

**EVALUATION OF LOAD CHARACTERISTICS OF
I-89 BRIDGE 58N, RICHMOND**

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Abstract

The Vermont Agency of Transportation has some concerns about the structural integrity of Bridges 58 North and South on I-89 located in the town of Richmond Vermont. Specifically, the steel stringers under very heavy and extraordinary traffic loads. Material testing has indicated that the stringers are of lower grade steel than specified in the design. The objective of this research initiative was to instrument a portion of the deck in Bridge 58 North in an effort to estimate its static and dynamic characteristics. The bridge deck was instrumented with 10 accelerometers and 28 dynamic strain sensors. Two aspects of particular interest are: (i) the live load distribution among the various stringers and girders and (ii) the degree of composite action taking place between the steel stringers and girders and the reinforced concrete slab. Currently AASHTO distribution factors are used to determine load ratings on the bridges, which lead to possibly conservative estimates, thus restricting some overweight load passage. Accurate determination of the load bearing characteristics would allow for as-tested values to be used in lieu of the AASHTO distribution factors and therefore lead to a more accurate load rating. Based on the examination of the vibration data measured on the bridge for a span of two years, we concluded that the bridge deck exhibits a low level of composite action between the concrete deck and the stringers, but a significant level of composite action between the deck and the main girders. We also concluded that the load distribution for each stringer are best described by transverse influence lines shown in Fig. 4.15. These influence lines were computed using a finite element model updated based on the identified vibration characteristics of the bridge.

Chapter 1

Introduction

1.1 Objective

The objective of this research initiative is to instrument bridge number 58 (north) on Interstate 89 in the town of Richmond, VT in an effort to estimate its static and dynamic characteristics. Two aspects of particular interest are (i) the live load distribution characteristics and (ii) the degree of composite action taking place between the steel girders and the reinforced concrete slab. Currently AASHTO distribution factors are used to determine load ratings on the bridges, which lead to possibly conservative estimates, thus restricting some overweight load passage. Accurate determination of the load bearing characteristics would allow for as-tested values to be used in lieu of the AASHTO distribution factors and therefore lead to a more accurate load rating. Regarding composite action, the bridge structural drawings do not show explicit use of shear connectors, however if measurements indicate that the stringers are behaving as composite (or partially composite) an increased load carrying capacity can be attributed to them.

Determination of the load bearing characteristics of this bridge will be done through the use a series of remain-in-place strain and acceleration sensors installed on deck stringers; one near and abutment, one near a pier, and one in a negative moment region. The system will be capable of recording continuous vibration data during the operation of the bridge, thus displaying characteristics over a wide range of traffic types and streams. Information will be used in an effort to determine whether or not special care need be taken when overweight loads cross the bridge, and to possibly revise bridge load ratings.

1.2 Motivation

The Vermont Agency of Transportation (VTrans) is concerned with the structural integrity of the deck of Bridges 58 N and S on I-89 located in the town of Richmond Vermont. Specifically the steel stringers under very heavy and extraordinary traffic loads. Material testing has indicated that the stringers are of lower grade steel than specified in the design.

VTrans has expressed uncertainty regarding the composite action between the stringers and concrete slab. The structural plan drawings do not include details on shear stud connec-

tors which enable composite behavior through shear stress transfer between concrete deck and steel beams. These concerns coupled with an atypical structural system lead to further uncertainties in the load distribution factors of the stringers used in load rating analysis. Initial estimates based on standard codified procedures indicate a low rating for this bridge; however the deck structural system is unusual and the possibility exists that the codified procedures are not applicable to this unusual structural system. VTrans has an overload vehicle restriction on the bridge. In order for an overload vehicle to traverse the bridge, all traffic is stopped and the overload vehicle must cross down the center of the bridge at a speed of 5 miles per hour. This creates significant back up on the I-89 highway, which if possible, could be reduced or eliminated given the information derived from this study.

1.2.1 Specific Project Objectives

The University of Vermont Structural Monitoring, Diagnostics and Prognosis Lab, under the direction of Dr. Eric Hernandez will conduct a Structural Health Monitoring (SHM) investigation using state-of-the-art sensor technology and signal processing algorithms to estimate the load distribution factors of the steel stringers and their degree of composite action. This goal will be achieved by means of the following objectives:

- Instrument a portion of Bridge 58N deck with 24 strain sensors and 10 accelerometers. This constitute a relatively low number of sensors in comparison with similar type of investigations carried out by other researchers. This is possible due to the use of advanced algorithms capable of extracting maximum amount of information from the measured data.
- Record multiple sets of vibration data for one year under varying traffic and weather conditions.
- Formulate a global model of the bridge and a detailed finite element model (FEM) of the deck structure of Bridge 58N.
- Calibrate the FEM to match the dynamic behavior identified from processing vibration measurements. Estimation of the degree of composite action will result from this analysis.
- Estimate live load distribution factors (LDF) for deck stringers and girders. This will be done using two independent methods, one relies on direct measurement of strains and the other relies on the use of the updated finite element model.

1.3 Literature Review

Based on the latest data from the National Bridge Inventory approximately 24% of all multi-span bridges in the United States are constructed using steel girders and a concrete slab [1]. This percentage is higher in regions such as the Northeast where it reaches 63%. One

important component in this type of deck construction is shear connectors. Shear connectors enable composite behavior by transferring horizontal shear stresses between the steel beam and the concrete slab. Composite decks possess a significantly larger flexural strength and stiffness compared to a non-composite one [2]. Shear connectors are typically constructed by welding vertical steel studs to the top face of the top flange in steel girders prior to pouring of the concrete slab. The design of shear connectors is governed by two criteria; static strength and fatigue. Shear connectors are first designed for fatigue loads due to moving vehicles and then checked for static ultimate strength. Girders are checked for static ultimate strength assuming full composite action, i.e. the number of shear connectors is enough to transfer the horizontal shear force at the interface that results when the steel girder has fully yielded and the concrete slab has simultaneously reached its maximum compressive capacity. AASHTO LRFD Specifications require that steel girder/concrete slab decks be designed as fully composite [8]. If a beam does not have enough connectors to guarantee fully-composite behavior, then it is categorized as partially composite and its ultimate load capacity is typically governed by the failure of shear connectors.

Whenever the structural integrity of an existing bridge deck needs to be assessed; the presence and effectiveness of shear connectors becomes a central issue. Due to unknown construction practices, lack of as-built drawings and cumulative damage effects such as fatigue the effectiveness of shear connectors is uncertain. The most widely used approach in the practice of structural assessment of bridge decks with uncertain composite action consists in assuming no interaction between the concrete deck and the steel beam. This practice is typically conservative but results in a diagnosis that is not cost effective and inconsistent with the fact that over the years of service some of these decks have withstood traffic loading beyond the strength provided by the non-composite assumption. An overly conservative diagnosis regarding a bridge deck could result in an unnecessary decision to replace, retrofit, or to reduce the load rating of the deck. On the other hand, assuming that the deck is non-composite could result in a model that underestimates stresses in secondary structural elements prone to fatigue. Development of technologies capable of assessing the effectiveness of shear connectors and the degree of composite action in uncertain bridge decks would prove useful for engineers and public transportation decision makers.

A reasonable approach to assess the effectiveness of composite action in a deck is to instrument it with sensors capable of measuring the strains in the vicinity of the steel-concrete interface. If the strain measurements in the steel and concrete near the interface are close, then one can infer that there is negligible relative slip between the two surfaces and composite behavior is verified (at least within the range of loading consistent with the measurements). As an alternative, one can measure the strain at various points along the depth of the steel girder and estimate the location of the neutral axis. Using principles from solid mechanics, the level of composite action can be inferred from the estimated location of the neutral axis. This last approach is only valid if no net axial force is present in the deck. One drawback of strain-based approaches is that they require significant instrumentation and can only assess composite action at a local level, i.e. at the section where the strain is measured.

Recent examples of the strain measurement approach can be found in the literature. In [4] Breña et al. present results from monitoring an I-girder type highway overpass under a controlled live load test. A total of 60 strain measurements were used to estimate load distribution factors and these results were compared with the results from a finite element model (FEM). The researchers found that although the deck was designed as non-composite, the strain measurements across the cross section (assuming Bernoulli's hypothesis of linear strain distribution) were consistent with the condition of I-girders acting as composite with the reinforced concrete slab. In [5] Chakraborty and DeWolf developed and implemented a continuous strain monitoring system on a three-span composite I-girder overpass. The instrumentation consisted of 20 uniaxial strain gages. The study reported on data over a period of 5 months. Among other things, the study included the determination of the location of the height of the neutral axes of various structural members when large trucks travel across the bridge. One of the conclusions of this study was that the measured strain levels are typically significantly below those recommended by AASHTO. The authors stated that this is a byproduct of conservative simplifications typically used in conventional designs, such as not including redundancies, connection restraints, and the way in which loads are distributed to different parts of the structure. This conclusion is in agreement with previous studies [6].

Jauregui et al. [7] conducted a series of controlled field loading tests on a standard I-girder bridge built in the late 1950's and assigned for decommission. Measurements consisted of strains and vertical deformations at various points. Results of the investigation show that the deck behaved as if partially composite right up to the onset of yielding. Partial composite action occurred in spite of the lack of shear connectors between the girder top flanges and the concrete slab. This suggests that partially composite action of the girders can be attributed to friction due to the slab bearing down on the girder top flanges and mechanical interlock at the girder-deck interface. Jauregui et al. argued that these two forms of shear restraint are dependable if not overcome and thus may be used to arrive at a better measure of the bending stiffness and resistance of the deck.

Alternatively, global acceleration measurements induced by operational traffic loads can be used to estimate the stiffness provided by the presence of composite action in bridge decks. The logical grounding of this hypothesis stems from the fact that vibration measurements carry information about mode shapes and frequencies, mode shapes and frequencies are related to stiffness of the bridge deck and the overall stiffness of the bridge deck is affected by the presence of composite action, both of these last statements have been observed experimentally by previous researchers [9, 10].

Morassi et al. [9], performed a theoretical and experimental laboratory investigation into the behavior of isolated, free-free steel beams-concrete slab composite beams. They found that if shear connectors are damaged their effect can be seen in the changes in vibration frequencies. It is expected that their general conclusions extrapolate to cases with different boundary conditions. Finally, Kwon et al. [10] performed a series of controlled laboratory experiments aimed at testing the effectiveness of post-installed shear connectors. After examination of their experimental results, it is possible to conclude that steel girders with concrete

slabs that do not possess explicit shear transfer mechanisms in the form of shear connectors; exhibit a flexural stiffness that lies between the fully-composite and non-composite assumptions. It can also be concluded that the difference between the overall stiffness of a composite beam with shear connectors versus an identical one without shear connectors can be observed even within the range of linear stresses expected induced by operational loads.

We propose the use of a sensitivity-based weighted finite element model updating to determine the degree of composite behavior in operational bridge decks with unknown/uncertain installation of shear connectors. The free parameters of the model are the rigidity per unit length of the beam-slab interface and the elastic modulus of the concrete slab. The features used in the model updating procedure are the identified modal frequencies and their corresponding mode shapes extracted from global acceleration measurements. A sequential weighted least-squares solution was implemented with a diagonal weighting matrix on which each element is inversely proportional to the variance of the identified modal features.

