

BRIDGE LOAD TESTING AND MONITORING USING CONVENTIONAL AND IMAGE-
BASED MEASUREMENT STRATEGIES

BY

ISAIAS ANDRES COLOMBANI

THESIS

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Adviser:

Professor Bassem Andrawes

ABSTRACT

Bridge load rating analysis has been used for decades to assess the safe load-carrying capacity of bridges in the United States. As time progressed, bridges that were rated using the American Association of State Highway and Transportation Officials (AASHTO) analytical load rating methods began to show lower load capacities than those that the bridges have proven to carry through load testing alternatives. This is especially true for reinforced concrete (RC) slab bridges which have complicated load path distributions and often benefit from load tests. This has resulted in unnecessary roadway restrictions, bridge closures, and other inconveniences to the communities that rely on these bridges for daily commute. Consequently, to provide better accuracy, the AASHTO Manual for Bridge Evaluation (MBE) offers the option of conducting load rating using field testing as an alternative to analytical computing rating factors for bridges.

In this study, diagnostic and proof load field tests are performed on two RC slab bridges to study the feasibility and constraints of using bridge load testing as a load rating method. AASHTO MBE and Effective Strip Width (EW) load rating approaches are used to compare results with AASHTO analytical load rating procedures. The results demonstrate the advantages to load testing over less reliable analytical load rating procedures. To reduce the complexity, time, and cost associated with instrumenting bridges for load testing, the study explores the feasibility of performing load tests using image-based techniques. Images of loaded bridges captured using consumer-grade cameras are used to monitor the displacement of the bridges during testing. The results from the field and laboratory studies show compelling results when compared to conventional Linear Variable Differential Transformer (LVDT) sensors. Furthermore, the studies show that image-based bridge monitoring techniques have great potential in conducting quick and cost-effective field-based load ratings of bridges. The research also investigates alternative

approaches to field tests for structural monitoring of bridges. A satellite image-based technique is explored to evaluate the potential of this technology to measure small displacements in bridges. Moreover, the technique is applied to three different bridges to validate the approach and the simplicity of its application for bridge monitoring. The results show promising potential in the method to be used as a structural health monitoring strategy for bridges.

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CHAPTER 1: INTRODUCTION

1.1. Problem Description

Analytical load rating procedures following the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation (MBE) guidelines have been the prominent means for load rating bridges for structural capacity evaluations (AASHTO, 2018). During this process, each bridge is analyzed using one or several hypothetical legal rating vehicles as defined by the MBE to determine if the bridge must be load posted to a smaller allowable load capacity and to what extent the bridge must be posted (AASHTO, 2018). These load postings are often encountered on a white roadway sign as drivers approach a bridge as shown in Figure 1.1. The signs show the maximum tonnage (weight in tons) that vehicles can have to safely cross the bridge. While these load postings are often overlooked by the general public, several larger vehicles used for cargo, agriculture, and other essentials, suffer many inconveniences. These consequences have led to problems concerning load rating analysis procedures. If the analysis is too conservative, the load postings may unnecessarily restrict the weight of the vehicles that are allowed to use the bridge, which would impact the communities that the bridge serves.



Figure 1.1. Load posting of a bridge in McLean County, IL

In 2021, the Federal Highway Administration (FHWA) reported that 7.5% of the United States' bridges are classified as being structurally deficient, meaning that one or several of their critical members is in poor condition and need of repair (FHWA, 2021). This amounts to over 46,000 bridges which have load ratings below today's design standards, resulting in load restrictions to numerous busy roadways throughout the United States. Today, many of these bridges are still being evaluated using visual inspection methods and analytical rating procedures following the AASHTO MBE guidelines (AASHTO, 2018). These methods have been shown to conservatively underestimate the true capacity for bridges through field testing approaches (Saraf, 1998; Jáuregui et al., 2010; Davids et al., 2013). Consequently, there has been a shift towards load rating using field testing methods for improving the credibility in the bridge's capacity to carry the specified design loads.

Among the commonly used bridge types that have been serving rural and agricultural regions in the U.S. are reinforced concrete (RC) slab bridges. According to the FHWA's 2016

National Bridge Inventory data, RC slab bridges account for 10.3% of the U.S. bridge inventory classified to be structurally deficient (FHWA, 2016). Many of these bridges are older RC slab bridges located in rural areas and are often overlooked because of their remote location. Because older bridges have been designed for lesser demands than the currently adopted HL-93 design truck used in the AASHTO specifications (AASHTO, 2020), these bridges are often deemed to be structurally inadequate to carry the design loads imposed today and are posted to smaller load capacities than these bridges have proven to carry through load testing methods (Jorgenson and Larson, 1976; Azizinamini et al., 1994; Miller et al., 1994). Although RC slab bridges are seemingly simple, these bridges have complicated load path distributions which often result in increased load rating factors (RFs) from those obtained using analytical rating procedures (Commander, 2019). Extensive research has been performed on these bridges to better understand the influence of edge beams, aspect ratio, and skew angle, which were identified to be prominent factors in obtaining more accurate and favorable load ratings for these bridges (Arockiasamy and Amer, 1997, Menassa et al., 2007, Freeman and Vasconcelos, 2018). Because each RC slab bridge is unique, often having a combination of distinctive skew angle, aspect ratio, and railing conditions, there are still several uncertainties that make these bridges great candidates for load rating using field testing.

Despite the several advantages of using field testing as a bridge load rating method, field testing can be expensive, time consuming, and challenging, making field testing impractical for bridge owners to employ. However, the costs associated with load testing can be significantly reduced if the conventional sensing system often used for these tests are replaced with optical procedures for measuring deformations. With the advent of computer vision, numerous image-based deformation measurement techniques have been developed which facilitate the field-testing

procedure for bridges. The most advantageous is the Feature-Based Image Registration (FBIR) technique developed by Lucas and Kanade (1981) because it utilizes automated feature extraction and matching techniques which have been shown to give comparable results to conventional sensors. Furthermore, FBIR has the potential to enhance the load testing strategy to be implemented by bridge owners at much lower costs and with minimal complication and field setup.

Still, field testing procedures for load rating often require roadway closures and proper coordination among stakeholders, making them difficult to employ on busy bridges. More advanced methods have been explored for structural monitoring of bridges at a larger scale with fewer potential costs and field work. These methods are also image-based but utilize radar signals from satellites to measure displacements of surface deformations with high spatial detail (Galloway et al., 2000). The capabilities of these methods to capture small-scale displacements have improved significantly in recent years leading to its successful application on structural health monitoring of bridges (Milillo et al., 2019). When combined with structural engineering expertise, these satellite-image techniques can be used to evaluate the condition of a bridge under service load conditions, which gives this near-real time structural health monitoring technique an advantage over traditional load rating and load testing methods.

1.2. Scope of Research

This research is motivated and supported by the Illinois Department of Transportation (IDOT) in pursuit of improving load rating procedures throughout Illinois using load testing and to study the feasibility of using bridge load testing as a load rating method. To familiarize the reader with the significance of load testing as a viable alternative to analytical load rating procedures of the AASHTO MBE, diagnostic and proof load field tests were performed on two distinct reinforced concrete (RC) slab bridges which were posted to lesser load capacities using

analytical load rating methods. The analytical load rating results were compared to the load ratings attained using load testing alternatives to demonstrate the advantages of load testing.

To present the potential for a more cost-effective field testing strategy, an image-based displacement measurement method was used during the load tests on the RC slab bridges to compare with the less convenient, more costly, linear variable differential transformer (LVDT) sensors which are traditionally used for these load tests. The image-based displacement strategy was validated during the field tests of both bridges. Moreover, the technique was explored further in the laboratory to evaluate the effects of different camera configurations and intrinsic algorithm parameters on the accuracy of the results.

A novel displacement measurement technique based on satellite imagery was also introduced to show the potential of a more automated and advanced method for structural health monitoring of bridges. The technology can be used to identify damage in a bridge by monitoring millimeter-scale displacements over time and can potentially offer a near-real time continuous structural health monitoring approach for bridges all over the world. To help validate this technology, analyses was carried out on three different bridges in the United States. The displacement behavior of each bridge was tracked using satellite images to identify the damage or correlate the displacements with temperature change throughout the acquisition period.

1.3. Thesis Outline

The scope of each chapter in this thesis is summarized as follows: Chapter 1 introduces the reader to the purpose of this research and the primary research scope. Chapter 2 provides a background on load rating methods, discusses the topic of load testing as a viable alternative to analytical load rating procedures of the AASHTO MBE, and presents the load rating results of two different bridge load tests. Chapter 3 describes an alternative image-based displacement

measurement method in place of using more costly conventional LVDT sensors for field tests. Chapter 4 introduces a novel displacement measurement technique using satellite imagery that can be applied to monitor the structural integrity of bridges in near-real time. Chapter 5 summarizes the key findings of this research.

CHAPTER 2: LOAD RATING THROUGH FIELD TESTING

2.1. Literature Review

The MBE provides guidelines for performing load rating analysis using Allowable Stress Rating (ASR), Load Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR) methods (AASHTO, 2018). The simplest and most common approach for conducting load rating analysis implements AASHTO's beam-line analysis approach (AASHTO, 2020). However, research has proven this method to significantly underestimate the true capacity for bridges (Saraf, 1998; Jáuregui et al., 2010; Davids et al., 2013). More comprehensive analytical methods can be employed to solve for the load effects, such as Finite Element Analysis (FEA), which demonstrated improved accuracy when compared to the beam-line analysis (Armendariz and Bowman, 2018; Khoury et al., 2019). However, FEA is quite complex and in many cases still suffers from numerous uncertainties that lead to making major modeling assumptions. For example, for a bridge that has been in service for several decades sustaining excessive deteriorations, it is quite complicated to determine with high certainty the mechanical properties of the materials, the interfacial bond between concrete and reinforcement, the existence of composite action between the superstructure girders and concrete deck, the level of rotational restraints provided by the supports, and the transverse load distribution (Beal, 1998). For this reason, AASHTO offered field load testing procedures as a supplement for calculating RFs for bridges (AASHTO, 2018).

There are two types of field tests that have been incorporated into the AASHTO MBE for load rating. The first is the diagnostic load testing approach which is applied to detect sources of distress in the bridge and identify the load distribution (Bakht & Csagoly, 1980). This test has demonstrated its ability to improve load ratings for several bridges of different physical characteristics. Arroyo et al., (2019) developed an extensive report discussing the diagnostic

testing procedure followed using the AASHTO MBE guidelines. Results in their diagnostic field test of a four-span RC slab bridge show that the bridge met the rating criteria for all the AASHTO design loads (Arroyo et al., 2019). Test findings in Hag-Elsafi and Kunin (2006) using this approach revealed that the simple span multi-girder test bridge provided more fixity at the supports than the original design intended, which resulted in a 100% increase in load rating from the diagnostic test analysis. Peiris and Harik (2016) compare diagnostic test results on a steel beam bridge with analytical load ratings. Results show as much as a 400% improvement in load rating from the analytical results. Freeman and Vasconcelos (2019) conducted diagnostic load tests on five RC slab bridges, each revealing greater load capacities compared to analytical load rating procedures of the AASHTO Standard Specifications.

Proof load testing is the second field testing approach discussed in the AASHTO MBE for load rating of bridges. These tests are conducted to prove the minimum capacity for bridges by loading the bridge to magnified design loads from those it is expected to carry (Beal, 1998). Proof load testing is also used to better identify critical sections and verify the distribution of the load in bridges (Nowak & Saraf, 1996). Arroyo et al., (2019) also discussed the details of the proof load test procedure using the AASHTO MBE in their comprehensive report on load testing. The report includes a proof load field test example on a four-span RC slab bridge which was found to have enough capacity to safely carry the design live loads. Research conducted by the University of Nebraska-Lincoln shows that proof load tests on a steel girder bridge could increase the RF by 2.3 times the analytically computed load rating for the bridge (Szerszen, 2015). Zhou and Guzda (2020) further demonstrate that a prestressed concrete girder bridge under proof load could carry the lower bound load rating, meaning no need for a load posting. These studies, among several others, demonstrate the advantages of proof load testing for load rating bridges.

2.2. Load Rating Procedures for RC Slab Bridges

2.2.1. AASHTO LFR Analytical Rating Procedure

A rating factor (RF) can be calculated using one-dimensional (1D) beam-line analysis and applying the following Load Factor Rating (LFR) equation provided by the AASHTO MBE (AASHTO, 2018)

$$RF = \frac{C - A_1 D}{A_2 L (1 + I)} \quad (\text{Eq. 2-1})$$

where C is the nominal capacity of the bridge, L and D are the maximum live and dead load effects on the bridge, respectively, I is the impact factor calculated as a function of the span length with a maximum value of 0.3 (AASHTO, 2002), and A_1 and A_2 are the loading factors for operating and inventory level conditions as per the MBE (AASHTO, 2018). Because the MBE refers back to the AASHTO Standard Specifications (2002), these specifications are used for the analytical load rating. The dead load effects are determined by distributing the weight of the curbs and/or railings equally along the width of the slab. The live load effects are obtained by maximizing the effects of the HS20 design truck wheel loads on a unit-wide strip of the slab. For simple span bridges with reinforcement running parallel to the span, AASHTO provides approximate approaches for calculating the live load effects on the bridge. These include Appendix A in the AASHTO Standard Specifications and the following equation for spans up to and including 50 ft

$$M_{LL} = 900S \leq 45,000 \quad (\text{Eq. 2-2})$$

where M_{LL} is the live load moment in foot-pounds for HS20 loading and S is the effective span length in feet (AASHTO, 2002). The following distribution width equation was used in this thesis to determine the effects of the design truck wheel load per unit width

$$E = 4 + 0.06S \leq 7 \quad (\text{Eq. 2-3})$$

where E is the distribution width for a single wheel in feet. The total effective width was calculated by centering the HS20 design truck(s) on a 10 ft wide design lane and considering the distribution width for two wheels. The effective width for a single lane load (SLL) is calculated by multiplying the wheel load distribution calculated using Eq. 2-3 by 2 as specified in AASHTO (2002). The wheel spacing used in calculating the effective width for a multilane load (MLL) was 4 ft representing the spacing of two HS20 design trucks side-by-side as shown in Figure 2.1.

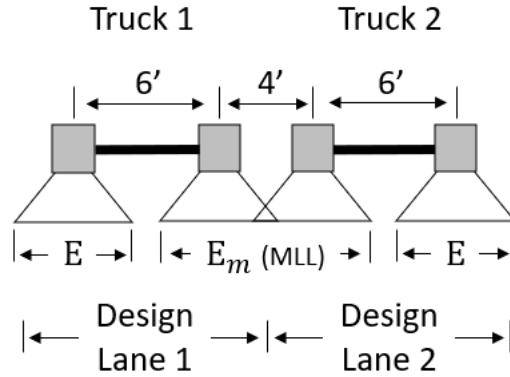


Figure 2.1. Wheel spacing used for effective width calculation of bridge under MLL (E_m)

2.2.2. AASHTO MBE Diagnostic Testing Procedure

Diagnostic load testing can be used to determine a modified load rating using the AASHTO MBE approach. The procedure requires that a modified rating factor (RF_T) be calculated using the following equation.

$$RF_T = RF_C \times K \quad (\text{Eq. 2-4})$$

where RF_C is the analytical RF and K is an adjustment factor determined from the diagnostic test results in accordance with Eq. 2-5 shown below.

$$K = 1 + K_a K_b \quad (\text{Eq. 2-5})$$

In Eq. 2-5, K_a accounts for the benefit or loss derived from the load test and consideration of the section factor resisting the applied load, and K_b accounts for the understanding of the load test results when compared with those predicted by theory (AASHTO, 2018). The MBE provides the following general expression to use for calculating K_a

$$K_a = \frac{\epsilon_c}{\epsilon_T} - 1 \quad (\text{Eq. 2-6})$$

where ϵ_T is the maximum member strain measured during the load test, and ϵ_c is the corresponding calculated strain due to the test vehicle, at its position on the bridge which produced ϵ_T (AASHTO, 2018). Moreover, the MBE also provides guidance on selecting the appropriate value for K_b , which utilizes the ratio T (the unfactored vehicle effect) by W (the unfactored gross rating load effect) to evaluate whether a linear structural response can be extended to amplified load levels. This evaluation is performed to ensure that the structure has enough capacity beyond its rating load level W .

2.2.3. AASHTO MBE Proof Load Testing Procedure

The first step for determining the modified load rating using this approach is to determine the target proof load to achieve an RF of 1.0. This is attained by utilizing a magnification factor (X_p) for establishing a margin of safety due to the current bridge conditions and uncertainties in the test. The MBE recommends a value of 1.4 be used as the modification factor with the possibility of adjusting this factor based on the structure type, redundancy, condition, average daily truck traffic (ADTT), and controlling load case (AASHTO, 2018). The new adjusted magnification factor (X_{pA}) is determined as follows

$$X_{pA} = X_p \left(1 + \frac{\sum \%}{100}\right) \quad (\text{Eq. 2-7})$$

where $\sum \%$ is the summation of the percentage increase and/or reduction recommended by the MBE for the above parameters. The target proof load (L_T) can then be determined from the following equation

$$L_T = X_{pA} L_R (1 + I) \quad (\text{Eq. 2-8})$$

where L_R is the comparable unfactored live load due to the rating vehicle for the lanes loaded.

After the target load is calculated, the details of the loading steps and procedure must be determined to safely load the bridge. The loads are applied in stages where each stage is a fraction of L_T applied incrementally and may be loaded and unloaded multiple times to confirm repeatability (AASHTO, 2018). The applied loads and bridge response are carefully monitored during the test to check for any signs of distress in the structure, to which the load testing should be terminated. Once the proof test is completed or terminated, the maximum applied live load (L_p) on the bridge is known and the operating level load rating (RF_o) is determined as follows

$$RF_o = \frac{\frac{k_o L_p}{X_{pA}}}{L_R (1 + I)} \quad (\text{Eq. 2-9})$$

where k_o is 1.0 when the target proof load is achieved and 0.88 if the test was terminated because of observed distress in the bridge prior to reaching L_T .

2.2.4. Structural Effective Strip Width Procedure

The structural effective strip width (EW) approach was another method employed in this study to compute a modified load rating. The basis of this method is to replace the AASHTO LFR EW from the analytical load rating analysis with a new measured EW from the test data to solve for a modified RF using Eq. 2-1. The purpose of including this method is to validate whether this

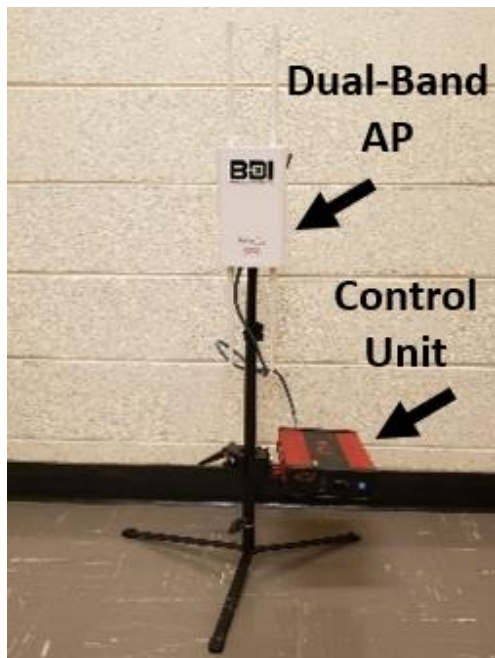
approach yields more favorable load ratings than the more conservative AASHTO analytical approach as verified by several studies (Saraf, 1998; Amer et al., 1999; Jáuregui et al., 2010; Shenton III and Jones, 2012). Also, the EW approach is compared with the AASHTO MBE load rating through load testing approaches discussed previously.

In this analysis, the critical moment distribution of the RC slab bridge is used to determine an EW. The edge beams are traditionally considered nonstructural members and were not included in the AASHTO EW calculations (Freedman and Vasconcelos, 2019). Therefore, they have been excluded from this simplified analysis despite demonstrating their favorable influence on the RFs for RC slab bridges in previous studies (Azizinamini et al., 1994; Mabsout et al., 2004). The moments are calculated from the measured strains from the load tests utilizing the elastic stress-strain relation and applying Euler–Bernoulli theory. An EW is calculated by dividing the area under the curve by the maximum moment observed for the controlling load case. This EW is then used to determine a modified load rating for the bridge.

