INFLUENCE OF FRP DECKING AS MEASURED THROUGH IMPACT TEST DATA

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ABSTRACT

Fiber reinforced polymer (FRP) materials have been introduced into the infrastructure engineering field as potential bridge components. A continuous five-span steel-girder bridge located over the Great Miami River in Dayton, Ohio recently had its original reinforced concrete deck replaced with an interlocking system of FRP panels. However, prior to this retrofit operation, multi-reference impact tests were performed on the original bridge (with reinforced concrete deck) to establish baseline signatures such as natural frequencies, mode shapes, and flexibility. Follow-up multi-reference impact tests were performed after the original deck was replaced with the interlocking FRP panel system. This paper discusses the various aspects that were involved with impact testing this large-scale, multiple span bridge as well as the results obtained from both the baseline and follow-up tests. Natural frequencies, mode shapes, and flexibilities identified from the baseline and follow-up tests are compared and general conclusions regarding how the deck replacement operations have influenced overall bridge behavior are drawn from these comparisons.

NOMENCLATURE

$[H(❻)]$	FRF matrix
$[K]$	stiffness matrix
$[\ f\ ]$	flexibility matrix
$\{❺\}_r$	rth mode shape
$\{❺\}_r^T$	transpose of rth mode
$\{❺\}_r^*$	complex conjugate of rth mode
$\{❺\}_r^{*T}$	Hermitian of rth mode
$②_r$	rth eigenvalue ($=⑨_r + j❻_r$)
M_{Ar}	Modal A (scaling) for rth mode

1. INTRODUCTION

Recent investigations regarding fiber-reinforced polymer (FRP) composites have revealed that FRP material is higher in strength, lighter in weight, and more corrosion resistant than reinforced concrete [1]. Due to its lighter weight, FRP deck systems should reduce the dead load (self-weight) of a bridge superstructure thus allowing the respective girders to carry/support higher live loads (i.e., trucks, traffic). In addition, the high resistance of FRP deck systems to environmental effects and corrosion should improve long-term performance of the bridge deck. This, in turn, should contribute to a longer service-life for the bridge as well as lower maintenance costs. FRP deck systems therefore possess the ability to enhance the structural and serviceability characteristics of a bridge. For such reasons, FRP composite deck systems have gained acceptance within the bridge engineering community as viable alternatives for the construction of new bridges and retrofit of existing bridges.

Replacing a reinforced concrete (RC) deck with a system of FRP deck panels constitutes a change in superstructure stiffness and thus a change in condition. Past research involving RC deck on steel-girder bridges has demonstrated that multi-reference impact test methods provide a proper and accurate measure of in-situ structural flexibility which has been used to effectively assess the condition of, and/or identify damage on, such bridges [2,3,4]. Consequently, it was proposed that multi-reference impact methods, as well as truck-load tests, be performed on a bridge whose existing reinforced concrete deck was to be replaced with a system of interlocking FRP deck panels. The purpose of such tests would be to identify, or reveal, any changes in superstructure condition brought about by the FRP retrofit operations. The proposed testing program included (a) testing of the original bridge with RC deck, (b) initial testing of the bridge with FRP panels, and (c) repetition of controlled tests every quarter for a period of two years. Discussions of the test bridge, the impact test methods, test results, and what the results imply are presented throughout this paper.

2. DESCRIPTION OF TEST BRIDGE

The bridge whose reinforced concrete deck was replaced with interlocking FRP panel systems is located in Dayton, Ohio and is referred to as MOT-49-10 (Figure 1). This bridge consists of five spans, all of which cross the Great Miami River and its flood plain. Span lengths are, respectively, 130 ft., 137 ft., 145 ft., 137 ft., and 130 ft. and each span consists of six built-up steel girders (i.e., plate girders). Due to the long spans of the bridge, FRP deck systems from four manufacturers were installed. This was done in part to permit evaluation of the individual FRP systems when subjected to similar loadings and environmental conditions. Approximate locations of the various decks are shown schematically in Figure 2.