From the perspective of model updating, the fundamental challenges are: (a) To show that the concrete modulus of elasticity and the interface stiffness are independently identifiable from a subset of modal frequencies and(or) mode shapes and (b) to show that the sensitivity of mode shapes and frequencies to changes in the free parameters is large enough to overcome the “noise” in the identified modal parameters. The identification noise is important because bridges are subjected to variations in environmental conditions (temperature and humidity) which affect boundary conditions and stiffness properties of the material and get reflected as changes in modal properties.

Chapter 2

Description of Bridge and Instrumentation

2.1 Geographical Location and Description

Bridge 58N is located along interstate highway 89 in the town on Richmond, Vermont (Fig.2.1). The bridge has two lanes traveling north and it provides an overpass over the Winooski river. The structure of the bridge consist in reinforced concrete columns and abutments supporting two built-up steel girders of variable cross section. The two girders are connected in the transverse direction by floor beams which also support longitudinal continuous steel stringers. A reinforced concrete slab serves as direct support to the asphalt surface and traveling vehicles. A typical cross section of the bridge with dimensions is shown in Fig.2.4.



Figure 2.1: Geographical localization of Bridge 58N with respect to Burlington, VT.



Figure 2.2: Overall view of bridge 58N from Winooski river



Figure 2.3: View of underside of bridge deck with stringers indicated.

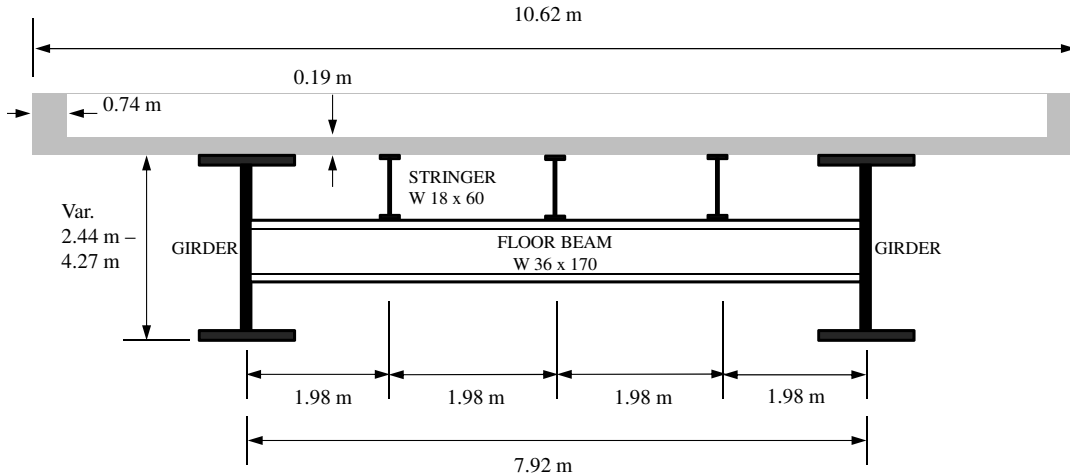


Figure 2.4: Typical cross section of bridge deck

2.2 Sensors and Data Acquisition

Two types of sensors were used in this project: (1) accelerometers and (2) dynamic strain sensors. All sensors were fixed on the bridge and data was recorded using a mobile data acquisition station. Due to lack of electricity and internet on the site, measurements consisted of 1-hour continuous measurements on selected days.

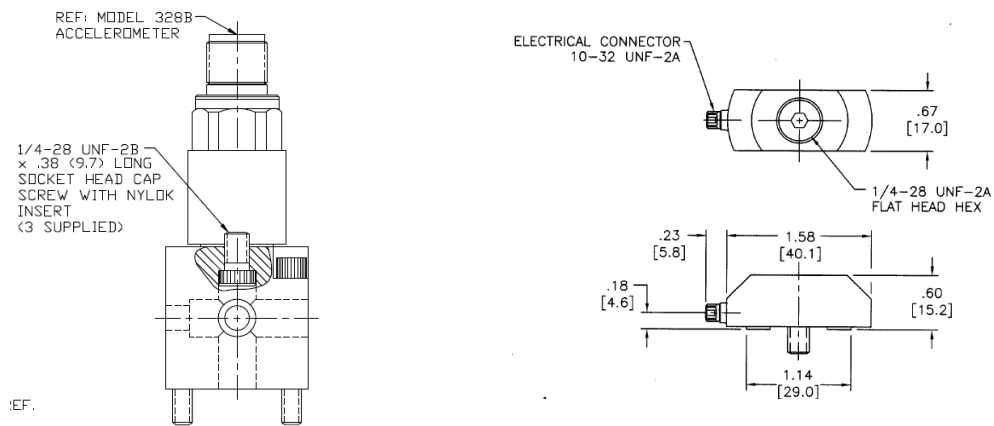


Figure 2.5: Sensors used in the instrumentation of bridge 58N. On the left the accelerometer PCB Piezotronics Model 393A03 and on the right the strain sensor PCB Piezotronics Model 240A02

Instrumentation on the bridge deck consists of 10 accelerometers (PCB Model 393A03) and 28 strain sensors (PCB model 242A02) as shown in Fig.2.5. To perform the data acquisition we used the LMS SCADAS MOBILE SCM05 with a uniform sample rate of 200 Hz for all sensors (see Fig.2.6).

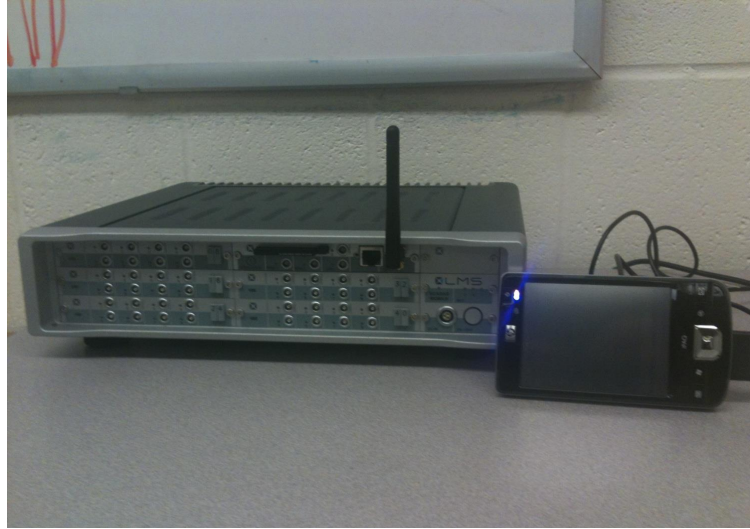


Figure 2.6: Data adquisition system used in this project. LMS SCADAS Mobile SCM05 and HQ iPAQ 214 PDA

2.3 Instrumentation

The main interest in this research project is the structural behavior of the deck. For instrumentation we selected (in coordination with VTrans) a representative portion of the deck. The instrumented zone is shown in Fig.2.7. This is the part of the bridge closest to the north abutment. The instrumentation of the bridge deck took place in Spring of 2012. The installation was carried out by a team from UVM and Vtrans under the coordination of Jason Tremblay (VTrans) and Eric Hernandez (UVM).

Fig.2.8 indicates the location of the accelerometers. Fig.2.9 presents the locations of the strain sensors. Fig.2.10 shows a picture of the mounting of a typical accelerometer and a strain sensor on a particular location in the deck.



Figure 2.7: Instrumentation zone in Bridge 58 North

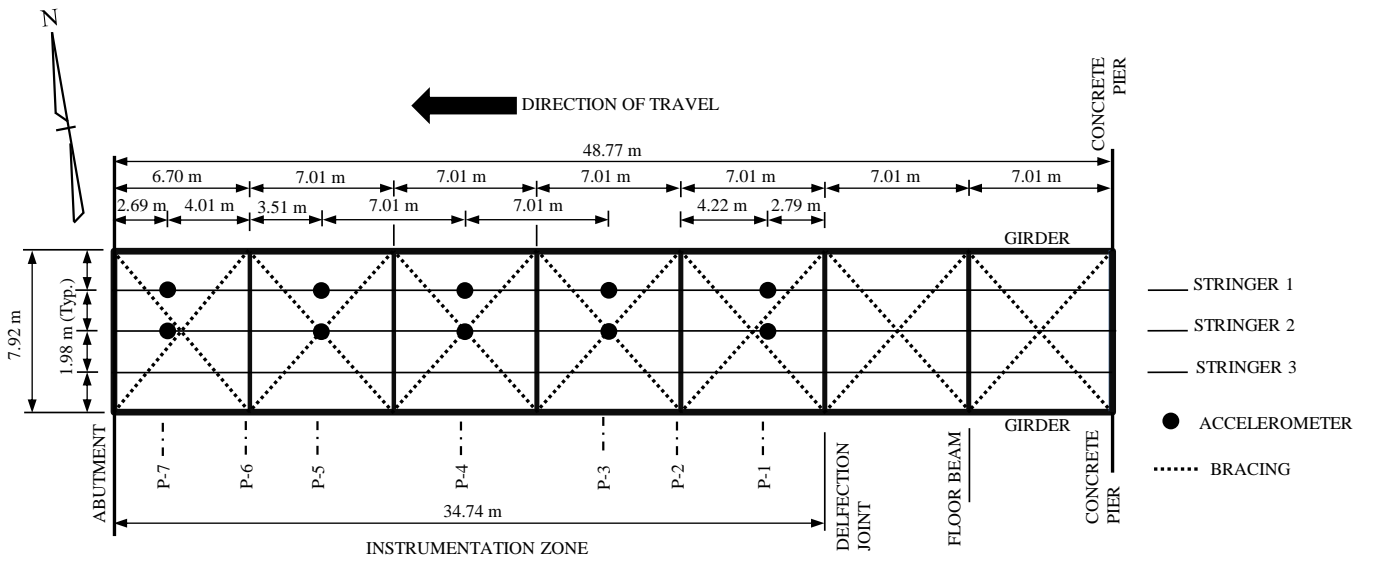


Figure 2.8: Plan view of accelerometer layout on instrumented portion of bridge deck