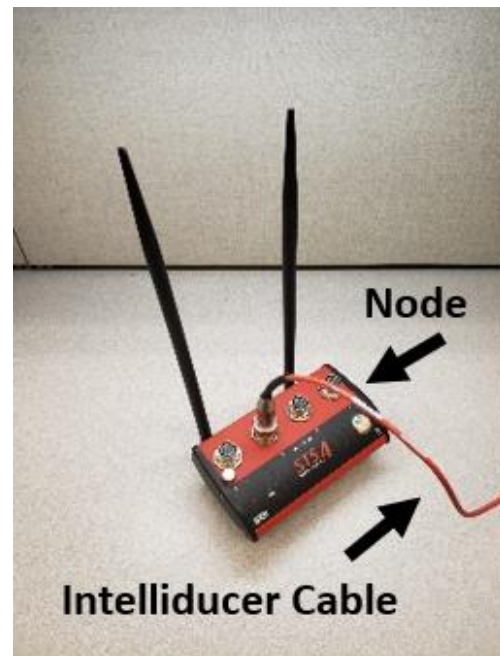
2.3. Field Testing Instrumentation

The instrumentation used to conduct the field tests was the Bridge Diagnostics Inc. (BDI) 4th generation Structural Testing System (STS4) shown in Figure 2.2. The system functions with various components that communicate via a wireless link to transmit the data to the computer. The system is built for rugged outdoor use to minimize environmental effects and prevent damage to the equipment during field tests. The main components of the system are the Wireless Base Station (WBS) used to provide the wireless interface to the data acquisition system (DAQ) and the computer, the STS4 nodes which transmit the data to the computer and provide power to the connecting sensors, the sensors used for measuring the desired load effects of the bridge, and the computer which is used to store and visualize the field data. Each component is essential to the

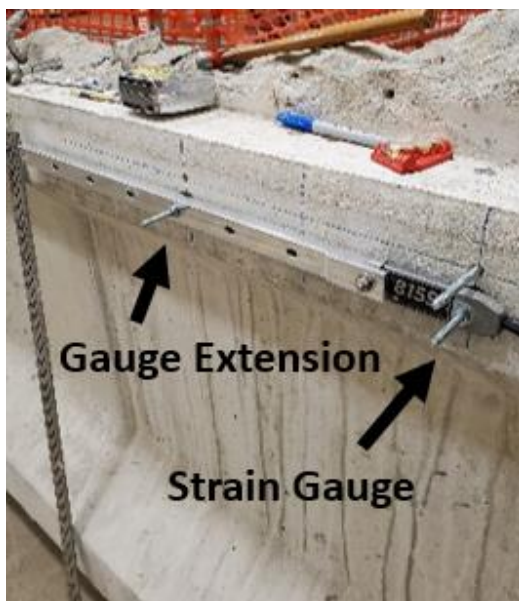
operation of the STS4 system and will be discussed in more detail regarding its installation, operation, and limitations.



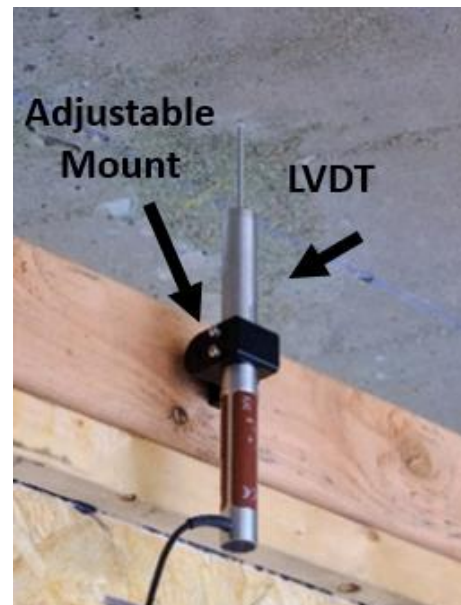
(a)



(b)



(c)



(d)

Figure 2.2. Images showing BDI STS4 instrumentation (a) WBS (b) STS4 node (c) ST350 strain gauge (d) LVDT

As aforementioned, the WBS provides the wireless Access Point (AP) for the system. The WBS has a control unit which supplies the dual-band wireless AP with the power needed to function remotely. The control unit has a battery life of approximately 16 hours and is rechargeable. The control unit also functions on an external (wired) 24 Vdc power supply in case of long-term monitoring applications. The WBS has ethernet ports for when a wired interface is preferred. In the most ideal conditions, the WBS has a range of 4 miles. However, this range is reliant on the line-of-site and is reduced with physical obstructions between the components. The WBS is mounted on a tripod where the dual-band AP and control unit can be installed for convenient positioning on the field as shown in Figure 2.2.a.

The STS4 nodes can accommodate up to four sensors each. A maximum of 128 nodes can be connected using a single WBS for the test. These nodes are equipped with a rechargeable internal battery that can supply power to the sensors for over 40 hours when used at the maximum sampling rate of 1000 samples per second (S/s). The STS4 nodes have an internal memory of 8 Gigabytes (GB) for backup data storage. They also have temperature sensor inputs for each channel. The node can operate via a wireless connection to the WBS or ethernet cable. The nodes are lightweight and compact for easy transport during field tests. An image of the STS4 node is shown in Figure 2.2.b.

Various sensors can be used for bridge monitoring and field-testing applications. However, the most common are linear variable differential transformers (LVDTs) and strain gauges (ST350s) for measuring displacements and strains, respectively. BDI supplies both types of sensors which are compatible with the STS4 nodes. The LVDTs purchased for the field tests have a range of ± 1.0 in. Their total length is 10.8 inches and they have a supply voltage of 6 to 18 Vdc.

They were purchased with adjustable aluminum mounting brackets that can be installed on a tripod or other fixture for field tests as shown in Figure 2.2.c.

The ST350s are encased in a sealed aluminum shell making them ideal for outdoor use. They have an effective gauge length of 3 inches with the option to add an extension for RC installations up to 24 inches. The ST350s can be installed with the typical adhesive mounting technique by applying the adhesive to the reusable mounting tabs that came with the ST350s. They can also be installed using anchors because they have ¼ inch mounting holes at both ends for easy installation on RC members as shown in Figure 2.2.c. To facilitate the installation of the extensions and ensure that the extensions are properly attached, a jig was purchased to align the extensions to the ST350. This is done prior to installing the ST350s to the concrete. This makes the installation procedure simpler and quicker.

The recommended gauge extension length used for the test is a function of the member type, member depth and span length. Table 2.1 provides guidelines for choosing the appropriate gauge length based on these parameters.

Table 2.1. Ext. Multiplier for recommended gauge extension length

RECOMMENDED LOWER AND UPPER GAGE LIMITS		
STRUCTURE TYPE	LOWER LIMIT	UPPER LIMIT
SLABS & RECTANGULAR BEAMS	1.0 X DEPTH OF MEMBER	LENGTH OF SPAN / 20
T-BEAMS	1.5 X DEPTH OF MEMBER	LENGTH OF SPAN / 20

After obtaining the appropriate gauge length, the measurements must be reduced to account for the extension effects on the readings. The new reading can be determined from the following equation

$$Reading = \frac{Reading}{Ext. Multiplier} \times Amplification Factor \quad (Eq. 2-10)$$

where the amplification factor is taken as 1.1 to account for the extension effect and the Ext. Multiplier is given in Table 2.2.

Table 2.2. Corresponding Ext. Multiplier and Amplification Factor for desired gauge length

GAGE EXTENSION LENGTH	EXT. MULTIPLIER	AMPLIFICATION FACTOR
6 in	2	1.1
9 in	3	1.1
12 in	4	1.1
15 in	5	1.1
18 in	6	1.1
21 in	7	1.1
24 in	8	1.1

To minimize the force on the ST350s, BDI recommends keeping the maximum strain below $\pm 1,000 \mu\epsilon$. There are also maximum strain ranges which depend on the extension and properties of the concrete. These maximum values are provided to ensure accuracy for the given gauge length. Greater strain values can be measured. However, BDI suggests that special attention be given to the gain settings on the DAQ being used when exceeding these ranges. These maximum strain ranges are provided in Table 2.3.

Table 2.3. Maximum strain gauge range for concrete and gauge length

EXT. MULT.	ACTUAL GAGE LENGTH WITH EXTENSION	MAXIMUM STRAIN RANGE	APPROX. CONC. STRESS FOR F'C = 3,000 PSI (20.7 MPa)	APPROX. CONC. STRESS FOR F'C = 4,000 PSI (27.6 MPa)	APPROX. CONC. STRESS FOR F'C = 5,000 PSI (34.5 MPa)	APPROX. STEEL RE-BAR STRESS
1	3 in (76.2 mm)	$\pm 1000 \mu\epsilon$	3.1 ksi (21.4 MPa)	3.6 ksi (24.8 MPa)	4.0 ksi (27.6 MPa)	30 ksi (207 MPa)
2	6 in (152.4 mm)	$\pm 500 \mu\epsilon$	1.6 ksi (11.0 MPa)	1.8 ksi (12.4 MPa)	2.0 ksi (13.8 MPa)	15 ksi (103 MPa)
3	9 in (228.6 mm)	$\pm 330 \mu\epsilon$	xx ksi (6.9 MPa)	1.2 ksi (8.3 MPa)	1.3 ksi (9.0 MPa)	9.9 ksi (68.3 MPa)
4	12 in (304.8 mm)	$\pm 250 \mu\epsilon$	780 psi (5.3 MPa)	900 psi (6.2 MPa)	xx ksi (6.9 MPa)	7.5 ksi (51.7 MPa)
5	15 in (381.0 mm)	$\pm 200 \mu\epsilon$	625 psi (4.3 MPa)	720 psi (5.0 MPa)	800 psi (5.5 MPa)	6.0 ksi (41.4 MPa)
6	18 in (457.2 mm)	$\pm 160 \mu\epsilon$	500 psi (3.4 MPa)	575 psi (4.0 MPa)	650 psi (4.5 MPa)	4.8 ksi (33.1 MPa)
7	21 in (533.4 mm)	$\pm 140 \mu\epsilon$	440 psi (3.0 MPa)	500 psi (3.4 MPa)	560 psi (3.9 MPa)	4.2 ksi (29.0 MPa)
8	24 in (609.6 mm)	$\pm 125 \mu\epsilon$	390 psi (2.7 MPa)	450 psi (3.1 MPa)	500 psi (3.4 MPa)	3.8 ksi (26.2 MPa)

The STS4 system is not complete without a laptop computer for data visualization and processing. For this, BDI gives some recommendations for the computer requirements. The computer used for the testing was equipped with an Intel Core i7 processor and 16GB RAM. The computer has 512 GB SSD internal storage capacity. It also has up to 13 hours of battery run time. Windows 10 Pro was installed and recommended by BDI for using their software applications. The computer will run BDI's STS-LIVE software that is used for visualizing the sensor readings during the test. The computer was also equipped with the BDI's STS-VIEW software for post-processing the results.

2.4. Piatt County Bridge Test

2.4.1. Bridge Description

The first bridge to be considered for this study was a two-lane 23 ft single span RC slab bridge constructed in 1925 in Piatt County, IL. Figure 2.3.a shows an image of the bridge and Figure 2.3.b illustrates its dimensions in plan view with a cross sectional view of the slab and RC barriers. The bridge was chosen because of its proximity to the university, availability of the design drawings, old age, and its low RF. The bridge slab varies in depth from 12.5 in at the face of the curb to 15.5 in at the crown with 19 in wide curbs and integral RC railings. The slab has a total width of 32.2 ft perpendicular to the direction of traffic and has a 24-degree skew angle. The design drawings indicate that the slab was made of standard IDOT Class A concrete specified prior to 1930 which has a compressive strength equal to 3,000 psi. The primary longitudinal reinforcing bars were 1.13 in diameter. A coring sample test was conducted in 2007 to test the compressive strength of the concrete slab. Test results indicated an average compressive strength of 8077 psi.

During a field inspection, spalling of the concrete slab was noticed along the southeast abutment in several locations, fully exposing the reinforcement. Figure 2.3.b shows the spalling of

the bottom slab as indicated by the hatched areas in the plan view schematic of the bridge. More spalling was evident along the southwest portion of the slab and spanning the south lane in the longitudinal direction. Figure 2.3.c shows photographs of the damage corresponding to the locations (1), (2) and (3) shown in Figure 2.3.b.

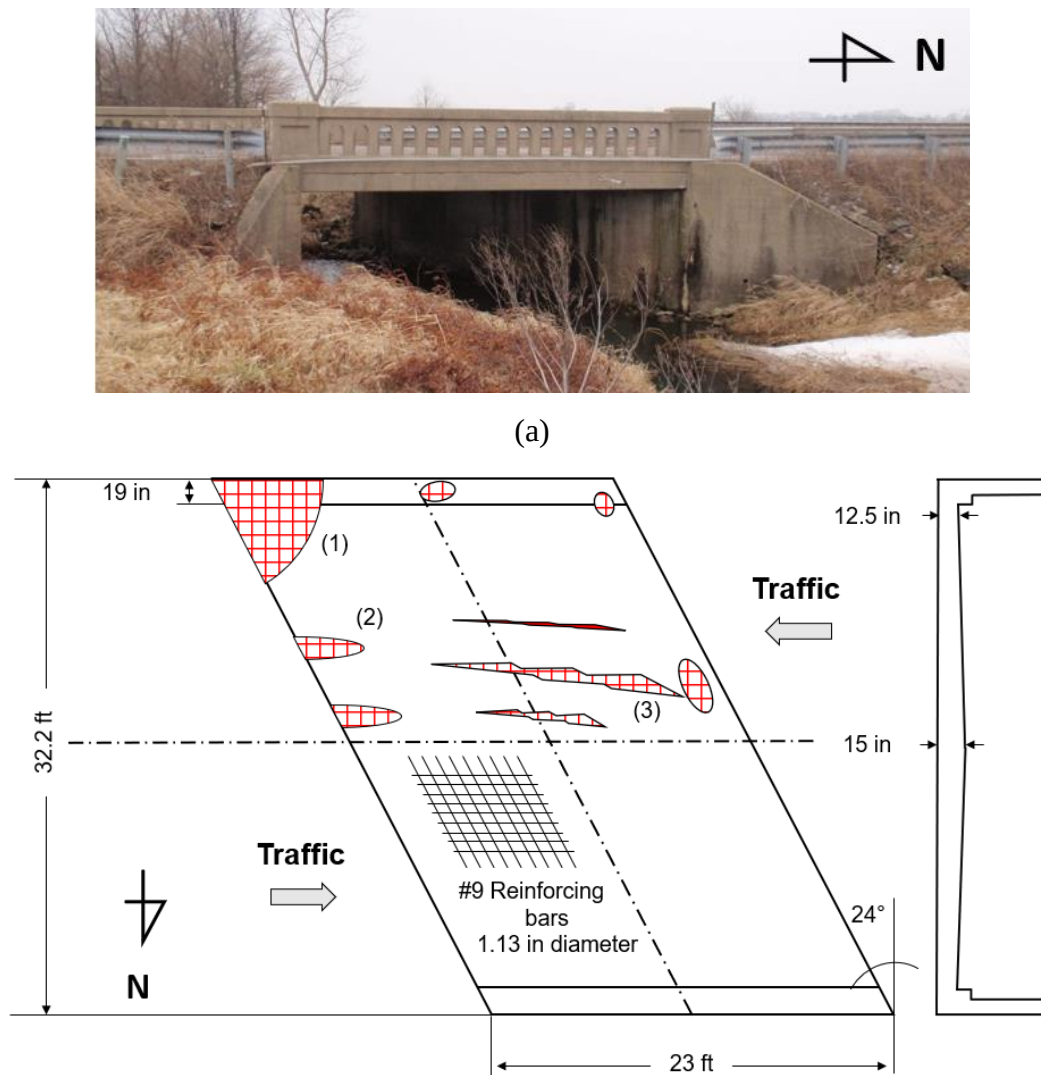


Figure 2.3. Tested bridge in Piatt County, IL (a) bridge photograph, (b) plan view illustrating localized damage and section view showing cross section, and (c) photographs of the damage

Figure 2.3 cont.



(c)

2.4.2. Analytical Load Rating Results

The live load effect which controlled the load rating analysis was the positive moment at the midspan. The bridge was evaluated under SLL and MLL to determine which would produce the greatest load effects. The slab depth used in the analysis was 12.5 in. The area of steel reinforcement was reduced by 10% when calculating the section capacity to account for the localized delamination of the reinforcement observed during the inspection (Figure 2.3.c). The 10% reduction was assumed based on the IDOT Structural Service Manual procedures and for direct comparison with the IDOT load rating procedures (IDOT, 2017).

The analytical RFs calculated using the concrete properties specified in the design plans and tested core samples are summarized in Table 2.4. The MLL condition governed the load rating for the bridge. The analytical load rating improved by approximately 11% when applying the tested concrete properties in the analysis. These properties are used for subsequent calculations.

Table 2.4. Analytical load rating for Piatt County bridge using design and tested concrete properties for single lane loaded (SLL) and multilane loaded (MLL)

Lane Loading	AASHTO LFR using Design Properties	AASHTO LFR using Tested Properties
SLL	0.588	0.655
MLL	0.519	0.578

2.4.3. Diagnostic Test Sensor Plan and Loading Plan

The STS4 instrumentation was chosen for this project because of its robustness, simple setup, and wireless sensor network. Nine strain gauges, labeled S1 through S9 in Figure 2.4, were distributed incrementally along the critical section of the bridge and oriented to capture the longitudinal strain profile from the test truck loads. The strain gauges were mounted using 12-inch gauge extensions and concrete studs which were embedded into the slab.

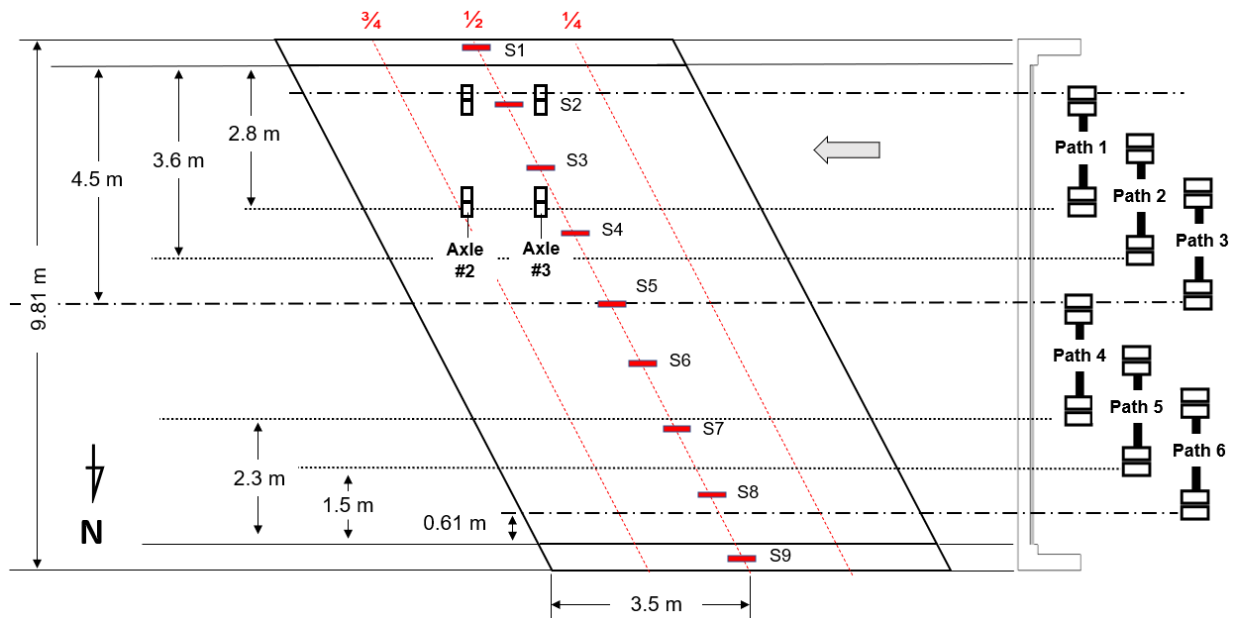


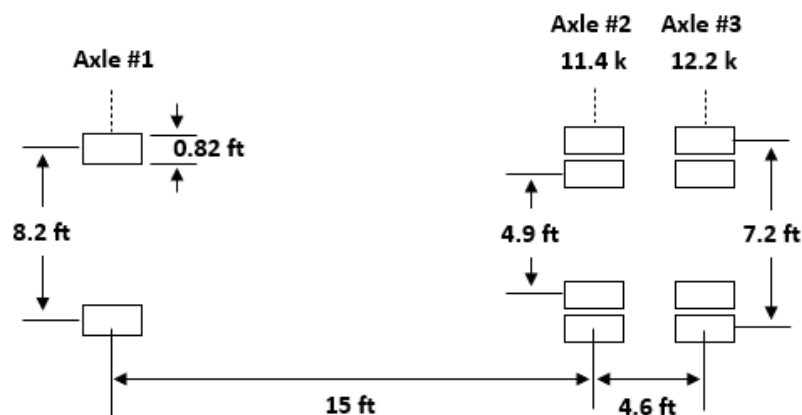
Figure 2.4. Strain gauge and loading plans for the diagnostic test

A single truck was used for the diagnostic test. The reason for using a single truck was two-fold, the first was to avoid long detours to the immoderately trafficked route, and the second was due to the sufficiency of a single truckload to provide a reasonable assessment of the condition of

the bridge in preparation for the proof load test. The truck selected for this test was a conventional IDOT dump truck with an axle configuration shown in Figure 2.5. The truck was loaded such that its effect on the bridge was approximately 60% of the maximum HS20 bending moment. This load is sufficiently high to prevent any damage to the bridge and produce responses along the transverse width that can be accurately measured by the sensors. The test truck axle loads were measured using movable scales just prior to loading the bridge. Because of the short span of the bridge, only the rear tandem axle fits on the bridge span to produce the desired load effect. Hence, only the rear tandem axle loads were relevant for the analysis and are shown in Figure 2.5.b.