2.1. Description of FRP Deck Panel Systems

Reising et. al. [5] provides a description of each individual FRP deck system used to replace the existing RC deck. These descriptions are repeated here. Deck system FRP-1 is made of pultruded components which in the factory are bonded and interlocked to form the deck panel. The second system (FRP-2) is comprised of upper and lower fiberglass fabric skin faces with multiple wrapped cells that form the stiffening webs in the longitudinal and transverse directions. These panels are fabricated using cell core technology in conjunction with SCRIMP (Seeman Composite Resin Infusion Molding Process). The core craft corrugated core sandwich system is used for fabrication of the third system (FRP-3). The basic system for these panels is a single-tier sandwich panel with a standard core configuration. The fourth deck type (FRP-4) is a hybrid system consisting of concrete deck poured over pultruded glass fiber-reinforced polymer (GFRP) panels reinforced with GFRP tubular sections. The pultruded section is intended to act as stay-in-place formwork and provide the required flexural tensile reinforcement. Additional GFRP reinforcing bars are used in the concrete deck similar to conventional RC decks. A description of how each individual FRP deck system was installed at the bridge site may be found in Reising et. al. [5].

3. MODAL RESEARCH ASPECTS

3.1. Impact Test Procedue

Due to the overall length of the bridge and other experimental constraints, the impact test grid for MOT-49-10 was divided into two sub-grids (Figure 3a). 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. Through the use of a specialized, mobile impact hammer (Figure 3b), impact force was applied near six sensor positions in each respective sub-grid to ensure excitation of pertinent superstructure modal characteristics (mode shapes and natural frequencies). Impact force and acceleration response signals were acquired using a Hewlett Packard (HP) VXI data acquisition system (Figure 3c) and processed into frequency response functions through a MATLAB-based operating software. Note that the test grids shown in Figure 3a permit modal evaluation of each span and formation of global results by splicing the overlapping response points in span 3 (the middle span). The techniques required to accomplish this splicing are discussed in detail by Lenett [6] and Brown [7]. The sub-grids in Figure 3a as well as the aforementioned splicing process were utilized for both the original bridge (with RC deck) and retrofitted bridge (with interlocking FRP deck panel systems).

3.2. Natural Frequencies and Mode Shapes

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, through MAC values, how well the respective modes correlate. 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 FRP-1, FRP-2, and FRP-3 panels – have slightly different modal characteristics than the northernmost spans which utilize the FRP-4 panel system. 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 RC 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. Investigations of modal flexibility are the topic of the next section.

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 1: 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	N/A
	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 2: Correlation between original bridge (w/ RC deck) South test modes and Retrofitted bridge (w/FRP deck) South test modes

3.3. Modal Flexibility

North and South modes identified for a particular test and displaying good MAC correlation (e.g., MAC⊄0.90 or greater) were linked, or spliced, together using the overlap points in span 3 and the methods discussed by Lenett [6] and Brown [7]. The resulting spliced modes contained modal, or dynamic, information for all five spans and were therefore considered global modes. These spliced modal characteristics were used to compute modal flexibility (Equations 1 and 2), which provided a measure of in-situ structural flexibility for the tested region of the superstructure.

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

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

The in-situ flexibilities established in this manner for both the original bridge and retrofitted bridge were subsequently compared to evaluate what effects the installed FRP systems had on superstructure stiffness. Previous research has shown that the deflected shapes of a girder line are sensitive to damage and condition [2, 8], therefore, to permit a simple yet direct comparison, superstructure displacements along a particular girder line were used as the basis for all comparisons between the original and retrofitted bridge. The particular displacement profiles used for these comparisons were obtained by multiplying the respective modal flexibility matrices with a load vector comprised of virtual downward loads of unit magnitude positioned at each measurement location along the girder line of interest (Figure 5). Comparing the original and retrofitted displacement profiles (Figure 5) revealed an increase in flexibility, or decrease in stiffness, in the first three southernmost spans (using FRP-1, FRP-2, and FRP-3 systems) and a consistent measure of flexibility in the two northernmost spans (with FRP-4 deck system).

