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Dynamic response evaluation of fiber-reinforced composite bridge decks and bridges

Chandra Sekhar Jinka
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Dynamic Response Evaluation of Fiber Reinforced Composite Bridge Decks and Bridges

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Thesis submitted to the College of
Engineering and Mineral Resources at
West Virginia University in partial
Fulfillment of the requirements for the
Degree of

Masters of Science

In

Mechanical Engineering

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Dynamic Response Evaluation of Fiber Reinforced Polymer Composite Bridge Decks and Bridges

Chandra Sekhar Jinka

Fiber Reinforced Polymer (FRP) composite bridge decks are gaining attention of bridge owners because of their light self weight, corrosion resistance and ease of installation. The lighter weight and lower stiffness of the FRP decks combined with the lower level of damping (than concrete) can lead to excessive deck vibrations and may substantially increase the dynamic amplification of induced stress and deflections. Also, a global NDE technique using dynamic characterization is necessary to ensure the long term performance of FRP bridges. Therefore, an extensive study on FRP decks and FRP bridges is carried out both at laboratory and in the field to evaluate their dynamic characteristics including natural frequencies, mode shapes, acceleration values, dynamic load allowance (DLA) factors, and damping ratios.

Test results of FRP deck modules revealed low moment transfer across the modules of the FRP deck. FRP deck stiffened with steel stringers was tested using forced vibration technique and a standard test procedure was developed including custom LabVIEW programs for automation of the test procedure. The developed procedure was used to conduct forced vibration testing on a small FRP bridge (Wickwire Run). Effect of temperature is found to have significant influence on the dynamics of FRP bridges. Further research is needed to study the effect of temperature on FRP bridge dynamics before applying vibration based damage detection techniques on FRP bridges.

Controlled truck load tests are also conducted on three FRP bridges namely Katy Truss Bridge, Market Street Bridge, and Laurel Lick Bridge in the state of West Virginia. The dynamic response parameters evaluated for the three bridges include DLA factors, natural frequencies, damping ratios and deck accelerations caused by moving test trucks. It was found that the DLA factors for Katy Truss and Market Street bridges are within the AASHTO 1998 LRFD Specifications, but the deck accelerations were found to be high for both these bridges. DLA factors for Laurel Lick Bridge were found to be as high as 93% (0.93) against the design value of 33% (0.33). Test results showed low damping in the FRP bridges and a technique to improve damping in FRP decks is suggested and tested in the laboratory.

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Chapter 1

Introduction

1.0 Background

Bridge owners are being challenged to upgrade existing bridges to accommodate heavier truck loads of higher frequency of travel. According to 2002 National Bridge Inventory maintained by Federal Highway Administration (FHWA), 163,000 bridges need repair or replacement. This staggering deficiency is attributed primarily to corrosion caused by the natural environment and the use of de-icing chemicals. It is imperative that new bridge construction and repair methods be made economical, longer lasting, lower maintenance and more rapidly deployable to minimize users' inconvenience. In that regard, FRP (glass or carbon fabrics with polymer binder) bridges have been found to be structurally durable (GangaRao et al., 2003) and economical (Sahirman et al., 2003) resulting in better utilization of public funds available for rehabilitation and replacement of bridges.

Constructed Facilities Center (CFC), West Virginia University has been in the forefront of developing new FRP bridge decks, and repairing and rehabilitating bridge structures using FRP composite materials. CFC in partnership with West Virginia Department of Transportation (WVDOT) and the Federal Highway Administration (FHWA) has been developing several FRP bridge super-structural systems and the state of West Virginia has over 24 new or rehabilitated FRP bridges (GangaRao et al., 2002).

FRP bridges are significantly lighter than the conventional bridges, first generation FRP decks weigh only about 25 to 30 lb/ ft² compared to about 135-lb/ ft² for a comparable 8-inch thick concrete with wearing surface. The cost of an FRP deck is directly proportional to the deck's self weight. Therefore, CFC has successfully developed a lightweight 8" deep FRP deck, weighing about 16-lb/ft² and a 4" deep low profile deck weighing about 11-lb/ft². The lighter weight and lower stiffness ($\cong$ 4 million psi) of the FRP decks combined with the lower level of damping (than concrete) can lead to excessive deck vibrations and may substantially increase the dynamic amplification of induced stress and deflections. Also, a global NDE technique using dynamic characterization is necessary to monitor the long term performance of FRP bridges and develop early distress detection methods. The following section reviews the importance of studying the dynamic response of bridges.

1.1 Importance of Dynamic Response of Bridges

Dynamic response of bridges is an important parameter both for design and maintenance of bridges to quantify: 1) dynamic amplification of peak stresses for considerations of strength, 2) dynamic deflections and accelerations causing discomfort to bridge users, 3) fatigue response and potential deck distress under dynamic loads and 4) damage detection.

Dynamic response (acceleration or velocity versus time) of bridges can be acquired under different types of excitations: 1) moving traffic (ambient excitation); 2) controlled moving truck loads; or 3) artificial excitation using impact hammer or electrodynamic shakers. Response of bridges due to moving traffic and truck load tests i.e., vehicle-bridge interaction can provide information regarding the dynamic amplification of strains or deflections (DLA), and deck accelerations for checking against human response limit states (OHBDC, 1983). Artificial excitation techniques are usually employed for evaluation of natural frequencies and mode shapes of bridges. Mode shapes are primary source of information to detect damage.

Vehicle-bridge interaction is one of the most complex problems faced by bridge engineers. To account for vehicle induced dynamic loads design codes have increased the static stresses/ loads on bridge super structures by a factor known as dynamic load allowance factor or impact factor. The AASHTO LRFD Bridge Design Specifications (1998) defines DLA as “an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles”. Other effect of dynamic loads due to moving vehicles is the possible human discomfort due to bridge vibrations. Bridge design codes pay less attention to the issue of human response to vibration; one of the high profile examples is the closure of the London Millennium Bridge due to excessive lateral vibrations.

Figure 1.1 Importance of Bridge Dynamics

Dynamic characteristics namely natural frequencies, mode shapes and damping ratios are called modal parameters. Many studies in the past have shown that vibration measurement can be effectively used to study the structural integrity of the structures (Burgueno et al., 2001; Salawu and Williams, 1995). Researchers at CFC have developed a strain energy algorithm for vibration based damage detection, and successfully applied it for complex structures such as an Armored Vehicle Launch Bridge (Aluri, 2001). The primary input for the damage detection algorithm is mode shape information; accuracy of algorithm depends on the quality of mode shapes acquired. Figure 1.1 illustrates the various reasons for

understanding the dynamic response of bridges. With emphasis on the issues showed in Figure 1.1 experimental investigation is conducted on FRP bridge decks and FRP bridges both in laboratory and field by addressing the following objectives.

1.2 Objective

The primary objective of this study is to establish the dynamic characteristics of FRP bridges including Dynamic Load Allowance (DLA) factors, natural frequencies, mode shapes, and damping ratios.

The objective was accomplished through several steps: 1) perform dynamic testing of FRP deck modules (Component Level) in the laboratory; 2) conduct modal tests on FRP decks stiffened with steel stringers (System Level) and FRP deck bridges in the field through a standard procedure; 3) conduct additional field tests on several FRP bridges to evaluate the DLA factors, natural frequencies and damping ratios; and 4) develop techniques to increase damping in FRP bridge decks based on laboratory testing.

To accomplish the above said objectives, this study was divided into the following:

1. Establish dynamic characteristics of FRP Bridge deck modules.

2. Establish standardized test procedure for laboratory and field testing of FRP bridge decks under steady state excitation.
3. Modal test FRP deck stiffened with steel stringers and the Wickwire Run Bridge.
4. Field monitor three FRP bridges to evaluate DLA, frequencies and damping ratios.
5. Develop a technique to improve damping in FRP bridge decks and verify in laboratory.

1.3 Scope

The report follows various steps ranging from a critical review of existing work, to the presentation of our test results. Additionally a brief overview of other chapters of this report is provided.

Extensive literature reviews in the field of dynamic analysis both on FRP bridge structures and bridge structures made of conventional materials like concrete, steel and others is presented in Chapter 2.

In the third chapter, the dynamic response including data collection and analysis are presented for the bridge decks, at the component level, with emphasis on dynamic characteristics like natural frequencies, mode shapes and damping ratios of the FRP component.

Chapter 4 deals with the experimental investigation of the FRP deck stiffened with steel stringers at the laboratory level. A standard procedure for establishing the dynamic characteristics of the FRP bridges is established through laboratory testing. Experimental analysis is done at this stage for a better understanding of the effects of some parameters like location of shaker on the deck.

Fifth chapter deals with forced vibration testing and analysis of the Wickwire Run Bridge with FRP deck and steel stringers. The experimental procedure developed in Chapter 4 is applied to this bridge to evaluate its dynamic characteristics.

Field monitoring of the Katy Truss, Wickwire Run, and Market Street bridges is discussed in Chapter 6. The parameters evaluated for the three bridges include: 1) dynamic load allowance factors, 2) natural frequencies, 3) damping ratios, and 4) deck accelerations. The field results are checked against The American Association of State Highway and Transportation Officials (AASHTO) and Ontario Highway Bridge Design Code (OHDBC), DLA and human response limit states.

Monitoring of the two FRP bridges revealed low damping levels, therefore a passive damping technique was developed and tests are undertaken on FRP deck modules to evaluate the proposed effectiveness of the technique, while Chapter 7 discussed the development of this technique and laboratory testing.

Chapter 8 summarizes the work with conclusions and some recommendations are made for further research.

Chapter 2

Literature Review

2.0 Introduction

Dynamic response of bridges is an important parameter both for design and maintenance of bridges especially for FRP bridges due to their low self-weight and damping. This chapter provides a summary of the state-of-the art of bridge dynamics. Over one hundred recent journal articles and technical papers were reviewed using sources such as Mountain Lynx (WVU library and statewide library system), Compendex, ASCE, and the Applied Science & Technology database. In addition, several thesis/dissertations from different universities were reviewed. The state-of-the-art review includes: 1) vibration provisions in bridge design codes, 2) various methods of analyses to determine the bridge dynamic characteristics (frequencies, mode shapes), 3) overview of testing methods both for verifying the design assumptions such as DLA factors and for full scale modal testing of bridges under different forms of excitation, and 4) finally a brief review on damage detection techniques on bridges using modal information.

2.1 Overview of Vibration Provisions of Design Codes

There are several factors that influence the response of a bridge to dynamic loads such as, the surface roughness, vehicle type, load and vehicle speed, type of bridge, span length and number of spans (GangaRao, 1992). All bridge design codes account for the effects of dynamic loading on bridges by incrementing the static loads by a factor known as dynamic load allowance (DLA) factor.

Other effect of dynamic loads due to moving vehicles is the human response to bridge vibration. Bridge design codes pay less attention to the issue of human response to vibration. The closure of the London Millennium Bridge was the result of excessive lateral vibrations. In this section, DLA and vibration serviceability limit state provisions from bridge design codes are reviewed.

2.1.1 Dynamic Load Allowance (DLA)

The 1998 AASHTO LRFD Bridge Design Specifications (AASHTO 1998) defines DLA as “an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles”. The 1996 American Association of Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges (AASHTO, 1996) addressed the dynamic effects on bridges due to moving loads by mandating an increase in live load by an “impact factor” (similar to DLA factor) which was determined by the formula $I = 50 / (L + 125)$ where I = impact fraction not to exceed 30%, and L is the length in feet for the portion of span that is loaded to produce the maximum stress in the member. The 1998 AASHTO LRFD Bridge Design Specification requires that the static load be incremented by a factor of 0.75(75%) for deck joints, and 0.33(33%) for all other bridge superstructure members (AASHTO, 1998).

The 1983 Ontario Highway Bridge Design Code (OHBDC, 1983), specifies DLA factor based on the first flexural frequency of the bridge (Figure 2.1), with DLA

values ranging from 0.2 (20%) -0.4 (40%). In the 1991 OHBDC (OHBDC, 1991), DLA as a function of number of axles per span was introduced, i.e., DLA of 0.40 for one axle, 0.30 for 2 axle, and 0.25 for 3 or more axles.

Figure 2.1. 1983 OHBDC dynamic load allowance based on First Flexural Frequency (OHBDC 1983)

2.1.2 Vibration Serviceability Limit of Bridges

It is well known that excessive floor vibrations can cause human distress (Lenzen, 1966; Wright & Green, 1959) depending upon the frequency and amplitude of vibration. Similarly, excessive vibrations of bridge can cause discomfort to bridge users. The vibration limits established to ensure comfort to bridge users can be stated as vibration serviceability limit states. The vibration serviceability issue for FRP bridges having lower damping and self-weight maybe more critical than the steel girder-concrete deck system.

The 1996 AASHTO Standard Specification (AASHTO, 1996) for Highway Bridges and 1998 AASHTO LRFD Bridge Design Specification (AASHTO, 1998) do not

directly address the issue of human response to vibration for highway bridges. However, the 1998 AASHTO LRFD Bridge Design Specifications (Section 2.5.2.6-Deformations) does state that, “Bridges should be designed to avoid undesirable structural or physiological effects due to their deformations.” It also states, “There are yet no simple definitive guidelines for limits of static deflection or dynamic motion. Among the current specifications, the Ontario Highway Bridge Design Code of 1991 contains the most comprehensive provisions regarding vibrations tolerable to humans.”

OHDBC (1983) addressed the issue of excessive vibration by restricting the allowable static deflection of the bridge based on the first flexural frequency. The 1983, OHDBC vibration provisions were not modified in the OHDBC of 1991. Figure 2.2 illustrates the allowable static deflection values to prevent discomfort to bridge users. This chart is particularly useful during design of a bridge, since it is easier to evaluate static bridge deflections than acceleration. Explaining the rationale for providing static deflection limit to restrict vibration, the OHDBC commentary 1983 states “Values of acceleration of a typical superstructure with typical approach irregularities under the action of a typical vehicle are not easily calculated” Hence, the criteria for acceleration are expressed in terms of a calculated static deflection against first flexural frequency”. It further stated “the conversion of acceleration to static deflection was made on the basis of field observations of dynamic response of bridges to traffic in traveled lane.”

Figure 2.2 1983 OHBDC static deflection limits to avoid bridge user discomfort (OHBDC, 1983)

Extensive research has been conducted on human response to floor vibrations and numerous charts for human response to vibration were developed. The OHBDC (1983) adapted the limit states developed for floor vibrations and provided limits to acceleration of pedestrian bridge serviceability; Figure 2.3 shows the chart for acceleration limits based on first flexural frequency. Acceleration of the bridge deck due to standard trucks traveling at different speeds is a measure of vibration serviceability. Figure 2.3 is useful in verifying whether the bridge deck accelerations measured in the field are within the acceptable range.

Demitz et al., 2003 developed a design methodology for composite bridges which essentially limits the deflection of these bridges based on limiting acceleration response, produced by passage of truck traffic. The design methodology relies

on the 1983 OHBDC vibration serviceability charts, which are discussed earlier in this section.

Figure 2.3 OHBDC acceleration limits for pedestrian bridges (OHBDC 1983)

2.2 Simplified Analysis Methods of Bridge Dynamics

Obtaining the natural frequencies is the first step for designing the bridges to meet the vibration serviceability criteria. The 1983 Ontario Highway Bridge Design Code (OHBDC, 1983), specifies static deflection limits based on the first flexural frequency of the bridge (Figure 2.1, 2.2 and 2.3). Also, experimental evaluation of the bridge frequencies without an estimate of bridge frequencies is highly complicated and time consuming. Hence different theories for estimating the fundamental frequencies of the bridge are discussed below.

2.2.1 Single Beam Analogy

This is one of the simplest and least time consuming types of analyses. In this method the bridge is idealized as a single beam. The transverse stiffness of the bridge is assumed to be infinite. Euler-Bernoulli's beam equation is used to estimate the bending frequencies of the idealized bridge. This equation is shown below (Equation 2.1).

$$f_n = \frac{\pi^2}{2L^2} \left(\frac{EI}{m} \right)^{1/2} \quad \dots 2.1$$

where n = the mode number
 L = span length
 E = Young's modulus
 I = second moment of area
 m = Mass per unit length

The Young's modulus value of the girders is usually taken as the bending Young's modulus of the idealized bridge beam about its strong axis. The moment of inertia of an idealized bridge beam depends on the assumption of degree of composite action, e.g., if 100% composite action is assumed, contribution of the deck is taken into account in computing the moment of inertia of the beam.