(a)



(b)

Figure 2.5. Test truck used for the diagnostic test (a) truck photograph, and (b) plan view dimensions of wheel and axle configuration showing measured axle loads

As shown earlier in Figure 2.4, the loading plan comprised six truck paths in the traverse direction of the bridge. The bridge was loaded in a quasi-static manner where the truck crosses the bridge at a crawling speed of about 5 mph. This allows numerous loading conditions to be recorded along the bridge for each truck pass, covering most of the bridge loading conditions in a relatively short amount of time. Each path was repeated three times to ensure the reliability of the measured responses. The tandem axle position shown in the plan view schematic of Figure 2.4 was the position producing the maximum load effects recorded during the field test. The dotted transverse skew lines represent the quarter points of the span, and the arrow represents the direction the truck traveled for all load paths.

2.4.4. Diagnostic Test Load Rating Results

The maximum measured member strain (ϵ_T) occurred along Path 1 when the north side of Axle #2 of the truck coincided with the three-quarter section of the bridge as shown in Figure 2.4. Since the strain gauges producing the maximum recorded values were located along the south edge of the slab, the edge strip section presented in Figure 2.6 was used to approximate the appropriate section factor resisting the applied load (AASHTO, 2018). The section factor includes the portion of slab, curb and integral RC railing contributing to the stiffness of the element. This assumption was made based on previous research which has shown the edge beam to contribute significantly to the moment distribution along the edges of RC slab bridges (Azizinamini et al., 1994; Amer et al., 1999; Mabsout et al., 2004). The width of the edge strip was determined from Eq. 2-3, representing the distribution width of a single line of wheels for the rating vehicle. The response at S1 was assumed to be resisted by the edge beam, which consisted of the integral curb and RC railing (AASHTO, 2002). Therefore, only the peak strain value from S2 beneath the slab (see

Figure 2.6), i.e. the response recorded by the first interior strain gauge, was used for the calculation of K_a . Applying Eq. 2-6, a K_a value equal to -0.310 was obtained, implying a reduction in load capacity from the analytical results.

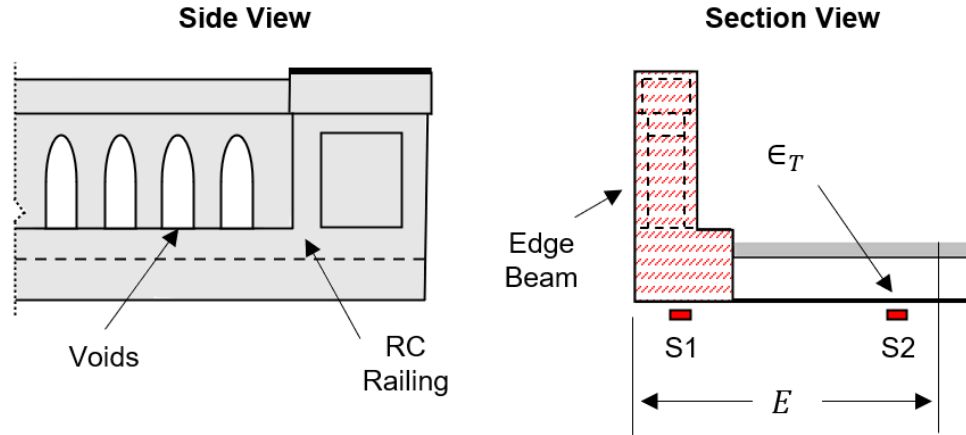


Figure 2.6. Side view railing detail and cross section considered for calculating K_a

The K_b factor was obtained using the cracking strain of the concrete as a threshold to determine if the linear behavior of the bridge could be extended to a load level of $1.33W$ as defined in the MBE. A strain limit of $132\mu\epsilon$ was calculated using the tensile strength of the concrete. The theoretical strain calculated for the applied truck load was greater than the strain limit, yielding a K_b factor equal to 0. This indicated that there would be no enhancement to the load rating of the bridge from the field test. Plugging the factors into Eq. 2-4 and Eq. 2-5 returns a modified RF of 0.655 showing no improvement from the analytical RF shown in Table 1.

2.4.4.1. Load Rating using Calibrated Model

The 1D beam-line model used for the analytical load rating was calibrated based on the diagnostic test results to obtain a modified RF for the MLL case. To calibrate the model, the material properties of the section shown in Figure 2.6 were adjusted to match the results of the

maximum measured strain (ϵ_T) during the field test. The following expression was used to calculate the modified modulus of elasticity E of the concrete

$$E = \frac{M}{S \epsilon_T} \quad (\text{Eq. 2-11})$$

where M is the theoretical moment determined from the 1D beam-line model for the SLL case, S is the section modulus, and ϵ_T is equal to $10.5\mu\epsilon$ located at S2 from the diagnostic test results. Eq. 2-11 yielded a modified modulus of elasticity equal to 2760 ksi, a 46% reduction in the material properties from the coring sample report and 12.5% reduction from the design plans. These properties were then used to calibrate the model. The calibrated model was analyzed under MLL to determine a modified RF for the bridge. The resulting theoretical RF was calculated as 0.488, which is a 15% reduction from the calculated MLL case analytical RF shown in Table 2.4.

2.4.4.2. Load Rating using Structural Effective Strip Width Approach

The calculated moment distribution from the diagnostic test was used to determine a modified EW for the bridge. The critical moment distribution is illustrated in Figure 2.7 showing the cross-sectional location of the truck axles relative to the edge of the curb. The moments between the discrete sensor values were assumed to be linear. Moreover, the sum of the moments under the curve was assumed to be equal to the externally applied moment from the idealized truck load (Arockiasamy & Amer, 1997). As discussed earlier, the edge beam strain values were neglected, leaving just the shaded region in Figure 2.7 under consideration in the analysis. Since the edge beam moment was neglected, the peak moment for this case was taken as 1.6 kip-ft/ft and located approximately 4.6 ft from the outer edge of the bridge. The area of the shaded region was then divided by this value to determine the modified EW for the bridge.

The modified structural EW calculated from the diagnostic load test was equal to 11.3 ft. The measured EW was 5% more than AASHTO's, indicating that a larger portion of the slab was resisting the applied moment and that less stress was being imposed in the longitudinal direction of the slab. Consequently, the load rating for the SLL case was also increased to 0.711 from the analytical RF computed using AASHTO.

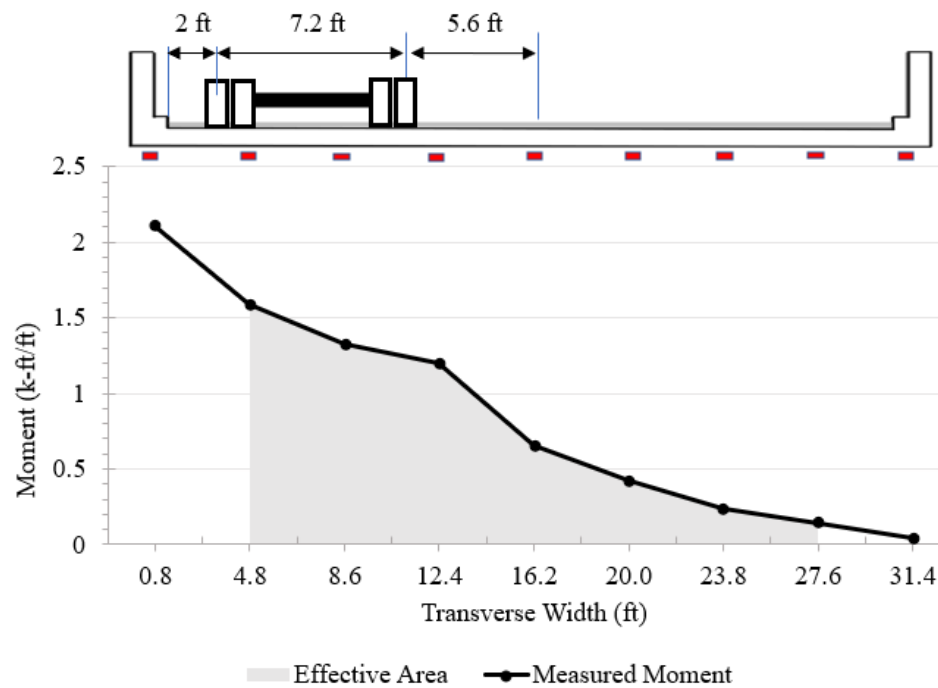


Figure 2.7. Moment distribution for critical load path from the diagnostic test

2.4.5. Proof Load Test Sensor Plan and Loading Plan

For the proof load test, the strain gauges were kept in the same position and orientation as for the diagnostic load test (see Figure 2.4). As shown in Figure 2.8, eight linear variable differential transformers (LVDTs) labeled L1 through L8 were placed at the same location as the strain gauges along the critical section to capture the displacement profile in the transverse direction during testing. A plan view schematic of the fixture used to support the LVDTs, and a picture of the fixture taken under the slab bridge during the proof load test is shown in Figure 2.8.

For multilane bridges, at least two lanes are required to be loaded during the proof load test (AASHTO, 2018). A target proof moment of 651 kip-ft/ft was calculated using Eq. 2-8. This value accounts for an ADTT below 1000 (-10%) and considers that the bridge is a non-redundant structure (+10%). Moreover, because an in-depth inspection was performed by the IDOT, X_p in Eq. 2-7 was reduced by 5%.

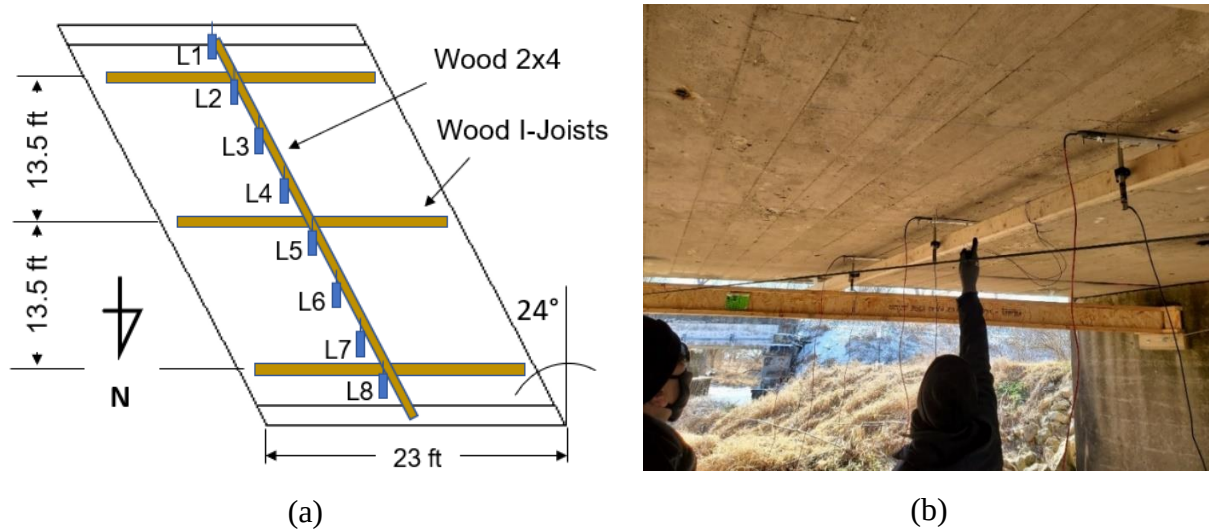


Figure 2.8. LVDT distribution and the fixture constructed using wood I-Joists for proof load test (a) plan view schematic, and (b) photograph of the fixture with installed LVDT sensors

Conducting the proof load test required the bridge to be closed. To avoid added costs and complications associated with extending the proof test to multiple days, the load testing was organized to last a single day. Because of constraints with test truck dimensions and loading capacities, it was not possible to load the bridge using the truck used for diagnostic testing. Preliminary calculations were necessary to determine an adequate and safe testing mechanism for loading the bridge to the required magnified loads. Furthermore, the loading would need to be easily adjustable to allow for incremental loading steps that are neither too large nor too small.

A 275-ton capacity crane was chosen to load the bridge in incremental loading stages using 22-kip stackable slab loads positioned directly on the bridge. Figure 2.9.a shows an image of the

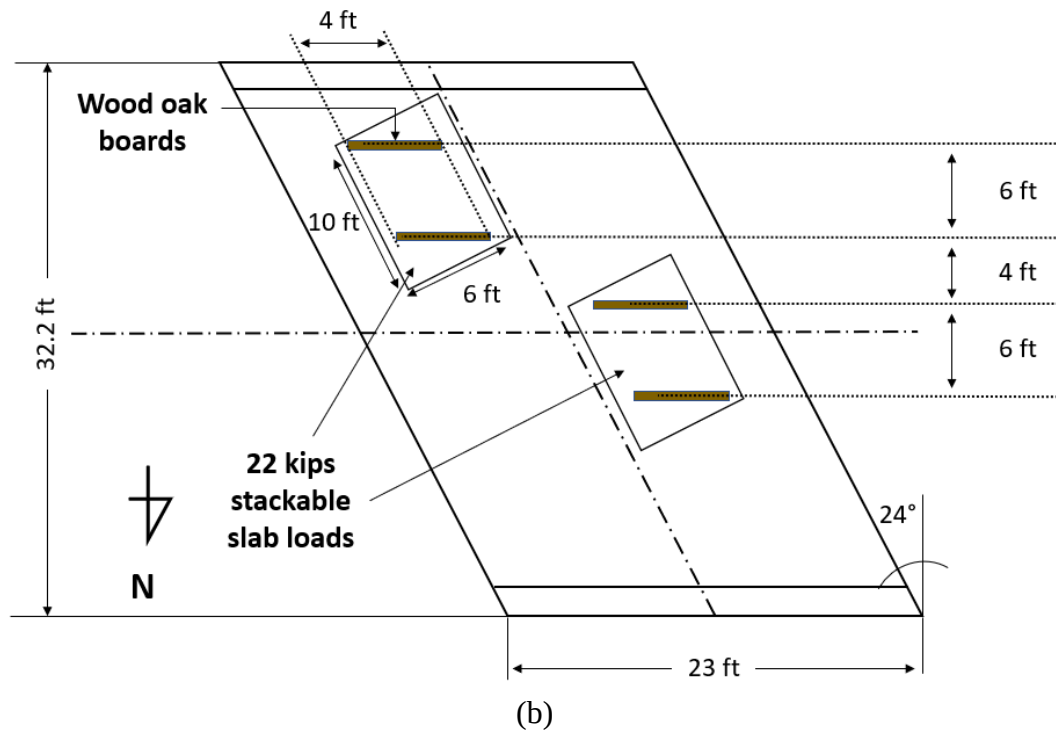
crane and the slab loads used for loading the bridge. Figure 2.9.b illustrates the plan view configuration of the slab loads being transferred by wood oak boards beneath the slabs to load the bridge. The boards were 4 ft in length and 6 in wide. They were spaced such that they represent HS20 axle spacing in the transverse direction (6 ft) and spaced 4 ft from each other between lanes. The transverse positioning of the slab pair was decided based on varying the configuration along the transverse width and finding the location which produced the maximum effects. This transverse positioning of the oak boards was then fixed for the remainder of the test. The loading stages began with one slab load in each lane positioned on the bridge to produce a theoretical moment that is 25% of L_T as determined from the preliminary 1D beam-line analysis. The loading was amplified in subsequent stages by adding a combination of more slab loads and shifting the position of the loads towards midspan. Each loading stage was repeated two times to assess the consistency of the measured responses.



(a)

Figure 2.9. Loading crane and loading plan for proof load testing (a) photographs of the crane, and (b) plan view configuration of slab loads and wood oak boards used to transfer the loads

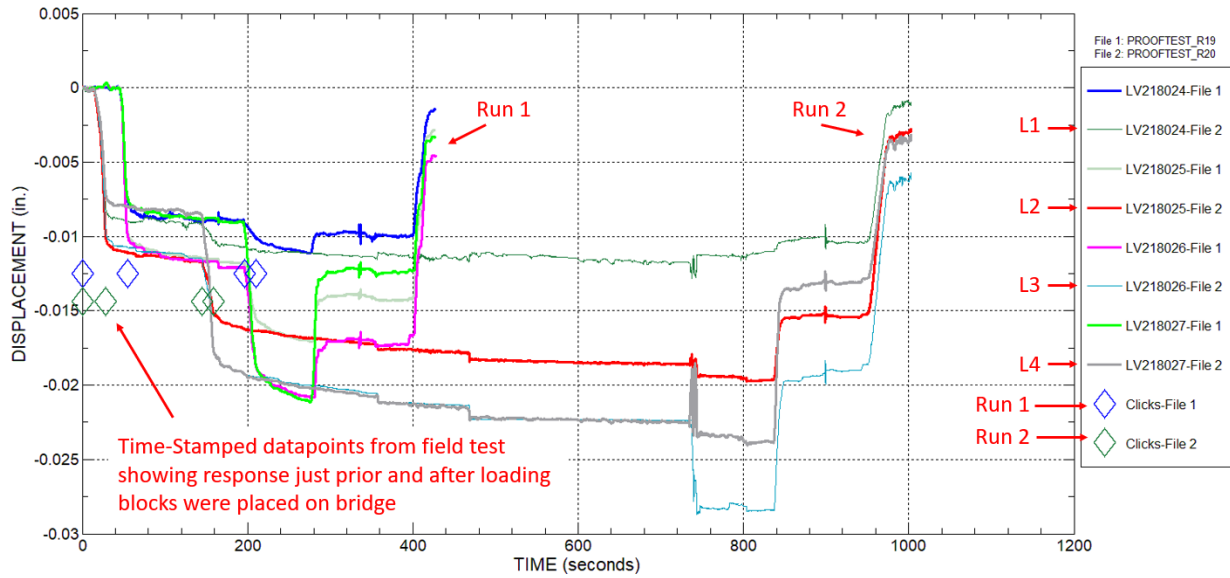
Figure 2.9 cont.