Original and retrofitted bridge strain profiles acquired through truck-load testing (Figure 6) also revealed changes in superstructure behavior. In particular, strain readings in the southern spans revealed that installation of FRP-1, FRP-2, and FRP-3 systems caused a loss of composite action between the deck and girders. However, original and retrofitted strain readings from the two northernmost spans, which used FRP-4 deck systems, identified consistent levels of deck-girder composite action. Consequently, modal flexibility and truck test results correlated well. When multi-reference impact test (modal flexibility) results are used in conjunction with the acquired strain profiles of Figure 6, the following changes in superstructure behavior, due to installation of FRP deck systems, can be identified – an increase in flexibility (e.g., decrease in stiffness) caused by a loss of composite behavior is observed in the spans with FRP-1, FRP-2, and FRP-3 systems whereas consistent flexibility/stiffness due to no significant change in composite behavior is observed in the northern spans with FRP-4 systems. Essentially, the stay-in-place formwork utilized with the FRP-4 deck system established a level of composite action between the deck and girders similar to the level observed with the original RC deck.

4. CONCLUSIONS

The response of MOT-49-10 will be monitored for 2 years through quarterly truck load and multi-reference impact tests. According to available truck and impact test results, the overall stiffness of the bridge has been reduced for the sections using FRP-1, FRP-2, and FRP-3 systems. However, those regions using the FRP-4 system with stay-in-place FRP formwork have not experienced any reduction in stiffness. Any loss of stiffness may be attributed primarily to loss of composite action and thus the quality of connection details between the panels and girders as well as between the panels themselves.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

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Figure 1: Overview of MOT-49-10.

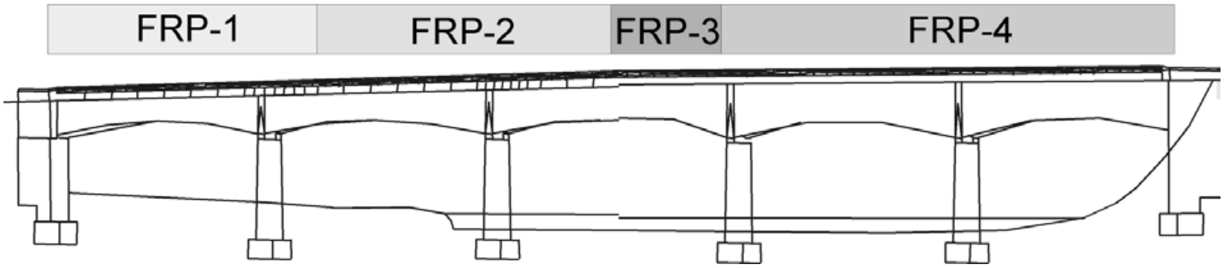


Figure 2: Approximate locations of FRP deck panel systems.