The limitation of this method is that torsional and bending-torsion coupled mode frequencies cannot be estimated. This approach is more useful to estimate the natural frequencies of an idealized bridge beam.

2.2.2 Grillage Analogy

For evaluating the static stresses values and dynamic characteristics (natural frequencies and mode shapes) bridges can be divided into a grillage or a grid. Then different types of analysis (closed form-solutions, FE Analysis and Matrix Analysis) can be applied to this grid to obtain the desired results.

Hambly (1976) offers comprehensive guidelines on idealizing the bridge as a grillage. Some of the recommendations for determining a suitable grillage mesh for a beam-and-slab deck include:

1. The structural behavior of the bridge should drive the building of the grid rather the application of set of rules.
2. If deck has longitudinal beams at center lines than lane widths, it is convenient and reasonable to place longitudinal grillage beams coincident with the center lines of the bridge.
3. If the bridge has no diaphragms, the spacing ranges from $\frac{1}{4}$ to $\frac{1}{8}$ of span.
4. If the bridge has diaphragms, then a transverse grillage member should be coincident with the diaphragm location.

The grillages can be analyzed using finite element analysis, mathematical models for grids, matrix methods, and Rayleigh method. Memory et al., (1995) modeled a steel beam and concrete deck bridge, span of 37 ft. as a grillage for analysis. Natural frequencies and mode shapes of the grillage were calculated

using both Rayleigh and FE methods. The bridge was also idealized as a single beam and the frequencies were calculated. The conclusion of his study was, “For a simply supported bridge with no-skew, the frequencies calculated using single beam analogy are always an overestimation than the frequencies calculated based on 2-dimensional or 3-dimensional analysis. In case of non-availability of FE packages, Rayleigh’s method applied to a grillage give as quick and accurate estimate of the fundamental frequency.”

GangaRao and Smith (1972) developed a governing mathematical model for torsionless grids, subjected to dynamic loads utilizing the macro field approach. Free and steady state force vibration equations for uniform two-dimensional torsionless grids for both simply and bridge type boundary conditions were presented. Closed form solutions, which are simple, exact and, very efficient to program were provided. The complexities involved in formulating the problem by the standard modal analysis are avoided with this approach.

2.3 Mathematical Models for Evaluating DLA

Chatterjee et al. (1993) presented a continuum approach to obtain transverse vibration of trussed bridges traversed by a single moving load and concluded that

- Type of truss (keeping the weight, number of panels, span and member properties same) has significant influence on the DLA.
- The variation of DLA with the speed parameter does not follow a consistent pattern.

- The DLA for the case of a multi-span continuous trussed bridge is generally higher than for the same trussed bridge discontinuous over the supports.

Wang et al. (1992) designed a multi-girder bridge as a grillage beam system in order to study its dynamic behavior due to vehicles moving across at different speeds and at different conditions of the bridge. The test vehicles were simulated as two non-linear vehicle models with seven and twelve degrees of freedom according to AASHTO H20-44 and HS20-44 truck design loadings. The road surface roughness was simulated using power spectral density based on International Standards Organization specifications. The analytical data was compared to previous experimental data and there is satisfactory agreement. The authors also conclude:

- The impact factors of the bridge loaded by two trucks in transverse direction (side by side) are smaller than those loaded by one truck
- With central loading, the impact factors of exterior girders are generally larger than those of interior girder. This issue was also highlighted by Bakht and Pinjarkar (1992).
- Road surface roughness greatly influences the impact of bridges. Impact factors (similar to DLA) can reach a high value of approximately 60% when the bridge road surface is poor.

Zhu et al. (2002) studied the dynamic loading on a multi-lane continuous bridge deck due to vehicles moving at constant velocity. The bridge was modeled as a orthotropic rectangular plate with intermediate line supports to simulate the multi span condition. A non-linear vehicle model was used with 7 degrees of freedom under H20-44 truck loading. Dynamic Impact factors of the bridge deck were evaluated for (similar to DLA) different load combinations. The study concluded that the transverse vehicle position has an important effect on the impact factor and that the road surface roughness is more important to the impact factors than the moving speed of the vehicle.

GangaRao (1992) conducted an extensive review of impact on DLA factors as implemented by the bridge design codes worldwide. Two steel stringer concrete deck bridges were tested to evaluate DLA and natural frequencies. A deterministic procedure, based on orthotropic plate theory was used to model a simply supported slab-stringer bridge under harmonic loads. Using the developed model, impact factors (DLA) were computed and compared with the test results. The theoretical values were about 20% less than experimental values, which was attributed to lack of 100% composite action between bridge deck and girders.

Humar and Kasif (1995) developed a model of bridge, idealizing it as an isotropic or orthotropic plate and modeled the vehicle as a single sprung mass moving along the deck. The plate is discretized by finite elements and solved using numerical integration. Using the developed model the DLA were evaluated for

single vehicle and multiple vehicles. The DLA due to multiple vehicles is lower than single vehicle.

2.4 Experimental Investigation of DLA

Often there is confusion in correlating the DLA factors obtained from the design codes and the DLA factors evaluated from field tests (Bakht & Pinjarkar, 1989). Several definitions for DLA were in use for experimental evaluation of the DLAs. Bakht and Pinjarkar (1989) critically reviewed the various definitions for calculating DLA and suggested the following method for calculation of DLA factors consistent with the definition of DLA based on structural dynamics theory. The DLA was computed as:

$$DLA = \left(\frac{\delta_{dyn} - \delta_{stat}}{\delta_{stat}} \right)$$

Where DLA= dynamic load allowance based on strain

δ_{dyn} = maximum deflection under the vehicle traveling at test speed

δ_{stat} = maximum deflection under the vehicle traveling at crawl speed

The appropriateness of the above DLA definition can be clearly illustrated using deflection measurements of a simple-span bridge by Biggs and Suer (1955). It can be clearly seen from Figure 2.4 that the maximum static response and the maximum dynamic response of the bridge occurred for different positions of the truck at different times.

Figure 2.4. Measured deflection of simple-span bridge (Biggs & Suer, 1955)

The maximum DLA should be calculated for the maximum amplification of deflections regardless of when the maximum responses occur. It will be shown later in Chapter 6 from Laurel Lick bridge test data, that the bridge response at 2 mph may not be a “true static” response, and therefore the bridge response at 2 mph cannot be used as a baseline value for computation of DLA for all FRP bridges.

There are many parameters that affect the measurement of DLA of the bridge. The effect of various parameters on DLA of a bridge is tabulated in Table 2.1, which is based on available literature.

Table 2.1. Influence of Parameters on the DLA

Parameters	Effect on DLA
Span of the bridge	Varies as described in figure 2.5
Surface roughness	DLA increases with increase in roughness (FHWA, 1995)
Approach conditions	DLA increases with rough approach conditions (Biggs & Suer, 1955)
No. of axles	DLA decreases with increase in no. of axles (AASHTO, 1996)
Weight of vehicle	DLA decreases with increase in vehicle weight (FHWA, 1995)
Speed of vehicle	Varies as described in Figure 2.6
Bridge vibration frequency	Varies as described in Figure 2.1 (OHBD, 1983)
Damping	DLA decreases with increase in damping (NCHRP)
Suspension	Unsprung vehicles produce higher DLA compared to the one with good suspension system (Biggs & Suer, 1955)
Location of measurement	DLA increases with the increase in the distance from the point of loading to the point of measurement (Bakht & Pinjarkar, 1989)

Figure 2.5. Effect of Span on DLA (FHWA, 1995)

Figure 2.6. Effect of Speed of truck on DLA (FHWA, 1995)

Bakht and Pinjarkar (1989) discussed the various factors responsible for misleading interpretations of DLA's and provided some recommendations for instrumentation, calibration and on data synthesis. They concluded that "There is lack of consistency in the manner in which the test data are interpreted to obtain the values of dynamic amplification factors." They further conclude that "Impact factor is not a tangible entity susceptible to deterministic evaluation; it can be accounted for in the design and evaluation of bridges only by a probabilistic approach."

Calculation of DLA's based on deflections, strains and reaction have been studied by many researchers. Fafard et al., (1998) concluded that for bridges with very good surface roughness DLA's obtained from displacements are 20% lower than those obtained from the reactions or strains, this is contrary to "The AASHO Road Test" (1962) which suggests that DLA factors obtained from measured deflection are always higher than DLA obtained from measured strains.

Wipf et al. (1999) conducted dynamic tests using a heavily loaded truck on sixteen timber highway bridges. The goal of this study was to determine the dynamic behavior of wooden bridges for estimating reliable design specifications. The study concluded by saying, "while DLA data are not exclusive indicators of dynamic behavior, the experimental values represent actual field performance under the test conditions."

Laman et al. (1999) collected dynamic strain data under controlled and normal traffic conditions on three steel through-truss bridges with similar characteristics. Dynamic strain histories were processed to obtain bridge peak static and peak dynamic response to evaluate DLA for each instrumented member. Based on the testing the authors found, "the measured DLA is dependent on truck location, component location, component type, and component peak static stresses but is nearly independent of vehicle speed."

2.5 Modal Testing Techniques of Bridges

Modal testing of bridge involves evaluating the modal properties (natural frequencies, mode shapes and damping ratios) of the bridge. However this is not feasible to use controlled excitation on bridge structures. Therefore sophisticated signal processing techniques have been developed to analyze the bridge vibration data collected under ambient or normal traffic conditions. This section discusses the work done by various authors previously in determining the dynamic characteristics of bridges with different excitation techniques. Based on the type of excitation there are different procedures for evaluating the dynamic characteristics of bridges. Ideally, the dynamic characterization technique in routine structural inspection can yield good results. In reality, technical effectiveness must be balanced with the practical requirements of a routine inspection tool. That is, the approach must be simple, fast, and inexpensive, and non-interfering with normal operations of the structure. Hence the advantages and disadvantages of each technique depending on portability, compatibility, and availability of equipment used for testing are illustrated in this section.

Farrar et al. (1999) presented an extensive review of different excitation methods for bridge super-structures and discussed different aspects regarding each method of excitation. Important conclusions of their review are listed below:

- Ambient excitation is the only means of exciting large bridge structures as the ability to input significant energy into the structure, particularly at

higher frequencies. (by some mechanical), device becomes impractical due to the size of the structure. Ambient excitation is also used with smaller bridges when other constraints prevent the bridge from being taken out of service during the tests.

- Impact excitation offers the advantage of quick setup time, mobility, and the ability to excite a broad range of frequencies. Precautions must be taken to avoid multiple impacts.
- Shakers offer the advantage of being able to vary the input waveform like harmonic, random or swept-sine signals. Shakers typically have a considerable amount of additional equipment for their operation such as power supplies, control hardware and cooling systems, which are not generally portable and are relatively expensive.

Salawu and Williams (1993) conducted forced vibration tests on bridges and concluded that impact testing is the easiest to conduct but probably produces less reliable data compared to tests using rotating mass electro-hydraulic vibrators. It is also possible that not all modes of interest will be sufficiently manifested by transient excitation. Salawu and Williams (1993) also conferred that ambient vibration testing is easier to conduct since the structural response can be measured while the structure is still in service. Artificial excitation systems, which could sometimes be complex and expensive, are not required. The disadvantage being errors in damping measurement since their values depend on the unknown excitation level. Salawu and Williams (1995) used a

hydraulic vibrator built for purpose of conducting full scale vibration tests on a bridge multispan reinforced-concrete highway bridge. Natural frequencies and modal damping ratios were evaluated.

2.5.1 Shaker Excitation

The dynamic characteristics of civil engineering structures, such as FRP bridges can be readily obtained using forced vibration tests. Eccentric rotating mass vibrators, electro hydraulic vibrators and electrodynamics shakers are used as excitation sources in forced vibration testing of bridges.

This excitation approach is unattractive due to (a) high cost of excitation equipment, (b) inconvenience of moving this equipment from structure to structure, and (c) interference of the structure with normal function of the structure

2.5.2 Impact Excitation

Dynamic characteristics of a bridge can be obtained from the response generated by an impact load. Impact excitation is commonly provided by using an instrumented sledge hammer or an instrumented drop weight hammer. The advantage of impact excitation is a whole range of frequencies are excited and the equipment is portable. The disadvantages are that getting good impact depends on the skill of the operator and the data cannot be processed if the bridge response changes with environmental changes.

Alampalli (2000) conducted a full scale modal test on Fiber Reinforced Polymer (FRP) composite bridge. Excitation was provided by an instrumented impact hammer and the response was acquired using a single accelerometer. Natural frequencies, mode shapes and damping ratios of the FRP Bridge were evaluated. The mode shapes indicated the effectiveness of the longitudinal shear key in load transfer between the two panels.

Venkatappa (2000) conducted full scale modal testing of the Armored Vehicle Launched Bridge (AVLB, a mobile bridge used by US Army) using instrumented impact hammer and a single accelerometer. Natural frequencies and mode shapes of the AVLB were evaluated.

2.5.3 Ambient Excitation

Bridges vibrate continuously under traffic, wind, and also under seismic excitations. All these excitation sources are commonly referred to as ambient. Bridge response can be acquired due to ambient excitation and processed to obtain frequencies and mode shapes.

Yen and Lai (1989) evaluated the dynamic characteristics of girder bridges from measured live load strains and resulting from regular truck traffic (ambient excitation). They used strain readings because they are easier to measure and

engineers are familiar with strains rather than accelerations. The conclusions of their study are

- Frequencies and damping ratios of bridges can be derived from measured strain-time data of the bridges.
- Single span bridge and a three-span continuous bridge tested in this study appear to vibrate predominantly in one frequency.

Ventura et al. (2000) conducted ambient vibration testing on Crowchild Trail Bridge. Frequencies and mode shapes of the bridge were evaluated and used to calibrate a number of computer models. In order to determine the natural frequency of a structure from ambient measurements, the spectral derivative normalization and averaging functions introduced by Felber (1993) was used. The study concluded “in order to properly calibrate a model of the bridge for structural dynamic analysis, it is necessary to have not only a good match of experimental and analytical natural frequencies but also a good match of experimental and analytical mode shapes.” They further suggested that improper determination of dynamic characteristics of a bridge using analytical models could lead to erroneous conclusions on its expected behavior under dynamic loading.

Beolchini et al. (1997) conducted dynamic tests of two bridge decks of a multi span bridge both under impact and ambient excitations. The tests on two decks were conducted to explain the different dynamic behavior. The amplitude of

response based on impact testing was not different indicating no difference in structural integrity of the two decks. However, the response under traffic excitation differed significantly. This was attributed to coupled frequencies, wherein dynamic responses become sensitive to the characteristics of the system and the excitation.

2.6 Vibration Based Damage Detection Techniques

Modal parameters of bridge like mode shapes and natural frequencies are the primary inputs for detecting damages in bridge super-structures. Structural researchers in the past have developed different techniques for detecting the damages using vibration measurements a few of them are discussed below

Lenett et al (2000) conducted multireference impact tests on an FRP deck panel specimen prior and after inducing damage and concluded that:

- FRP deck panel specimen could be modeled as a linear, observable, and time-invariant system, thus permitting application of impact test methods.
- Modal flexibility acquired through impact test methods provided a good measure of FRP systems in-situ structural flexibility.
- Displacements simulated with modal flexibility were sensitive to damage within the FRP structural system

Venkatappa (1997) developed a strain energy damage detection using vibration measurements and concluded that the strain energy based damage detection is

much more sensitive to damage than shifts in natural frequencies. Based on the laboratory investigations the author concluded that the method is straight forward, sensitive to damage, suitable for automation, ideal for scanning system and does not rely on numerical analysis or FE update. The strain energy was automated and validated through several experiments on aluminum rods.

Aluri (2000) conducted resonant sine dwell test on an Armored Vehicle Launch Bridge (AVLB) using automated laser sensor system. Four different damage scenarios were simulated and modal testing was done for all four damage states. Strain energy algorithm developed by Venkatappa (1997) was used on the mode shape data of the AVLB. The conclusions of the study were:

- Damage was located in two of the four simulated cases.
- Theoretically, the increase in spatial density (number of points defining a mode shape) of the points on the mode shapes increases the accuracy of the strain energy algorithm. Increase in number of data points acquired using laser vibrometer does not necessarily increase the accuracy of damage detection.