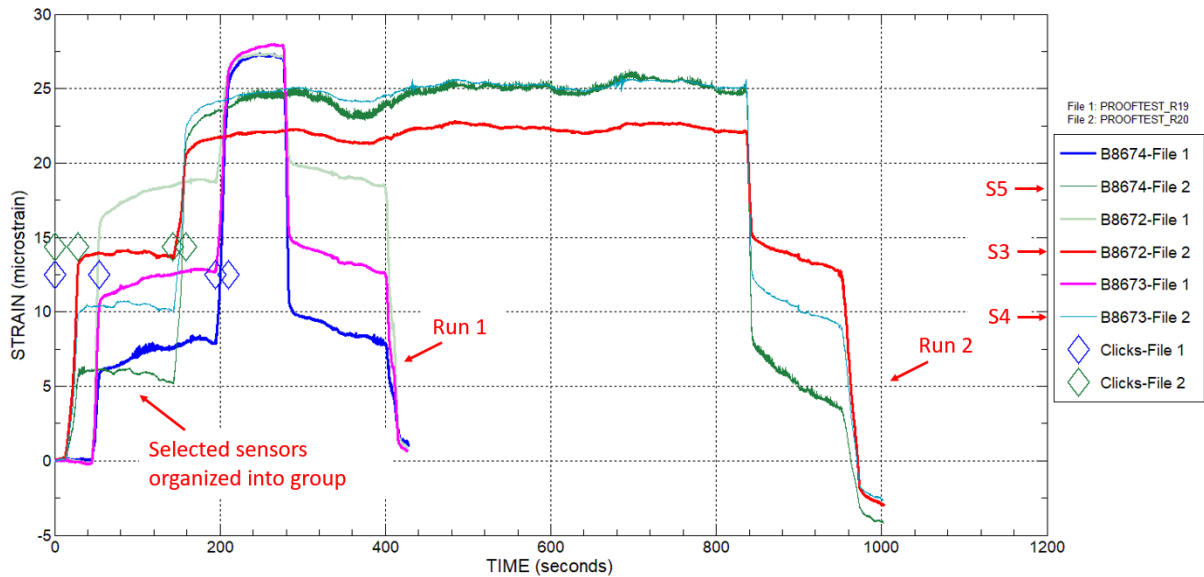
2.4.6. Proof Load Test Load Rating Results

The proof load test data processing was performed with the sensor results exported directly from the BDI STS-LIVE software. A plot of the response histories for the critical LVDTs and strain gauges recorded during the proof load test is shown in Figure 2.10. The figures were processed in the STS-VIEW software where the data can quickly be visualized as a function of time. Both Run 1 and Run 2 of Stage 3 are shown in the plots. Also shown in the plots are the time-stamped data points denoted by the trapezoidal shapes on the figures. These are the points just prior and after the slab loads were placed on the bridge. These points were added during the load test using the comment feature of the STS-LIVE software. The peak responses processed for the load rating evaluation were the instantaneous sensor responses. These were the points just after the slab loads were placed on the bridge. Some time-dependent displacements and strains were

apparent from the plots. However, these are attributed to other effects such as the wind's influence on the sensors which were not relevant for the load rating analysis.



(a)



(b)

Figure 2.10. (a) Displacement response history plot for L1 through L4 and (b) strain response history for S3 through S5 recorded in STS-LIVE for Run 1 and Run 2 of Stage 3 loading of the proof load test

A plot of the displacement history including the residual displacements for LVDT 3 producing the peak values on the bridge is shown in Figure 2.11. This plot helps illustrate the loading sequence, the difference between each loading run, and the residual displacements upon unloading for every loading step. At Stage 1 loading (25% L_T), the two bridge runs were nearly identical with no residual displacements upon unloading. A residual displacement approximately equal to 10% was observed for run 1 of Stage 2 loading (50% L_T). However, when the loading was repeated for run 2, there was no increase in the residual displacement and the peak displacements were almost equal. The displacement increased by 5.5% for run 2 compared to run 1 of Stage 3 (67% L_T), implying the bridge was displacing more than anticipated for the applied theoretical moment. Moreover, a residual displacement of approximately 12% was observed during the final run of Stage 3 leading to the conclusion that the bridge has undergone some permanent deformation. Hence, it was decided to terminate the test after the second run of Stage 3. Because the observed distress occurred during Stage 3 loading, AASHTO requires that the L_p used in Eq. 2-9 be the load just prior to reaching the load causing distress in the bridge. Applying Eq. 2-9 with a k_0 equal to 0.88 and an L_p corresponding to the Stage 2 loading, a modified RF equal to 0.425 was calculated, which is 26% less in load rating compared to the analytical results.

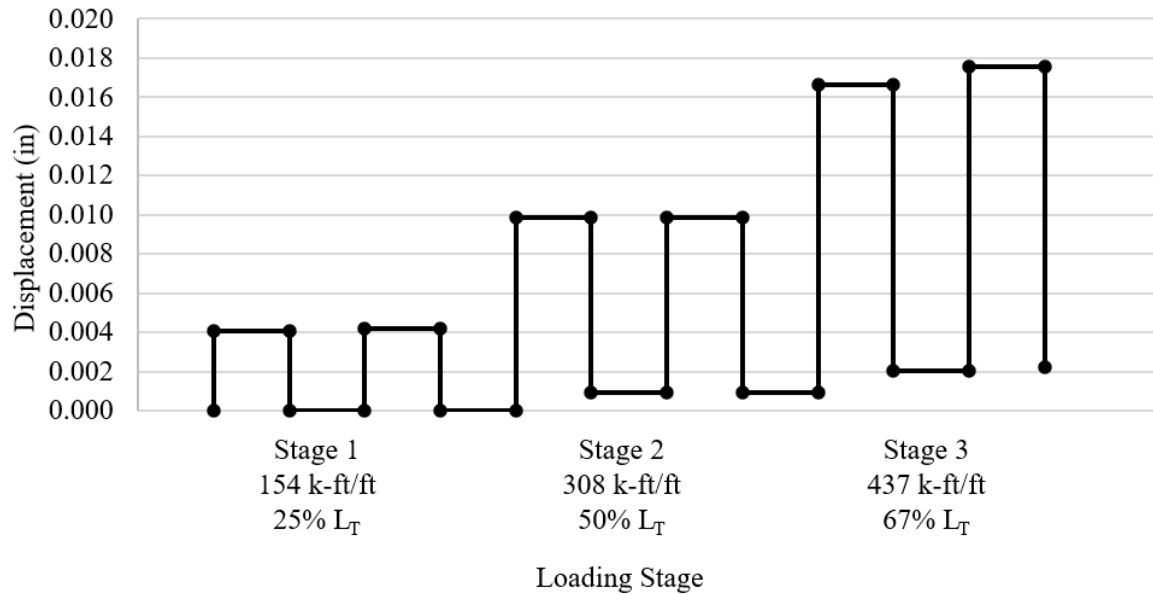


Figure 2.11. Load-displacement history for proof load test recorded by LVDT 3

2.4.6.1. Load Rating using Structural Effective Strip Width Approach

The transverse moment distribution from the proof load test for the critical load case is shown in Figure 2.12. The figure also illustrates the location of the loads along the transverse width of the bridge producing the maximum effects. The peak strain value was observed under the south edge beam as predicted from studying the diagnostic test results. No strain gauge was placed under the north curb of the bridge because of limitations in the number of data channels available for the test. It was therefore conservatively assumed to be zero. The structural EW was determined as 9.13 ft, which is 3% less compared to the effective width calculated using Eq. 2-3. The resulting modified RF for the MLL case was calculated as 0.560.

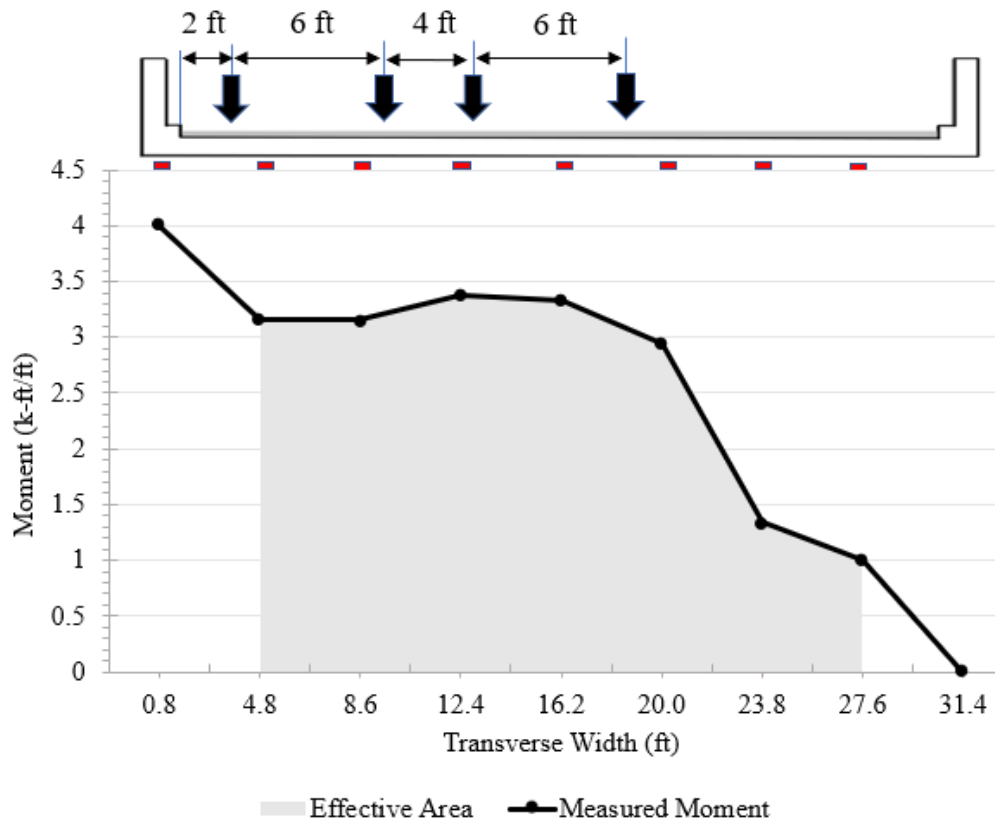


Figure 2.12. Moment distribution for critical load path from the proof load test

2.4.7. Results Comparison and Discussion for Piatt Bridge Test

A summary of the load rating results is presented in Table 2.5. A summary of the load rating results is presented in Table 3. The modified RF using the diagnostic load test MBE procedure suggests that the test results cannot be directly extrapolated to performance at greater loads, although the calculated K_a factor shows that the load effects on the bridge were more significant than obtained from the theoretical model. The modified RF using the diagnostic test EW approach shows an improvement from the analytical RF by about 8%. This result was not representative of the actual rating since the test was conducted under SLL. An MLL diagnostic test would give a better approximation of the modified load capacity for the bridge. Furthermore, neglecting the edge beams, skew angle and the effects of SLL in the analysis distorts the effective width calculations and final RF. The modified RFs using the diagnostic test calibrated model and

the proof load test approaches give better estimations of the bridge load capacity and condition. Results show that the load rating for the critical MLL case decreased by as much as 26% in the case of the proof load test using the MBE approach, by about 16% using the calibrated model, and by about 3% using the EW approach.

The calibrated model resulted in a 32% improvement in the RF compared to the MBE proof load test. Although both EW and MBE methods using the proof test resulted in smaller RFs compared to that of the analytical approach, the MBE approach proved to be more conservative than the EW approach, showing a 26% smaller RF compared to the EW approach. This was also evident in the diagnostic test results where the MBE approach gave an 8% smaller RF than that computed using the EW approach. When compared to the MBE methods, the EW approach was more involved, requiring each strain measurement along the cross section of the bridge to be considered to compute an EW for the bridge. The MBE procedure was straight forward, simply requiring the peak strain of the critical load case to be used in the diagnostic test analysis and the maximum applied live load (L_p) for the proof load test analysis.

The reductions in load rating are attributed to the significant damage observed on the bridge. The localized damage under the south curb and extensive longitudinal cracks observed during the visual inspection were significantly altering the load distribution in the slab. Moreover, the modified load ratings shown in Table 2.5 illustrate that more conservative analytical load ratings may be necessary for older RC slab bridges which have noticeable damage and unique geometric properties.

Table 2.5. Summary of modified load ratings using different methods

Load Rating Approach		Modified RF	Analytical RF	Loading Type
Diagnostic	EW	0.711	0.655	SLL
	MBE	0.655		
Proof	EW	0.560	0.578	MLL
	MBE	0.425		
Calibrated Model		0.488		

2.4.8. Conclusions of Piatt County Bridge Test

This comprehensive study presented a load rating analysis on an old RC slab skewed bridge using several analytical and experimental methods. Diagnostic load testing and proof load testing were employed on the bridge for calculating modified RFs to compare with the AASHTO analytical load rating results. Two approaches, namely, AASHTO's MBE approach and the EW approach, were applied and discussed for each load test. A summary of the main findings of this bridge study are as follows:

- Load testing may reduce the load ratings for RC slab bridges that have noticeable damage and unique geometric properties. In such cases, conservative assumptions may be necessary when estimating the material properties and section properties of the concrete slab for load rating similar bridges.
- The MBE approach gave more conservative load ratings showing a 14% and 31% smaller RF compared to the EW approach for the diagnostic test and proof load test, respectively.
- The load rating for the bridge was reduced by an average of 26% using the SLL diagnostic test MBE and EW approaches compared to the analytical load rating procedure.

- The controlling load rating for the bridge was reduced by an average of 24% using the MLL calibrated model and proof load test MBE and EW approaches compared to the analytical load rating procedure.
- The calibrated 1D beam-line model from the SLL diagnostic test conveyed a 25% greater RF than that calculated using AASHTO's MBE proof test procedure and a 13% lesser RF than calculated using the EW approach. This shows that the calibrated model can be used to estimate a reasonable load RF for the bridge at greater load levels.

This load rating study has shown that the load rating for bridges can vary by as much as 38% depending on the method being employed for the load rating analysis. The load rating using the MBE approach was simpler to apply while the EW approach was less conservative. The diagnostic and proof load testing procedures used on the bridge were expensive compared to the analytical rating procedure. The conclusions made in this study are solely based on the results of a single bridge. Hence, a greater inventory of RC slab bridges should be considered in future work to further verify the outcomes of this investigation.

2.5. McLean County Bridge Test

2.5.1. Bridge Description

Figure 2.13 illustrates an image of Bridge 2 chosen for load testing located in McLean County, IL. The bridge was constructed in 1954 and was decommissioned and scheduled for replacement because of its age and load posting. The tested bridge was a two-lane continuous two-span RC slab bridge with span lengths of 32 ft, a width of 24 ft, and 12 in wide curbs. The design drawings specify a concrete compressive strength equal to 3500 psi and 1.27 in diameter longitudinal rebars. A coring sample test was performed for the slab resulting in average

compressive strength of 4450 psi. The bridge had several longitudinal microcracks along the bottom of the slab and a long transverse crack located about 12.5 feet from the abutment face.



Figure 2.13. McLean County bridge

2.5.2. Analytical Load Rating Results

The analytical load RFs were determined using the concrete properties specified in the design plans and tested core samples and are summarized in Table 2.6. The load rating was controlled by the MLL condition along the positive moment region located 11 ft from the abutment face. Although the bridge capacity was reduced along the negative moment region where there was a discontinuity in the primary reinforcement, the dead load effects were much less significant in this region, leading to the controlling load rating in the positive moment region. The analytical load rating improved when applying the tested concrete properties in the analysis. These properties are used in subsequent calculations.

Table 2.6. Analytical load rating for McLean County bridge

Critical Section	Lane Loading	AASHTO LFR using Design Properties	AASHTO LFR using Tested Properties
Positive Moment	SLL	0.925	0.946
	MLL	0.858	0.879
Negative Moment	SLL	0.969	0.991
	MLL	0.899	0.920

2.5.3. Diagnostic Test Sensor Plan and Loading Plan

Eight strain gauges labeled S1 through S8 in Figure 2.14 were positioned incrementally along the critical section. The instrumentation was the same used for the previous bridge study and oriented to capture the longitudinal strain profile for the bridge under diagnostic test loading. The strain gauges were attached using the stud mounting technique with 18-inch gauge extensions. The tested span was chosen based on inspection of the bridge. The north span was shown to have more extensive cracks, including a 13 ft transverse crack close to the critical section. Furthermore, the span was more convenient for sensor installation and testing purposes.

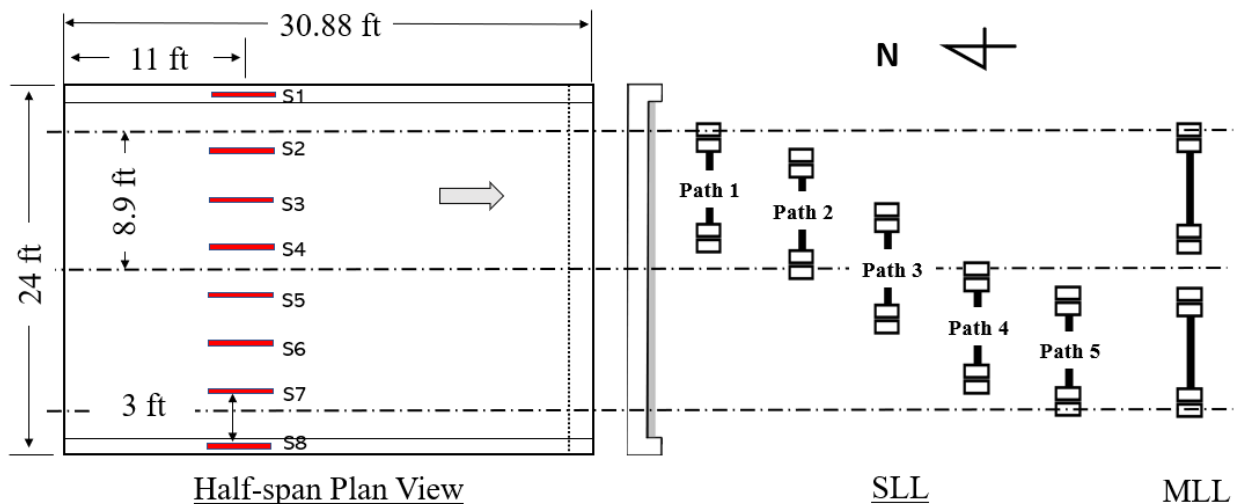


Figure 2.14. Half-span plan view schematic of McLean County bridge showing the strain gauge sensor configuration and load paths for SLL and MLL

Both SLL and MLL were considered for the diagnostic test loading conditions to obtain an RF from the diagnostic test for the controlling MLL case. The trucks selected for this test are shown in Figure 2.15. Truck 1 was used for the SLL case. The trucks were loaded such that their effect on the bridge was approximately 80% of the maximum HS20 bending moment to allow for a modified load rating from the test results regardless of whether the bridge behavior cannot be extrapolated to the amplified loads of 1.33W specified in the AASHTO MBE (AASHTO, 2018). The test truck axle loads were measured using movable scales prior to loading the bridge. The measured axle loads are shown in Figure 2.15.b.

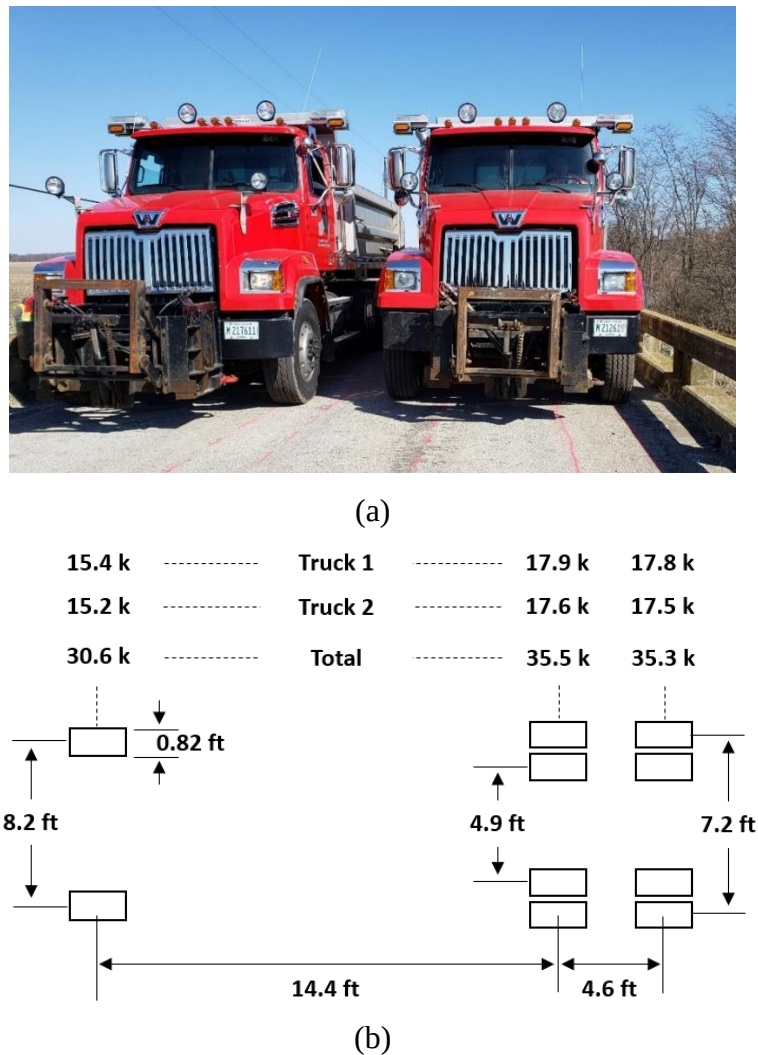


Figure 2.15. Loading trucks for the diagnostic test showing (a) image of the trucks and (b) plan view axle configuration with measured axle loads for both trucks

As shown earlier in Figure 2.14, the loading plan comprised five truck paths in the traverse direction of the bridge under SLL and one truck path under MLL. The bridge was loaded with the trucks going at a crawl speed of approximately 5 mph. Each path was repeated two times to ensure the reliability of the measured responses. The truck path producing the greatest load effects under SLL was along Path 3 at S4. The maximum response under MLL was also at S4. The trucks traveled south for all load paths.

