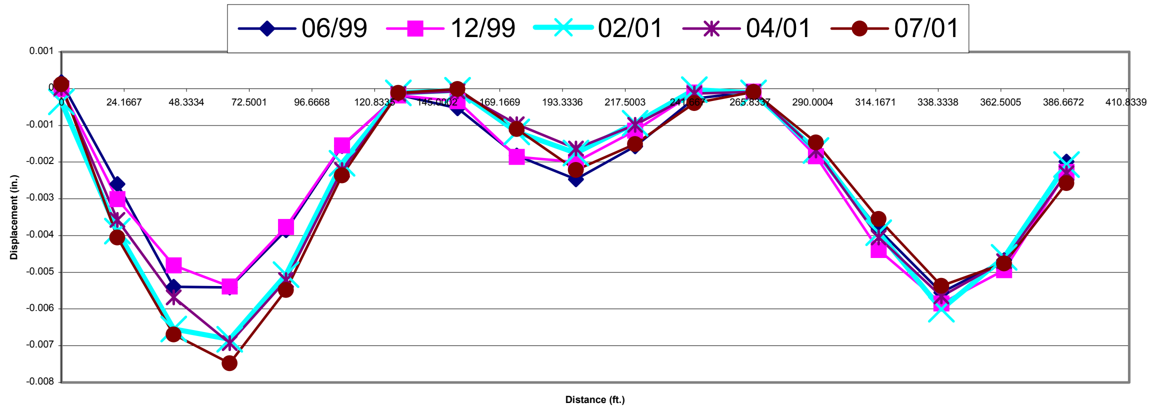

Figure 5a: Flexibility for MOT-49 South Side

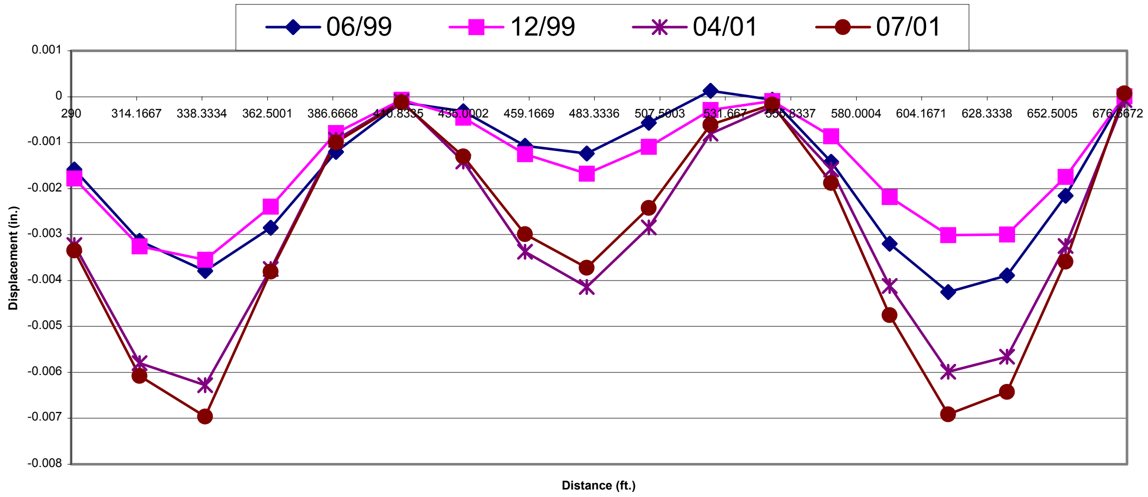

Figure 5b: Flexibility for MOT-49 North Side

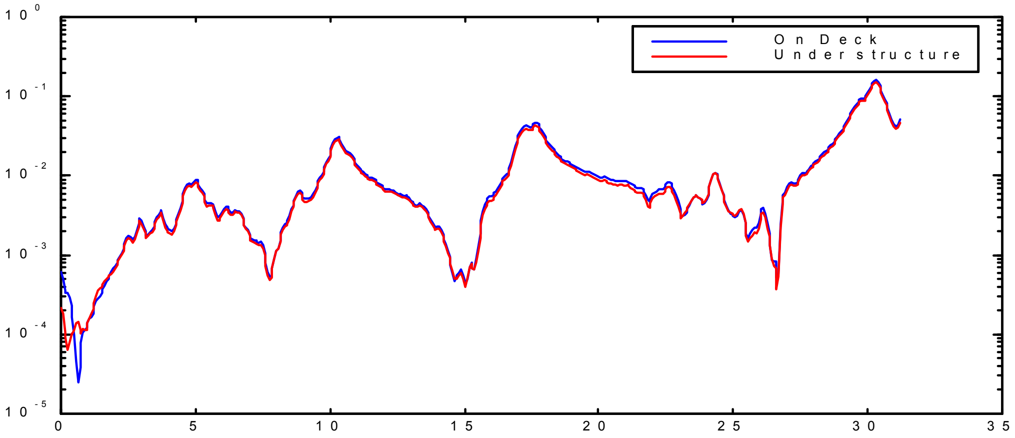

Figure 6: FRF comparison of deck and girder 3 in southernmost span

Mode Number and Natural Frequency		MAC Value
Original (RC Deck)	**Retrofitted (FRP Deck)**	
Mode #1- 2.47 Hz	Mode #1 – 2.65 Hz	0.989
Mode #2- 2.77 Hz	Mode #2 – 2.98 Hz	0.931
Mode #3– 3.47 Hz	Mode #3 – 3.86 Hz	0.939
Mode #4– 4.51 Hz	Mode #4 – 4.85 Hz	0.910
Mode #5– 5.24 Hz	Mode #5 – 5.41 Hz	0.898
	Mode #6 – 5.60 Hz	N/A
Mode #6– 8.17 Hz		N/A
Mode #7– 8.61 Hz		N/A
Mode #8– 9.42 Hz	Mode #7 – 6.66 Hz	0.869

Table 1a: Correlation between original bridge (w/ RC deck) North test modes and Retrofitted bridge (w/ FRP deck) North test modes

Mode Number and Natural Frequency		MAC Value
Original (RC Deck)	**Retrofitted (FRP Deck)**	
Mode #1- 2.47 Hz	Mode #1 – 2.65 Hz	0.972
Mode #2- 2.95 Hz	Mode #2 – 3.05 Hz	0.983
Mode #3– 3.53 Hz	Mode #3 – 3.82 Hz	0.960
Mode #4– 4.53 Hz		N/A
	Mode #4 – 4.08 Hz	0.898
	Mode #5 – 4.86 Hz	N/A
Mode #5– 5.29 Hz	Mode# 6 – 5.56 Hz	0.967