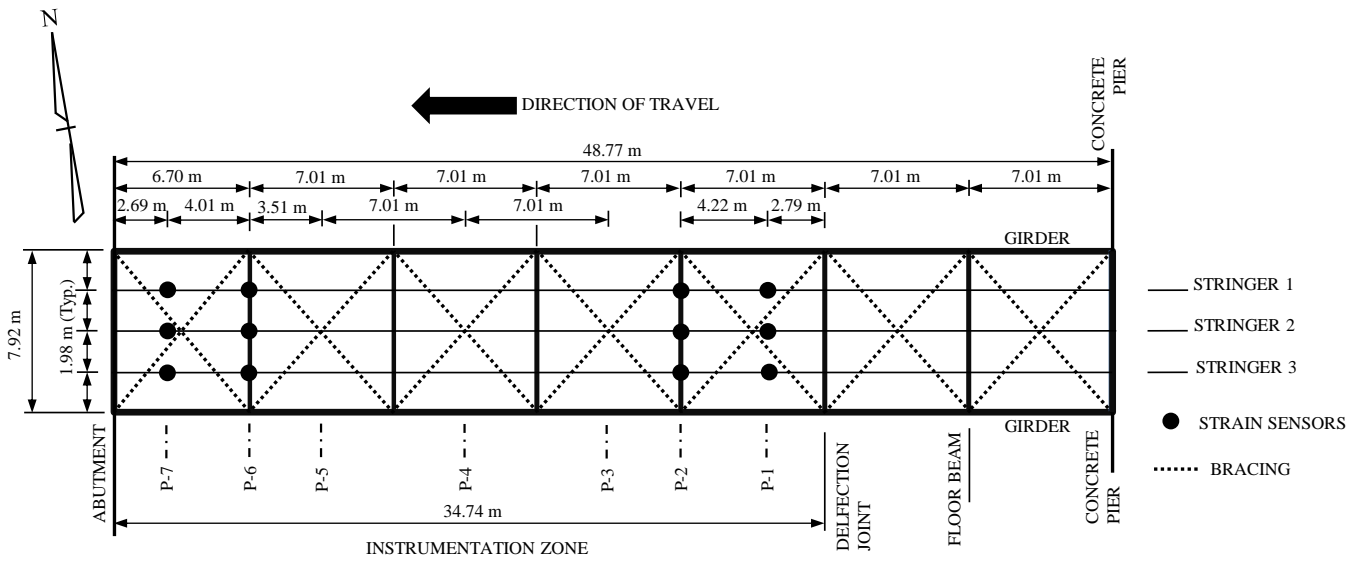


Figure 2.9: Plan view of strain sensor layout on instrumented portion of bridge deck

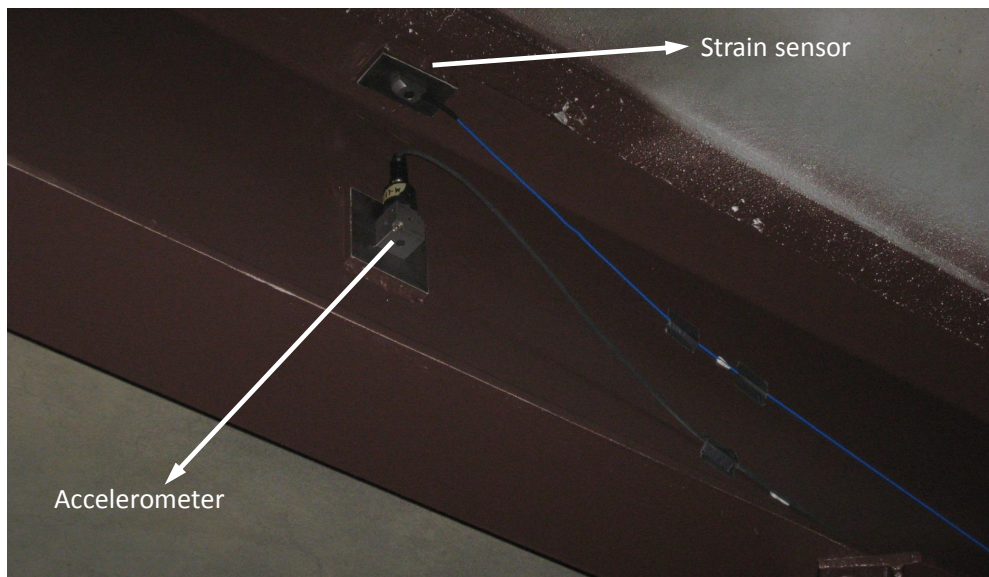


Figure 2.10: Accelerometer and strain sensor mounted on a particular location of the deck stringers

Chapter 3

Signal Processing and Data Analysis

In this section we present the signal processing and data analysis carried out as part of this research project. A description of the sensors and their location has been given in section 2.2.

The data collection presented in this study begins April 12, 2012 and concludes November 20, 2013, with a hiatus between August 27, 2012 and August 28, 2013. Therefore, we can consider that data spans between April and November of a nominal year. Measurements consist of 1-hour long records recorded sporadically during this interval. To reduce the effect of input uncertainty (weight, speed or lane of travel of the vehicles) only free vibration measurements were used in this analysis. A representative sample of 184 free vibration intervals were selected for analysis from the total measured data. The criteria for selection was that the length of the record be longer than 10 seconds after the vehicle left the bridge while no other vehicle entered the bridge during that time. The measured temperature during the selected intervals ranged from $15^{\circ}F$ to $87^{\circ}F$.

Typical strain measurements during two different truck events is shown in Fig.3.2 and 3.3. Typical acceleration measurements during the passing of a heavy truck are shown in Fig.3.4.

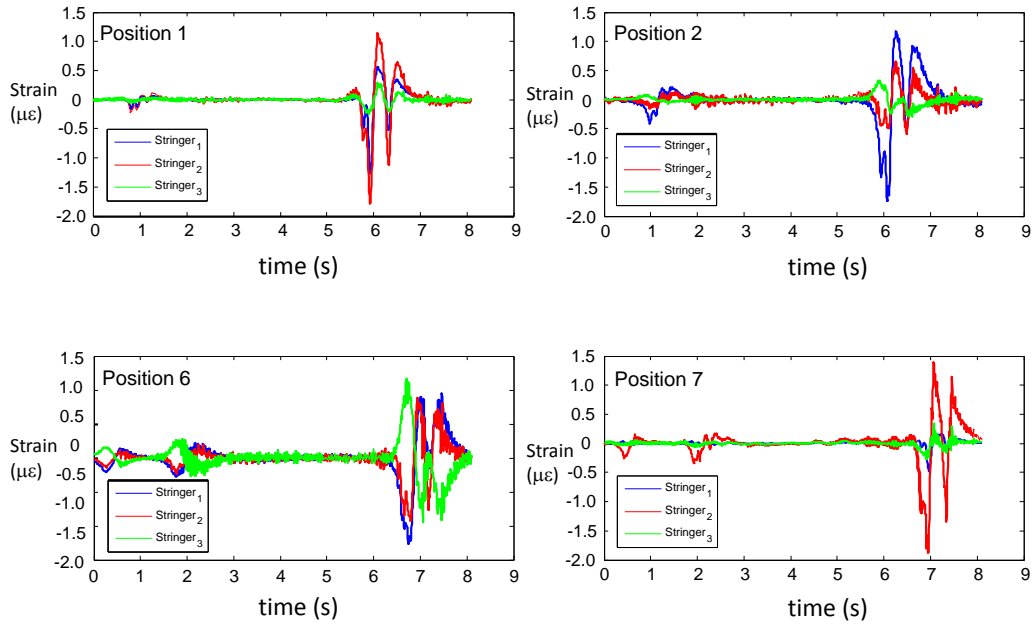


Figure 3.1: Strain measurements in the bottom flange of all stringers at various positions (see Fig.2.9 for strain sensor locations). The strains are measured at the welded plate and not directly on the stringer flange.

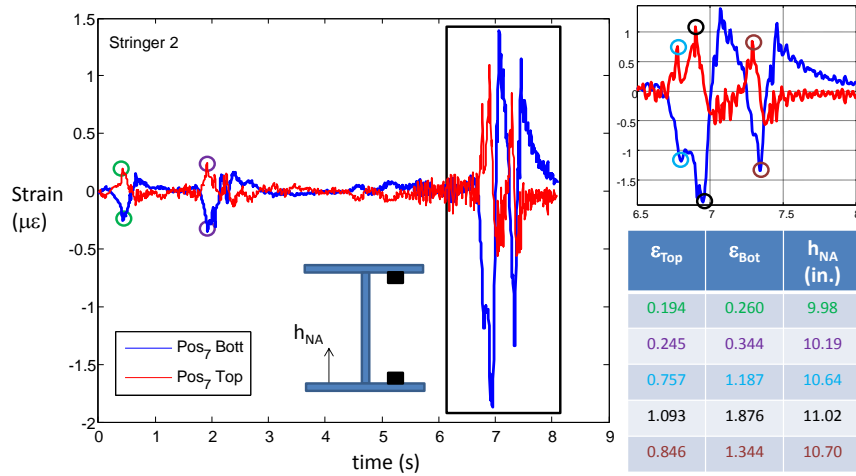


Figure 3.2: Strain measurements in stringer 2 (central stringer) as a vehicle traverses the bridge. The adjoint table presents the calculation of the neutral axis.

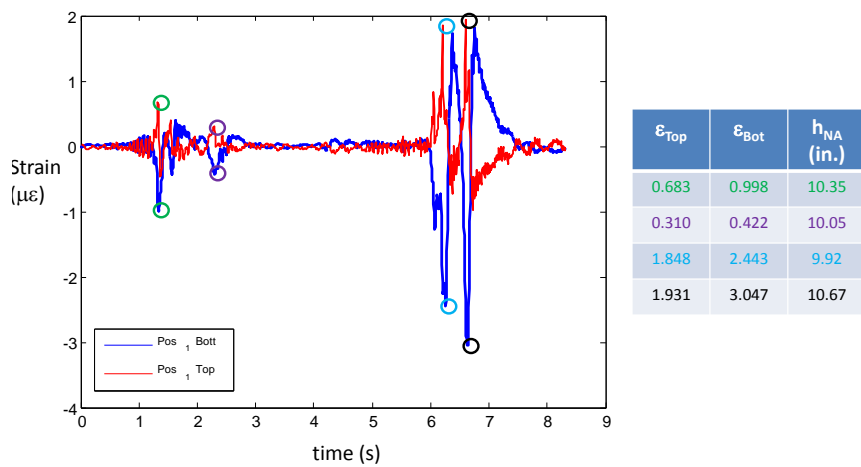


Figure 3.3: Strain measurements in stringer 2 (central stringer) as a vehicle traverses the bridge. The adjoint table presents the calculation of the neutral axis.