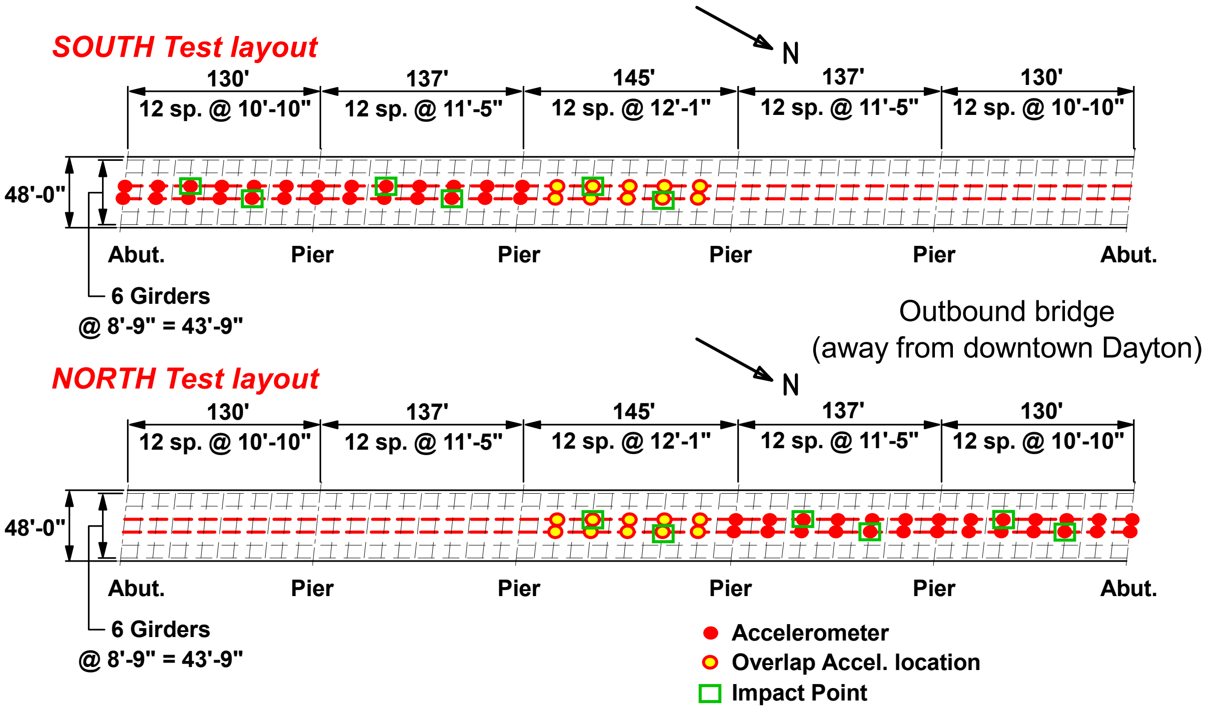


Figure 3a: MOT-49 impact test grids.

Figure 3b: Impact Hammer.

Figure 3c: VXI system and South test setup.

Figure 3: Impact Testing.