Burgueno et al. (2001) conducted forced vibration testing of FRP bridge system before and after damage. Analytical models combined with vibration tests were found to be a basis of health monitoring. They further stated that “The clearest indicators of the state of structural condition were the fundamental frequencies and the vibration mode shapes and not the damping as the measure of structural

degradation can be complicated by the difficulty in determining the exact nature of the damping and how it changes with structural damage.”

Salawu and Williams (1995), conducted full scale vibration tests on the bridge before and after repair and stated that, “repair works caused a slight reduction in the natural frequencies but no definite trend in the changes to the modal damping ratios. Comparison of the mode shapes before and after repairs using modal analysis procedure was found to give an indication of the location of repairs.”

Chapter 3

Lab Evaluation of Dynamic Characteristics of FRP Deck Modules (Component Level)

3.0 Introduction

Dynamic tests on fiber reinforced polymer (FRP) composite modules are conducted to evaluate their dynamic characteristics including natural frequencies, mode shapes and damping ratios in the laboratory. The objective of these tests is to understand the dynamics of FRP deck modules. The efficacy and ease involved in testing these components with available testing equipment required for evaluating frequencies, mode shapes and damping ratios is studied. Bedford Reinforced Plastics (BRP), Inc. supplied the pultruded deck modules required for conducting dynamic tests in the laboratory. Two types of FRP decks, one made of polyester resin and the other made of vinyl ester resin, were tested. Both deck modules are made using E-glass fibers and fabrics, but their fiber architecture and cross section is slightly different (Howard 2002). Details of specimens used for testing are shown in Table 3.1

Dynamic tests were performed on these specimens in the laboratory in order to determine:

1. Natural Frequencies
2. Mode Shapes
3. Damping Ratios

Table 3.1 Details of specimen tested in laboratory

Type of deck	Length
Polyester deck module	12 ft.
Vinyl ester deck module #1	6 ft.
Vinyl ester deck module #2	6 ft.
Transverse vinyl ester deck module	5 ft. 8 in.

Figure 3.1 Transverse Deck Specimen with three modules

Static test results of the 3 contiguous lightweight deck modules (as in figure 3.1) at the component level indicated that joints are unable to transfer adequate bending moment (Howard, 2002). For example, transverse (perpendicular to cell) bending stiffness ($E_y I_y$) obtained from FRP components under static bending is about 0.90×10^8 lb-in². The transverse (perpendicular to cell) bending stiffness is an order of magnitude less than the bending stiffness in longitudinal (cell- see Figure 3.1) direction ($E_x I_x$), which was found to be 1.14×10^9 lb-in². Therefore dynamic testing was also done on the transverse deck specimen to identify joint rotation.

The test method, equipment and instrumentation required for conducting dynamic tests on FRP components have to be carefully selected in order to achieve the desired test goals and these issues were elaborated in Chapter 2. A sinusoidal test was found to be the best option for testing the modules for the following reasons:

- 1) Sinusoidal testing is particularly useful in identifying closely spaced natural frequencies, mode shapes and any non-linear behavior.
- 2) Accuracy of test results is not dependent on skill of the operator as is the case in impact hammer testing.
- 3) No curve fitting is needed for obtaining mode shapes.
- 4) Provides better excitation for large specimens/structures (compared to impact and random excitation), since all the energy of the shaker is used to excite one mode of vibration at a time. The downside of this is long test times.
- 5) Combined with a laser vibrometer, the process of dynamic testing can be automated. Also, “drop-outs” from laser vibrometer can be detected easily and filtered out.
- 6) Ready availability of electrodynamic shaker, data acquisition system, and a laser vibrometer makes it easier to conduct tests under sinusoidal load input.

The equipment and instrumentation required for performing modal analysis in the laboratory is discussed in the following section.

3.1 Equipment and Instrumentation

For carrying out modal analysis on fiber reinforced polymer composite specimens described in Table 3.1 the following equipment is used: 1) Electrodynamic Shaker and Power Amplifier, 2) Data Acquisition System, 3) Scanning Laser Vibrometer and 4) Control Program in LabVIEW.

3.1.1 Electrodynamic Shaker and Power Amplifier

An electrodynamic shaker, Model ET-140 manufactured by Labworks Inc., is used to excite the deck. The shaker can generate an output of 100 lbf peak, with a frequency range of excitation from DC-6 kHz (Labworks Inc, ET-140 User Manual). This shaker needs a power amplifier (Model No. PA-141 from Labworks Inc.) to drive its armature and also a vacuum pump to constantly dissipate heat produced during the course of the shakers operation. The signal generated (voltage) from the data acquisition system is given as input to the amplifier. The amplifier generates an output voltage and corresponding current that drives the armature of the shaker. The power amplifier has two modes of vibration: 1) voltage mode, where the voltage generated from the amplifier is proportional to the voltage generated from the data acquisition system, and 2) current mode, where the current generated from the power amplifier is proportional to the voltage generated from the data acquisition system. The electrodynamic shaker converts the electrical power into mechanical power,

which is transmitted to the test structure, and heat, which is dissipated by a vacuum pump.

3.1.2 Data Acquisition System

Signal generation for the shakers and data acquisition from the laser vibrometer are carried out using National Instruments (NI) PCI-4551 dynamic signal analyzer (DSA) board for swept sine testing of the 12 ft polyester deck. The NI PCI-4551 has two outputs and two inputs, with on board FFT. An example program from National Instruments Swept Sine Audio Tester.llb was used to control the NI PCI-4551 board. The details of the software program are discussed in section 3.1.4. The NI PCI-4551 malfunctioned after testing the polyester deck and was replaced with NI PCI-6031E data acquisition (DAQ) board.

The PCI-6031E has two output channels and thirty two differential inputs channels with 16-bit resolution and a maximum sampling rate of 100k Samples/sec. The PCI-6031E board has only one A/D converter and uses' multiplexing to acquire data from all the channels, therefore this DAQ board is not ideal for measuring phase difference between signals.

3.1.3 Scanning Laser Vibrometer

An Ometron scanning laser vibrometer (VPI 4000) is used to collect the response from the specimen. The laser vibrometer works on principle of Doppler Effect, wherein the laser beam is divided into reference and signal beams. The signal

beam is directed onto the vibrating test specimen, and the back-reflected light is re-combined with the reference beam. There is a shift in frequency of the signal beam due to the vibrating specimen, the frequency shift is converted to an analog voltage representing the instantaneous velocity of the moving surface by the vibrometer (Ometron, 1995). The accuracy of the velocity measurement depends on quality of the reflected signal and retro reflective coating may have to be applied to improve the signal accuracy.

The scanning laser vibrometer VPI 4000 consists of a red Helium-Neon laser capable of measuring vibrations up to 200m over a frequency range of 0.01 Hz to over 300 kHz. The scanning capability of the vibrometer comes from two rotating mirrors reflecting the laser beam in X and Y directions, the position of mirrors can be controlled using a ± 5 V analog voltage.

3.1.4 Control Program in LabVIEW

LabVIEW or Laboratory Virtual Instrument Engineering Workbench is a graphical programming language used for developing data acquisition control and analysis programs. As mentioned earlier, swept sine testing on the polyester deck module was conducted using an example program package from National Instruments called Swept Sine Audio Tester.llb written in LabVIEW. The package contains two VI's (VI's or virtual instruments are the graphical programs designed in LabVIEW), the Swept Sine Audio Tester.vi program was used for testing, and the inputs included the start frequency, end frequency, number of steps, settle time,

sampling rate and amplitude of excitation (Figure 3.2). The program code and manual for its operation can be found at www.ni.com (Search Keyword: Swept Sine Audio Tester). This program will only work with NI PCI-4551 board. The PCI-4551 board malfunctioned after testing the polyester deck module therefore a new PCI 6031E board was acquired and a custom LabVIEW program is developed for conducting the swept sine testing using the NI PCI-6031E data acquisition board on the vinylester deck specimens.

This program, called “Swept sine.vi” has four inputs (start frequency, end frequency, number of steps and dwell time) for signal generation and two inputs for data acquisition (sampling rate and number of samples). Once these parameters are given, the swept sine program starts generating the signal through the PCI-6031E board from start frequency and increments the frequency by a fraction, which is equal to the $(\text{end freq} - \text{start freq}) / \text{number of steps}$ and continues generation at each frequency for the dwell time specified. In other words, the specimen will be excited at each frequency for a period of dwell time starting from start frequency till the end frequency in a given number of steps.

Figure 3.2 Front Panel of Swept Sine Audio Tester.vi

The response from the deck is collected at each frequency interval with given inputs (sampling rate and number of samples). Amplitude obtained from laser vibrometer (at each frequency) is plotted on Y-axis with corresponding frequency on X-axis. The peak in this graph corresponds to the natural frequency of the specimen. The front panel of the Swept sine.vi is shown in Figure 3.3. The LabVIEW circuit diagram for this program is shown in Figure D-1 (Appendix-D). Figures D-2 and D-3 show the remaining two loops of this program.

Figure 3.3. Front Panel of Swept sine.vi

3.2 Test Setup for Module Testing

Dynamic testing of structures is usually performed under ‘free-free’ or ‘ground’ boundary conditions. To simulate free-free vibration condition, the test specimen is usually suspended on soft springs, elastic cords, or airbags. The reasons for adopting a free-free boundary condition are: 1) better excitation of the structure with minimal interference from boundaries, 2) ease of simulation through finite element software. Taking into consideration the complexity involved in evaluating the dynamic characteristics of FRP modules, a decision was made to test the modules with a free-free boundary condition. It is impossible to get an ideal free-free setup, but high strength elastic cords (50% elongation for 125 lbs) were used

to closely approximate the free-free boundary condition by suspending the deck module from a frame as shown in Figure 3.4.

The deck modules are suspended from a square tube (1/2" thick walls) supported over heavy steel frames, this support system is significantly stiffer and heavier than the test module thereby eliminating any support vibrations. Attachment of the shaker to the test structures requires two precautions: 1) to have adequate connection strength between the shaker and the test structure for safe operation of the shaker, and 2) to restrict the movement of shaker relative to the test structure. The shaker is attached to the structure using a stinger which is stiff in one direction (intended direction of excitation) and be flexible enough in other five directions (Ewins 1984). The armature of the shaker is attached to the deck surface by using a 1/2" threaded plastic rod (stinger), with a one inch long 1/4" dia. flexible plastic rod one end of which is screwed in to a piece of brass disc, and is glued to the deck surface using high strength epoxy. The other end of the plastic rod is screwed into the armature of the shaker. This is done to ensure that the smaller plastic rod will transmit the shaker force in one direction and also fail before causing damage to the shaker armature due to any lateral loads. The shaker to deck module connection is illustrated clearly in Figure 3.5. Proper care was taken to ensure the stinger connection was perpendicular to the module surface and connected at middle of deck width to minimize the excitation of any torsional modes.

Figure 3.4 Experimental Setup for Testing FRP composite modules in Laboratory

Figure 3.5 Experimental Setup with Shaker- Deck attachment

Ideal arrangement to restrict the movement of shaker relative to the test structure, would be to suspend the test specimen using soft springs and mount the shaker on ground or rigid support (Ewins, 1984). The ET-140 electrodynamic shaker is mounted on a trunnion for excitation in inclined and horizontal orientations. Testing of the deck module required the use of horizontal orientation of the shaker (Figure 3.5). To eliminate any movement of the shaker due to excitation of deck module, the shaker was mounted on solid concrete block measuring 18" x 18" x 24" weighing about 200 lbs.

As mentioned earlier, accuracy of the laser vibrometer depends on the amount of light reflected from the deck surface. To ensure an accurate velocity reading, the deck surface is painted with retro reflective paint at each data pick up point. The scanning laser vibrometer is ideal for collecting mode shapes information with high spatial density. The accuracy of damage detection using strain energy improves with the number of data points defining the mode shapes, up to a certain limit (Venkatappa 1997; Aluri 2001). Since there were plans for using the deck module data for damage detection, the only consideration in limiting the number of data points was the cost of the retro reflective paint. Only 33 “patches” were painted on all three specimens to acquire the mode shapes. For the transverse deck specimen, entire surface was painted.

3.3 Test Procedure

3.3.1 Identifying the Frequencies Using Swept Sine Method

Determining the frequencies is the first step in modal analysis of any specimen. Because of the availability of the equipment, steady state excitation technique is used at this stage to evaluate the dynamic characteristics of the FRP deck modules. A good approximation of frequencies by using analytical methods will help in carrying out the experiment. The analytical equation used for finding the bending frequencies of an isotropic beam is

$$f_i = \frac{\lambda_i^2}{2\pi l^2} \left(\frac{EI}{m} \right)^{1/2} \dots\dots\dots (3.1)$$

Where f = Frequency in Hertz

i = No. of mode

E = Modulus of Elasticity of material

I = Moment of Inertia

m = Mass per unit length

l = Length of specimen in inches

λ_i = Frequency constant (4.73 for $i=1$; 7.8 for $i=2$)

Although the equation used is only valid for isotropic beams, it will be later shown that bending frequencies of orthotropic FRP deck modules using equations corresponding to isotropic materials can be predicted reasonably close to the

values obtained from experiments. Once the frequencies are estimated a swept sine test is carried out in a $\pm 10\text{Hz}$ range of the predicted frequency.

The Swept Sine Audio Tester.vi program which controls the NI PCI-4551 is used to perform the swept sine testing on the polyester deck module. The custom program Swept sine.vi which controls the NI PCI-6031E board is used to perform the swept sine testing on the vinyl ester deck modules. The Swept Sine test is conducted at different shaker locations and the response is collected at different points. Primary focus was to find the first three frequencies of the deck. The location of shaker and the location of laser spot are important for carrying out the swept sine test. Precautions are taken to see that the point at which the shaker is attached and the laser spot does not coincide with the nodal points of that particular mode for which the frequency is being evaluated. For instance, while evaluating the first natural frequency the shaker is attached exactly at the center of the specimen locating maximum deflection. And the data is collected at the center of the deck on the other side.

3.3.2 Evaluating the Mode Shapes Using Resonant Sine Dwell Testing

Once the frequencies are obtained, the deck is excited constantly at its natural frequency and the response from the deck is collected along its length at constant intervals (either 6 inches or 12 inches depending on the length of specimen) using the OS4000 Laser Vibrometer software.

Figure 3.6 Patches of Reflective Paint on 12 ft. Polyester FRP deck.

This software takes the velocity from the laser and the signal generated as reference (two inputs) and calculates the phase difference and signal level of the laser velocity with respect to the reference signal. The sign of the signal level depends on the cosine function of the phase difference. If this cosine is either positive or zero, the sign of signal level remains unchanged but if this cosine is negative then the sign of signal level changes from positive to negative or vice versa. The deck is divided into three lines with discrete points with spacing of either 6" or 12" depending on length of the specimen (Figure 3.6). The sign corrected amplitudes at each point are plotted to get the mode shape of the specimen at that particular frequency.

3.3.3 Evaluating Damping Using Exponential Decay Method

The free vibration response of a specimen is used to evaluate damping ratios. The specimen is excited at its natural frequency and the specimen response in amplitude versus time is collected. After few seconds, the excitation is stopped and the response is collected until the signal dies down completely (Figure 3.7)

Figure 3.7 Forced and free vibration response of the FRP deck module.

The forced vibration response is removed and the free vibration decay response data are used for calculating the damping ratio of the specimen. The exponential decay envelope of the free vibration data are given by $Ae^{-\xi\omega t}$, equating this to the

experimentally obtained exponential decay term we can evaluate the damping ratio (ξ).

3.4 Test Results

3.4.1 Natural Frequencies

Using the isotropic beam equation (defined in Section 3.2) and the static properties of the 12ft. polyester deck module from Howard (2002), the first three theoretical frequencies of the polyester deck module are calculated (Table 3.2).

The swept sine plots for the polyester deck obtained at four locations of interest are shown in Figure 3.8. The spectrum illustrates the complex behavior of the deck component, there are about twenty vibration modes between the first bending mode 72.29 Hz and second bending 199.5 Hz. Swept sine tests are done near the fundamental frequency with higher frequency resolution (0.01Hz resolution) in order to get accurate frequencies of vibration of the deck, which are listed in Table 3.2.