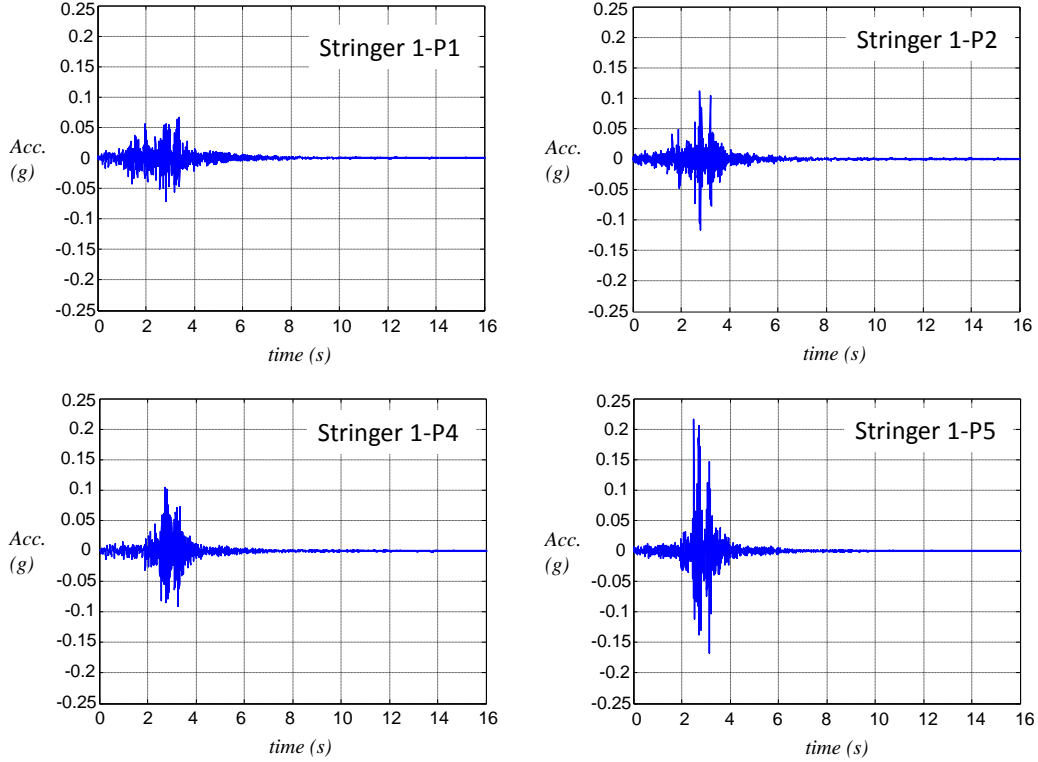


Figure 3.4: Acceleration measurements at various positions along Stringer-1 as a truck traverses the bridge (see Fig.2.8 for sensor location coordinates)

3.0.1 System Identification

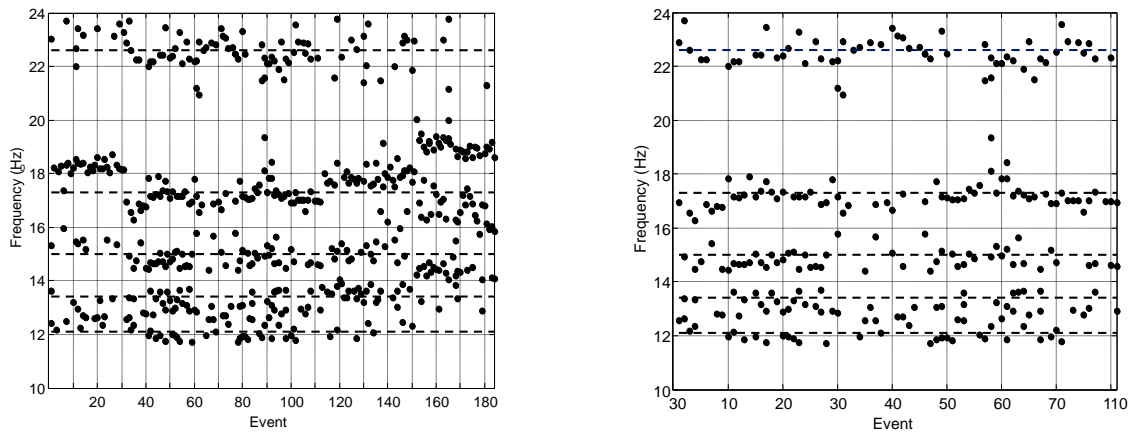
Knowledge of the characteristics of the traffic experienced by the bridge, vehicle speed, weight and traveling lane, is rarely available. As a way to reduce uncertainty related to the traffic induced excitation, free vibration responses are used for the system identification. The acceleration intervals were processed using the Eigensystem Realization Algorithm (ERA) [23]. The ERA identifies a linear model of the form

$$x(k+1) = \mathbf{A}x(k) + \mathbf{B}u(k) \quad (3.1a)$$

$$y(k) = \mathbf{C}x(k) + \mathbf{D}u(k) \quad (3.1b)$$

where $x(k)$ is the internal state at time $t = k\Delta t$, $u(k)$ is the input, $y(k)$ is the output. The \mathbf{A} matrix, also known in linear system theory as the state transition matrix, carries information about the system eigenvalues and mode shapes. The mathematical formulation

to extract the system frequencies from the eigenvalues of \mathbf{A} can be found in [23]. A summary of the identified modal frequencies from the selected data set is shown on Fig. 3.5.



((a)) Identified frequencies from events between April and November
 ((b)) Identified frequencies from events between July and October

Figure 3.5: Identified frequencies from global acceleration measurements

From Fig.3.5 several trends can be observed. First, it is clear that during colder months the natural frequencies experience an increase with respect to the warmer summer months. This change occurs in all mode shapes and it appears to be to some degree reversible, so it is not due to a structural damage. We attribute it to temperature and humidity variations which affect the mechanical properties of the deck and boundary conditions.

It can also be observed that there is appreciable variation of the identified frequencies even within the summer months. This can be seen in Fig.3.5(b). In general this variability can be seen to be in the order of 5 – 10%.

Chapter 4

Finite Element Modeling

4.1 Model Formulation

This section describes the various models of the bridge deck that were formulated and how these models were updated(modified) in order to represent or better match the identified structural properties from the acceleration measurements.

We began by formulating a global model to represent the fundamental vibration properties of the bridge deck. In this model the stringer play an insignificant role and were omitted for simplicity. Since the deck is symmetric with respect to the center, only half the bridge deck was modeled. The model is depicted in Fig.4.1. From this model we extracted a coupling stiffness matrix that will enable us to model the instrumented portion of the bridge in more detail without having to model the complete deck, while maintaining its compatibility at the boundary.

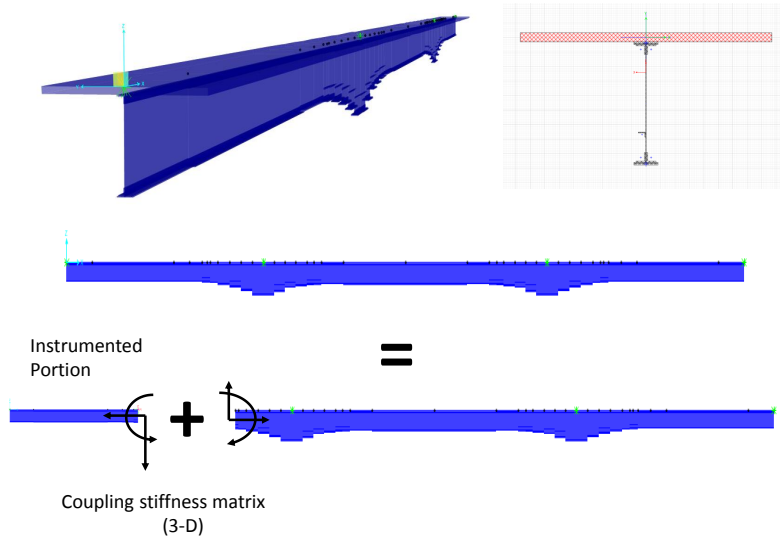
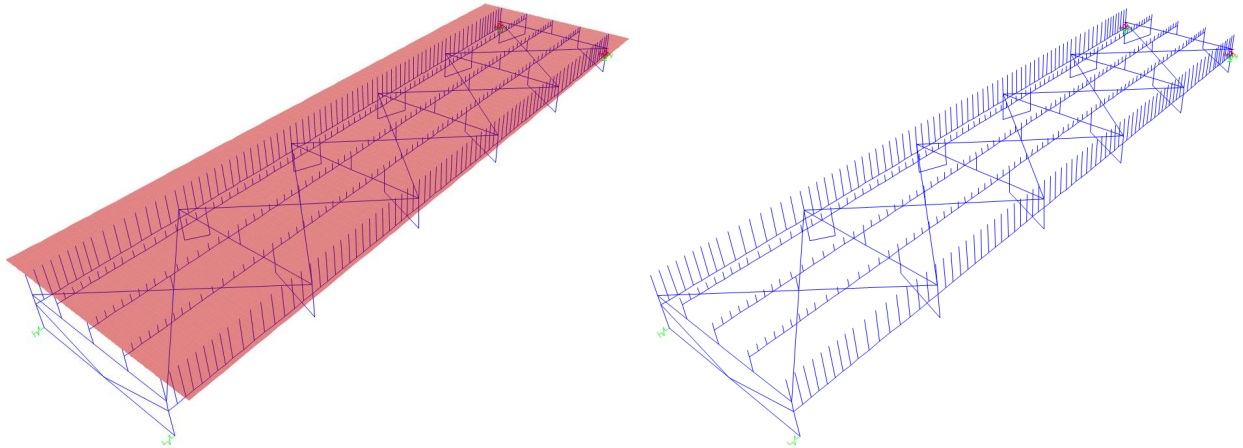
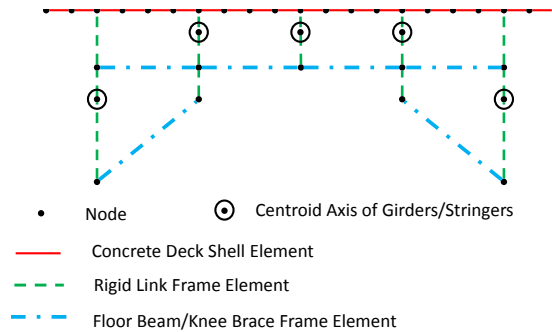


Figure 4.1: Global Model

A three-dimensional linear finite element model of the instrumented portion of the bridge deck was formulated. The model explicitly includes the main girders, floor beams, girders, bracings and slab. The model has 83,026 degrees of freedom, 932 frame members that represent the steel elements (girder, stringers, floor beams, bracings) and shear connectors, and 13,120 shell areas to model the concrete slab. The model is shown in Fig. 4.12(a), 4.12(b) and 4.12(c).



((a)) Three-dimensional finite element model of bridge ((b)) Three-dimensional FEM of bridge deck (no slab) deck



((c)) Cross section of three dimensional FEM

Figure 4.2: Three dimensional FEM

4.2 Sensitivity-based model updating

In this section we present the procedure we used to modify or update the finite element models in order to match the dynamical properties of the real structure identified by the processing of the sensor data.

4.2.1 Definitions and Theoretical Background

Finite element model updating can be defined as a series of computational steps, in which a preselected set of model parameters within a particular model class are modified to minimize a function of the difference between response measurements on the system and model predictions.

More formally this can be stated as: Given a model class $\mathcal{M}(\theta)$ with response feature vector $y_{\mathcal{M}} \in \mathbb{R}^m$ and corresponding system response feature vector $y_{\mathcal{S}} \in \mathbb{R}^m$, modify an f -dimensional subset $\theta_f \subset \theta$ such that a local minimum of the function $J = g(\Delta y)$ is attained,

where $\Delta y = y_S - y_M$. The subset θ_f is typically referred to as the free parameter space. In general, the response features and model parameters have a non-linear relationship and sequential linearization is typically required.

The relation between the response features and perturbations in the free model parameters $\Delta\theta_f$ can be written as

$$y_M(\theta_f + \Delta\theta_f) - y_M(\theta_f) = \mathbf{S}\Delta\theta_f + H.O.T. \quad (4.1)$$

where $H.O.T$ represents higher order terms in $\Delta\theta_f$, the matrix $\mathbf{S} \in \mathbb{R}^{m \times f}$ and each component of it is defined as

$$S_{ij} = \frac{\partial y_{M,i}}{\partial \theta_{f,j}} \quad (4.2)$$

The objective function J is typically selected as a quadratic form

$$J = \epsilon^T \mathbf{W} \epsilon \quad (4.3)$$

where

$$\epsilon = \Delta y - \mathbf{S}\Delta\hat{\theta}_f \quad (4.4)$$

$\mathbf{W} \in \mathbb{R}^{m \times m}$ is a weighting matrix and $\Delta\hat{\theta}_f$ is the estimated change in the free parameters. If $m \geq f$ then the solution that minimizes J is obtained by

$$\Delta\hat{\theta}_f = \alpha (\mathbf{S}^T \mathbf{W} \mathbf{S})^{-1} \mathbf{S}^T \mathbf{W} \{\Delta y\} \quad (4.5)$$

where $0 < \alpha \leq 1$ is a scalar. The purpose of α is to reduce the estimated change in the model parameters to avoid unrealistic variations (overshooting) byproduct of the linearization in eq.4.1.