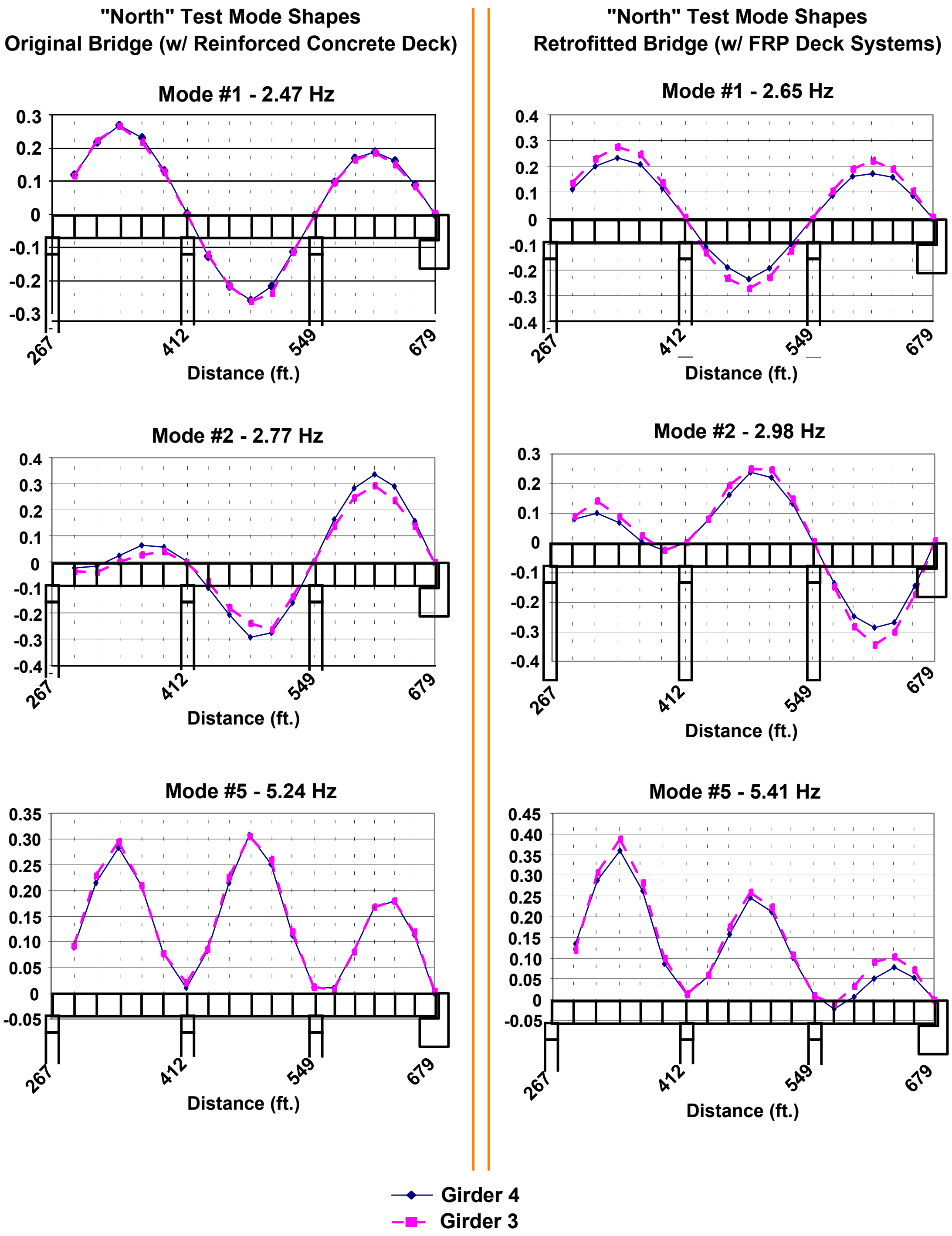


Figure 4: North test mode shapes identified for original and retrofitted bridge.