Table 3.2 Frequencies of FRP Deck Modules

Module	First Bending Frequency (Hz)		Second Bending Frequency (Hz)		Third Bending Frequency (Hz)	
	Theory	Expt.	Theory	Expt.	Theory	Expt.
12 ft Polyester deck	73.21	74.29	203.74	199.5	399.03	361.5
6 ft Vinyl Ester #1	295.66	258.25	-	-	-	-
6 ft Vinyl Ester #2	295.66	244	-	-	-	-

First three frequencies were evaluated experimentally for the 12ft. polyester deck module. For the 6ft. vinyl ester deck modules only first frequency is evaluated experimentally, which is 258.25 Hz for module 1 and 244 Hz for module 2. The swept sine spectrum for vinyl ester decks are shown in Figures 3.9 and 3.10 respectively. Swept sine testing was done on the transverse deck specimen to identify the bending mode. The frequency of first bending mode for the transverse deck specimen was identified as 76.84 Hz.

Figure 3.8 Frequency Spectrum of 12 ft. polyester

Figure 3.9 Swept sine spectrum for Vinyl ester deck module #1

Figure 3.10 Swept sine spectrum for Vinyl ester deck module #2

3.4.2 Mode Shapes

A resonant sine dwell test is conducted at first, second and third bending modes using the setup illustrated in Figure 3.4. The processed data obtained (as explained in Section 3.3) from all 33 points (shown as reflective paint patches in Figure 3.6) for polyester deck module on the surface are plotted as mode shapes in Figure 3.12.

As can be seen from Figure 3.8, as the frequency of vibration increases the FRP deck module behaves more like a plate structure instead of a beam. This is not surprising, since the FRP deck module is essentially made up of thin flanges and webs (Figure 3.11). Though 72.29 Hz can be clearly identified as first bending mode by looking at the mode shape, the second and third bending modes are attributed to 199.5 Hz and 361.5 Hz based solely on the theoretical calculations. Strictly speaking, for a complex structure such as an FRP deck it is not possible to get true “bending” modes for higher order frequencies without any local vibrations of flanges. In fact, partial plate like behavior of the deck modules is evident even for first bending mode of the two vinyl ester deck modules. Mode shapes of vinyl ester deck modules are illustrated in Figures 3.13 and 3.14 respectively. Also, the effects of boundary conditions are seen on the second and the third bending modes of polyester deck.

First mode of vibration of any structural system is always about an axis of least stiffness, and the higher order modes of vibrations are about an axis of higher

stiffness. For example, the first mode of vibration for a long steel box beam with small cross sectional area will likely be a bending mode since closed sections are torsionally rigid. One of the benefits of acquiring all possible modes of vibration of a structure is to identify the relative stiffness of a structure in different directions. This is especially important for FRP deck modules, since they have different stiffnesses in different directions. For the 12 ft polyester deck module and the two 6ft vinyl ester deck modules, the first mode of vibration is the bending mode, without significant local vibration of flanges.

Figure 3.11 Cross Section Lightweight Composite Bridge Deck

Figure 3.12 Mode shapes of polyester deck (a) First bending mode-72.29 Hz, (b) Second bending mode-199.5 Hz, (c) Third bending mode-361.5 Hz

Figure 3.13 Mode shape of Vinyl Ester deck #1 at 258.25 Hz

Figure 3.14 Mode shape of Vinyl Ester deck #2 at 244 Hz

Figure 3.15 First bending mode of the transverse specimen at 76.84 Hz.

Mode shape for the transverse deck specimen at 76.84 Hz was evaluated (Figure 3.15). It can be clearly seen from the mode shape that there is little

moment transfer between the modules at the joints. The dynamic test results reinforce the conclusions of Howard (2002) that the low $E_y I_y$ value (perpendicular to the cell direction) is due to the lack of moment transfer at the joints. The disadvantage of modular deck systems such as the FRP decks is that 100% moment transfer between modules cannot be achieved thereby reducing the effectiveness of the deck to act as plate structure. In order to improve the joint efficiency several techniques are used including application of glass fabrics at joints and use of rivets.

Static test methods can only indicate change in $E_y I_y$ value due to joint strengthening. However the transverse specimen results indicated that dynamic tests can be used to quantify the joint efficiency, especially with the use of curvature based methods such as strain energy method developed by CFC researchers (Venkatappa 1997). Lack of 100% moment transfer at the joints indicates a change in curvature of mode shape at the location of joint, joint strengthening (by some method) will improve the joint stiffness, thereby reducing the curvature at its location. This conclusion can be arrived at by using the before and after strengthening mode shape information and the strain energy algorithm, joint efficiency can be predicted. Based on the results obtained in this study, dynamic characterization will be used to evaluate joint efficiency improvements due to various strengthening techniques.

3.4.3 Damping

The damping of vinyl ester deck is evaluated using the exponential decay method explained in Section 3.2.

Figure 3.16 Exponential decay of vinyl ester deck module#1 at 258.25Hz

The free vibration data along with the exponential decay curve of vinyl ester module#1 at its first frequency i.e. 258.25 Hz is shown in Figure 3.16. From the above plot, the damping ratio is evaluated for the module using the following relation

$$Ae^{-\xi\omega t} = 1.97738e^{-19.487t}$$

Or

$$\xi\omega = 19.487$$

where $\omega = 2\pi f$

$$= 2\pi(258.25)$$

=1622.633

Therefore Damping ratio $\xi = \frac{19.487}{1622.63} = 0.012$ or 1.2%

The damping ratios obtained from the current test setup are slightly on the higher side, since the value includes damping of the shaker armature and that of elastic suspension cords.

Figure 3.17 Decay of vinyl ester deck module #2 at 244Hz

The free vibration data of the vinyl ester deck #2 is shown in Figure 3.17, which illustrates the complexity involved in calculating the damping ratio based on exponential decay curve. In this case it is not possible to fit the exponential curve, as the free vibration amplitude is not completely dying down gradually; however some ripples are noted. Hence the damping ratio is not evaluated for vinyl ester module # 2. Similar problems were encountered while evaluating the damping ratio of polyester deck.

Test results of the FRP deck modules reveal that local vibrations of the flanges are predominant part of the response, especially at higher order modes. Mode shapes of transverse specimen indicated low moment transfer between the modules. Because of these two reasons the joints of vinyl ester deck required for FRP deck and stringer system is fortified with an extra layer of fabric.

Chapter 4

Forced Vibration Testing of FRP Deck Stiffened By Steel Stringers (System Level)

4.0 Introduction

Dynamic characterization of FRP composite deck with stiffened stringers is performed in the laboratory for two reasons: 1) one is to understand the dynamic behavior of the FRP deck - Steel stringer system and 2) to establish a standard test procedure for evaluating the dynamic characteristics of the system such that it can be used in the field. Testing was performed on vinyl ester deck and stringer system with wearing surface. The five modules, each made of E-glass fabrics and vinyl ester resin, are joined together at the manufacturing plant to form a 120 in. x 100 in. deck. Howard (2002) prepared the test specimen including surface preparation of deck and bonding of stringers to deck for conducting static load tests. The same specimen was used for conducting dynamic tests. The only addition to the deck is the ½ inch thick polymer concrete wearing surface on the deck surface.

Dynamic test was performed on the FRP deck - Steel stringer system to evaluate

1. Natural frequencies of the system
2. Mode shapes of the system
3. Effect of stringer spacing on the dynamics of the system.

4.1 Preliminary Setup and Testing

Dynamic testing of any structure involves three aspects: 1) selection of support conditions and excitation mechanism, 2) selection of appropriate transducers, and 3) processing of signals and curve fitting of data.

Considering the size and weight of the FRP deck- Steel Stringer system it was concluded that the system cannot be suspended using elastic cords (similar to modules as in Chapter 3). The use of Airmount® airbags (available at CFC) between the system and the supports was also ruled out as the deck and stringer system was too light to have effective operation of the airbags. Hence the option of testing the system in a “free-free” vibration test condition was not possible. Therefore the deck is tested with “simply supported” boundary conditions i.e., the stringers are simply supported over a span of 88 in. Furthermore, these “simply supported” boundary conditions closely simulate the conditions of an actual bridge in the field.

Initially the system testing was done using impact excitation technique. A PCB Model 086C50 impact hammer manufactured by PCB Piezotronics was used for evaluating the dynamic characteristics of the deck and stringer system. This hammer weighs about 12 pounds and has 1mV/lbf sensitivity with a maximum operational frequency range of 500Hz. A single accelerometer was used to acquire the output response of the deck; this is done so as to apply single degree

of freedom (SDOF) analysis methods for evaluating the modal parameters such as frequencies and mode shapes.

This approach did not yield the desired results for the following reasons: 1) the 12 lb impact hammer available with CFC was too big for this application. The impact excitation imparted into the structure had to be done very slowly so that acceleration of deck can not exceed the limits of the accelerometer; in addition, stringers were bouncing off the supports due to the impact 2) use of impact hammer technique on the lightweight FRP deck resulted in exciting local modes (of that particular flange), picked by the accelerometer and complicated the analysis. 3) The deck and stringer system was about 4 ft above the ground making excitation using impact hammer very difficult, and 4) no modal analysis software was available to process the frequency response functions and extract the frequencies and mode shapes.

Steady state excitation technique using an electrodynamic shaker was, therefore, chosen for conducting dynamic testing of the deck. The 110 lbf electrodynamic shaker (Model ET-140) used for dynamic testing of the FRP deck modules (explained in Chapter 3) was attached at the bottom of the stringer for exciting the deck. The force generated by this shaker was not sufficient to excite the deck, which was evident by repeated failure of the plastic stinger between the shaker armature and the steel stringer. To counteract this problem an electrodynamic reaction mass shaker was used. This shaker was manufactured

by APS dynamics under the registered name of ELECTRO-SEIS Model 113 The power for the shaker was supplied from a Dual-Mode Power amplifier (Model 124 from APS Dynamics).

Contact (accelerometers) and non-contact sensors (laser vibrometer) were available for acquiring the vibration response of the deck stringer system. Non-contact sensors such as a laser vibrometer are preferred because there is no need to mount the sensors and they have no mass loading effect in a localized sense on the vibrating structure. However, based on the operational requirements of the laser vibrometer, it was calculated that the laser vibrometer needs to be installed at least 27ft. perpendicularly above the surface of the deck to scan the entire deck. This is due to the restriction on the sweep operating angle of the laser beam through its aperture. This requires a special arrangement to hold the laser at constant position, which can be expensive, time consuming and less stable. Therefore piezoelectric accelerometers are chosen for acquiring the dynamic response on the deck and stringers system. Two types of accelerometers were used which are discussed in detail later in this chapter.

4.2 Equipment & Instrumentation

The equipment required for carrying out dynamic testing on the fiber-reinforced polymer composite deck stiffened by steel stringers is discussed briefly in this section. The following equipment is used for the entire testing of the system

1. Electrodynamic Shaker and Power amplifier

2. Accelerometers
3. Data Acquisition System
4. LabVIEW programs

4.2.1 Electrodynamic Shaker & Power Amplifier

A reaction mass shaker Model 113 manufactured by APS Dynamics, Inc. under the registered name of ELECTRO-SEIS is used to excite the FRP deck stiffened by steel stringers. The power for the shaker is supplied from a Dual-Mode power amplifier (Model 114-EP; APS Dynamics, Inc.). This shaker is designed specifically for driving structures at their natural resonance frequencies with maximum power to resonant loads and minimum total shaker weight and drive power. The shaker is operated in the free armature mode which allows the shaker to be placed any where on the surface of the deck and operated as shown in Figure 4.6. In addition the force required for exciting the deck and stringer system can be increased by adding reaction mass to the armature of the shaker. This shaker has an operational range of 0-2000Hz with a maximum force rating of 30 lb peak above 0.1Hz. Further details about this shaker can be found in “Instructional Manual ELECTRO-SEIS® Model 113”. The dual mode power amplifier is designed to provide drive power for the shaker. The voltage mode of the amplifier produces high internal damping in the shaker armature due to low amplifier source impedance whereas the current mode has minimum effect on system damping which is useful in modal testing. Hence the current mode is used through the entire testing process. The maximum input to this amplifier is ± 2 V and the power output from the amplifier is 250 V-A peak.

4.2.2 Accelerometers

Piezoelectric accelerometers (PCB 393B12 & PCB 393A03) are used for acquiring the response of the deck. The signal conditioning required for the accelerometers is supplied by ICP[®] sensor power unit (Model 480C02). Model 393B12 accelerometer has 10 V/g sensitivity and an operational range of 0.15-2000Hz where as model 393A03 has 1 V/g sensitivity with an operational range of 0.5-2000Hz. The resolution of the accelerometer, which defines the least sensible measurements, for PCB 393A03 is 0.00001g and for PCB 393B12 is 0.000008g.

4.2.3 Data Acquisition System

NI PCI-6031 board is used for evaluating the frequencies and the mode shapes of the system. The PCI-6031E has two output channels and thirty two differential inputs channels with 16-bit resolution and a maximum sampling rate of 100k Samples/sec. The PCI-6031E board has only one A/D converter and uses' multiplexing to acquire data from all the channels, therefore this DAQ board is not ideal for measuring phase difference between signals.

A peculiar problem was encountered during the testing process; it may take hours for the signal conditioner to settle down due to the resulting coupling time constant of the data acquisition system. To overcome this 1 Mega Ohm resistor is added in parallel with the input to the acquisition system. This reduces the overall resistance between the signal and ground, making the coupling time

constant more manageable in 10 seconds (Personal correspondence with PCB technical support). Hence all the inputs are added with 1 Mega Ohm at the data acquisition BNC as shown in Figure 4.1.

Figure 4.1 $1\text{M}\Omega$ resistor in parallel with signal acquired from accelerometer

4.2.4 Control Programs in LabVIEW

Swept sine test is conducted to identify the natural frequencies of the system, using “Swept sine.vi” program which is discussed in Chapter 3. A new LabVIEW program was written to acquire the mode shapes of the system titled, “Modeshape for N channels.vi.” The program requires 6 inputs, they are: 1) Channel numbers to which the accelerometer were connected separated by commas, 2) Number of samples to acquire, 3) Sampling Frequency, 4) Modal frequency, 5) Number of rows, and 6) Number of columns. This program also requires specifying the primary reference channel in its diagram window, which must be one of the channels listed previously in the channels tab. The primary reference channel is the signal from the accelerometer mounted on the armature

of the shaker, which is used to calculate the phase difference between primary reference channel and other accelerometers. The front panel of this program is shown in Figure 4.2. The LabVIEW circuit diagram for this program is shown in Figure D-4 (Appendix-D). Figures D-5 and D-6 show the two sub-vi's of this program which are used in building the Modeshape for N channels.vi program.

Figure 4.2 Front Panel of Modeshape for N channels.vi

The excitation signal for the power amplifier is provided by another LabVIEW program called “Continuous Sine Wave Generation.vi,” which can be found in the

example library of LabVIEW software under the DAQ section. The excitation and data acquisition are carried out by two separate programs. First the system is excited using the “Continuous Sine Wave Generation.vi” and then the “Modeshape for N channels .vi” is executed.

The “Modeshape for N channels.vi” program acquires the data from specified accelerometers including the primary reference at the specified sampling rate. A sampling rate is selected to avoid aliasing. The program evaluates the amplitude of vibration of all accelerometers and computes the phase difference between the primary reference accelerometer and the remaining accelerometers. The sign of the amplitude of each accelerometer is decided based on the value of Cosine function of the phase difference between that accelerometer and primary reference. If the Cosine of the phase difference is less than zero, then the amplitude of that particular accelerometer is multiplied with negative 1 (-1), if the Cosine function is zero then the amplitude of corresponding accelerometer is multiplied with zero, and if the Cosine function is greater than zero the amplitude of corresponding accelerometer is multiplied with positive 1 (+1). Finally the program prompts the user to store all these phase corrected amplitudes for all the accelerometers to a file.

4.3 Test Specimen

One vinyl ester FRP composite deck is tested with two different combinations of stringers, one with two steel stringers (136” stringer spacing, Figure A-1) and the

same deck with three steel stringers (68" spacing, Figure A-9). The specimen preparation was done by Howard (2002) as he conducted static load testing on the same deck. Two W10X39 steel stringers are attached at the longitudinal ends of the deck using PLYOGRIP®-7779/300 making the effective spacing between them to be 136 in (Howard 2002). After static testing, a half inch thick polymer concrete wearing surface is added over the deck. Once the frequencies and mode shapes are evaluated for the deck with two stringers, a third W10X39 stringer is attached at the center of the deck using ½ inch bolt and nuts and the testing is repeated to find the frequencies and mode shapes.