2.5.4. Diagnostic Test Load Rating Results

The maximum measured member strain (ϵ_T) used for the post-processing occurred at S4 when the first of the rear tandem truck axles crossed over the critical section of the bridge. The section factor assumed to resist the applied load was based on the properties of the slab. Applying Eq. 2-6, a K_a value equal to 0.89 was obtained, implying an improvement in load capacity from the analytical results.

The K_b factor was obtained using the cracking strain of the concrete as a threshold to determine if the linear behavior of the bridge could be extended to a load level of 1.33W. A strain limit of $132\mu\epsilon$ was calculated using the tensile strength of the concrete. The theoretical strain calculated for the applied truck load was greater than the strain limit, yielding a K_b factor equal to 0.5 since the truck load was sufficiently high to obtain some meaningful results from this load test (AASHTO, 2018). Plugging the factors into Eq. 2-4 and Eq. 2-5 returns a modified RF of 1.27 indicating a 44% improvement from the controlling analytical RF shown in Table 1.

2.5.5. Proof Load Test Sensor Plan and Loading Plan

A fixture was constructed using wood I-Joists to support the LVDTs along the critical location during the field test. A photograph of the fixture is illustrated in Figure 2.16 along with

the half-span plan view showing the LVDT locations. The LVDTs were located in the same positions as the strain gauges used for the diagnostic tests. The strain gauges were kept in the same position and orientation as for the diagnostic load test (see Figure 2.14). As shown in Figure 2.16, eight LVDTs labeled L1 through L8 were positioned along the critical section to capture the displacement profile along the transverse direction during testing. A plan view schematic of the LVDTs and strain gauges positioned on the north span and a picture of the fixture taken under the slab bridge during the proof load test is shown in Figure 2.16.

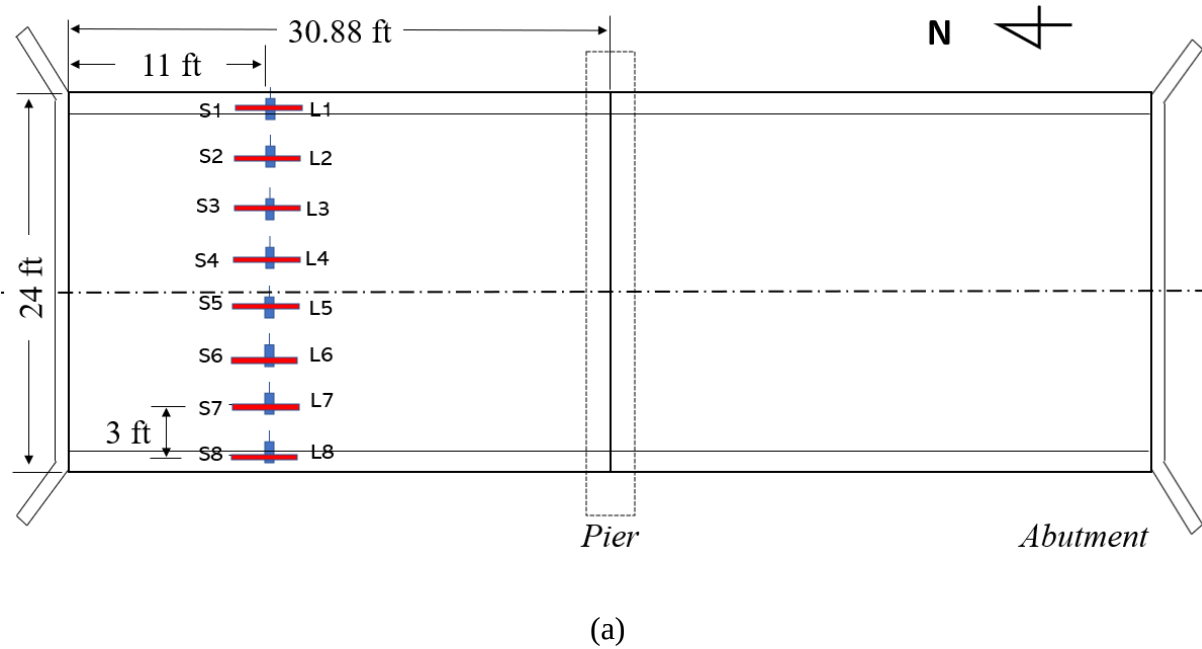


Figure 2.16. Instrumentation plan for proof load test of Bridge 2 (a) plan view schematic, and (b) photograph of the fixture with sensors installed on bridge

Figure 2.16 cont.



(b)

Conducting the proof load test required the bridge to be closed. The load testing was organized to last a single day using two trailer trucks which would be driven simultaneously onto the bridge as live load. The trailer trucks were nearly identical with slight differences in the trailer bed length which had a minor influence on the loading effects. The trailers were loaded using a small crawler crane which stacked the 3,200-pound bin blocks used for weights onto the trailer beds of the trucks. An image of one of the trailer trucks used for the proof load test is shown in Figure 2.17.a along with the crawler crane and bin blocks used for the test. The axle weights were measured on the field before loading the bridge using digital scales. An image of the movable scales used for measuring the axle weights is presented in Figure 2.17.b.



(a)



(b)

Figure 2.17. Image of one of the trailer trucks, (a) the concrete bin blocks used for weight and the crawler crane used to load the trailers, and (b) the movable scales used for measuring the axle loads during the proof load test

The location of the trucks which produced the maximum moment at the critical positive moment region of the bridge was determined from the 1D beam-line analysis. Only the rear tandem axles of the test trucks were on the bridge span to generate this moment. A target proof moment (L_T) of 717 kip-ft/ft was calculated using Eq. 2-8. This value accounts for an ADTT below 100 (-15%) and considers that an in-depth inspection was performed on the bridge (-5%). The desired loading increments were then established based on the percentage of the target proof load. A figure showing the bin blocks' arrangement on the trailers used for each loading stage and the corresponding applied moment on the bridge is illustrated in Figure 2.18. The percentage of the target proof load is also shown in the figure. For example, at loading Stage 8, there were six rows of bin blocks spanning the length of the trailer and three rows across the width of the trailer. There was an additional stack of bin blocks placed on top of the rear two rows of the trailer. The total quantity of bin blocks on each trailer was 24 for Stage 8 loading. This produced an applied moment of 639 kip-ft equal to 89.2% of L_T . This was the maximum moment achieved during the proof load test used for modifying the RF. Each loading stage was repeated two times to assess the consistency of the measured responses by backing the trucks onto and off of the bridge.

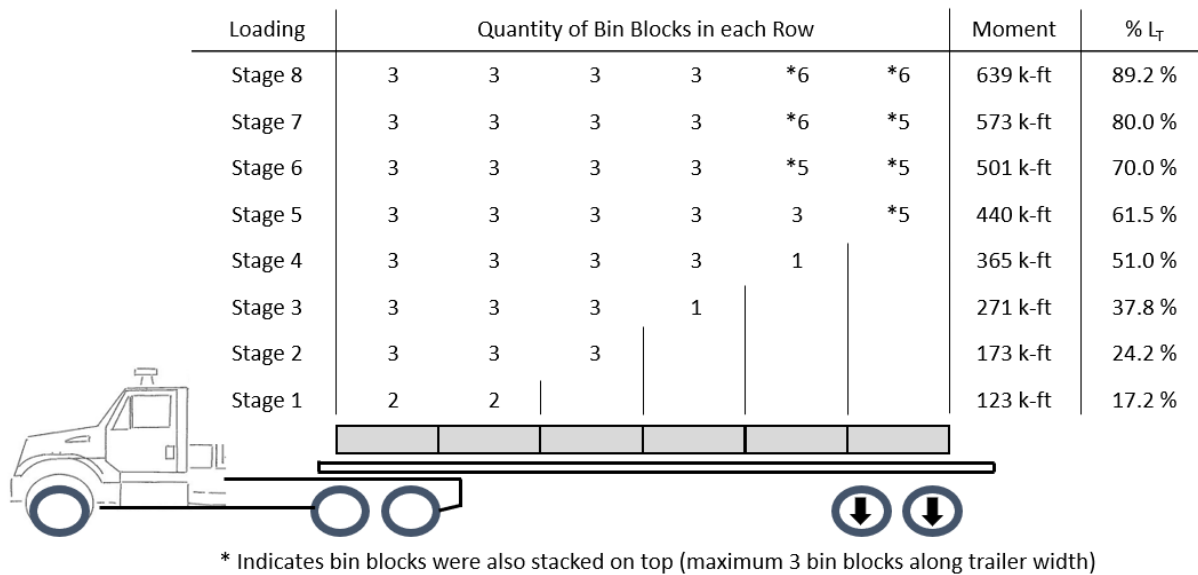


Figure 2.18. Loading sequence for both trailers showing the number of bin blocks and the corresponding location on trailer per loading stage during the proof load test

2.5.6. Proof Load Test Load Rating Results

A spreadsheet using Microsoft Excel (2021) was created for evaluating the linearity of the bridge responses during the test. The results recorded in the STS-LIVE software were copied into Excel and processed between runs during the test. For each strain gauge, a plot of the magnitude with respect to the loading was generated. For each LVDT, a load-displacement plot was created, and a displacement history plot was created to show the residuals of the displacements after every loading stage. A plot of the displacement history at LVDT 4 recorded during the proof load test is shown in Figure 2.19.a. This was the LVDT which recorded the largest displacements for the bridge during the proof load test. A plot of the strain history for strain gauges S3, S4, and S5 recorded during the proof load test is shown in Figure 2.19.b. As observed from the data, there were no significant signs of nonlinearity in the bridge response. There were some differences between runs. However, the residuals were all less than 10% of the measured response, which was used as an indication for permanent deformation to the bridge. Also, the strain data never exceeded

the estimated cracking strain of $132\mu\epsilon$. This estimated cracking strain is presented in Figure 2.19.b as a line representative of the limit where the measured strain of the slab has not exceeded the cracking strain. The strain gauge producing the maximum responses was S4 located at the same location as L4.

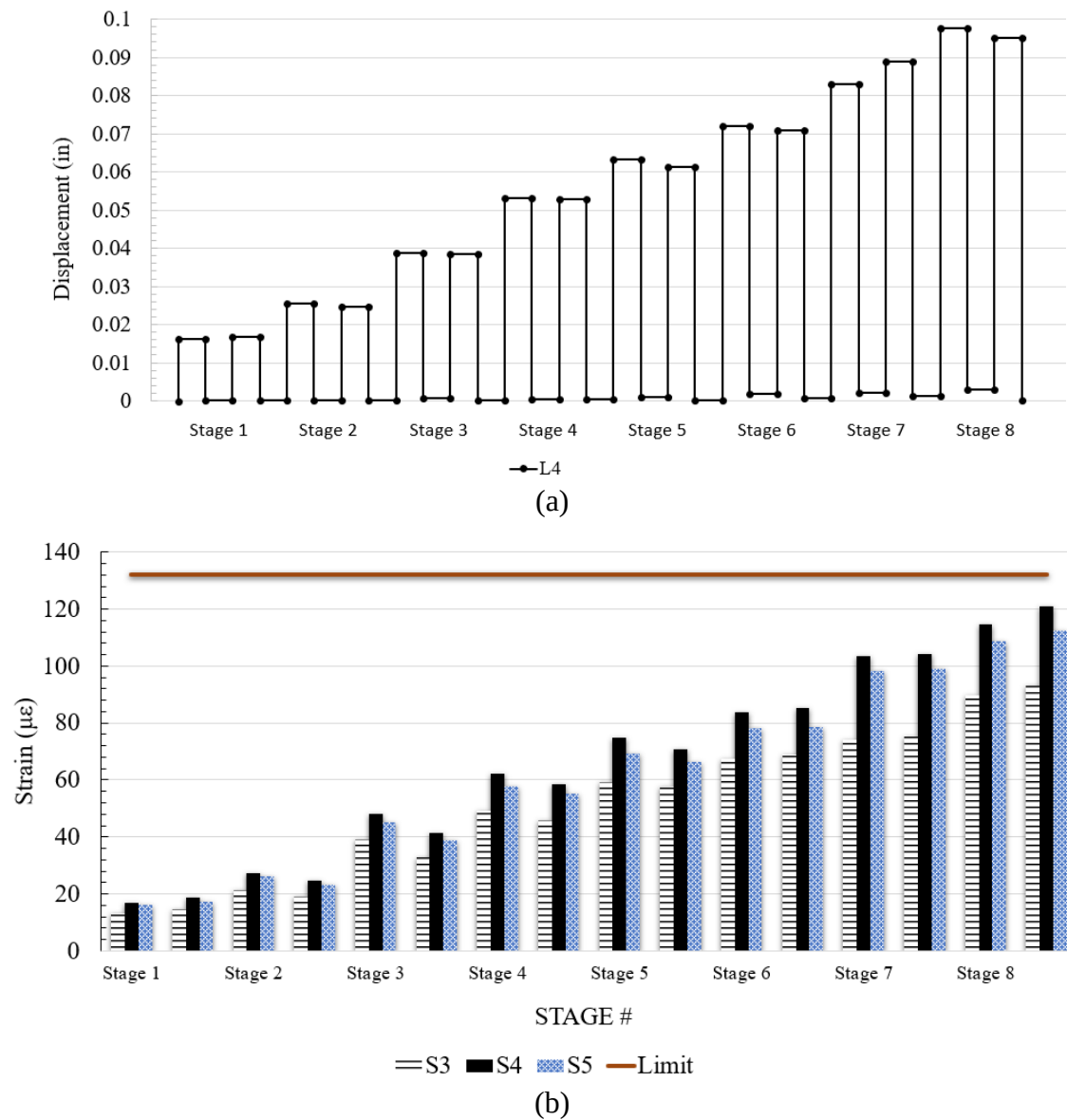


Figure 2.19. Proof load test response histories for (a) displacement of LVDT 4 and (b) strain gauge magnitudes of S3, S4 and S5

The proof load test was terminated after Stage 8. No signs of distress in the bridge were apparent. However, the trucks were no longer capable of driving onto the bridge because of the heavy axle loads carving a large indentation into the roadway at the approach of the bridge. Images of the roadway damage are illustrated in Figure 2.20. Because the test was terminated due to non-structural aspects of the load test and not because of observed distress in the bridge, the k_0 used in Eq. 2-9 was taken as 1.0. A modified RF equal to 0.912 was calculated, which is a 4% improvement from the analytical results.



Figure 2.20. Damage of the road due to heavy axle loads during the proof load test

2.5.7. Results and Discussion of McLean Bridge Test

A summary of the load rating results is presented in Table 2.7. The modified RF using the diagnostic load test MBE procedure suggests that the bridge has a much greater load capacity than expected from the analytical load rating results. The percentage improvement is as much as 44%. The modified RF using the proof load test shows only a minor improvement from the analytical

RF of about 4%. The RF was not greater for the proof load test because of the need to stop the test earlier due to the damage on the roadway. Because there were no signs of distress in the bridge at nearly 90% L_T , it was expected that the bridge can carry the target proof load.

Table 2.7. Summary of RFs computed for McLean County bridge

Lane Loading	Analytical RF	Diagnostic Test RF	Proof Load Test RF
MLL	0.876	1.27	0.912

Overall, the field test results show that the bridge performed better than suggested by the analytical load rating. This shows that the analytical rating procedure using AASHTO LFD may give conservative RFs, which in this case, resulted in a load posting for the bridge that may not be necessary. Furthermore, the bridge was decommissioned and replaced which could have been avoided or postponed to a much later time since the bridge has proven to show good performance under amplified loads. These expenditures could have been used for other more critical bridges in the Illinois bridge inventory. Hence, why these load tests are being investigated as an alternative load rating method in this research.

It should be noted that the EW approach and modified RF using a calibrated model were not performed for this study. The primary purpose of this study was to carry out the proof load test on the bridge and establish a loading protocol that can be used for subsequent bridge tests. The diagnostic load test was conducted under MLL and the calibrated model was not necessary in this case for establishing a modified RF for the bridge. The results for the modified RFs using EW procedure were not consistent with the results obtained using AASHTO MBE method. The narrow width of the bridge combined with the MLL of the trucks during both load tests resulted in much smaller effective widths for the bridge. This is because the area under the moment distribution

curve is much smaller with a narrow width which yields a smaller EW result in the calculation. This analysis was outside of the scope of this research and left out in this study.

2.5.8. Conclusions of McLean County Bridge Test

This study presented a load rating analysis using diagnostic and proof load tests for a two-span RC slab bridge that was decommissioned and scheduled for replacement. AASHTO MBE procedures were used to obtain a modified load rating for the bridge. A summary of the main findings of this bridge study are as follows:

- Load rating using diagnostic and proof load testing showed that the bridge was capable of carrying the design loads and that no load posting should be necessary.
- The load rating improved by as much as 44% using load rating through diagnostic testing methods of the AASHTO MBE.
- The load rating improved by as little as 4% following load rating through proof load testing procedures of the AASHTO MBE. The proof test was terminated because of non-structural damage of the roadway. Hence, a greater load rating using this test was expected.
- The proof load testing strategy was effective at demonstrating the minimum capacity for the bridge. Although, methods for decreasing the time to load the trailers using the concrete bin blocks are recommended to improve the efficiency of using this load test for load rating.

2.6. Conclusions

This section presented a comprehensive load rating study on two different RC slab bridges. The purpose of this research was to perform load testing on RC slab bridges to obtain modified load ratings to compare with the analytical AASHTO load rating procedures. Diagnostic load testing and proof load testing were employed following on both bridges. The results obtained for

each bridge was different. However, the load testing did prove to yield more reliable load ratings than the analytical results, giving insightful information with regards to the condition and structural capacity of both bridges. The load testing was effective. Yet, there is room for improvement as it pertains to the costs for conducting these tests. The content of in the next chapter aims at improving the load testing strategy employed in this study by exploring a potential alternative for measuring displacements of these slab bridges. The method utilizes image-based technology which can significantly reduce costs, installation time, and challenges associated with using conventional LVDT sensors during load tests.

CHAPTER 3: IMAGE-BASED DISPLACEMENT MEASUREMENTS FOR FIELD TESTING OF BRIDGES

Bridge load testing relies on deformation measurements to make conclusions about the structural integrity of a bridge. Two of the most common and useful field measurements during bridge tests are strains and displacements because they give understanding to the load path distribution of a bridge, which is important for load rating analysis. Although strain measurements are important for translating bridge deformations to bending moments, this is challenging and often impossible to do precisely when testing reinforced concrete (RC) slab bridges which have unique sectional and material properties. Moreover, these sensors are highly influenced by environmental effects and very sensitive to surface irregularities such as microcracks common in concrete components that make strain measurements less reliable when performing proof load testing on these bridges. A more dependable method for proof load testing of RC slab bridges is using displacement measurements for capturing the more global response of the bridge. The process of using bridge displacements as the driving indicator of structural damage and capacity during proof load tests has been successfully used for several bridge studies in the literature (Saraf, 1998; Arroyo et al., 2019; Lantsoght et al., 2017). Furthermore, numerous improvements and alternatives to conventional displacement sensors have been studied which have made load testing more feasible for bridge owners to employ. This chapter explores the potential of using image-based displacement measurement procedures for field testing of bridges as an alternative to less convenient, traditional displacement sensors.

3.1. Literature Review

Linear variable differential transformers (LVDTs) are sensors extensively used for measuring displacements during field tests. These sensors are capable of capturing displacements

with thousandths of an inch accuracy under ideal conditions. However, their accuracy depends on a stable reference point from which the displacements are measured making them less ideal for field testing applications (Khuc & Catbas, 2017). These sensors are also limited to capturing displacements for a specified range, in a single direction, and at a single point. Hence, several sensors are often necessary to properly capture the load distribution and loading effects on bridges, which increases costs for field testing.

With the advent of computer vision and high-resolution consumer-grade cameras, there has been a great deal of research utilizing this technology for structural monitoring applications (Yoon et al., 2016; Alavi & Buttlar, 2019; Dong et al., 2020). Three major developments in computer vision have driven the application of image-based processing for deformation monitoring. One method is Digital Image Correlation (DIC), another is the optical flow-based approach, and another will be referred to as the Feature-Based Image Registration (FBIR) method (Peters & Ranson, 1982; Horn & Schunk, 1981; Lucas & Kanade, 1981).