In this study the measurement features will consist of a subset of modal frequencies and(or) their corresponding mode shape amplitudes at sparse locations. Analytical closed-form expressions of the sensitivity of eigenvalues and mode shapes in undamped multi degree of freedom systems can be found in the literature [20, 22]. The sensitivity of eigenvalues to parameters that define the mass and(or) stiffness is given by

$$\frac{\partial \lambda_j}{\partial \theta_k} = \phi_j^T \left[-\lambda_j \frac{\partial \mathbf{M}}{\partial \theta_k} + \frac{\partial \mathbf{K}}{\partial \theta_k} \right] \phi_j \quad (4.6)$$

This expression is very convenient because it only involves the mode shape corresponding to the frequency of interest. In the case of eigenvectors the sensitivity is given by

$$\frac{\partial \phi_j}{\partial \theta_k} = \sum_{h=1}^H a_{jkh} \phi_h \quad (4.7)$$

$$a_{jkh} = \frac{\phi_h^T \left(-\lambda_j \frac{\partial \mathbf{M}}{\partial \theta_k} + \frac{\partial \mathbf{K}}{\partial \theta_k} \right) \phi_j}{\lambda_j - \lambda_h} \quad \text{for } h \neq j \quad (4.8)$$

and

$$a_{jkh} = -\frac{1}{2}\phi_j^T \left(\frac{\partial \mathbf{M}}{\partial \theta_k} \right) \phi_j \quad \text{for} \quad h = j \quad (4.9)$$

In the case of eigenvector sensitivity, the result depends on all other mode shapes and frequencies, with heavier influence by the modes that are closer in frequency to the mode of interest. Alternative expressions for mode shape sensitivity can be found in [21]. In cases where the computation of closed-form sensitivities becomes computationally expensive or prohibitive one can always resort to a less elegant computation given by

$$S_{ij} \approx \frac{\Delta y_{\mathcal{M},i}}{\Delta \theta_{f,j}} \quad (4.10)$$

This requires careful selection of $\Delta \theta_{f,j}$ and the solution of multiple eigenvalue problems in order to compute the changes in eigenvalues and eigenvectors.

4.2.2 Computational Verification of Procedure

The mathematical theory describing the behavior of composite beams with weak shear connectors subjected to unidirectional bending on a symmetry plane has been studied in depth by various researchers [11], [12], [13] and [14], just to mention a few. More recently, Dall'Asta [16] developed a more complete theoretical formulation for three-dimensional cases. The author included out-of-plane bending and torsion based on Kirchhoff bending theory and Vlasov torsion theory. Ranzi et al. [17] performed a two-dimensional comparative study using four different formulations to analyze partially composite two-layer beams subject to symmetric bending, namely; the exact analytical solution, direct stiffness method, the finite element method and finite differences. The authors concluded that the direct stiffness method, formulated using basis functions from the exact solution, provides the best accuracy followed by the finite element method and the finite differences. In this project, we are interested in complex three-dimensional structures and we do not know the basis functions from the exact solution, therefore we resort to a finite element model (FEM) formulation.

All finite element models (FEM) to be considered in this project are linear with stiffness, lumped mass and classical damping. The set of parameters θ consist of all the material properties necessary to formulate the stiffness, mass and damping matrices. The set of free parameters θ_f will be the elastic modulus of the reinforced concrete slab and the stiffness of the connecting elements representing the rigidity per linear unit of length of the interface between the steel girder and the concrete slab. The stiffness of the connecting elements will provide an indication of the overall degree of composite action between steel and concrete. In both cases the parameters represent homogenized averaged properties.

As mentioned previously the two fundamental problems to address are: (i) identifiability of parameters, i.e. Is it possible to separately identify the elastic modulus of concrete and the stiffness of the interface? (ii) Can these parameters be identified in the presence of noise or bias in the identified modal features. To investigate these two aspects, we will postulate various scenarios and models with increasing complexity.

We begin with the simplified model of a 2-dimensional continuous beam with sections and dimensions analogous to the stringers (W18x60) shown in Fig.4.3 and tributary portion of the slab (1.98m). This model consists of two parallel longitudinal rows of elements that represent the concrete slab (top) and the steel stringer (bottom). The longitudinal elements are interconnected at intermediate nodes by massless perpendicular vertical elements spaced at 0.20m. The vertical elements represent the stiffness of the interface that enables the composite behavior. Figure 4.3 shows the dimensions and general details of the specific model considered. The initial values of the free model parameters are 3630 kN/m for the rigidity per unit length of the rigid connectors and 28 GPa for the elastic modulus of concrete.

After this, we study a 3-dimensional model analogous to the portion of the bridge deck that is instrumented as indicated in Fig.4.12(a) and 4.12(b).

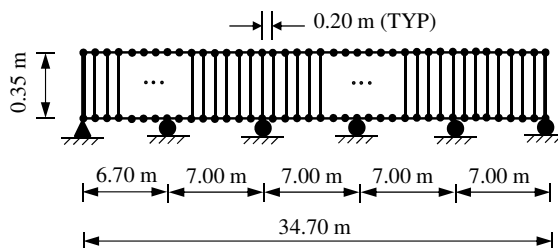


Figure 4.3: Two-dimensional finite element model of composite beam

Two Dimensional Model - Simulated Cases

In this section we describe the simulation results corresponding to the application of sensitivity-based model updating to the 2-D semi-composite beam shown in Fig.4.3. The main interest is to determine if the two free parameters (concrete modulus and shear connector stiffness) are distinguishable based on a subset of modal features; in this case the lowest 6 eigenvalues.

Various cases were considered:

1. CASE 1: Identify a simultaneous increase of 20% in the rigidity per unit length of the vertical connectors and in the elastic modulus of concrete.
2. CASE 2: Identify a decrease in the rigidity per unit length of the vertical connectors and an increase of 20% in the elastic modulus of concrete.
3. CASE 3: Identify an increase of 100% in the rigidity per unit length of the vertical connectors while the elastic modulus of concrete remains unchanged. This was induced by reducing their separation in half while keeping the stiffness of the individual connectors the same. This case involves the presence of model error since the model does not match the system used to generate the “identified” modal features.

The first two cases are intended to investigate the capability of the methodology to distinguish separate changes in the free parameters. For both cases the separation between

Table 4.1: System and initial frequencies (prior to updating) for Cases 1, 2 and 3 in the 2D Model

Frequency (#)	SYSTEM (Hz)	CASE 1 (Hz)	CASE 2 (Hz)	CASE 3 (Hz)
1	16.24	16.80	16.17	17.20
2	17.40	18.042	17.30	18.63
3	20.28	21.13	20.21	21.85
4	22.86	24.13	22.84	24.92
5	25.79	27.05	25.58	28.39

the vertical connectors was selected as $s_m = 0.20 \text{ m}$. Case 3 examines the effect of model error. Here the separation of the connectors is inconsistent with the model and we investigate if the correct stiffness per unit length can still be estimated. For this case the system had a separation of vertical connectors $s_\lambda = 0.10 \text{ m}$ and the model $s_m = 0.20 \text{ m}$. For all cases, the sensitivity approach was implemented using only the discrepancies in the first six eigenvalues. In all cases the initial values of the free model parameters are $3,630 \text{ kN/m/m}$ for the rigidity per unit meter of the rigid connectors and 28 GPa for the elastic modulus of concrete.

Figs. 4.4, 4.5 and 4.6 show the evolution of the free model parameters as the number of iterations increases for cases 1,2 and 3 respectively. It can be seen that in all cases the selected free parameters converge to the target values. This suggests that the parameters are independently identifiable from modal information.

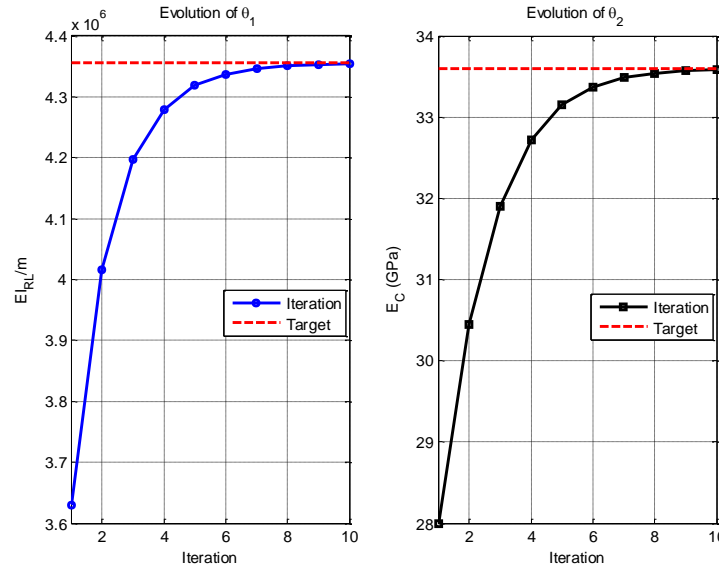


Figure 4.4: Evolution of model parameters a) Rigidity per linear meter and b) elastic modulus of concrete for Case 1 of 2D FEM

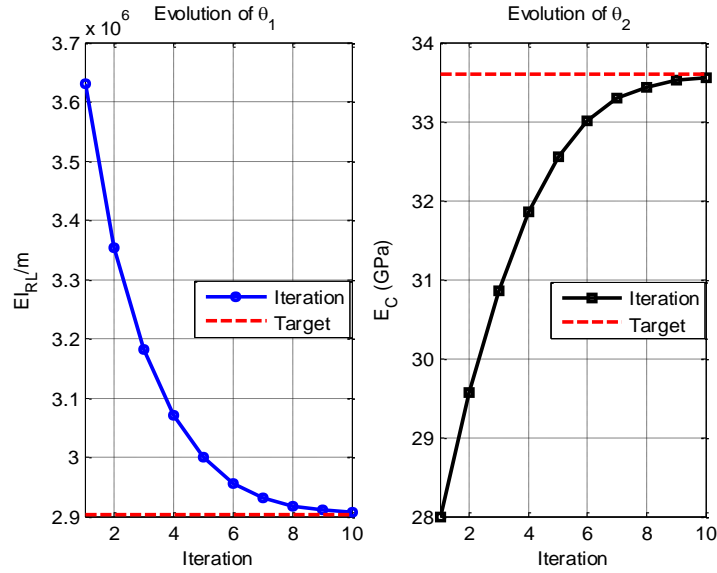


Figure 4.5: Evolution of model parameters a) Rigidity per linear meter and b) elastic modulus of concrete for Case 2 of 2D FEM