4.4 Test Setup

As mentioned earlier, testing the FRP deck stringer system in "free-free" vibration condition was not feasible, therefore the system was tested in simply supported condition. Initial swept sine tests conducted on the system revealed that the deck bounced off the supports as it reached its natural frequencies. Therefore the stringers are clamped near all four support locations (six support locations in case of three stringers) using C-clamps as shown in Figure 4.3. Clamping down the stringers to the supports changes the simply supported boundary conditions to partially fixed boundary conditions, this change in boundary conditions will change the dynamics of the system. The total span of the system is 88 inches. The location of accelerometer mounted on the vinyl ester deck with two stringers is shown in Figure 4.4.

Figure 4.3 Clamping of the Steel Stringer to the Support using C-clamp

Figure 4.4 Test setup for FRP deck stiffened by two steel stringers

The reaction mass electrodynamic shaker is placed on the surface of the deck as shown in Figure 4.4. The deck is excited for different shaker locations and the

response is collected for different accelerometer locations for purpose of identifying the frequencies, one such position is shown in Figure 4.4.

4.5 Test Procedure

4.5.1 Identifying the Frequencies Using Swept Sine Testing

Swept sine tests are conducted using the LabVIEW program “Swept sine.vi,” which is explained in Chapter 3. This custom built LabVIEW program controls the PCI-6031 board, which generates the frequency from start frequency to end frequency with given number of intervals dwelling at each frequency for given number of seconds. There are four inputs (start frequency, end frequency, no. of steps and dwell time) for signal generation and two inputs (no. of samples and sampling frequency) for data acquisition. Precautions are taken to sample the data with a proper sampling rate to avoid aliasing. The signal generated from the NI PCI-6031 board is connected as an input to the dual-power amplifier, which supplies the power for the shaker. The response of the deck collected by the accelerometer is acquired through the BNC board, which acts as interface between the NI PCI-6031 board and the sensors.

Two test cases are considered to study the effect of number of stringers on the dynamic behavior of the system. One is the deck with two stringers (Figure 4.4) and the other with three stringers (Figure A-9). In either case the frequencies of the system are not evaluated analytically due to the complexities involved and

hence much attention was paid to conduct the swept sine tests for different shaker and accelerometer locations on the surface of the deck. As the range of the frequencies is unknown swept sine tests are conducted from 1Hz to 200Hz with different shaker and accelerometer locations. The different shaker and accelerometer locations are required for identifying maximum number of frequencies of the system. Each shaker location excites a particular set of frequencies of the system. Complete list of shaker and accelerometer positions for both test cases are given in Appendix A.

4.5.2 Evaluating the Mode Shapes using Resonant Sine Dwell Test

For different locations of shaker and accelerometer the swept sine test results showed the peaks in spectrum of frequencies, which represent the longitudinal and transverse bending modes, torsion modes or a combination of both. A sine dwell test is conducted for each frequency to evaluate the mode shape of the system at that frequency. The custom LabVIEW program, “Modeshape for N channels.vi” is used for carrying out this task.

Figure 4.5 Location of Accelerometers on the FRP deck for Mode Shape Evaluation

Only ten high sensitivity 393B12 (10 V/g) accelerometers were available, therefore the deck was divided into five rows (Figure 4.5) and data were collected from one row at a time as shown in Figure 4.6. To obtain a 2D mode shape by using all five line scans, a secondary reference accelerometer is mounted, which is fixed for all five line scans. The procedure for acquiring mode shape for each row (line scan) is as follows:

1. Shaker is placed in a particular position to excite a set of frequencies (e.g., Position 1 will excite the modes near 9.5 Hz, 14.75 Hz, 20 Hz, 29.25 Hz)

2. Continuous Sine Wave Generation.vi program is executed and the shaker excites the deck at one frequency (say 9.5 Hz).
3. Mode Shape for N Channels.vi program is executed and phase corrected data from eleven accelerometers (Ten 393B12's and secondary reference accelerometer) is acquired and stored.

The excitation is stopped and accelerometers are moved to the adjacent row and steps 1 to 3 are repeated to obtain line scans for each row. Since each line scan is acquired at different trials (i.e., possibly different excitation levels), the data from secondary reference accelerometer are used to normalize each line scan data, which is illustrated with an example in Appendix C.

Figure 4.6 Line scan with 10 accelerometers.

4.6 Test results

4.6.1 Natural Frequencies

Three combinations of shaker and accelerometer locations were used to identify natural frequencies of the system. One such position for the FRP deck stiffened by two steel stringers is shown in Figure 4.4. The swept sine frequency spectrum is shown in Figure 4.7. Swept sine spectrum for FRP deck stiffened by three steel stringers is shown in Figure 4.9 and the corresponding location of shaker and accelerometer is shown in Figure 4.8, for a three stringer system.

Figure 4.7 Swept sine frequency spectrum corresponding to position 3

Figure 4.8 Position 3 for FRP deck with three steel stringers.

Figure 4.9 Frequency spectrum corresponding to position 3

Table 4.1 Frequencies of Vinyl Ester Deck with Two W10X39 Stringers

Position1	Position2	Position3	Position4
9.5 Hz	-	10 Hz	-
14.75 Hz	14.75 Hz	15 Hz	-
20 Hz	-	-	-
29.25 Hz	29.5 Hz	29.75	-
-	90 Hz	-	-
-	-	-	123 Hz
-	-	-	168 Hz

Table 4.2 Frequencies of Vinyl Ester Deck with Three W10X39 Stringers

Position 1	Position 2	Position 3
43.8 Hz	43.8 Hz	43.8 Hz
51.6	-	-
-	-	68.2 Hz
82 Hz	-	-
96.9 Hz	-	-
-	104.7 Hz	-
-	-	107.7 Hz
	-	119.8 Hz
-	123.3 Hz	-
126.3 Hz	-	-
-	-	145.1
-	-	155.8
-	158.6 Hz	-
-	167.1 Hz	-
169.6	-	-
-	-	176.2 Hz
-	190 Hz	-
211.9 Hz	-	-
-	-	229 Hz

The frequencies obtained from all the trials for the FRP deck and stringer system with two stringers and three stringers are tabulated in Table 4.1 and Table 4.2 respectively. At each frequency, further swept sine tests with higher resolution are conducted to get accurate estimates of frequencies because of which there might be a change in frequency between the modal frequencies (where the mode shapes were acquired) and the frequencies shown in Table 4.1. All positions listed in Tables 4.1 and 4.2 are shown in detail in Appendix A, including the frequency spectrum for each position.

4.6.2 Mode Shapes

After getting frequencies of reasonable accuracy the mode shapes are evaluated at all the frequencies using the custom developed “modeshape for N channels.vi” LabVIEW program. The mode shapes obtained for vinyl ester deck stiffened by two steel stringers are shown in Figure 4.10. The modeshapes of the deck and stringer system with three stringers is also evaluated using the “modeshape for N channels .vi” program. The mode shapes are shown in Figure 4.10 and Figure 4.11(a) and Figure 4.11-(b).

Figure 4.10 Mode Shapes of Vinyl Ester Deck Stiffened by Two Steel Stringers

Figure 4.11-(a). Mode Shapes of Vinyl Ester Deck Stiffened by Three Steel Stringers at different frequencies

Figure 4.11-(b) Mode Shape of Vinyl Ester Deck Stiffened by Three Steel Stringers at Different Frequencies.

As mentioned in Chapter 3, the first mode of vibration of a system is always in the axis of least stiffness. The first bending mode for the FRP deck and stringer system, both in 2 and 3 stringer cases, occurred in the direction of stringer spacing, which is not the primary bending direction of the system. This is due to the fact that the span of the system (88in) is less than the spacing of the system.

Modeshapes for higher order modes are complex and cannot be described as bending or torsion modes. Because the number of points in the direction of span on the FRP deck and stringer system defining the mode shape are only five, whereas over the spacing they are ten. Due to less number of data points over the span, higher order modes appear as lower order modes. This phenomenon is called “spatial aliasing” which is analogous to aliasing of signals due to under sampling.

Chapter 5

Forced Vibration Testing of Wickwire Run Bridge

5.0 Introduction

Forced vibration testing of bridges is conducted to evaluate modal properties such as natural frequencies and mode shapes. Forced excitation, whenever possible is preferable over ambient excitation because the force input into the structure can be controlled and analyzed easily. As discussed in Chapter 2, forced vibration tests are restricted primarily to small bridges. Recently, researchers have focused their efforts on detecting damage based on the change in modal properties due to damage (Womack and Halling, 1999). Also, several damage detection algorithms which use mode shapes as primary inputs have been developed and implemented (Venkatappa 1997; Aluri 2001; Lennet, 2000).

One of objectives of this study is to evaluate the modal properties of a bridge using forced vibration testing technique, as a first step in the long term goal of developing and implementing damage detection algorithms for health assessment of FRP bridges. The criteria for selecting the Wickwire Run Bridge for full-scale forced vibration testing are as follows:

- 1) This bridge superstructure is made up of FRP deck and steel stringers, similar to the test specimen discussed in Chapter 4. Also, FRP deck with steel stringers is more common type of construction in FRP bridges.
- 2) The dimensions of this bridge are best suited for conducting forced vibration tests

using the reaction mass electrodynamic shaker.

- 3) The traffic on this bridge can be diverted without causing much trouble to the users.
- 4) There are no other sources of vibration like an Interstate Highway or Railroad near the bridge there by eliminating any secondary vibrations.

Dynamic characteristics of Wickwire Run Bridge including natural frequencies and mode shapes are evaluated using steady state excitation technique. As concluded in Chapter 4 all the test equipment and instrumentation including the custom LabVIEW programs required for carrying out forced vibration tests on FRP deck and stringer system are tested thoroughly in the laboratory. Therefore similar testing procedure (discussed in Chapter 4) is adopted for conducting dynamic tests on the Wickwire Run bridge. The tests are conducted in mid summer, to accommodate the study on thermal behavior of the deck conducted by Krit Laosiriphong. Deck temperature measurements were made as part of the thermal study, which were useful in explaining the time variance of modal properties, this issue is discussed in later sections of this chapter.

5.1 Bridge Description

Wickwire Run Bridge located in Taylor County, West Virginia is built using 8 in. FRP composite bridge deck, designed by Constructed Facilities Center, West Virginia University and fabricated by the Creative Pultrusion, Inc. under the trade name of SuperdeckTM. A double trapezoidal and hexagonal cross section made of E-glass multi-axial stitched fabrics with chopped strand mat and continuous roving is

pultruded with weather-resistant vinyl ester resin in preparation of the deck module. These modules were placed perpendicular to the traffic flow on the stringers and joined in the field using shear keys to provide mechanical interlocking in addition to the adhesive bonding. The deck is chemically bonded to the stringers with Pliogrip® and mechanically joined with Huck bolts for extra strength to the bond. The total span of the bridge is 30 ft and the width is 22 ft. W 24X104 type steel girders which are equally spaced along the flow of traffic at 6ft. support the deck.

5.2 Equipment and Instrumentation

The equipment required for conducting dynamic tests on the Wickwire Run Bridge is same as the equipment used for testing the FRP deck and stringer system discussed in Chapter 4 except for the electrodynamic shaker and power amplifier. The list of equipment and instrumentation used for testing the Wickwire Run Bridge is given below:

- 1) Electrodynamics Shaker and Power Amplifier
- 2) Accelerometers
- 3) Data acquisition system
- 4) Custom LabVIEW Programs

5.2.1 Electrodynamics Shaker and Amplifier

The Wickwire Run Bridge with a span of 30ft and 22 ft. width is considerably heavier and bigger than the FRP deck and stringer system tested in the laboratory. Based on the experience gained through laboratory testing of FRP deck and stringer system and

through the discussions held with Kenneth Metzger of APS Dynamics Inc, a Model 400 electrodynamic shaker was acquired to test small bridges such as the Wickwire Run bridge. The total weight of Model 400 electrodynamic shaker including reaction mass is 232 pounds. The power for the shaker is supplied from a dual-mode power amplifier (Model 124-EP, APS Dynamics, Inc.). The working principle of this shaker is similar to the Model 113 shaker, which is explained in Section 4.1.1. This shaker has an operational range of 0-200Hz with a maximum force rating of 100 lb peak. The only restriction in the operational conditions of the shaker with the reaction mass is that it can impart the rated maximum force of 100 lb peak starting from 2.2 Hz. The power amplifier is used with current mode to avoid any internal damping in the shaker. The maximum input to this amplifier is ± 2 V and the power output from the amplifier is 750 V-A peak.

Ten PCB393B12 accelerometer and one PCB393A03 accelerometers are used to acquire the dynamic response of the bridge. Custom LabVIEW programs were used to conduct the swept sine and resonant sine dwell testing. Details of the accelerometers, data acquisition system and custom programs are described in Chapters 3 and 4.

5.3 Test Procedure

The test procedure for conducting forced vibration tests is similar to the procedure followed for the FRP deck and stringer system in the laboratory which is discussed in Chapter 4. As the time available for conducting the test was limited, a decision was

made to limit the mode identification and mode shape acquisition to the first two longitudinal (direction of traffic flow) bending modes. In order to conduct the sine sweep, the first two frequencies of the bridge are estimated using single beam analogy. The step-by-step testing procedure is as listed below:

1. Estimation of frequencies using single beam analogy.
2. Identifying the frequencies using swept sine tests.
3. Evaluating the mode shapes using resonant sine dwell tests.

Each of the above three steps are discussed briefly in the following sections.

5.3.1 Estimation of frequencies using single beam analogy.

Estimation of the frequencies of the FRP deck and stringer system using single beam analogy is done by idealizing the entire bridge as a single beam. The moment of inertia of the FRP deck is ignored for calculations assuming that there is no composite action between the deck and the stringers but the weight of the deck is considered while calculating the frequencies. The equation for calculating the frequencies of vibration using single beam analogy is given as:

$$f_i = \frac{\lambda_i^2}{2\pi L^2} \left(\frac{EI}{m} \right)^{1/2} \quad i= 1,2,3 \dots \dots \dots (5.1)$$

where $\lambda_i = i\pi$

i = mode number.

E = modulus of elasticity of the stringer.

I = combined moment of inertia of all the four stringers of the bridge.

m = mass per unit length of the bridge including deck and stringer system.

L = span of the bridge from center to center.

5.3.2 Identifying the Frequencies of Vibration Using Swept Sine Testing

In order to identify the first and second longitudinal bending frequencies of the bridge, the location of the shaker and the accelerometer are very important. The location of shaker is decided based on experiments done in laboratory on lightweight 8 in. FRP deck stiffened by steel girders for finding the natural frequencies.

The Wickwire Run bridge has 4 steel stringers (Figure 5.2), the shaker is located near the center span of second stringer and the accelerometer is located at the center of the span of third stringer to get the first frequency. As the wearing surface was not uniform near the center of the second stringer the shaker is placed 9 inches away from the centre so that its surface can be in complete contact with the deck and thereby avoiding any secondary vibration during the course of excitation. At this position (named as position-1) a swept sine test is conducted from 8 Hz to 25 Hz with 0.5 Hz resolution using the “Swept sine.vi” LabVIEW program. This test was conducted at 9:50 am when the deck temperature was 111°F. The accelerometer was moved nearer to center of the second stringer on which the shaker was placed earlier, but 2 feet away from the shaker to avoid any over excitation of the accelerometer due to the shaker (This position is named as position-2). A swept sine test was conducted from 8 Hz to 25 Hz with 0.5 Hz resolution. This test was carried at 11:15 am when the deck temperature

was 125°F.

For the second mode the maximum displacement in the mode shape occurred about one quarter of the distance from either end of the bridge. Hence each accelerometer and the shaker are placed 7.5 feet (as the total length of the bridge is 30 feet) from the end of second stringer and the accelerometer is placed 7.5 feet from the other end on the same stringer (this position is named as position-3). Again a swept sine test was conducted from 63 Hz to 75 Hz with 0.5 Hz resolution. The temperature of the deck during this test was around 155°F. Swept sine tests were conducted further near the peaks obtained at each trial with 0.1 Hz resolution for obtaining accurate frequencies.

The location of shaker and accelerometer for the three positions (position-1, position-2 and position-3) discussed above for conducting swept sine tests are shown in Appendix-B.