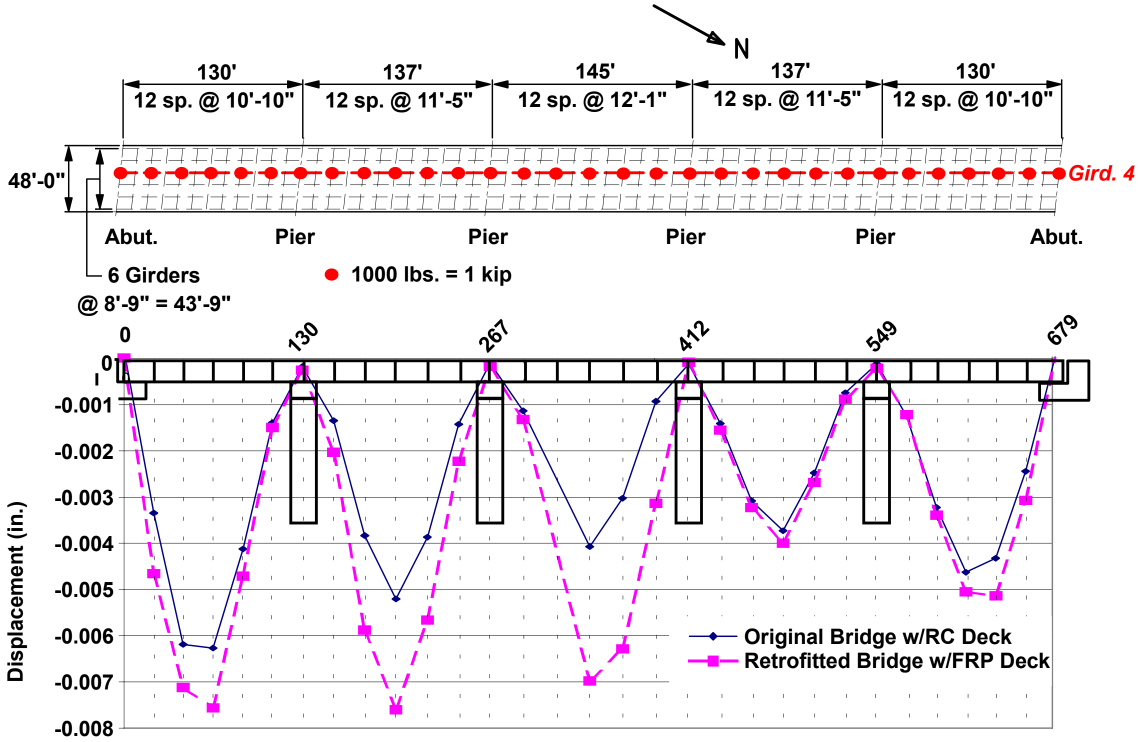
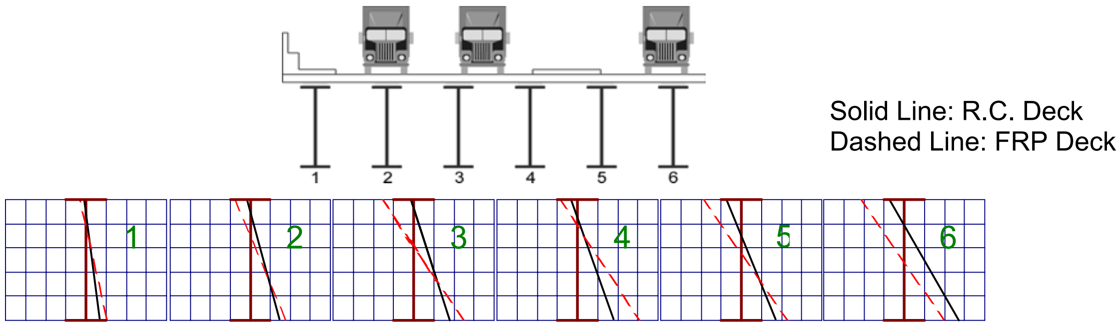
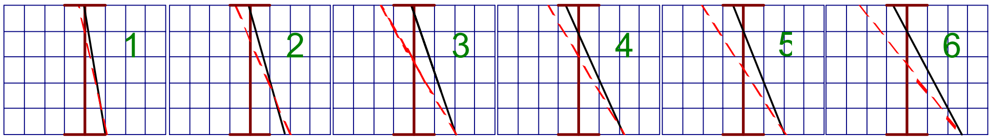


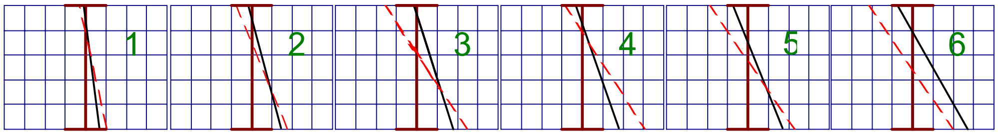
Figure 5: Comparison of original and retrofitted simulated displacements.



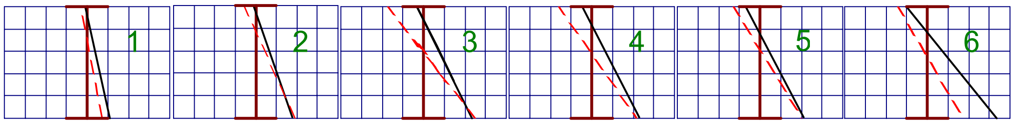
(a) Section 1 (FRP-1)



(b) Section 2 (FRP-2)



(c) Section 3 (FRP-3)



(d) Section 4 (FRP-4)

Figure 6: Girder strain profiles.