The innovative DIC method first published by Peters and Ranson (1982) was among the first in computer vision to be applied to civil structures for the use of testing and monitoring (McCormick & Lord, 2010). This method can capture submillimeter deformations using images from consumer-grade cameras (Aghlara & Tahir, 2018) and was utilized early on for monitoring crack openings in concrete (Sutton et al., 1983). DIC has also been used and field validated in several bridge studies proving the potential of this technology for field testing applications (Yoneyama & Ueda, 2012; Murray, 2013; Murray et al., 2015; Cai et al., 2019). However, the DIC methodology has a major drawback. DIC requires markers or patterns to be installed on the surface of the object of interest which can be difficult to do on many bridges. Also, this limits the measurements taken during the field tests to the discrete locations which have the applied pattern.

Optical flow-based approaches have become increasingly popular for various computer vision applications because of their wide range of functions and good performance. Several optical flow-based approaches exist for estimating displacements. One of the first is Horn and Schunk's (1981) intensity-based optical flow approach which defines optical flow as a distribution of apparent velocities of movement of brightness patterns in an image. This method is dependent on constant image illumination intensity which is a major drawback for bridge monitoring applications. While improvements to this original optical flow-based approach are many (Brox & Malik, 2011), the phase-based optical flow approach has gained attention since it is less sensitive to variations in illumination and contrast while producing even more accurate results than popular optical flow procedures found in the literature (Derpanis, 2005; Beauchemin & Barron, 1995). However, the phase-based optical flow technique relies on well-defined edges of the target object to calculate accurate displacements (Cha et al., 2017). Since RC slab bridges do not typically have well-defined edges along their critical section, artificial edges would need to be placed on the structure to obtain the measurements accurately at target locations, thus presenting the same issue as with DIC when it comes to obtaining bridge displacement measurements.

Consequently, this research puts an emphasis on recent FBIR methods which utilize feature identification and matching algorithms to find and track points of an object in the image frame (Lucas & Kanade, 1981). This method has been applied before for bridge monitoring applications. Lydon et al. (2019) presented in their study a low-cost image-based sensing system for displacement monitoring of civil structures which was validated in the field on a three-span RC beam bridge. However, the study only considered a single location within the image frame as a target for the displacement without uncovering the potential of measuring multiple target locations within the frame of view. Dong et al. (2020) presented a field test-based method using computer

vision to identify the lateral load distribution factors for existing highway bridges to perform load rating. While multiple displacements were measured, the girders had targets attached to facilitate the feature identification and matching which does not fully take advantage of the natural feature identification benefit using FBIR approach. Another field study by Feng et al. (2015) showed that FBIR can be used effectively to capture dynamic displacements of a railway bridge at several camera distances to the target object. The procedure proved to be effective, but not much work was done in the submillimeter scale during the field tests. Moreover, the tests also verified only a single target in the image frame.

3.1.1. Background on FBIR Measurement Method

The goal of the FBIR method is to capture and quantify surface deformations of objects captured in a set of image frames by detecting the exact physical points of the objects in each image. Figure 3.1 shows a schematic of a target object captured in two separate images for two different instances. The image on the left of Figure 3.1 shows the undeformed object in its unloaded state. The image on the right shows the target object in its fully loaded state, where the object is displaced as indicated in the figure. The region of interest (ROI) is identified in each image as the boxed section in the frame. This is the region where the displacements are desired to be obtained. Several points are identified by a feature-detection algorithm within the selected ROI. The images are compared to one another to determine the relative movement of the object points between the image frames.

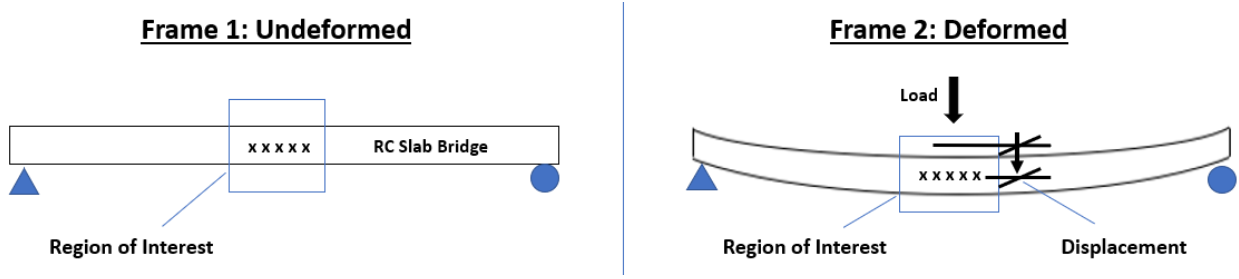


Figure 3.1. Visual application of FBIR for displacement monitoring

Movements of detected objects in the image can then be converted to engineering displacements by scaling the pixels to the desired physical units. A scaling factor (SF) can be determined from the following relation presented in Khuc and Catbas (2017)

$$SF = \frac{L_{object}}{L_{pixel}} \quad (\text{Eq. 3-1})$$

where L_{object} is the length of a known dimension in the image frame and L_{pixel} is the corresponding pixel length of that known dimension.

Like DIC, FBIR benefits from the arrival of high-resolution cameras, and perhaps even more so because of its ability to identify natural features in images, eliminating the need to apply patterns to a target object. The processing scheme is similar to DIC with a few distinctions in how displacements are measured. FBIR differs in that it utilizes feature-based algorithms for tracking deformations (Wang et al., 2015). An example of the feature identification algorithm used in this study is the popular SURF method presented by Bay et al. (2008) used for detecting distinct features in each image. The algorithm can identify features in an image that are invariant to scaling, translation, and rotation while being efficient computationally (Lowe, 1999; Bay et al., 2008). The M-estimator Sample Consensus (MSAC) algorithm was used to find the optimum set of matched points in the ROI by rejecting outliers in the dataset (Torr & Murray, 1997). The feature points can be filtered further by applying thresholds to exclude ambiguous matches, increase the feature

point density, and improve the confidence of the feature matches. The set of identified feature points can then be averaged to obtain a displacement for the selected region.

Figure 3.2 shows the components of the FBIR technique illustrating the reference and deformed object images with their corresponding identified and matched feature points within the selected ROI. The strongest feature points in each frame were identified and matched from the SURF algorithm. The matched feature points were then filtered using the MSAC algorithm to distinguish the inliers from the outliers. The results are presented as a montage image in Figure 3.2.d.

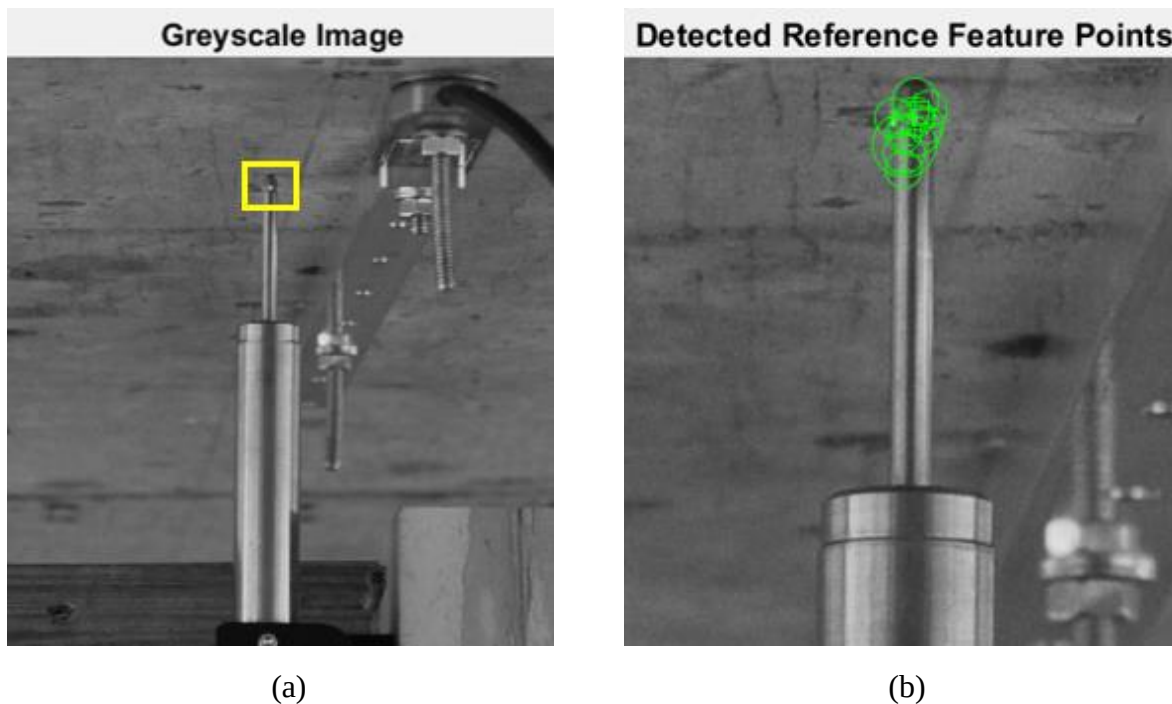
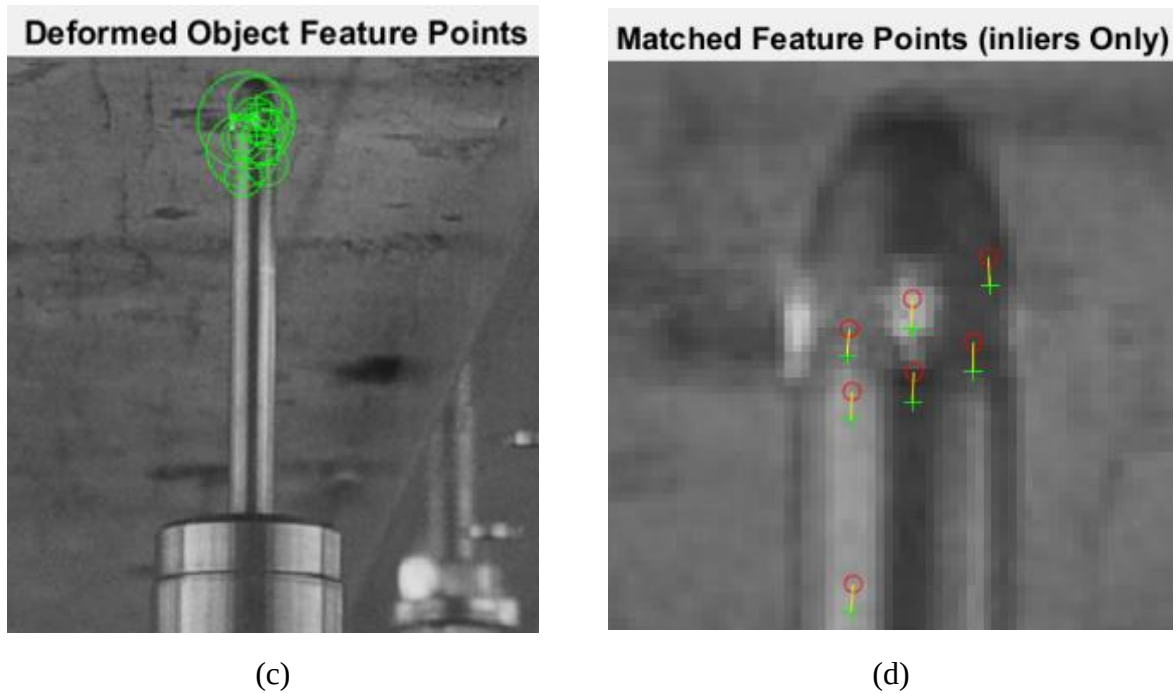


Figure 3.2. FBIR process which illustrates (a) ROI (b) identified feature points in reference image (c) feature points in deformed object image (d) matched inlier points in both images

Figure 3.2 cont.



3.2. FBIR Code Procedure for Field and Laboratory Tests

MATLAB software was used to develop a code for processing the images and to carry out the FBIR steps for the displacement estimation (MathWorks, 2021). The code utilizes built-in MATLAB functions that can perform the SURF and MSAC algorithms to identify, match and filter the feature points in the image frame. After running the algorithms for a specified ROI, the position vectors of the matching feature points were converted to the appropriate dimensions using Eq. 3-1. The final set of matched feature points was then averaged to estimate the total displacement of the region and compare it with the LVDT values.

To account for camera movement from wind and other sources, a stable object in the frame, which was assumed to be stationary between the images, was used to differentiate the movement associated with camera shake from the actual displacements of the bridge. This meant that the

camera movement did not have an influence on the final results, thus eliminating the need for the camera to be completely stable during the test to record accurate measurements. The same measurement procedure was used to trace the movement of stable objects in the image frame. The measured deformations of the target object were corrected by subtracting the motion of the stationary object in the frame from the motion of the target object. The camera movement in some instances was significantly larger than the displacements being measured. Therefore, the importance of this step to monitor such small-scale displacements using the FBIR procedure could not be overlooked.

3.3. Piatt County Bridge Test using FBIR

3.3.1. Field Test Setup

The FBIR method was investigated during the proof load test of the Piatt County bridge presented in section 2.4 of this thesis to validate this technique against the conventional LVDT approach. FBIR was applied on a set of images captured using a Galaxy Note 10 Plus cell phone with a 12-MP telephoto rear camera. The images were taken during Stage 3 of the proof load test. The measured displacements using the FBIR were then validated against conventional LVDT sensor values to evaluate the accuracy and potential for this technology to be applied to future proof load testing.

A schematic of the camera setup for the bridge with the distance to target and image of the frame of view is shown in Figure 3.3. Two LVDTs were captured in the frame of view of the camera, namely, L3 and L4. Since the FBIR approach relies on a minimum number of identified stable feature points in each image frame to accurately match and track pixels throughout each image, the collection of feature points was averaged over a region of interest (ROI) to determine

the final displacement. The ROI was selected such that it was the smallest possible while still capturing stable feature points that are representative of the LVDT results.

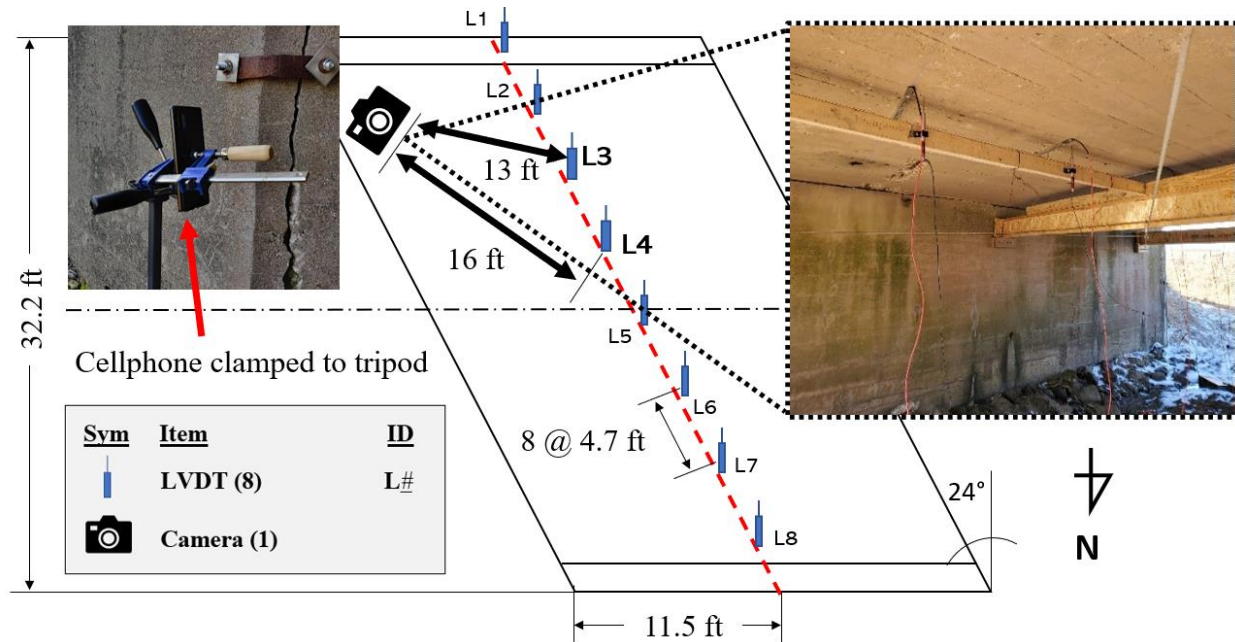


Figure 3.3. Schematic of Piatt County bridge showing cellphone camera, camera location, and field of view used during the proof load test

3.3.2. Results and Discussions

The processed results are presented in Table 3.1 showing the percent difference in displacement values between the LVDT and FBIR for L3 and L4. Differences in displacement values were just 2.67% between the FBIR and L3 located 13 ft from the camera whereas the error was nearly 7.5% for L4 located 16 ft from the camera. The results show that readily accessible consumer-grade cameras can be used to capture submillimeter displacements of RC slab bridges with relatively high accuracy. The greater error presented for the location at L4 is attributed to the obstruction of the strain gauge in the ROI due to the camera angle not providing a clear view of the slab above the LVDT and minimizing the number of reliable feature points identified on the target object. Furthermore, the error suggests that the pixel resolution of the cellphone camera at greater distances reduced the accuracy of the FBIR displacement measurements.

Table 3.1. Displacement results of Piatt County bridge study at Stage 3 of proof load test, inches

LVDT	Displacement		Error
	FBIR	LVDT	
L3	0.0181	0.0176	2.67%
L4	0.0180	0.0168	7.48%

3.4. McLean County Bridge Test using FBIR

3.4.1. Field Test Setup

The FBIR technology was also examined during the proof load test of the McLean County bridge presented in section 2.5 of this thesis to further test the accuracy and potential of this approach under a more ideal field condition. Figure 3.4 illustrates the half-span plan view of the bridge with the sensor and camera locations. The monitoring location for the camera was focused on LVDT 4 (L4) with the frame of view also shown in Figure 3.4. The image on the right side of Figure 3.4 shows the field setup after the sensors were installed and the location of the camera used for testing. The ground was sufficiently dry to support the camera used for the FBIR on a tripod mount located 6.6 ft from the target LVDT. The purpose of using this ideal field setup was to study the accuracy of the FBIR under environmental effects without having other extrinsic parameters such as camera distance and angle to target influence the results. These extrinsic parameters were studied later during the laboratory tests after confirming the performance of the FBIR procedure on the field.

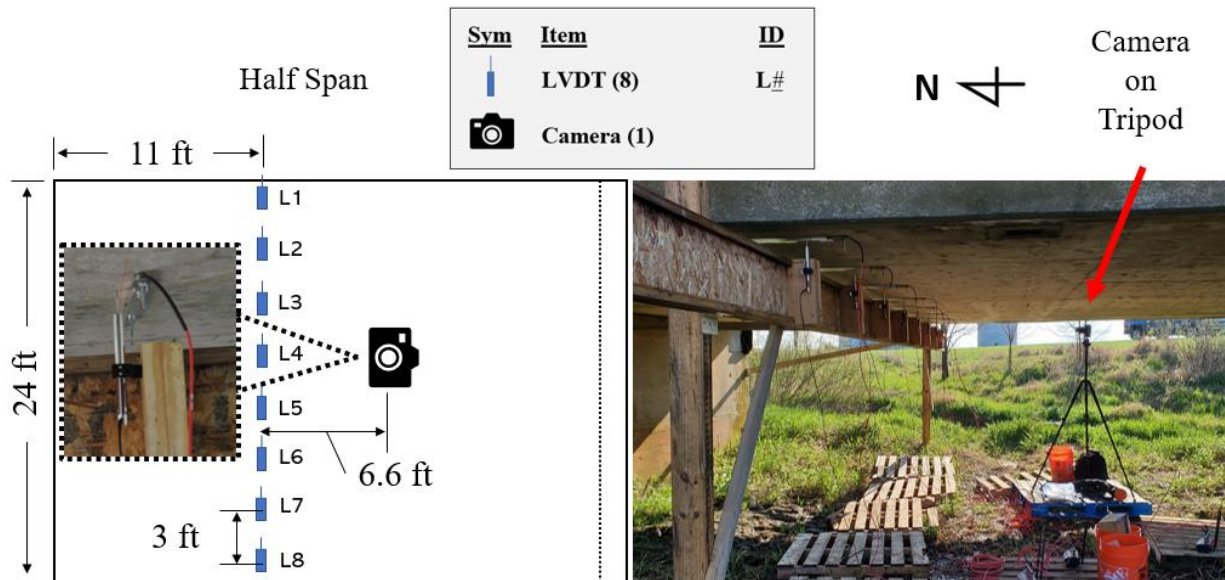


Figure 3.4. Half span plan view instrumentation setup for proof load test of McLean County bridge and image of the field setup

A Nikon D90 digital camera with a 2-inch telephoto lens was used to capture the images during the load test. The camera captures 12.3-MP images and was configured to the maximum zoom to minimize lens distortion in the photographs. Remote control using a wireless push button was used to take the images to minimize unwanted camera movement. The camera was set to a manual option after adjusting the focus for each loading increment during the load test. This ensured that the camera focus was not lost between loading stages and the camera focus would not change with change in lighting conditions. The first image was taken just before loading the bridge. Once the trucks were positioned at their designated locations, a second image was taken before unloading the bridge completely. The process was repeated for every loading stage. All the FBIR results were processed after the proof load test was completed.