5.3.3 Evaluation of Mode Shape using Resonant Sine Dwell Testing

For evaluating the mode shape of the deck nine accelerometers (PCB 393B12) were mounted on the top of the deck, above the second stringer at an equal spacing of 3.5 ft leaving one foot on both ends of the deck. One accelerometer (PCB 393B12) was placed at 10 feet (which is one third of the total span) from the end of third stringer for secondary reference and one accelerometer (PCB 393A03) was placed on top of the shaker armature for primary reference. The location of shaker is decided based on the mode of interest. The setup for second mode evaluation can be seen in Figures 5.1 &

5.2. This setup is for second mode where the shaker is located at 8 ft. from one end of the bridge on top of second stringer. For the first mode all the setup remains similar except that the shaker is located exactly at the center of the second stringer. The shaker is located at the center of second stringer as defined in Position-2 setup and is excited at 16Hz. After collecting the data from all the accelerometers using the LabVIEW program, the accelerometers were moved on top of the deck exactly above the third stringer and the deck is again excited at 16Hz (near 1st longitudinal bending mode). The response from all the accelerometers is again recorded by using the LabVIEW program, "Modeshape for N channels.vi." Similar procedure is followed for 16.8Hz excitation. Now the shaker is moved to position 3 (defined in swept sine test) and the deck is excited at 69.4 Hz and 69.8 Hz (near 2nd longitudinal bending mode) and the mode shape data are collected using the same program for each case from all the accelerometers

Figure 5.1 Location of shaker and Accelerometers

Figure 5.2 Location of accelerometers on deck surface

5.4 Test Results and Discussion

5.4.1 Single Beam Analogy Results

The frequencies of the entire bridge are estimated using the single beam analogy and are calculated by using Equation 5.1 as follows:

$$E = 30 \times 10^6 \text{ psi.}$$

$$I = 4 \times I_g \text{ (Assuming there is 0\% composite action between the FRP deck and stringers)}$$

where I_g moment of inertia of one W 24 x 104 steel stringer = 3100 in⁴

$$\text{Therefore } I = 12400 \text{ in}^4$$

Total weight of the bridge = weight of 4 stringers + weight of the deck including wearing surface

$$= 4 \times 104 \times 30 + 25 \times 22 \times 30$$

$$= 12480 + 16500$$

$$= 28980 \text{ lbs}$$

$$\text{Total mass of the bridge} = \frac{28980}{386} = 75.5 \text{ lb-sec}^2/\text{in}$$

$$\text{And the mass per unit length of the bridge} = \frac{75.5}{30 \times 12} = 0.20855 \text{ lb-sec}^2/\text{in. /in.}$$

$$f_1 = \frac{1 * \pi}{2 * (30 * 12)^2} \sqrt{\frac{30 * 10^6 * 12400}{0.20855}} = 16.18 \text{ Hz}$$

$$f_2 = \frac{2^2 * \pi}{2 * (30 * 12)^2} \sqrt{\frac{30 * 10^6 * 12400}{0.20855}} = 64.717 \text{ Hz}$$

5.4.2 Natural Frequencies

Swept sine tests conducted for position 1 from 8 Hz to 25 Hz with 0.5 Hz resolution is shown in Figure 5.3. and the peak was observed at 17 Hz and 18.5 Hz. The deck temperature was 111°F. To obtain accurate first frequency which is at 9.5 Hz another swept sine test with 0.1 Hz resolution is conducted as shown in Figure 5.4 and the peak is identified as 9.7 Hz, and the deck temperature was 151°F.

Figure 5.3. Swept sine 8Hz to 25 Hz with 0.5Hz resolution @ 111°F

Figure 5.4. Swept sine from 7 Hz to 11 Hz with 0.1 Hz resolution @ 151°F

Swept sine test conducted at position 2 from 8 Hz to 25 Hz with 0.5 Hz resolution is shown in Figure 5.5. Peaks were observed near 16 Hz and 17 Hz. This test was carried at 11:15 am, when the deck temperature is 130°F.

Figure 5.5. Swept sine 8 Hz to 25 Hz with 0.5 Hz resolution- Position 2

Figure 5.6. Swept sine 15 Hz to 20 Hz with 0.1 Hz resolution – Posiiton 2

Figure 5.7. Swept sine 64 Hz to 75 Hz with 0.5 Hz resolution- Position 3

With the same locations of shaker and accelerometer another swept sine test was conducted from 15 Hz to 20 Hz with 0.1 Hz resolution. Here two peaks were observed, one at 16 Hz and the other at 16.8 Hz as shown in Figure 5.6. This test was conducted at 12:30 pm when the deck temperature was 141°F. Swept sine results obtained for position 3 from 63 Hz to 75 Hz with 0.5 Hz resolution and the spectrum is shown in Figure 5.7. The temperature of the deck during this test was around 155°F. As the peak was observed at around 69.5 Hz, a swept sine test was performed again from 69 Hz to 70 Hz (near 2nd longitudinal bending mode) with 0.1 Hz resolution and the spectrum is shown in Figure 5.8. Two peaks, one at 69.4 Hz and other at 69.8 Hz were observed in this spectrum. The deck temperature was 160°F during this test.

Figure 5.8. Swept sine 69 Hz to 70 Hz with 0.1 Hz resolution- Position 3

In both cases of identifying the frequencies, the location of the shaker is marked for further evaluation of the mode shapes. The frequencies obtained in all trials are tabulated in Table 5.1.

Table 5.1 Natural Frequencies for Wickwire Run Bridge

Position 1	Position 2	Position 3
9.7 Hz		
18.5 Hz	16Hz	
	16.8 Hz	
		69.4 Hz
		69.8 Hz

It can be observed from Table 5.1, the first longitudinal bending frequency decreased from 18.5 Hz to 16 Hz, this is due to the increase in the temperature of the deck from 111°F to 141°F. The decrease in frequency indicates that the bridge is more flexible at higher temperatures. Modal analysis on any system can only be done if the system satisfies the conditions of reciprocity, linearity, and time invariance. It is clear that there is a considerable effect of deck temperature on the dynamic behavior of the bridge, and therefore Wickwire Run Bridge is a time varying system. Forced vibration testing using steady state excitation is the only option for evaluating the “operating” mode shapes of a time varying structure. Since the curve fitting techniques used to extract mode shapes from impact excitation or random vibration testing are only applicable for linear time invariant systems.

5.4.3 Mode Shapes

Mode shape information obtained at all trials from the two rows are corrected using the secondary reference accelerometer through a similar technique explained in Section 4.4.2. The 2D mode shapes obtained for the two rows are plotted in Figures 5.9, 5.10, 5.11 and 5.12.

Figure 5.9. Mode Shape at 16 Hz

Figure 5.10. Mode Shape at 16.8 Hz

Figure 5.11. Mode Shape at 69.4Hz

Figure 5.12. Mode Shape at 69.8 Hz

Forced vibration testing using shaker excitation is proven to be a viable method for extracting the modal parameters (mode shapes and natural frequencies) of a small bridge such as Wickwire Run bridge. The sensitivity of damage detection is enhanced

for higher modes of vibration since the change in curvature is rapid for higher order modes (Venkatappa, 1997). Second longitudinal bending modes could also be sufficiently excited for the accelerometers to sense the vibrations. Higher order modes could not be evaluated due to time constraints.

As mentioned earlier, modal parameters are primary input for strain energy damage detection algorithm. Based on the forced vibration tests conducted in this study, it is evident that ambient temperature at the time of testing has significant influence on the dynamic behavior of FRP bridges. Further research is needed to study effects of environmental conditions on FRP bridge dynamics. This can be accomplished through detailed Finite Element (FE) analysis of a FRP bridge wherein different temperature gradients can be applied as loads and eigen value analysis performed to evaluate the change in modal parameters. Influence of environmental conditions on bridges dynamics have to be “factored in”, in order to use modal parameters in damage detection algorithms. In other words, changes in modal parameters (thereby strain energy) due to damage should be delineated from changes in modal parameters due to temperature and other environmental effects. Forced vibration testing is a viable technique for evaluating the dynamic characteristics of small bridges like the Wickwire Run bridge. While frequencies and mode shapes of FRP bridges are utilized in damage detection methods, DLA factors, deck accelerations, damping and dynamic strains are required to assess the performance of FRP bridges under traffic loads. Three FRP bridges are tested using controlled truck tests and DLA, damping etc are evaluated, details are presented in Chapter 6.

CHAPTER 6

DYNAMIC LOAD RESPONSE OF FIBER REINFORCED POLYMER COMPOSITE BRIDGES

6.0 Introduction

FRP decks with lighter self weight and lower stiffness combined with the low level of damping can lead to excessive deck vibration causing discomfort to pedestrians and may even increase the dynamic amplification of induced stress and deflections, therefore dynamic performance of FRP bridges is equally important as their static performance. In Chapter 5 steady state forced vibration technique was used for evaluating the dynamic characteristics of a small bridge useful for vibration based damage detection. In this chapter three FRP bridges, Katy Truss, Market Street, and Laurel Lick bridges were tested to evaluate dynamic load allowance (DLA) factors, vibration serviceability and damping ratios. The parameters evaluated were checked against AASHTO and OHDBC design code provisions. A discussion on DLA and vibration serviceability provisions in various design codes was presented in Chapter 2.

The parameters evaluated for the three bridges include:

1. Natural frequencies,
2. Deck accelerations,
3. Damping ratios, and
4. Dynamic Load Allowance (DLA) factors.

Many variables affect the bridge dynamic response such as the speed of the vehicle crossing the bridge, bridge vibration frequency, weight of vehicle, suspension system of

the vehicle, surface roughness, approach conditions and damping of the bridge structure. This study did not focus on studying the effects of vehicle dynamics and surface roughness on the bridge dynamic response. Three axle dump trucks, typically weighing about 40,000-50,000 lbs, provided by the West Virginia Department of Transportation-Division of Highways, to evaluate the dynamic load allowance (DLA) factors of the three bridges were assumed to have similar dynamic characteristics. The FRP bridge deck used in the three bridges is manufactured in a controlled environment of a pultrusion plant, resulting in a deck made to tight tolerances. A half inch or three-eighth inch polymer concrete was used as overlay in the three bridges. It was observed that all three bridges had a smooth riding surface without too many undulations; therefore, surface roughness is not a major issue of concern in dynamic response of FRP deck bridges.

6.1 Bridge Description and Instrumentation

The three bridges discussed herein were built using FRP composite bridge decks, designed by Constructed Facilities Center, West Virginia University and fabricated by the Creative Pultrusion, Inc. under the trade name of SuperdeckTM. Based on extensive analysis and testing of other FRP structural shapes such as I-beams and box-beams, it was established that a cross-section made of a hexagon and double-trapezoids (Figure 6.1), enhanced the structural performance of a highway bridge, especially in terms of load carrying capacity. The superstructures for the three bridges are different from each other though the decks were of same type, summary of three bridges can be seen in Table 6.1.

Figure 6.1. Cross-Section of Composite Bridge Deck Components

Strain gages, LVDTs, accelerometers were used to measure the bridge dynamic response in terms of fundamental frequency and damping ratios. These data are needed to evaluate DLAs. The instrumentation for the each of the bridges was different due to accessibility and site conditions are described in this section.

6.1.1 Katy Truss Bridge

Katy Truss Bridge is a 90 ft single span trough truss bridge built with wooden deck but later in year 2001 the wooden deck was replaced with FRP bridge deck. The stringers transfer the load to six floor beams at 18 ft spacing. The bridge is designed for a HS-20 loading. Fourteen strain gages were installed on this bridge, eight gages were mounted on four floor beams, one on top and bottom of each floor beam. Gages 3, 4, 9, and 5,6,10 were used in computation of degree of structural compositeness between the FRP deck and floor beams (Figure 6.2). Gages 13 and 14 were mounted to monitor the strains in the FRP deck. An LVDT was mounted on the center of the bridge on the central stringer to acquire the static and dynamic deflections. A PCB 393A03 accelerometer was mounted on top of the bridge deck at the center of the bridge to

record the vibration response of the bridge. The output of the accelerometer is $\pm 5V$ with a sensitivity of 1000mV/g.

Figure 6.2. Sensor layout for Katy Truss bridge

6.1.2 Market Street Bridge

In the case of Market Street Bridge, the deck is supported over seven plate girders with a center-to-center spacing of 8'-6" (Figure 6.3). It is an integral abutment bridge with a single span of 177 ft. Fifteen strain gages were installed on this bridge, eight gages were mounted on four stringers, one each on top and bottom of each stringer (refer to fig 6.4). Gages 1, 2, 15, and 3, 4, 16 were used in computation of degree of structural compositeness between the FRP deck and stringers (Figure 6.4). Gages 11, 12, 13, 14 were used to measure the strains in the FRP deck. Deflection readings could not be taken due to accessibility problems. A PCB 393B12 accelerometer, with an output of the accelerometer is $\pm 5V$ with a sensitivity of 10 V/g was mounted on top of the bridge deck to record the vibration response of the bridge.

Figure 6.3. Market Street Bridge under construction

Figure 6.4. Sensor layout for Market Street bridge

6.1.3 Laurel Lick Bridge

Laurel Lick bridge deck was supported by six FRP girders (WF 12 x 12 x ½) with center-to-center spacing of 2'-6" (0.76 m). It is a single lane integral abutment with FRP columns and lag panels with a span of 19 ft. Ten strain gages were installed on this bridge, six gages were mounted on six stringers, one on bottom of each stringer. Gages 1, 2, 15, and 3, 4, 16 were used in computation of degree of structural compositeness between the FRP deck and stringers (Figure 6.5). Gages 9, 10, 11, 12 were used to

measure strains in the FRP deck. Deflections and accelerations were not measured.

Table 6.1 provides the summary of structural details for the three bridges tested.

Figure 6.5. Sensor layout for Laurel Lick bridge

Data from strain gages, accelerometers and LVDTs was acquired using a System 5100 scanner from Vishay Micro-Measurements controlled through a laptop computer. The scanner can acquire data from fifteen strain sensors and five LVDTs/accelerometers with 16-bit A/D resolution. The lead wires used to connect some of the strain gages to the System 5100 scanner are very long (up to 50ft in length), which will increase the resistance of the gage. The Systems 5100 scanner has a built-in mechanism to correct the lead wire resistance of the strain gages. The maximum sampling rate of 50 scans per second per (50Hz) channel was used to acquire all the dynamic test data. Typically, the first natural frequency of bridges are within 10 Hz, therefore 50 Hz sampling rate is more than adequate to prevent any aliasing.

Table 6.1 Summary of Three Bridges

	Katy Truss Bridge	Laurel Lick Bridge	Market Street Bridge
Type of Bridge structure	FRP deck on truss structure	FRP deck and FRP stringer	FRP deck on Steel Plate Girders
Year Built	2001	1997	2001
Design Load	HS-20	HS-25	HS-25
Deflection Limit	L/800	L/500	L/800
Span	90'	19'	177'
Width	14'	16'	56'
Deck Thickness	8"	8"	8"
No of Lanes	1	1	3 & 2 sidewalks
No. of stringers	3 @ 6' spacing with 6 floor beams @ 6' spacing	6 @ 2'-6" spacing	7 @ 8'-6" spacing
Sidewalks	No	No	Yes
Bridge Entrance Conditions	Rough	Smooth	Smooth

6.2 Testing Procedure

The testing procedure for conducting dynamic tests on three bridges varies slightly depending on the in situ conditions. Hence the detailed procedure is discussed in the following sections:

6.2.1 Dynamic Load Allowance Factors

Tests were conducted on every bridge with a dump truck traveling at a crawl speed of 2 mph, and the bridge response at 2 mph was taken as a baseline value for computation of DLA. Depending on the bridge site conditions, the loaded truck was allowed to travel at highway speeds and the dynamic strains and accelerations were measured. Katy Truss bridge response was acquired for four speeds 5, 10, 15 and 20 mph and at crawl

speed of 2 mph, which was taken as a static response. The Market Street bridge response was acquired for three truck speeds of 10 mph, 20 mph, and 30 mph and at crawl speed of 2 mph, which was taken as a static response. In the case of Market Street Bridge, additional impact tests were performed, by running a loaded truck over a 2" x 4" wooden plank to create multiple impacts as shown in Figure 6.6.

The Laurel Lick bridge response was acquired for 4 mph and at crawl speed of 2 mph, which was taken as a static response wherein the dynamic load allowance factors were computed for one speed. The dynamic test was restricted to one speed of 4 mph because the Laurel Lick Bridge is close to an intersection.