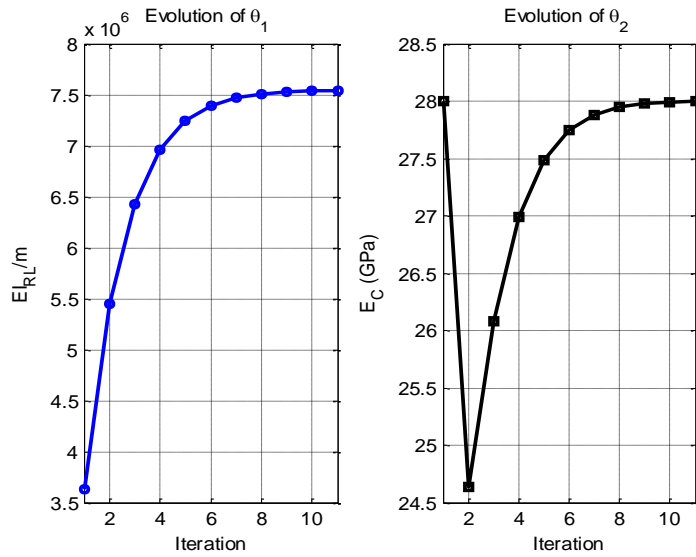


Figure 4.6: Evolution of model parameters a) Rigidity per linear meter and b) elastic modulus of concrete for Case 3 of 2D FEM

Three Dimensional Model - Simulated Cases

Following encouraging results from updating a 2D FEM of an isolated stringer, we now look into the verification of the sensitivity approach using a 3D FEM which simulates the

instrumented portion of the bridge deck shown in Fig.4.12(a) and 4.12(b). In this model we investigate cases 1 and 2 from the previous section, namely

1. CASE 1: Identify a simultaneous increase of 20% in the rigidity per unit length of the vertical connectors and in the elastic modulus of concrete.
2. CASE 2: Identify a decrease in the rigidity per unit length of the vertical connectors and an increase of 20% in the elastic modulus of concrete.

Table 4.2 shows the five modal frequencies used to perform the model updating. The table also shows the initial values of the frequencies prior to updating corresponding to each case. The sensitivity matrix was approximated using eq.4.10. The value of $\Delta\theta_f$ to compute the changes in modal parameters was selected as $0.01\theta_f$. the sensitivity matrix is shown in Fig.4.7. As expected the eigenvalues are more sensitive to changes in the concrete modulus, however the sensitivity due to changes in the stiffness of the shear links is not negligible.

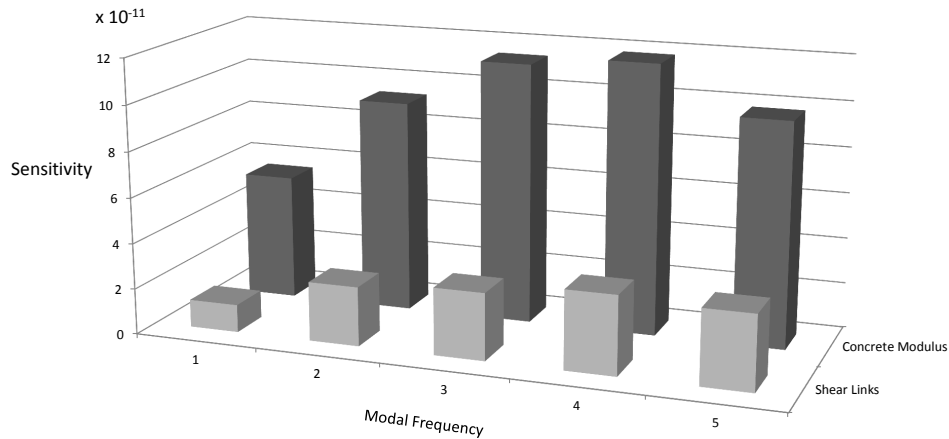


Figure 4.7: Sensitivity matrix of first five eigenvalues to concrete elastic modulus and shear links.

Figures 4.8 and 4.9 show the results for Cases 1 and 2 respectively. Similarly to the 2D FEM, the modal features and the model parameters converge to the target values.

Table 4.2: Comparison of frequencies for Cases 1 and 2 with the initial values for the 3D Model

Frequency (#)	SYSTEM (Hz)	CASE 1 (Hz)	CASE 2 (Hz)
1	14.05	14.35	14.30
2	17.16	17.69	17.58
3	18.15	18.80	18.67
4	18.83	19.51	19.37
5	22.19	19.51	19.37

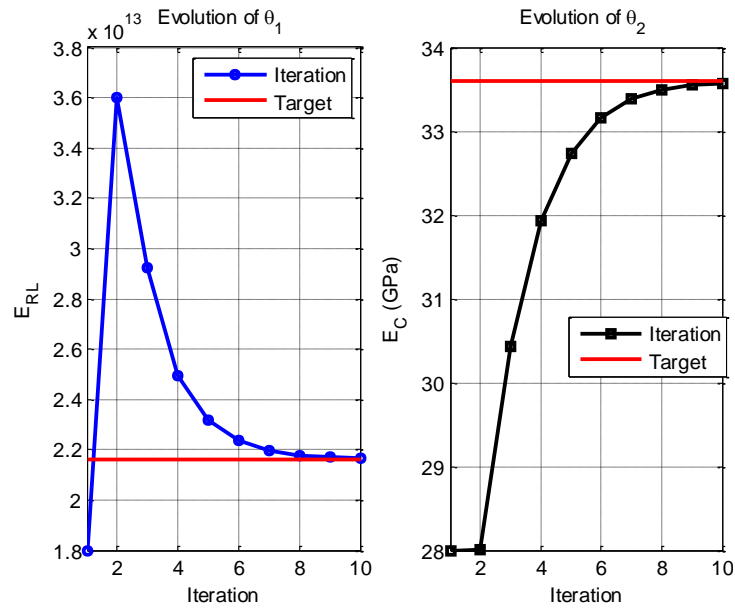


Figure 4.8: Evolution of model parameters a) Rigidity per linear meter and b) elastic modulus of concrete for Case 1 of 3D FEM

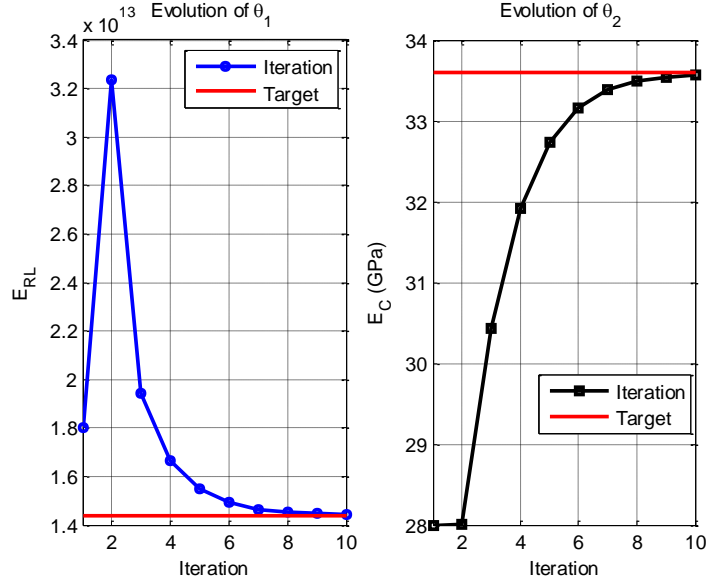


Figure 4.9: Evolution of model parameters a) Rigidity per linear meter and b) elastic modulus of concrete for Case 2 of 3D FEM

4.3 Model Updating of Global Model Using Measured Accelerations

In this section we apply the model updating procedure to the global finite element model shown in Fig.4.1. This model is symmetric around the central longitudinal axis and omits the stringers, since they ply an insignificant role in the global vibration properties of the bridge. The three main variables in this model are the lateral stiffness of supports, the modulus of elasticity of the reinforced concrete slab and the stiffness of the connections between the reinforced concrete slab and the steel girders. The fundamental question here is: Are the girders acting composite with the reinforced concrete slab?

This model was updated in order to match the first three fundamental frequencies of the deck as a whole. The identified frequencies from the acceleration measurements are shown in Fig.4.10. As can be seen from Fig.4.11, the model that better replicates the identified vibration frequencies is the composite model. From this we can conclude that the outer girders can be considered as composite with the reinforced concrete slab. This can be attributed to various factors: (i) the use rivets in the built-up girders, these can generate enough roughness to create the necessary bond between the steel and concrete, and (ii) friction generated by the weight of the slab on the steel girders.

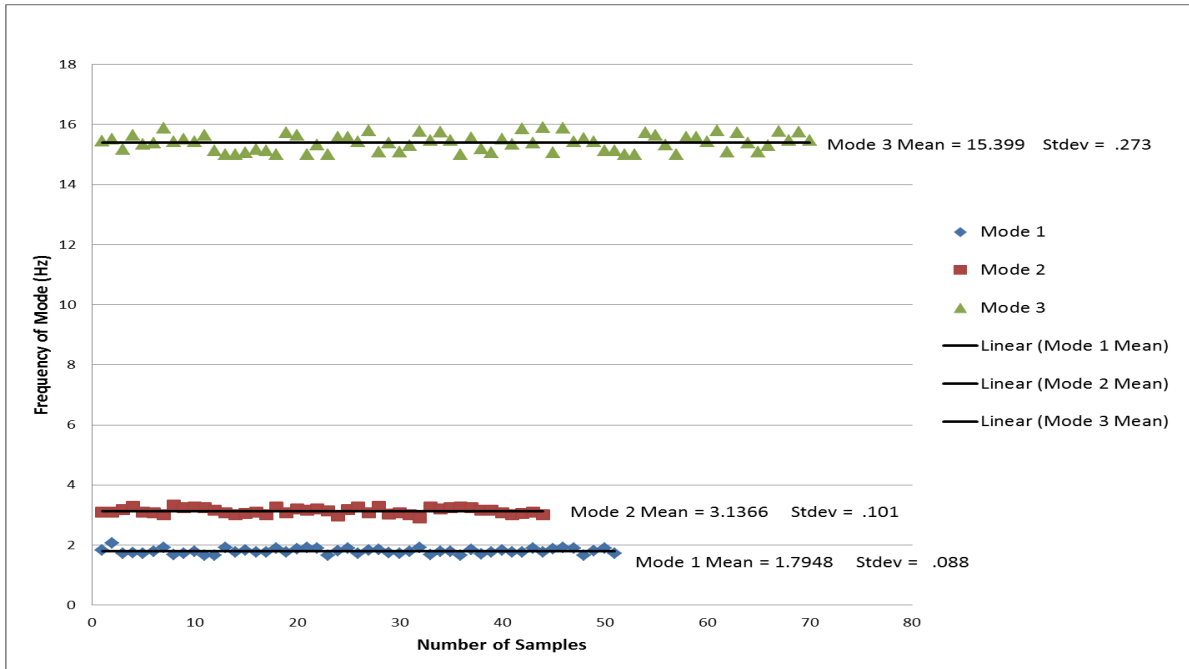


Figure 4.10: Identified global frequencies of bridge deck.