3.4.2. Results and Discussions

A load-displacement diagram was generated during the test to evaluate the linearity of the bridge response using the LVDTs, shown in Figure 3.5. A load-displacement plot was also created using the FBIR results during the post-processing for each loading increment to compare with the LVDT at location L4 shown in Figure 3.4. Both load-displacement responses are illustrated in Figure 3.5.

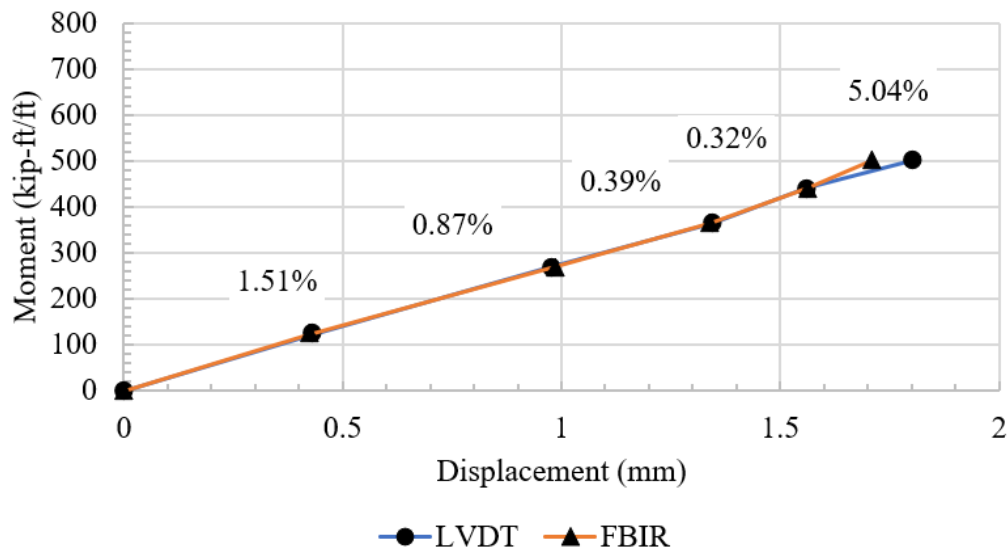


Figure 3.5. Moment-displacement plot for from field test showing the LVDT and FBIR sensor responses

The errors presented in Figure 3.5 varied for each loading stage. This is attributed to the nature of the SURF feature identification algorithm used in the FBIR procedure. Moreover, the averaging technique used to obtain these results introduces some errors when compared with the LVDT, which measures the displacement at a single point. The largest error was about 5% which occurred when the displacement was the largest (about 1.8 mm). When investigating the potential causes for this discrepancy, it was observed that the final loading stage had the most severe camera movement. Increased load levels of the truck also began to damage the roadway at the approach during this loading stage. Any movements of the fixture were not accounted for during the displacement correction. Therefore, there was no differentiation of the camera movement from the

fixture movement, which may account for this discrepancy during the comparison. This issue further demonstrates the disadvantage of needing a stable fixture for field tests.

3.4.2.1. FBIR Threshold Study

Despite the promising results obtained from the field tests, several parameters needed to be investigated to obtain the most accurate results. One influential factor for obtaining comparable and accurate deflection measurements using the FBIR approach was the selection of the ROI. For the results presented in Figure 3.5, the ROI was selected along the LVDT tip to obtain the most relevant comparison with the LVDT value. The LVDT gauge head was also the most distinctive object along the slab giving notably good contrast for the feature identification and consequently, a high probability of accurate matches. The selected rectangular ROI was small enough to avoid large deviations in feature point displacements attributed to three-dimensional (3D) surface changes normal to the plane of the image frame. Moreover, the size and location of the ROI were chosen such that it provided sufficient feature point density for the target location of interest.

To adjust the feature point density, the strongest feature threshold, labeled “metric threshold,” was changed between 100 and 1200 to identify more or fewer features for the estimation, respectively. Ambiguous matches were rejected by applying the nearest neighbor-based matching threshold discussed in Mikolajczyk and Schmid (2005) to use only the best matches below the specified threshold for the estimation. Finally, the feature points could be adjusted manually by excluding outliers that are outside of the percentiles specified by the user. The percentiles threshold was applied in all the cases presented in Figure 3.5 to eliminate irrelevant outliers in the dataset. Points outside the lower and upper percentile threshold were excluded as outliers. The chosen lower and upper percentiles were influenced by inspecting the deviation of the feature points from one another and their location with respect to the LVDT gauge head.

3.4.2.2. Slab Feature Identification Study

It was deemed important to consider the case where there are no LVDTs on the slab. The FBIR procedure should be able to detect feature points of the slab for the displacement estimation without having to rely on installing markers on the bridge. For this, an ROI was selected to capture the slab region alone, without relying on the distinctive LVDT tip which attracted more feature points. The ROI is shown in Figure 3.6 illustrating the displacements of the feature points in millimeters. The ROI selected for this study was located as close as possible to the LVDT tip to compare with the magnitude of the displacements in this region.

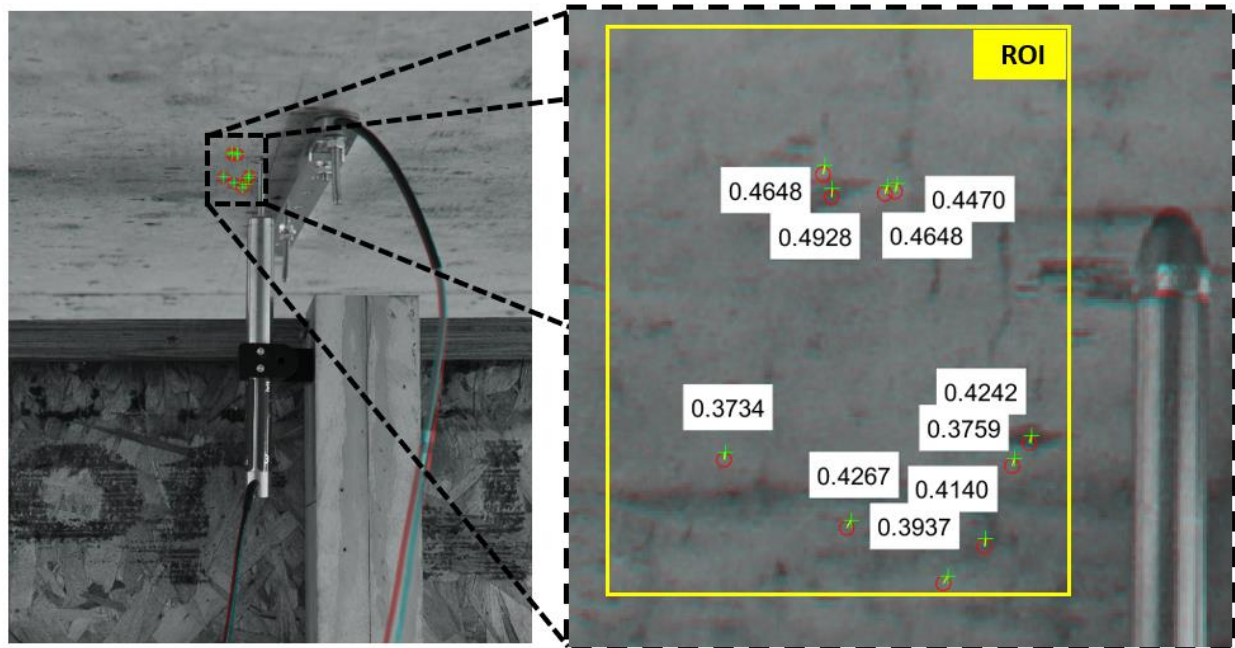


Figure 3.6. Selected ROI for feature identification study of the RC slab

The results from the analysis showed that there were several stable feature points identified on the slab surface. The fluctuation in magnitude of the displacements as one moves from the top of the ROI to the bottom is evident as the actual 3D surface of the slab delves deeper into the frame of the image. It so happens that in this case, the strongest feature points were either above or below

the target of interest. Because of the distinctiveness of the target and the utilization of natural features for the displacement comparison with the LVDT, the ROI was chosen to capture both feature dense areas that are above and below the target of interest to get a better average to compare with the LVDT. The averaging approach was found to yield relatively accurate results in comparison to the single-point displacement of the LVDT. The figure also illustrates the importance of selecting the ROI. If the ROI was chosen only along the top or the bottom feature dense areas, the approximation would yield different results.

3.4.2.3. Parameter Investigation Summary

Table 3.2 presents the displacement accuracy from the slab feature identification study and from changing the parameters based on percentiles threshold, high metric threshold (1200), and low metric threshold (100). For the threshold studies, the ROI was selected along the LVDT tip to compare the effect of the parameters within the same ROI for each loading stage. For the slab feature identification study, the ROI was held constant, and the metric threshold was maintained at a value of 100 for each loading stage to avoid any influence from external parameters.

Table 3.2. Summary of FBIR errors with respect to the LVDT for each approach

ROI	Threshold	Errors for each Loading Stage (%)				
		1	2	3	4	5
LVDT Gauge Head	Metric Threshold = 100	0.55	1.13	2.47	2.11	6.60
	Metric Threshold = 1200	1.64	0.63	3.39	0.33	5.66
	Percentiles Threshold	1.51	0.87	0.39	0.32	5.04
Slab	Metric Threshold = 100	0.31	3.87	3.17	3.24	17.0

Most errors, except for Stage 5 which had some movement in the LVDT fixture, were within 4% regardless of the parameters chosen for the estimation. The results do not give a clear indication of whether the smaller metric threshold performs better than the greater metric

threshold. However, the results do show that the slab features can be used for accurate displacement measurements using the FBIR approach. The FBIR errors presented in the table for each approach help give the user a better approximation of what to expect when working with a similar camera setup and similar field conditions. Still, this setup will likely not be possible for many bridges. Also, measuring several displacements with a single camera is more efficient. Thus, further studies of the parameters and the limitations for this FBIR approach were necessary.

3.5. Laboratory Test Application using FBIR

3.5.1. Laboratory Test Setup

After obtaining satisfactory results from the field tests, the FBIR approach was studied further to test the boundaries of this technology for bridge testing applications. A four-point flexure test was conducted in the laboratory for another IDOT project on a large-scale bridge deck specimen shown in Figure 3.7.a. The specimen consisted of a simply supported RC deck slab on a W14x90 steel girder. The clear span of the specimen was 17 ft. Displacements using conventional LVDTs were measured at approximately one-fifth and one-quarter of the span as shown in the plan view schematic of the field set up in Figure 3.7.b. The LVDT data was recorded using the Bridge Diagnostics Inc. structural testing system. The wireless base station (WBS), wireless node (DAQ), and LVDT sensors used for this test are also shown in Figure 3.7.a. A speckle pattern with a contrasting white background was applied to the surface of the specimen along the mid-span region for an unrelated study of the specimen capacity using DIC. A few pieces of tape were attached to the flange of the beam to use as “artificial” features to compare with the displacement results using the natural features of the beam.

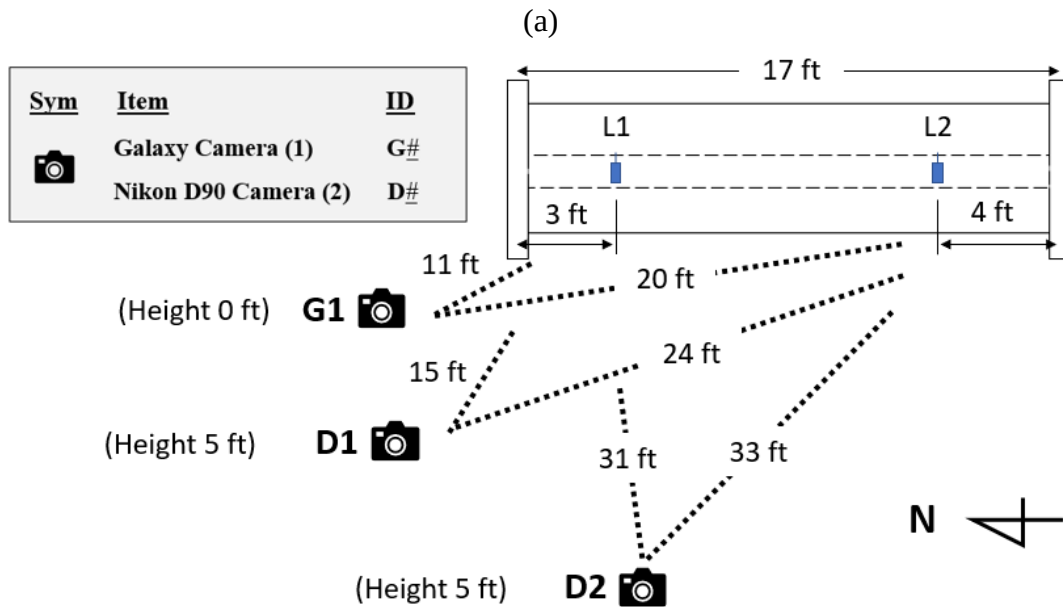
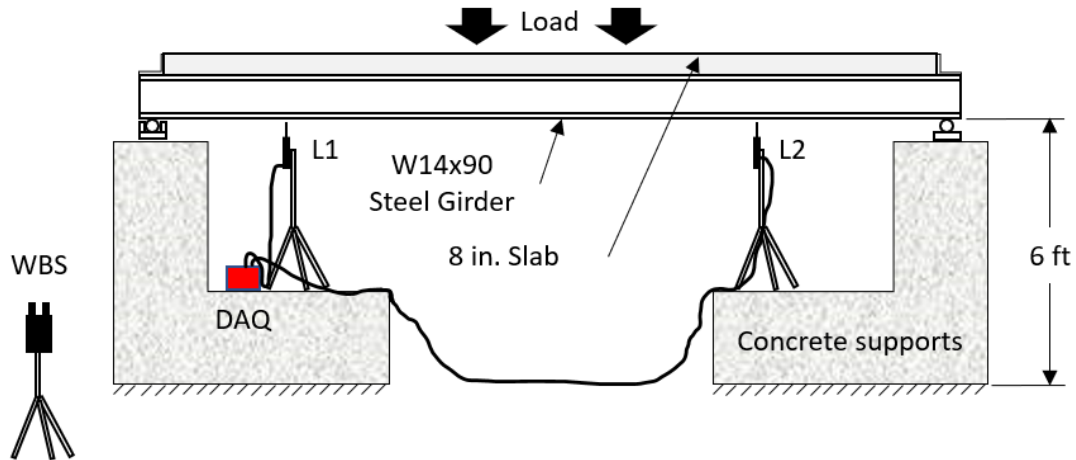


Figure 3.7. Laboratory test setup on bridge deck girder specimen instrumented with conventional sensors showing (a) side view of the test setup, and (b) plan view camera layout

The test specimen was loaded in stages using an actuator to apply the incrementally increasing load for each stage. Various camera configurations were used for each loading stage to assess the effect of camera position on the results. A Nikon D90 digital camera and a Galaxy Note 10 Plus cellphone camera were used for the test. The locations of the cameras for the lab test are shown in Figure 3.7.b. Images were captured using remote pushbuttons for each camera. The test commenced by taking one image from all cameras before the specimen was loaded to obtain a

reference image from which the specimen is assumed to have zero displacement. Several images were then captured as the specimen was loaded to the target load. Once the target load was reached, a final picture was captured from both cameras, then the specimen was unloaded completely. The process was repeated for each loading step.

The Galaxy camera was set up to a 4x zoom to capture the entire span of the specimen without distorting much of the frame using the wide-angle lens option. The camera was positioned on the ground and held via a clamp during this loading step. Because of the proximity of the camera to the target, the angle the camera made with the target locations was approximately 28 degrees. This configuration helped in investigating the potential of having a more elevated bridge and a large angle between the camera and target location during field testing.

The Nikon D90 camera was set up to the maximum zoom range, 3.1x (55 mm focal length), which was sufficiently wide to capture the entire span in the frame of the photo without distorting much of the image. The D90 camera was fixed on a tripod just below eye level to test the accuracy at a less severe angle to the target. The first position for this camera is indicated in Figure 3.7.b as D1 which is to the north of the specimen and close to the Galaxy camera. Both cameras were positioned along the north side of the specimen to imitate a real case scenario for short-span bridge testing.

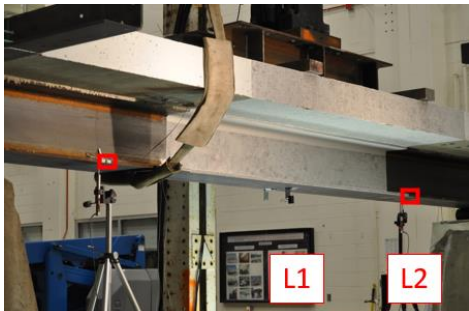
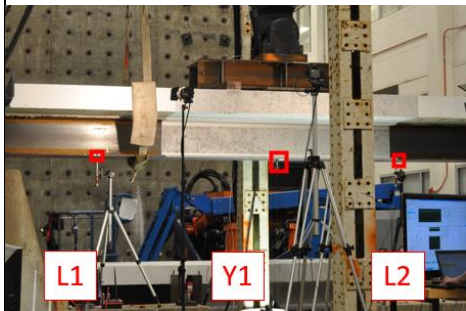
A second configuration was studied for the D90 camera at position D2 shown in Figure 3.7.b This position was used to test the accuracy at a longer distance with a perpendicular field of view with the span. This allowed for the displacement to be estimated anywhere along the entire span with just one iteration of code. This was possible because the scaling factor is the same along the entire span of the specimen. To prove this, the displacement values measured using the image-based approach were verified at three locations along the span with conventional sensors. These

monitoring locations gave a good representation of the overall displacement profile of the specimen.

3.5.2. Results and Discussions

Numerous displacement measurements were made for every loading stage. However, most of the results exceeded the submillimeter range. Therefore, the results yielding the smallest displacements for each camera and location are presented in Table 3.3. Three different errors are presented for each location and camera configuration. Like the field test, the metric threshold and percentiles threshold were adjusted, keeping the ROI constant, to visualize the effects of the parameters on the accuracy of the FBIR procedure.

Table 3.3. Displacement results comparison for laboratory test

D90 (D1)			Galaxy (G1)		D90 (D2)			
								
LVDT (mm)	0.3404	0.5334	0.3683	0.5766	1.1938	2.2733 ¹	1.7907	
Distance to Target	15 ft	24 ft	11 ft	20 ft	31 ft	32 ft	33 ft	
Percent Error (%)	Metric Threshold = 100	9.87	12.70	0.21	13.9	0.09	0.71	7.90
	Metric Threshold = 1200	2.25	5.78	8.32	N/A	7.40	1.13	10.11
	Percentiles Threshold	2.25	4.00	1.30	13.30	0.09	0.71	1.53

¹Recorded using cable-extension position transducer at midspan of the specimen

Figure 3.8 shows the feature matches after applying each of the three thresholds to the Galaxy camera at L1. Applying the metric threshold of 100 gave several mismatches in the dataset which are identified in Figure 3.8.a with a thick border. Applying the metric threshold of 1200 removed these outliers as shown in Figure 3.8.b. However, a single match was still comparatively outside of the median. The percentiles threshold was used to remove this outlier by specifying an upper threshold in the 85th percentile. This eliminated the outlier from the estimation, shown in Figure 3.8.c. The remaining matches were within an acceptable range and were averaged to obtain the final displacement in millimeters. The percent difference from the conventional LVDT sensor at L1 was 1.3% shown in Figure 3.8 under column G1. A similar process was repeated for all camera configurations.