Figure 6.6. Truck traveling over a 2"x4" wooden plank to create impact- Market Street Bridge

6.2.2 Vibration Serviceability

Vibration serviceability limit states of the Katy Truss and Market Street bridges were checked by evaluating the measured accelerations for various truck speeds. In order to check the measured bridge response of the two bridges against the 1983 OHBDC

acceleration limits for pedestrian serviceability (Figure 2.3), it is necessary to evaluate the first natural frequency of these two bridges. Power Spectral Densities (PSD) of acceleration response was calculated to obtain the first fundamental frequency of the bridge. The measured accelerations contained both the forced vibration response due to moving truck and free vibration response of the bridge after the truck has passed the bridge. The free vibration response was used to calculate the Power Spectral Density.

6.2.3 Damping

Damping is an important property of any bridge structure; it is a measure of how quickly a bridge can dissipate energy after excitation. Damping in a bridge structure has two components, i.e., material damping and structural damping. Damping of conventional bridge structures such as steel-concrete composite, concrete and timber are usually not measured, since the inherent damping of these conventional materials is sufficiently high enough to dissipate vibrations. The National Cooperative Highway Research Program (NCHRP) Synthesis on Dynamics Impact Factors for Bridges (NCHRP, 1998) states that, “Most bridges inherently do not have significantly different damping values, and thus damping is not a parameter that can be adjusted to control the response of bridges.” In addition, it states that “however, one reason for determining damping in bridges is to calibrate analytical models for which accurate damping values are necessary if accurate dynamic responses are to be determined.” One of the disadvantages of the lighter FRP decks with self weight is the lack of inherent material damping. As shown from the dynamic tests on the two FRP deck bridges, lack of

adequate self-weight (compared to concrete) and lower damping can cause excessive vibration in these bridges.

Since the two FRP bridges tested are predominantly vibrating in a single mode, damping ratios were evaluated by fitting an exponential decay envelope to the free vibration response of the bridge (See Figures 6.12 & 6.13). The peak amplitudes for each cycle of vibration were plotted and exponential curves were fitted to that data.

6.3 Results and Discussion

The results from the field tests conducted on the three bridges including the DLA factors, frequencies, accelerations and damping ratios are presented in this section. The DLA factors will help establish deflection and stress amplification due to moving truck loads whereas frequencies and acceleration are needed to check for human response to vibration limit states.

6.3.1 Dynamic Load Allowance Factors

The maximum DLA factors for various components of the bridges are reported in Table 6.2. The DLA factors for the Katy Truss and Market Street bridges are well within the 1998, AASHTO LRFD Code mandated DLA value of 33%. The DLA factors for Laurel Lick bridge are presented later in the report.

Table 6.2. Maximum DLA factors

Bridge	Katy Truss		Market Street	
Component	DLA	Truck Speed	DLA	Truck Speed
Deck	6.8 %	5 mph	30 %	30 mph
Stringer	9.4 %	10 mph	25 %	30 mph
Floor beams	9.4 %	5 mph	-	-

Additional impact tests were performed on Market Street bridge, by running a loaded truck over a 2" x 4" wooden plank to create multiple impacts as shown in Figure 6.6. The maximum DLA factors for various components of the bridge due to impact loads are reported in Table 6.3. Maximum DLA factors of 79% and 52% were obtained for impact load conditions, but the absolute values of strain after amplification due to truck impact are less than a 100 microstrains which is well within the failure strains of Steel and FRP. Moreover, the truck running over a 2" x 4" wooden block is an extreme test case scenario, which is not likely to be commonly encountered on bridges.

Table 6.3. Maximum DLA factors with truck crossing over 2"x4" –Market Street Bridge.

	Market Street Bridge	
Component	DLA	Truck Speed
Stringers	79%	2 mph - 2x4
Deck	52%	10 mph - 2x4

For Laurel Lick Bridge, it was observed that there was noticeable amplification of strains even for truck traveling at 2 mph (See Figure 6.7). Taking this amplification for 2 mph crawl test into consideration the DLA factors for Laurel Lick Bridge were recalculated based on a true “static” response i.e., when the center of gravity of the rear axle of a loaded test truck was located at the center of the bridge. A comparison of DLA factors evaluated based on 2 mph crawl speed test and a true “static” load case is presented in Table 6.4. The maximum amplification factors of 78% for stringers and 93% for deck were obtained when true static case was taken as baseline, which are much higher than 33% DLA mandated by the 1998 AASTHO LRFD Bridge Specifications. It should be noted that, even though the higher amplification of strains is a concern, as in case of the Market Street Bridge, the absolute strain values are much lower than the ultimate strain of FRP.

Figure 6.7. Strain-time response of gage 6 at 2 mph

Table 6.4. Maximum DLA factors for Laurel Lick bridge.

Components	2 mph as baseline		Static test as baseline	
	DLA	Truck Speed	DLA	Truck Speed
Stringers	14 %	4 mph	78%	2 mph
Deck	21 %	4 mph	93%	2 mph

Previous studies have shown that the DLA factors obtained from measured deflection are always higher than the DLAs obtained from measured strains (Paultre et al, 1992). DLA factors obtained from strain (Gage # 11) and deflection (see Figure 6.2) mounted at the same location on Katy Truss bridge are shown in Table 6.5. Test data from Katy Truss Bridge shows that DLA obtained from measured deflection is lower than DLA obtained from measured strains. This is true for 3 out of the 4 test cases corresponding to the Katy Truss bridge.

Table 6.5. DLA factors based on strains and deflections of Katy Truss bridge

Truck Speed	Strain	Deflection
5 mph	6.25 %	3.61 %
10 mph	0 %	0.41 %
15 mph	3.13 %	1.01 %
20 mph	6.25 %	4.58 %

It should be noted that the DLA's evaluated for three FRP deck bridges is based on global response and no efforts have been made to study the changes in local response due to dynamic loads e.g., local increase in stresses of FRP deck flanges and webs due to concentrated moving wheel loads. As seen from the dynamic test results of the lightweight deck (Chapter 3), FRP decks exhibit plate like behavior and local response needs to be studied to avoid any punching shear through the module flanges or buckling of webs due to wheel loads.

6.3.2 Vibration Serviceability

Figure 6.8 shows the acceleration response of Katy Truss Bridge under a truck traveling at 10 mph and Figure 6.9 shows the PSD of the acceleration data.

Figure 6.8. Acceleration of the Katy Truss Bridge at 10mph

Figure 6.9. PSD of the Katy Truss Bridge acceleration at 10mph

As seen in Figure 6.9 the bridges are vibrating predominantly in one particular mode, and frequencies were obtained for various speeds of the truck considering only the free vibration component. As shown in Table 6.6, the natural frequencies obtained at various speeds are slightly different, this is due to the frequency resolution of the PSD i.e., the difference is within one or two Δf 's (where Δf = sampling frequency/ no of samples) and not due to variation in bridge frequencies. It should be noted that the truck speeds are listed just to differentiate between various data sets; the speed of a truck has no bearing on free vibration response of structure except for the amplitudes of vibration.

Table 6.6. First fundamental frequency of Katy Truss Bridge.

Truck speed	2 mph	10 mph	15 mph	20 mph
Frequency	5.275 Hz	5.25 Hz	5.3 Hz	5.25 Hz

Figure 6.10 shows the acceleration of the Market Street bridge due to truck traveling at 10 mph. In order to obtain the PSD from the test acceleration data, a 10th order high pass Butterworth filter with a cutoff of 0.5 Hz was used to eliminate the DC component in the signal. The filtered free vibration response was used to calculate the PSD. Figure 6.13 shows the free vibration response of the bridge after filtering the acquired field data.

Figure 6.10. Acceleration of bridge due to truck traveling at 10 mph

Table 6.7 lists the natural frequencies of the Market Street Bridge obtained after analyzing the free vibration response of the bridge for various truck speeds. It should also be noted that the truck speeds are listed just to differentiate between various data sets; the speed of a truck has no bearing on free vibration response of structure except for the amplitudes of vibration. The second frequency in the two spectra (20 mph and 10 mph with 2x4) may not correspond to any vibration mode; it could be a multiple of first vibration mode.

Table 6.7. Natural frequencies of Market Street Bridge

Frequency	10mph	20mph	30mph	10mph with 2"x4"
First	3.19	3.0	2.97	3.09
Second	-	6.34	-	6.31

The maximum accelerations of the Katy Truss bridge due to a moving truck at various speeds are shown in Table 6.8. Maximum acceleration value of 0.12g (g being the acceleration due to gravity) was attained by the bridge due to truck traveling at 20 mph. Referring to the 1983 OHBDC acceleration limits chart (Figure 2.3), the maximum permissible acceleration for fundamental frequency of 5.25 Hz is 0.9 m/s^2 or 0.0917g. The acceleration of Katy Truss Bridge falls in the unacceptable range for 2 out of the 7 test cases (table 6.8).

It should be noted that there are no sidewalks on the Katy Truss bridge with little pedestrian traffic. A passenger traveling in a vehicle has a different threshold of bridge vibration sensitivity compared to pedestrian user of the bridge. At one end of the Katy Truss bridge, the approach was built with sharp grade (Figure 6.11) as the approach road is below the bridge deck and, thereby creating an impact load whenever a vehicle passes through the approach. The discrepancy in maximum accelerations for a truck traveling at same speeds at different times (Table 6.8) may be attributed to the rough approach conditions to the bridge.

Figure 6.11. Sudden grade in the approach for Katy Truss Bridge

Table 6.8. Maximum acceleration of Katy Truss bridge

Truck Speed	Maximum Acceleration	
	Trial 1	Trial 2
2 mph	0.0815 g (close to limit of 0.0917g)	-
10mph	0.081 g (close to limit of 0.0917g)	0.067 g
15mph	0.112 g (exceeds limit of 0.0917g)	0.058 g
20mph	0.0869 g (close to limit of 0.0917g)	0.12 g (exceeds limit of 0.0917g)

Vibration serviceability limit state of the Market Street Bridge was also checked by evaluating the measured accelerations for various truck speeds. Table 6.9 details the maximum acceleration of the bridge due to truck moving at different speeds. By checking against the 1983 OHBDC acceleration limits for pedestrian serviceability (Figure 2.3), the maximum permissible acceleration for Market Street Bridge with a frequency of 3 Hz is about 0.6 m/s^2 or 0.0611g. The data indicates that the acceleration is close to limiting values for 10 mph, 20 mph load cases, which significantly exceeds

the acceleration limits when truck is traveling at 30 mph, and when the truck is running over a 2x4 wooden plank at 2 mph and 10 mph.

Table 6.9. Maximum acceleration of Market Street Bridge

Truck Speed	Maximum Acceleration
2 mph	0.0361 g
10 mph	0.0609 g (close to limit of 0.0611 g)
20 mph	0.0595 g (close to limit of 0.0611 g)
30 mph	0.112 g (exceeds limit of 0.0611 g)
2 mph – 2x4 impact	0.1272 g (exceeds limit of 0.0611 g)
10 mph – 2x4 impact	0.2057 g (exceeds limit of 0.0611 g)

6.3.3 Damping

As shown in Figures 6.12 & 6.13 the exponential curves fit well to the acceleration data.

The damping ratio ξ can be obtained from equating $y=0.0669e^{-0.1804t}$ of the exponential envelope (experimental) to $y=Ae^{-\xi\omega t}$ where ω is the circular natural frequency and A is amplitude. Damping ratios of 0.54% of critical damping and 0.47 % of critical damping were obtained from two data sets for the Katy truss bridge revealing a very low damping of the bridge system.

Figure 6.12. Free vibration response of Katy Truss bridge with decay envelope

Figure 6.13. Free vibration response of Market Street bridge with decay envelope

Damping ratios for Market Street Bridge were calculated for two sets of data, 2.27% of critical damping and 1.67 % of critical damping were obtained from the two data sets. The exponential decay curve could not be obtained from other data sets because the free vibration response for other cases contained not one but, two dominant frequencies. Typical values of damping in FRP bridges measured in this study along

with damping values in conventional construction materials (steel and concrete) as reported by Paultre et al (1992), are shown in Table 6.10. The damping ratios are significantly lower than the concrete and steel-concrete bridges, this highlights the need for additional damping on FRP bridges, to reduce vibration related problems. A cost effective and easy-to-use passive damping technique for FRP bridges using of pour-in-place Urethane foams was developed, which is discussed in detail in Chapter 7.

Table 6.10. Typical damping ratios of bridges (Paultre et al., 1992)

Type of Bridge	Number of Bridges Tested	Average Damping Value	Lowest Damping Value
Concrete	213	7.9 %	2 %
Steel-Concrete Composite	12	8.4 %	5.5 %
Prestressed Concrete	4	2.2%	0.8 %
Steel	14	1.3%	0.4%
FRP-Current study	2	1.24%	0.5%

Uncertainty associated with the accelerometers used for acquiring the data in conducting these tests is only $\pm 1.5\%$ (considering 95% confidence level). Higher sensitivity and finer resolution (0.00004g for 393A03 & 0.000008g for 393B12) of the accelerometers yielded high accuracy in all the test results.

Chapter 7

Damping in FRP Decks

7.0 Introduction

Fiber reinforced polymer (FRP) composite bridge decks being considerably lighter in weight (10 lb/ft^2 to 25 lb/ft^2) than conventional constructional material like steel and concrete are subjected to high accelerations because of their low damping ratios. This phenomenon is observed in 6ft vinyl ester deck module#1 (explained in Section 3.4.3) which has low damping (1.2%) and in Katy Truss Bridge, which has high acceleration values. This can cause discomfort for pedestrians using the bridge, hence there is an urgent need for improving the damping properties of these FRP bridge decks. One such passive damping technique is developed in the laboratory using the pour-in-place urethane foam. This method is adopted for two reasons. 1) The foam doesn't add much weight to the structure and 2) due to its cellular structure the FRP deck can be filled with this foam. Dynamic characteristics for two 6 ft. vinyl ester FRP deck modules are readily available hence they are filled with urethane foam and their characteristics are evaluated after fill. The test results on damping are compared for scenarios of before and after filling the cells with foam.

As seen in Chapter 6, the first bending mode is the predominant mode of most of the bridges. Hence to optimize the amount of foam required to fill, the FRP deck modules at the center of the bridge span could be filled with urethane foam so that they can damp down the vibrations, and thereby reduce the deflection in the first bending mode. This

concept was tested on FRP deck modules by filling them with urethane foam only at the center for two feet as shown in Figure 7.1.

Figure 7.1. 6 ft. Vinyl Ester FRP deck with foam filled at its center.

7.1 Preparation of specimens

The two vinyl ester FRP composite deck modules discussed in Chapter 3 are tested to check the improvement in damping ratios due to pour-in-place urethane foam. As only first bending mode was evaluated for both specimens before fill the effect of foam was checked for first mode only. The deflection being more at the center of the specimen for the first bending mode, only 2ft of the deck at its center is filled with 4 lb/ft³ urethane foam.

Figure 7.2. Specific uses of pour-in-place method (Szycher, 1999).

There are a wide variety of foams available based on the density as shown in Figure 7.2. Keeping in mind the cost and self weight of the foam, medium density foam is used for testing the FRP deck modules. The module is filled in layers to allow sufficient airflow inside the module, which is very important for complete expansion of the foam. Figure 7.1 shows one such module filled with foam at its center.

7.2 Testing Procedure

7.2.1 Identifying Natural Frequencies using Swept Sine Testing

The test setup used here is same as the one discussed in Chapter 3. Swept sine tests are conducted on two modules separately after filling them with foam using the “Swept sine.vi” LabVIEW program. The peaks in the frequency spectrum showed the natural frequency of the specimen.

7.2.2 Evaluating the Modeshapes

Evaluation of the mode shapes is done using sine dwell excitation. The module is excited at its natural frequency and the response from the deck is collected from 33 points (11 points in three rows) over the surface of the module using the laser vibrometer. Only first bending mode is evaluated for two specimens.

7.2.3 Damping ratios

For evaluating the damping ratios the module is excited at its natural frequency and the data are collected from the center of the module. After few seconds the excitation is stopped and the data are collected until the signal completely died down. Only free vibration data are used for calculating the damping ratio based on the exponential decay method.

7.3 Test results

7.3.1 Natural Frequencies

Swept sine results for the two specimens are shown in Figures 7.3 and 7.4.