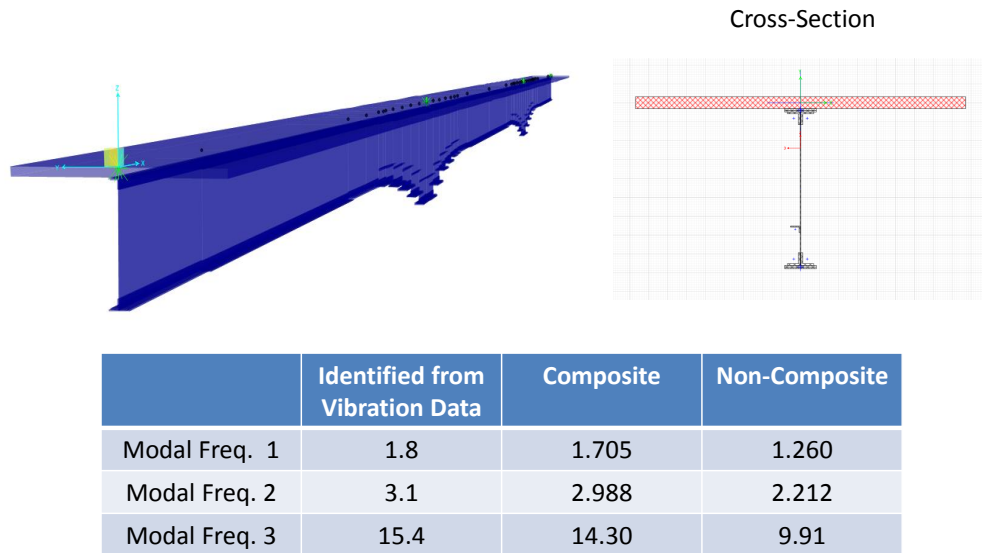
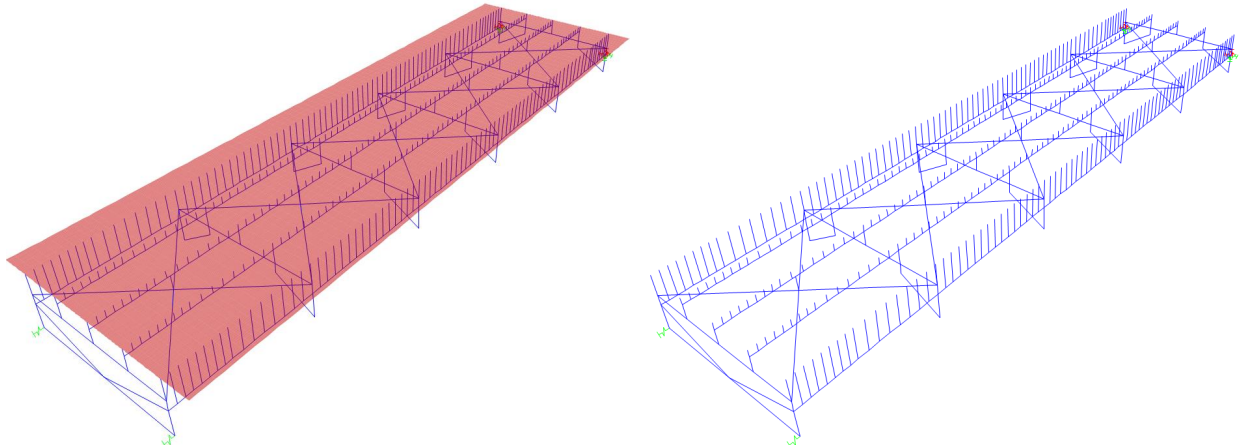


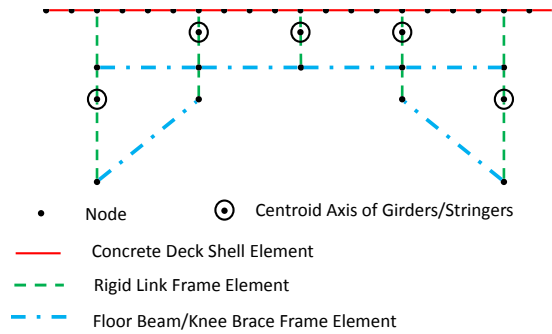
Figure 4.11: Comparison of modal properties between composite model, non-composite model and those identified from vibration data.

4.4 Model Updating of Detailed Deck Model Using Measured Accelerations

In this section we present a summary of the results from the implementation of the finite element model updating procedure on a detailed model of the instrumented portion of the bridge. This model is intended to represent the local behavior of the deck and will be heavily influenced by the composite action in the stringers and the stiffness of the reinforced concrete slab. The model is shown in Fig.4.12(b). From the results of model updating the global model, we will operate under the condition that the concrete slab is acting composite with the outer girders.



((a)) Three-dimensional finite element model of bridge ((b)) Three-dimensional FEM of bridge deck (no slab) deck



((c)) Cross section of three dimensional FEM

Figure 4.12: Three dimensional FEM

We compared two scenarios: (i) using all identified frequencies across the complete time interval of measurements and (ii) using only the subset of the data corresponding to the summer months (see Fig.3.5(a) and 3.5(b)). In the first case we would find yearly average values of the free parameters while in the second case we will identify lower bound values.

In both cases the variance of the identified frequencies is computed and a diagonal weighting matrix is computed. The i^{th} diagonal element of the weighting matrix is the inverse of the variance of the i^{th} identified frequency. For convenience purposes, the weighting matrix is scaled in such manner that the diagonal of the weighting matrix adds to unity. This is an arbitrary choice since the weighting matrix can be multiplied by any scalar without changing the result of the weighted least-squares solution. For each case, two different procedures were implemented. In the first one, the model updating algorithm was unconstrained while in the second one, the algorithm was constrained to operate within reasonable limits for the variables, specially the concrete modulus of elasticity. The selected lower and upper bound for the concrete elastic modulus were 21.52 GPa and 27.79 GPa respectively, this corresponds to a lower bound of a compressive strength of concrete of 20.68 MPa(3 ksi) and an upper

bound of 34.47 MPa(5 ksi).

The model updating results for the scenario where all the data was used is presented in Tables 4.3 and 4.4. The model updating results for the scenario where a subset of the data was used is presented in Tables 4.5 and 4.6.

Table 4.3: Model Parameters - Using complete data set

Parameter	Initial	Unconstrained	Constrained
$E_c(GPa)$	28	10.08	21.5
$k_L(MN/m/m)$	516.6	2,857.7	4.2

Table 4.4: Comparison of frequencies Bridge 58N - Using complete data set

Frequency (#)	ID (Hz)	σ^2	W	Initial (Hz)	Unconstrained (Hz)	Constrained (Hz)
1	12.04	0.65	0.09	14.04	12.64	12.94
2	13.37	0.14	0.43	17.16	14.90	14.40
3	15.02	0.30	0.28	18.15	15.58	14.52
4	17.80	0.69	0.08	18.83	16.24	14.84
5	22.57	0.42	0.11	22.19	19.58	19.72

Table 4.5: Model Parameters - Using reduced data set

Parameter	Initial	Unconstrained	Constrained
$E_c(GPa)$	28	15.3	21.5
$k_L(MN/m/m)$	516.6	26,889.8	5.9

Table 4.6: Comparison of frequencies Bridge 58N - Using reduced data set

Frequency (#)	ID (Hz)	σ^2	W	Initial (Hz)	Unconstrained (Hz)	Constrained (Hz)
1	12.15	0.11	0.26	14.04	13.33	12.96
2	13.20	0.10	0.29	17.16	16.08	14.49
3	14.86	0.16	0.22	18.15	16.86	14.60
4	17.21	0.21	0.13	18.83	17.56	14.94
5	22.49	0.30	0.10	22.19	21.08	19.76

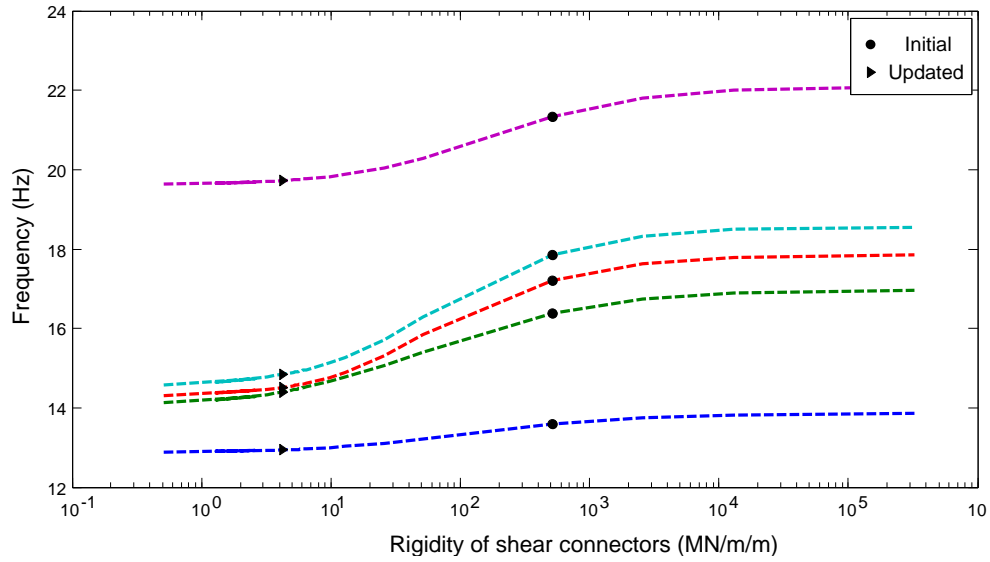


Figure 4.13: Frequencies of deck as a function of the effective stiffness of the shear connectors. Shown are the values of the initial stiffness (solid circle) and the updated value (solid triangle)

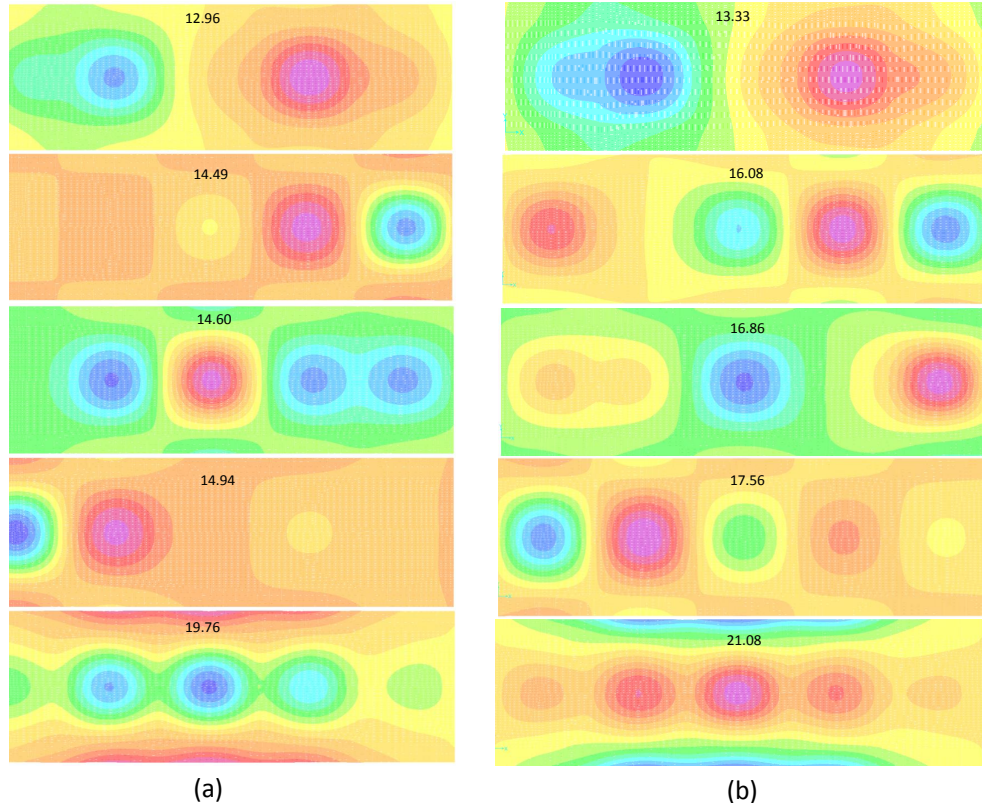


Figure 4.14: Mode shapes (with corresponding frequency) on the concrete slab corresponding to the updated model. On the left is the result of the constrained model updating and on the right the results of the unconstrained.