Mode #6– 5.37 Hz	Mode# 7 – 7.39 Hz	0.897
Mode #7– 8.22 Hz	Mode #8 – 10.49 Hz	0.853

Table 1b: Correlation between original bridge (w/ RC deck) South test modes and Retrofitted bridge (w/ FRP deck) South test modes

REFERENCES

1. **Reising, R., et. al.**, *Performance of a Five-Span Steel Bridge With Fiber-Reinforced Polymer Composite Deck Panels*, Transportation Research Board, 1770 (2001): 113-123.

2. **Lenett, M., et. al.**, *Condition Assessment of Commissioned Infrastructure Using Modal Analysis and Flexibility*, Proceedings, 17th International Modal Analysis Conference, February, 1999.

3. **Lenett, M.**, *Global Condition Assessment Using Modal Analysis and Flexibility*, Doctoral Dissertation, University of Cincinnati, 1998.

4. **Catbas, N.**, *Investigation Of Global Condition Assessment And Structural Damage Identification Of Bridges With Dynamic Testing And Modal Analysis*, Doctoral Dissertation, University of Cincinnati, 1997.

5. **Shahrooz, B., et. al.**, *Testing and Monitoring of a Five-Span Bridge with Fiber Reinforced Polymer Deck Systems*, to be presented at NDE/NDT for Highways and Bridges Topical Conference, September 10-13, 2002.