Figure 7.3 Swept sine test from 250Hz to 300Hz for vinyl ester deck#1

Figure 7.4 Swept sine test from 250Hz to 300Hz for vinyl ester deck#2

There is a considerable change in the frequencies of both specimens due to addition of the foam, which acts as damper. For the vinyl ester deck #1 the first bending frequency decreased from 258.25 Hz to 251.57 Hz where as for deck#2 it has increased from 244Hz to 251Hz. But for the deck #2 another peak is observed near 207Hz whose mode shape is also evaluated to check its significance. Table 7.1 shows the comparison of frequencies for two specimens. The significance of the peak observed at 207Hz for the vinyl ester deck #2 can be understood only after evaluating the mode shape at that frequency.

Table 7.1 Frequency comparisons of vinyl ester modules

Vinyl ester deck#1		Vinyl ester deck#2	
Before fill	After fill	Before fill	After fill
258.25 Hz	251.57 Hz	244 Hz	251Hz & 207Hz

7.3.2 Mode shapes

The mode shapes of two specimens at their natural frequencies are shown in following figures.

Figure 7.5 Mode Shape of vinyl ester deck#1 at 251.57Hz – After Fill

Figure 7.6 Mode Shape of vinyl ester deck#2 at 251Hz- After Fill

Figure 7.7 Mode Shape of vinyl ester deck#2 at 207Hz – After Fill

The mode shapes for two specimens with foam cannot be characterized as bending modes or otherwise, in the sense that the flanges of the deck module exhibit plate like behavior, which was not the case with them without foam. (Refer Figures 3.8 & 3.9). Hence additional testing is needed to correlate the first bending mode for the modules before and after fill. The mode shape of vinyl ester deck #2 at 207 Hz shows less deflection at its center compared to the mode shape of the same specimen at 251 Hz, which could be due to the foam present in its cells at the center of the specimen.

7.3.3 Damping

The free vibration decay data for the deck#1 at 251.57Hz is shown in Figure 7.8, with decay is in two stages. It is difficult to fit an exponential curve to this data for evaluating the damping ratio using exponential decay method. The objective was to study the effect of limited application (at the center) of urethane foam on the damping of modules.

In order to achieve the objective, dynamic tests are conducted on a 6" x 4" rectangular steel box tube. Steel being isotropic, does not exhibit the complex behavior of composite materials and thereby showing change in damping due to addition of foam. The rectangular cross section was chosen because for a closed section the bending mode occurs before any torsional mode which is required to check the effect of urethane foam on the damping ratios and natural frequencies

Figure 7.8 Free vibration decay curve for vinyl ester deck#1 at 251.57Hz

7.4 Test Procedure for Rectangular Steel Tube

The first three modal frequencies are estimated analytically for a free-free boundary condition. After getting an estimate of the frequencies a swept sine tests are conducted near each frequency using “Swept sine.vi” LabVIEW program to get an exact result. The test procedure comprises of three phases,

1. Identifying the Frequencies
2. Finding the Mode shapes at each frequency

3. Acquiring the free vibration data at each frequency

These three tests were conducted for the steel beam before and after filling it with urethane pour-in-place foam. The damping is estimated by using the exponential decay data of the steel beam. The test procedure adopted here is same as the procedure explained in Section 7.2. This procedure is applied of the specimen before and after filling it with foam.

7.5 Test Results

7.5.1 Natural Frequencies

The swept sine test results obtained for the rectangular steel tube before filling with foam are illustrated in Figures 7.9 and 7.10. And the swept sine test results for first and second modes of the rectangular tube after filling it with foam are shown in Figures 7.11 and 7.12. The natural frequencies of the rectangular steel tube are shown in Table 7.2

Figure 7.9 Swept sine test of steel beam before fill (1 Hz resolution)

Figure 7.10. Swept sine test for the steel beam before fill (1 Hz resolution)

Figure 7.11. Swept Sine test of Steel beam after fill- 185Hz to 250Hz with 1 Hz resolution

Figure 7.12. Swept sine test of the steel beam after fill from 510Hz to 530Hz with 0.25 Hz resolution

Table 7.2 Natural Frequencies of rectangular steel tube.

Before fill	After fill
210.7 Hz	206Hz
556.6 Hz	517.5Hz

7.5.2 Mode Shapes

The mode shapes at first and second natural frequencies of the rectangular steel tube before and after fill with urethane pour foam are shown in Figures 7.13 to 7.16.

Figure 7.13. Mode Shape of the Steel beam at 210.7Hz

Figure 7.14. Mode shape of the steel beam at 556.6Hz

Figure 7.15. Mode shape of the steel beam after fill at 206Hz

Figure 7.16. Mode shape of the steel beam after fill at 517.5Hz

7.5.3 Damping ratio

The free vibration and forced vibration response of the rectangular tube before filling with urethane foam at 206Hz is shown in Figure 7.17.

Figure 7.17. Forced and free vibration response of the beam at 210.7Hz

Figure 7.18. PSD of free vibration data at 210.7Hz

Figure 7.19. Free vibration response of the steel beam at 210.7Hz

Only free vibration part of the response is required for finding the frequency and the damping by using the exponential decay method of the free vibration data. The natural

frequency of the steel tube can be checked by finding out the PSD of the free vibration data, which is shown in Figure 7.18. The free vibration data is shown in Figure 7.19. The PSD shows a peak at 209.5Hz which is used to calculate damping ratio by exponential decay method. The decay envelope denoted by the red line in the figure whose equation is given by

$$\text{Exponential decay} = Ae^{-\xi\omega t} \dots\dots\dots (7.1)$$

Equating the two equations we have

$$Ae^{-\xi\omega t} = 6.1582e^{-22.44t}$$

$$\text{Or } \xi\omega = 22.44$$

$$\text{Where } \omega = 2\pi f$$

$$= 2\pi * 209.5$$

$$= 0.017047$$

Or the steel beam has 1.7% damping

Similar procedure is followed to calculate the damping ratio based on exponential decay data for the steel beam at remaining three frequencies (206Hz, 558Hz, and 517Hz) listed in Table 7.2 and the damping ratios are tabulated in Table 7.3.

Table 7.3 Damping ratios of Rectangular steel tube

At first natural frequency		At second natural frequency	
Before fill	After fill	Before fill	After fill
1.7% at 209.5Hz	2.4% at 206Hz	0.063% at 558Hz	0.4% at 517.5Hz

As seen in Table 7.3, damping of rectangular steel tube increased due to the urethane foam at first natural frequency and second natural frequency, providing proof for this concept. Extensive testing needs to be done on different modules and FRP deck and stringer systems utilizing this concept before any field implementation is attempted. Several issues such as thermal insulation provided by the foam thereby increasing the temperature differential between top and bottom of deck, water absorption and fire retardancy of foams also need to be addressed prior to any field implementation.

Chapter 8

Summary and Conclusions

8.0 Summary

Extensive study on FRP decks and FRP bridge decks stiffened with steel or FRP stringers is carried out both at the laboratory and field to evaluate their dynamic characteristics including natural frequencies, mode shapes, and damping ratios. Experimental investigation was started with FRP deck modules wherein their natural frequencies and mode shapes were evaluated (Chapter 3). This is followed by dynamic testing of FRP deck and stringer system in the laboratory (Chapter 4) using forced vibration excitation. A standard test procedure is established at this stage for further field applications. Using this test procedure, dynamic tests are performed on Wickwire Run bridge (Chapter 5) using forced vibration techniques. Controlled ambient vibration tests were conducted on three bridges to evaluate their natural frequencies, damping ratios, deck accelerations and DLA factors (Chapter 6). The damping ratios obtained by ambient vibration testing showed low damping in FRP bridges. Hence a technique for improving damping in FRP decks is suggested after experimentally evaluating the procedures in the laboratory (Chapter 7). The conclusions of the entire study are described briefly in the following section.

8.1 Conclusions

The following conclusions are drawn from dynamic response evaluation of FRP decks in the laboratory and of FRP bridge decks stiffened with stringers in the field.

1. Test results of the FRP deck modules reveal that local vibrations of the deck flanges are predominant part of deck response, especially at higher order modes. Also the mode shape of transverse (Figure 3.15) specimen indicated low moment transfer between the modules.
2. A standard test procedure for conducting forced vibration testing on FRP deck stiffened by steel stringers was established, including the development of custom LabVIEW programs for test automation (Section 3.1 and 3.2).
3. The test procedure established at the laboratory was used for conducting forced vibration tests on Wickwire Run Bridge. First two longitudinal bending modes were sufficiently excited and acquired. Test results of the Wickwire Run Bridge revealed a significant influence of temperature on bridge dynamic behavior.
4. Change in damping ratio of FRP components due to addition of urethane foam could not be determined consistently using the exponential decay method. Addition of urethane foam to the steel specimen increased its damping ratio (Chapter 7). Increase in damping is higher at second mode of vibration compared to the first mode.
5. Damping ratios of the Katy Truss and Market Street bridges indicate low levels of damping, which are a primary reason for excessive vibration. In the case of Katy Truss Bridge, one reason for the excessive vibration could be the rough approach to the bridge.
6. Dynamic Load Allowance factors for Katy Truss and Market Street bridge are within design specifications, however in case of Laurel Lick bridge the factors were as high as 93%.

7. Based on the Laurel Lick bridge test data, it was observed that proper care should be taken when selecting a baseline value for DLA computation for FRP bridges.

8.2 Contributions to the Field of Bridge Engineering

This report constitutes first comprehensive study on the dynamic response evaluation of FRP decks and FRP bridges. The contributions of this study include:

1. A thorough literature review on various issues related to the bridge dynamics and dynamic characterization of bridge decks. This review has lead to several conclusions such as 1) selection of test parameters, procedure and analysis for evaluating the dynamic load allowance (DLA) factors for the FRP bridges, 2) identification of various excitation methods for conducting dynamic tests on FRP bridges and illustration of their advantages and disadvantages, and 3) presentation of two methods of simplified analysis for bridge dynamics, which are useful at the design stage of bridges.
2. Standard test procedure was developed for forced vibration testing of small bridges using electrodynamic shakers. Most of the testing process has been automated using LabVIEW programs specifically designed to conduct swept sine tests and to evaluate the mode shapes. This procedure can be replicated to evaluate the modal parameters of any small bridge, by providing minimal training to any bridge engineer.
3. Effect of temperature was found to have significant influence on dynamics of FRP bridges, therefore a detailed investigation on the dynamic behavior of FRP bridges

with respect to temperature changes is recommended before employing vibration based damage detection techniques.

4. This study has found that dynamic response of FRP bridges is significantly different from the dynamic response of conventional bridges especially in vibration serviceability issues, which are not addressed by the AASHTO bridge design specifications. FRP bridges though designed according to AASHTO specifications were found to vibrate excessively due to truck traffic. This study recommends two solutions to reduce the excessive vibration problem: 1) vibration serviceability of FRP bridges should be incorporated in design process, and 2) additional damping has to be provided for FRP deck bridges.

8.3 Recommendations

1. Custom LabVIEW programs developed for the purpose of determining frequencies for bridges should be modified to acquire response from multiple accelerometers thereby reducing the time required for conducting swept sine tests.
2. Effect of temperature on the dynamic behavior of the FRP deck must be investigated before applying vibration based damage detection techniques on FRP bridges.
3. Issue of vibration serviceability must be addressed during the design of FRP bridges.
4. Additional tests should be conducted to improve damping in FRP decks using pour-in-place urethane foam.

5. Ambient vibration tests using two axle dump trucks should be conducted and compared with three axle dump trucks (current study) to see the improvement in behavior of FRP bridges.

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APPENDIX-A

A.1 Vinyl Ester Deck Stiffened by Two Steel Stringers

A.1.1 Position 1

Test setup:

Figure A-1. Position-1 of Shaker and Accelerometer for FRP deck and two steel stringer system

Test results:

Figure A-2. Swept sine results for Position 1

A.1.2 Position 2

Test setup

Figure A-3. Position-2 of Shaker and Accelerometer for FRP deck and two steel stringer system

Test results:

Figure A-4. Swept sine results for position 2

A.1.3 Position 3:

Test setup:

Figure A-5. Position-3 of Shaker and Accelerometer for FRP deck and two steel stringer system

Test results:

Figure A-6. Swept sine results for position 3

A.1.4 Position 4

Test setup:

Figure A-7. Position-4 of Shaker and Accelerometer for FRP deck and two steel stringer system

Test results:

Figure A-8. Swept sine results of position 4

A.2 Vinyl Ester Deck Stiffened by Three Steel Stringers

A.2.1 Position 1

Test setup:

Figure A-9. Position-1 of Shaker and Accelerometer for FRP deck and three steel stringer system

Test results:

Figure A-10. Swept sine results of Position 1

A.2.2 Position 2

Test setup:

Figure A-11. Position-2 of Shaker and Accelerometer for FRP deck and three steel stringer system

Test results:

Figure A-12. Swept sine results of Position2

A.2.3 Position 3

Test setup:

Figure A-13. Position-3 of Shaker and Accelerometer for FRP deck and three steel stringer system

Test results:

Figure A-14. Swept sine results of position 3

Appendix B

Figure B-1 Swept sine Position-1 for shaker and accelerometer

Figure B-2 Swept sine Position-2 for shaker and accelerometer

Figure B-3 Swept sine Position-3 for shaker and accelerometer

Appendix C

Accelerometer	Accel No.1	Accel No.2	Accel No.3	Accel No.4	Accel No.5	Accel No.6	Accel No.7	Accel No.8	Accel No.9	Accel No.10	Secondary Reference Accelerometer	Normalized Values of A11	Method of normalization of A11
	A1	A2	A3	A4	A5	A6	A7	A8	A9	A10	A11	A12	
Row 1(R1)	0.0008	0.0023	0.0034	0.0076	0.0049	0.0053	0.0055	0.0040	0.0040	0.0022	0.00058	1	$\leq (R1A11/R1A11)$
Row 2(R2)	0.0009	0.0027	0.0041	0.0055	0.0065	0.0062	0.0057	0.0052	0.0038	0.0019	0.00056	0.96552	$\leq (R2A11/R1A11)$
Row 3(R3)	0.0014	0.0031	0.0050	0.0064	0.0068	0.0069	0.0073	0.0064	0.0039	0.0021	0.00050	0.86207	$\leq (R3A11/R1A11)$
Row 4(R4)	0.0015	0.0027	0.0048	0.0067	0.0075	0.0088	0.0084	0.0065	0.0036	0.0013	0.00045	0.77586	$\leq (R4A11/R1A11)$
Row 5(R5)	0.0011	0.0030	0.0048	0.0075	0.0082	0.0078	0.0071	0.0058	0.0033	0.0006	0.00050	0.86207	$\leq (R5A11/R1A11)$
Normalized Acceleration values													
(R1*1)=>	0.0008	0.0023	0.0034	0.0076	0.0049	0.0053	0.0055	0.0040	0.0040	0.0022			
(R2*0.9655)=>	0.0008	0.0026	0.0040	0.0053	0.0062	0.0060	0.0055	0.0051	0.0036	0.0018			
(R3*0.8621)=>	0.0012	0.0027	0.0043	0.0055	0.0059	0.0059	0.0063	0.0055	0.0033	0.0018			
(R4*0.7759)=>	0.0012	0.0021	0.0037	0.0052	0.0058	0.0068	0.0065	0.0050	0.0028	0.0010			
(R5*0.8621)=>	0.0009	0.0026	0.0042	0.0064	0.0071	0.0068	0.0062	0.0050	0.0029	0.0005			

Table C-1. Normalization of line scans to obtain 2D Mode Shape

Procedure:

Each element in Row1 to Row 5 (from A1 to A10 in each row) is multiplied with the corresponding normalized value of A11 which is shown in A12. For example all the elements from A1 to A10 in Row 2 will be multiplied with 0.96552 to get the normalized values of that row with respect to the secondary reference accelerometer value. Similar procedure is followed for all rows to obtain 2D mode shape of the deck and stringer system.

Appendix - D

Figure D-1. Diagram for Loop 0 of swept sine.vi

Figure D-2. Diagram for Loop 1 of swept sine.vi

Figure D-3. Diagram for Loop 2 of swept sine.vi

Figure D-4. Diagram for Modeshape for N channels.vi

Figure D-5. Diagram for Sinewave amplitude.vi

Figure D-6. Diagram for Sinewave phase difference.vi