4.5 Load Distribution

To estimate the load distribution factors among the various stringers we compared two distinct and independent approaches:

- *Inferred from the Updated Finite Element Model:* We used the updated detailed finite element model and directly computed the influence lines from this model. In this approach we can compute the load distribution factors for the stringers and the girders.
- *Direct Measurement:* We compared the relative values of the strains measured in the stringers and this gives an estimate of the relative load distribution factor between the various stringers. In this approach we can not obtain the actual load in the stringer, only their relative values.

The two approaches previously described will be compared and if they provide consistent results, then reliable conclusions can be drawn.

4.5.1 Load Distribution from Updated Finite Element Model

Using the updated finite element model it is possible to compute the load distribution on each stringer and girder by computing the influence coefficient as a function the position of a unit. Fig. 4.15 shows the influence coefficient of every stringer when a unit load is placed at the center of a span and moved in the transverse direction. The figure also shows the sum of the influence coefficients, this indicates the fraction of the load that is directly transmitted to the stringers, and its complement, the load that is transferred to the outer girders directly from the slab.

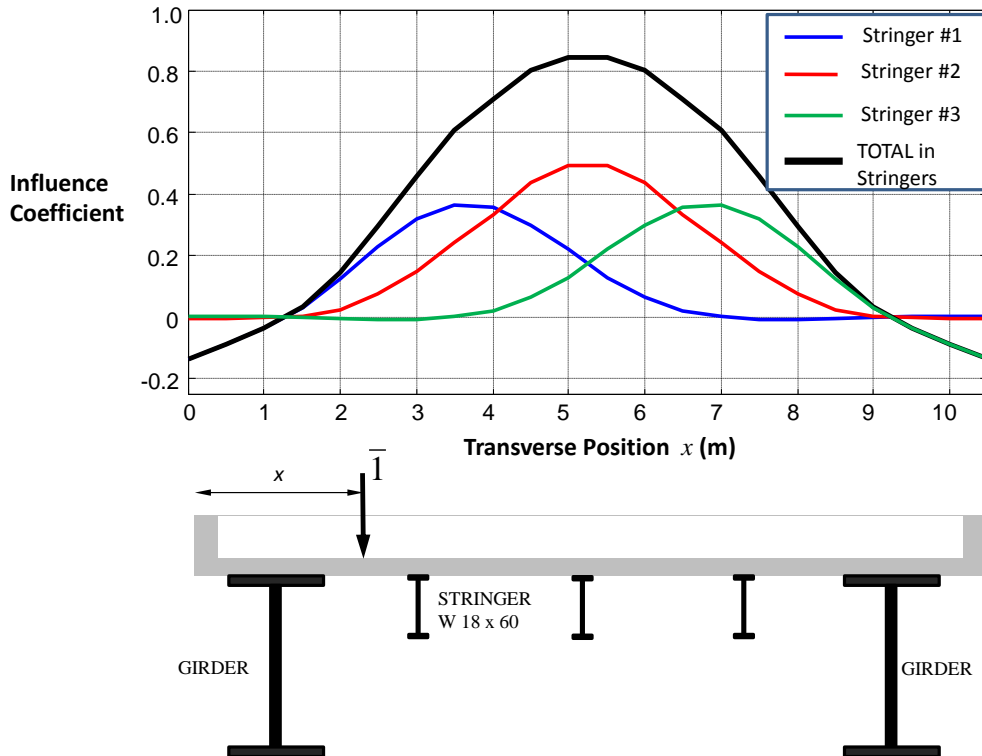


Figure 4.15: Transverse influence line for stringers 1, 2 and 3. These influence lines were computed at the center of a stringer span.

During the field testing of bridge, a loaded 2-axle truck was driven over the bridge at 60 mph. The back axle of this truck was weighted at 32 kips (142.34 kN). This load was applied to the model and the values for the maximum positive bending moments and total tributary loads are presented in Fig. 4.16 and Fig.4.17. Approximately 50% of the load goes to the stringers, with the two stringers on the loaded lane receiving most that 50%. The remaining 50% of the load goes to the girder closest to the load.

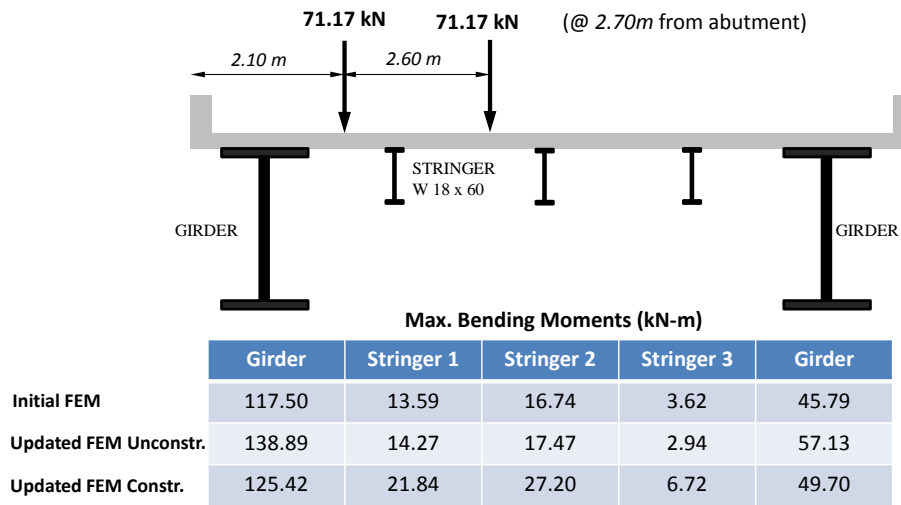


Figure 4.16: Comparison of maximum bending moments in various elements of the bridge deck

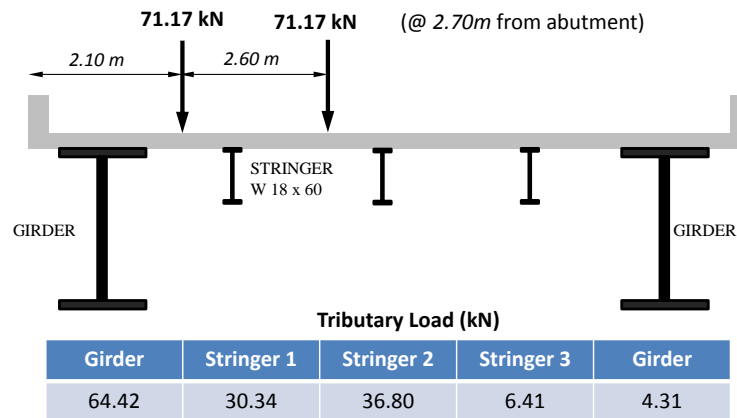


Figure 4.17: Tributary load distributed by the slab on the various elements of the deck

Finally, we also tested the distribution of a uniform load on the travel lane. This load corresponds to the AASHTO lane load [8]. The results are presented in Fig.4.18.

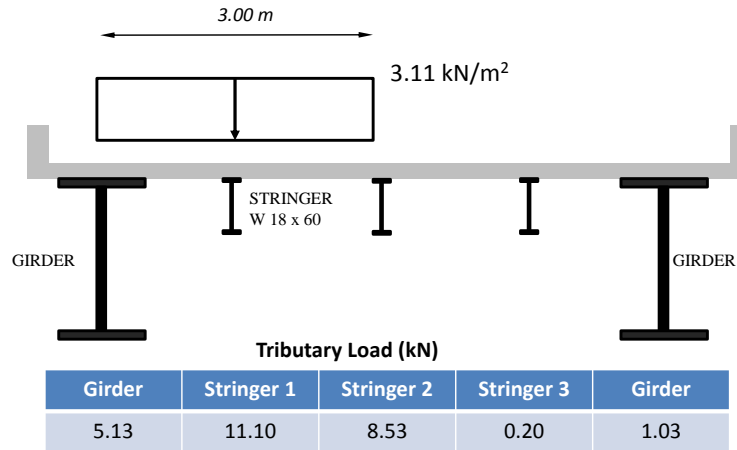


Figure 4.18: Tributary load distributed by the slab on the various elements of the deck due to AASHTO lane load

4.5.2 Direct Relative Measurement

Several field tests were performed in order to measure the relative load distribution factor among the various stringers. Table 4.7 shows the maximum strain measurements (relative to stringer 2) in the bottom flange of the stringers. Since the strain is directly proportional to the bending moment, these can be interpreted as a relative moment distribution factor. These values can be compared to those presented in Fig.4.17. The results indicate that for engineering purposes the updated FEM can be considered accurate (although it tends to slightly overdistribute the load away from the central stringer to the stringer on the lane with the applied load).

Table 4.7: Relative values of maximum measured strain in stringers during various field tests where the travel lane was loaded. Results from updated FEM are also shown.

Test (#)	Stringer 1	Stringer 2	Stringer 3
1	0.71	1.00	0.10
2	0.63	1.00	0.24
3	0.67	1.00	0.14
UFEM	0.82	1.00	0.17

Chapter 5

Conclusions

The main conclusions after instrumenting, processing the measurements and performing finite element model updating on various models of the deck of bridge 58N are as follows:

- Instrumentation was successfully conducted and the main dynamic response characteristics of the instrumented portion of the bridge were reliably identified from the measurements.
- An updated three-dimensional finite element model of the instrumented portion of the bridge deck was calibrated and found to accurately predict the bridge deck's response in the presence of moving loads.
- The stringers and the reinforced concrete deck were found to be acting partially-composite, although much closer to the fully non-composite behavior (see Fig.4.13). *We recommend that for stress analysis purposes, the stringers be considered as non-composite.*
- The vertical loads on the bridge deck distribute among the stringers as shown in Fig.4.15. This was tested and verified using two independent methods.
- This report can not provide conclusions regarding the ultimate strength capacity of the bridge deck or any of its components. The results and models obtained from this report can only be used for analysis in the linear elastic range of strains/stresses.

5.1 Recommendations

We propose the following recommendations:

- Continue to perform vibration monitoring on the bridge. To the best of our knowledge, all sensors are still functioning properly. We are open to collaborate with the Vermont Agency of Transportation on this regard.

- Provide electricity and internet on the bridge site so monitoring can occur in near real-time without the need to have personnel on-site.
- Visually monitor the welds used to attach the sensors to the bridge in case that small fatigue cracks develop with time.

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