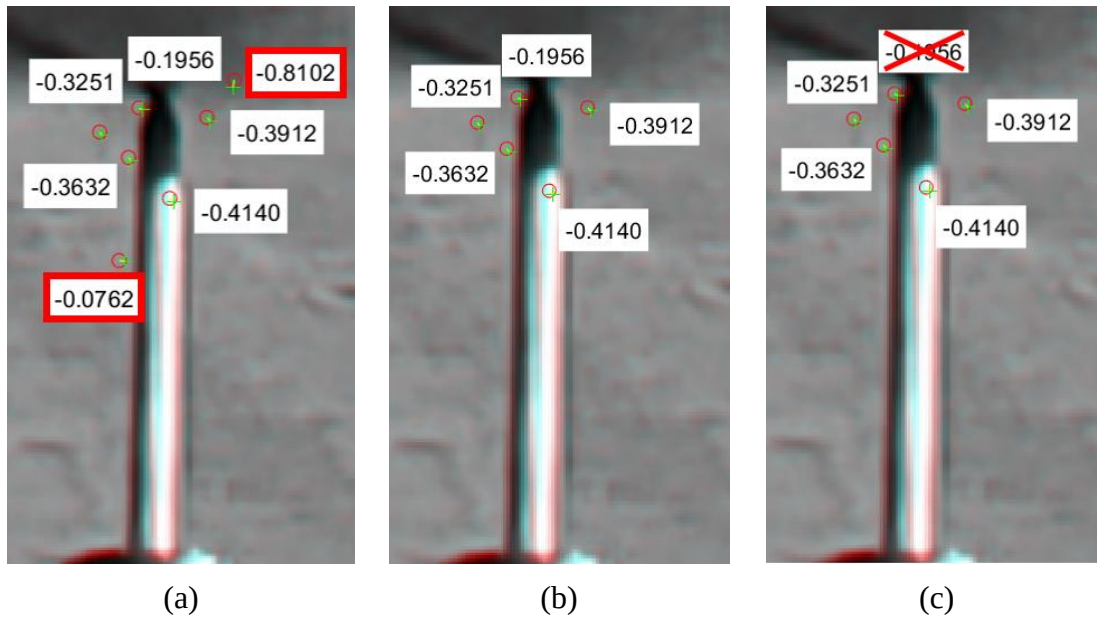


Figure 3.8. Threshold adjustment process leading to the final displacement estimation at L1 showing images of the matched feature points with (a) a metric threshold of 100, (b) a metric threshold of 1200, and (c) an 85th percentile threshold for the Galaxy camera

3.5.2.1. FBIR Threshold Study

From Table 3.3, it is shown that the Galaxy camera at L2 gave the largest errors. The resolution of the image at location L2 was noticeably blurry in comparison to L1. This occurred

because of the profound difference in distance between L1 and L2 within such a small frame of the camera. The locations of the LVDTs were at two opposite edges of the image frame. In such a case, the camera's single point autofocus was not able to focus on both locations at the same time. Therefore, the matching algorithm was not able to properly distinguish the pixels from one another at L2, returning a larger error. This was evident when applying the metric threshold of 1200. No feature point matches were identified using this threshold for L2. When applying the small threshold of 100, only three feature points were identified as inliers despite a relatively large ROI selected for the estimation. Because of this, the metric threshold was increased in intervals of 100 until enough feature point matches were available for the estimation. A metric threshold of 750 was applied for the final percentiles estimation. This ensured that the few features used for the final estimation would be the most reliable.

The Nikon D90 camera at D1 benefited from a greater metric threshold with less feature point density and more robust feature points for the displacement estimation. On the contrary, the greater distance and normal field of view position of D2 gave smaller errors using a smaller metric threshold. This observation appears to show that at greater distances to the target, a greater feature point density is preferred for a more accurate displacement estimation. This could be because the distance reduces the camera resolution which reduces the accuracy and quantity of the feature matches. Having a greater density of feature points for the estimation, in this case, may help improve the confidence in the results. When combining a smaller metric threshold with a greater ROI, a larger density of feature points is identified. The user can then have more confidence when filtering the feature points using the percentiles threshold.

3.5.2.2. FBIR Percentiles Threshold Study

Most errors using the final percentiles approximation were within 4% except for the low-resolution Galaxy cellphone image at L2. After determining the appropriate metric threshold for L2 of the Galaxy camera, the percentiles threshold was chosen based on the deviation of the feature point magnitudes. While incrementally increasing the metric threshold from 100 to 750 in the metric threshold study, two of the three features identified were of the same magnitude. The final metric threshold of 750 removed one of these feature points. However, because of this observation, an upper percentile threshold of 85% was applied to eliminate the third feature as an outlier. This gave the smallest error for the estimation presented in Table 3.3 for L2 under column G1.

The percentiles threshold helped reduce the errors or ensure that the approximations were the most reliable by eliminating the matches that were irrelevant for the estimation. This was done based on an inspection of the feature points. If the feature points were outside of the actual target of interest, they were excluded. If the feature point displacements were notably outside of the average, they were also excluded from the final estimation. Identifying the outliers based on inspection was not always obvious. Careful consideration of the feature point outliers was necessary. Hence, the improved accuracy and confidence using the percentiles threshold came at the cost of greater processing time.

3.5.2.3. Effect of Camera Position on FBIR Accuracy

No submillimeter data was available for position D2. Hence, the smallest displacement was recorded as 1.2 mm at L1 which gave a small error of 0.09%. The displacements of the feature points were of similar deviation and were considered reasonably consistent. This was possibly due to the positioning of the camera being normal to the plane of the beam. An added benefit to this camera position was that the entire displacement profile of the beam could be estimated using the

FBIR technique. To help validate this, a cable-extension position transducer located at the midspan of the specimen was used to verify a third displacement along the span of the specimen. The position of the sensor is labeled Y1 shown in Table 3.3 under column D2. The ROI was selected along the applied patterns because of the inability of the natural features of the steel beam to provide sufficient feature point density for this camera resolution. The average error between the conventional sensors and the image-based approach using the percentiles threshold was 0.78% for D2, which was deemed an acceptable error for field testing applications.

The effect of camera angle on the target was not apparent in this study when comparing position D1 to D2. Because of differences in the magnitude of the displacements, distances to the target object, and reflection of the light from the beam, the effects of this parameter on the accuracy of the FBIR could not definitively be addressed. However, the camera angle did influence the camera resolution for multi-point displacement measurements. This was evident for G1 which had low image resolution at L2 in comparison to the other two cameras which had a less severe camera angle to the target objects. It should be noted that due to the single-point autofocus features intrinsic to most digital cameras, this will be a common issue for multi-point displacement measurements using any camera. By selecting the mid-point of the targets of interest, the effect of reduced camera resolution can be minimized.

The SURF algorithm was much more effective at identifying natural features on the RC slab during the field tests than on the steel beam during the laboratory test. From observation, the distinctive features of the slab made for dependable feature point match in the SURF algorithm. The feature points were much less distinctive on the steel beam. For D1 and D2, the most accurate results were obtained from the artificial features placed on the beam. The camera angle and illumination for these positions made it difficult for the algorithm to achieve reliable feature

matches for the displacement estimation. The Galaxy camera position was low and close enough to the beam to get better distinctiveness around the LVDT gauge head to obtain good displacement estimations presented in Table 3.3.

3.6. Conclusions

Three different tests were presented in this chapter to prove the potential of the Feature-Based Image Registration (FBIR) to return accurate displacement measurements under various field setups and conditions. Field tests and laboratory tests were used to help validate this. The studies demonstrated that the FBIR approach can be very accurate when compared to the less convenient Linear Variable Differential Transformer (LVDT) sensors often used for field tests. The displacements obtained from the FBIR technique and those obtained from conventional LVDT sensors showed an average difference of approximately 5% during the Piatt County bridge study, 1.6% during the McLean County bridge study, and 3.3% during the laboratory tests. The laboratory tests studied the versatility of the FBIR approach by incorporating multiple targets measured from various camera configurations and distances. The ability for FBIR to capture the profile of the bridge displacements was also demonstrated during the final laboratory test, where an average error of 0.78% at distances up to 33 ft from the target was achieved. While the studies show promising results, it was concluded that improvements to the camera resolution and camera parameters could make this procedure much more accurate and versatile for field testing.

A parameter threshold investigation was conducted on the Speeded-Up Robust Features (SURF) algorithm used in the FBIR procedure to study the threshold parameters that are most suitable for deformation measurements using this approach. The metric threshold parameter was found to be the most influential parameter in the SURF algorithm towards the accuracy of the final results. From the laboratory test results, it was observed that a greater metric threshold should be

used when there is sufficient feature point density within the region of interest (ROI) to improve confidence in the displacement estimation and improve the accuracy of the results. A smaller metric threshold should be used when the density of the feature matches is low for the selected ROI. This was not the case for all the laboratory tests and does not reflect the results obtained during the field tests. Hence, a manual inspection of the feature points and their deviation from each other was necessary to choose an appropriate filter to eliminate outliers from the estimation. This improved the accuracy of the FBIR method. However, it came at the expense of more processing time.

Natural features of the slab surface identified using the SURF algorithm were reliably used to obtain displacements which were on average 5.5% from the LVDT results during the McLean County bridge test. Although these results were obtained under an ideal field setup, the results demonstrated the potential for natural features to be used effectively for RC slab-type structures in the FBIR displacement estimation. On the contrary, the laboratory test results showed that the FBIR was not as effective at capturing distinct natural features on the steel beam. The main reason was associated with the lighting conditions and camera resolution having an effect on the distinctiveness of beam features. In such cases, either more reflective components should be applied on the bridge or greater resolution cameras should be used.

It should be noted that the code developed in this study to conduct the necessary FBIR steps does not reflect the work of others. This research did not address the effects of weather and humidity on the results which are likely to affect the feasibility of the FBIR for field testing. The intrinsic parameters of the cameras were not studied which may improve the image quality for better performance using this approach. For simplicity, no attempts were made to implement improvements of the feature detection and matching algorithm in the code. These considerations

may help improve this method's accuracy and reliability as a relatively simple and easy tool for monitoring the displacement of bridges during field tests.

CHAPTER 4: SATELLITE IMAGING FOR STRUCTURAL MONITORING OF BRIDGES

The application of FBIR methods for measuring bridge displacements is an advantageous tool for bridge load testing. The method shows great potential and has much room for improvement for field testing applications. However, there are still several challenges which make load testing challenging and inconvenient for assessing the condition of a bridge. Load testing will almost always disrupt traffic, causing more delays and inconveniences to daily commuters, especially for more populated cities. Taking field measurements and conducting the load tests can also be disrupted by common weather conditions such as rain and snow in some regions, making coordination and planning difficult and producing less reliable data from the sensors. Hence, the satellite remote sensing technology is explored in this chapter as a novel displacement measurement strategy which can be more convenient for structural monitoring applications.

4.1. Literature Review

Satellite technology has been a powerful and useful Earth observation tool used for a multitude of applications in modern society. Today, methods for obtaining precise deformations of the Earth's crust have enabled us to monitor large and small regions of the Earth with high spatial detail. The technology driving this area of research is Synthetic Aperture Radar (SAR). SAR is a means of obtaining information about the Earth by measuring the distance between the sensor and the Earth (phase) and the strength of the radar response (amplitude) to any given target (Uys, 2016). However, the conventional SAR method is not sufficient for obtaining detailed Earth information since it measures three-dimensional (3D) targets in two-dimensions (2D). By using two SAR images over the same target at slightly different positions, an interferogram can be generated which shows a great correlation of phase with topographical information of the target

region (Ferretti et al., 2007). This process is recognized as Interferometric SAR (InSAR) which introduces a third dimension (time or space) to the system (Uys, 2016). By removing the shift in phase related to the topography, the remaining pattern shows surface deformations between the two acquisition dates. This procedure is known as Differential InSAR (DInSAR). Although a very useful strategy, the accuracy and reliability of DInSAR are significantly influenced by decorrelation and atmospheric artifacts (Songbo et al., 2020). The method has been overcome by an even more powerful method called Multi-Temporal InSAR (MT-InSAR) which mitigates the decorrelation and atmospheric effects by utilizing so-called Permanent Scatterers (PS). These PS are points with stable backscattering identified through a multi-temporal analysis and are unlikely to change throughout the analysis (Songbo et al., 2020). By using multiple images in time-series analysis, the PS can be used to estimate more reliable displacements of regions and objects.

In recent years, InSAR has seen rising interest from researchers and institutions across Europe and Asia for monitoring bridges. InSAR's recent advancements and methods have been employed and validated in several works for studying deformations prior to bridge collapse (Sousa & Bastos, 2013; Crosetto et al., 2016; Selvakumaran et al., 2018). The results of each method utilized in these studies suggest that InSAR satellite data can be used and processed to serve as an early warning technique for bridges experiencing abnormal deformation behavior or to detect damage in the structure. Several different methods have been developed and explored to obtain near-real time accurate displacement measurements of bridges from satellite imagery using InSAR. Some of these methods will be summarized in this review as they have significant potential for bridge displacement monitoring to minimize the risk of catastrophic collapse in the future.

4.1.1. Measurement Procedures using InSAR

To measure displacements with InSAR, a set (stack) of images should be chosen from one or several satellites pertaining to the same spatial location of the target. This technique is what is recognized as MT-InSAR which utilizes repeat pass radar signals from satellites to measure displacements of surface deformations with high spatial detail over a given period (Galloway et al., 2000). The target can be anything from the motion of the Earth's crust to deformations of infrastructure. From the stack of images, points are identified using a series of algorithms based on methods for converting radar signal responses to a digital elevation model (DEM). These points identified within the image pixel are then compared with a chosen reference (master) image from the stack to capture any changes in geographic location. These changes can be millimetric in magnitude. The deformations can then be adjusted for errors in atmospheric conditions and other effects that influence the accuracy of the results.

Several methods of analyzing coherent pixels from a network of SAR images for determining surface deformations were developed. One technique exploits a small subset of image pixels in an image framework to identify and measure permanent scatterers (PS), stable image points characterized by their coherence, for analyzing ground motion (Ferretti et al., 2000). This method is recognized as Persistent Scatterer Interferometry (PSI), which computes differential interferograms in reference to a master image for performing phase analysis on pixels demonstrating stable SAR response throughout the stack of images (Bovenga, 2018). Several different PSI approaches have been developed and are presented in Crosetto et al. (2016). The PSI method has proven to be reliable for measuring small displacements over time in urban environments where the point density of PS is high (Bovenga, 2018). The use of this technique becomes less reliable for monitoring deformations of the Earth's surface in rural areas where the density of PS is much less. However, this method can be reliable for monitoring bridges where the

density and coherence of these identified points on a bridge can be high and small-scale features may be of interest.

Some limitations to the PSI approaches are worth mentioning. Since these methods rely on reflective surface objects to identify as PS, PSI is limited by the amount of reflectivity the bridge components give off. Furthermore, the location of these PSs is not chosen, and the study is limited to the deformation of identified features at arbitrary locations on the bridge. One way around this is to install reflectors which can be placed at points of interest on the bridge to be tracked by the satellite as shown in Quin and Loreaux (2013). Greater resolution X-band SAR data also aid in diminishing this issue (Crosetto et al., 2016). PSI utilizes line of sight (LOS) information which is the one-dimensional (1D) distance between the 3D target to the satellite radar. This means deformations must be analyzed using multiple sensor time series from various satellite passes as presented in Milillo et al. (2019) or other means to obtain coordinate displacements in 3D.

Another method utilized for monitoring displacements is the Small Baseline Subset (SBAS) InSAR method proposed by Mora et al., (2002). This method analyzes temporal changes in the deformation of the Earth's surface using DInSAR and attempts to limit decorrelation phenomena often seen in PSI methods (Selvakumaran et al., 2018). The approach makes use of the single value decomposition (SVD) method applied for solving the values of non-linear deformation for each acquisition time for increasing the datasets used in the analysis. It also limits spatial decorrelation and topography errors by utilizing interferometric data pairs that have a small spatial separation between orbits (Mora et al., 2002). The original approach was developed to monitor larger spatial deformations with a dense set of data points such as the improved SBAS-InSAR approach proposed by Wang et al. (2019) for monitoring landslides in locations with high

vegetation coverage. However, the application of SBAS is less suitable for infrastructure monitoring.

A combined approach presented by Hooper (2008) considers both SBAS and PSI approaches which have been shown to maximize the spatial sampling of useable signal leading to more reliable deformation signal resolution. This is done by combining filtered SBAS pixels with unfiltered PSI pixels, thus improving the density of points in the image for processing (Hooper, 2008). A study which applies this proposed method for bridge monitoring is presented in Sousa and Bastos (2013) where they detect a high rate of motion in the tested bridge prior to its collapse.

4.1.2. Background on Selecting Suitable Satellite Data

There are several considerations when selecting a satellite mission to use for MT-InSAR analysis. The most suitable satellites for monitoring bridge deformations are those with high spatial resolution. Having greater resolution decreases decorrelation between images and increases the accuracy and density of the point displacements in the analysis. While bridge monitoring is possible using lower resolution C-band SAR such as the European Sentinel-1A/B satellites, the greater frequency X-band SAR sensor can detect more detail in an image. Some popular satellites which carry these X-band sensors are the TerraSAR-X/TanDEM-X (TSX/TDX) and COSMOS-SkyMed (CSK) satellites. These satellite missions have been applied extensively in the research for bridge monitoring applications. This is not just because of their sensor resolution, but also because of the several other benefits which make them suitable for infrastructure monitoring.

Another important feature of the satellites which makes them ideal for bridge monitoring is the satellite revisit time. This is the time it takes for the satellite to capture two consecutive acquisitions over the same target (Battagliere & Virelli, 2019). The CSK constellation mission for example is a combination of four satellites which orbit the earth synchronously, making it possible

to achieve a revisit time of 12 hours. This allows for an extremely large amount of data to be collected each day and night. In comparison, the TSX/TDM missions have a revisit time of 11 days and sun-synchronous orbit which takes images during the same time of day throughout the monitoring period. The shorter the revisit time, the more suitable the satellite constellation is for bridge monitoring and PSI analysis.

There are numerous imaging modes available for the satellite sensors. The imaging modes available for the TSX satellite are presented in Figure 4.1. Each mode comes with a different spatial resolution which makes the sensors useful for a variety of tasks. The spatial resolution is evident from the figure, showing the high-resolution spotlight mode in comparison to the lower resolution ScanSAR mode which covers more area. It should be noted that the scene size in the Spotlight imaging mode is highly dependent on the incidence angle (the observation angle between antenna direction and target). The most common and basic imaging mode is the StripMap (SM) mode which takes a sequence of pulses while the antenna is fixed in elevation and azimuth (lengthwise distance with the antenna direction).

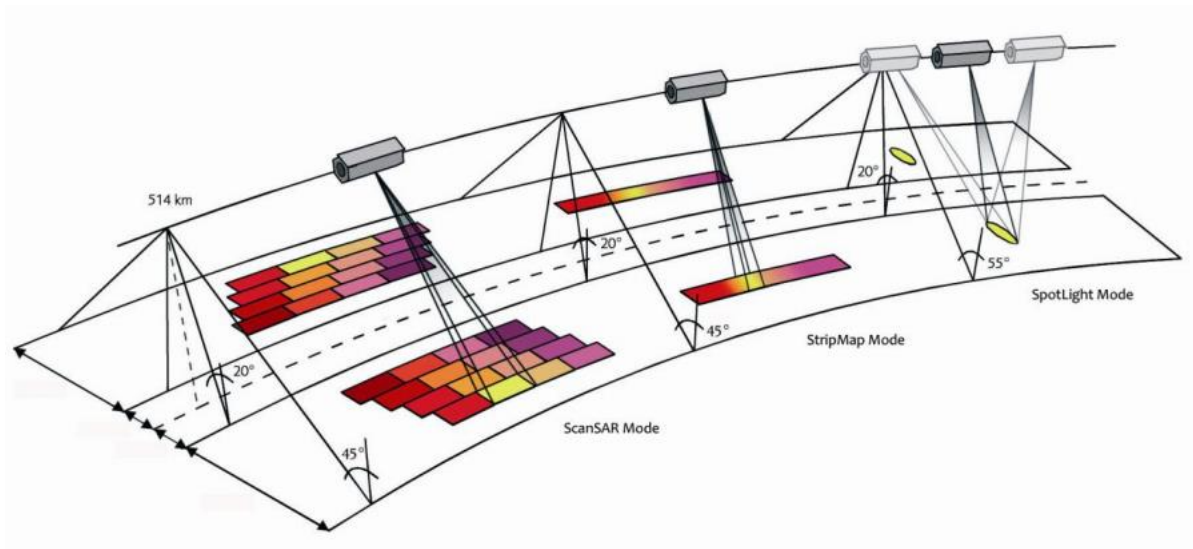


Figure 4.1. TSX satellite imaging modes (acquired from Airbus Defense and Space (2015))

When ordering data from any data provider, the images must be acquired with the same incidence angle and orbit to conduct interferometry. The chosen incidence angle depends on the displacements being monitored and the angle the satellite sensor makes with the bridge. Because LOS displacements are obtained from the results of a typical MT-InSAR analysis, it is useful to consider the location of the satellite at the acquisition time relative to the target. With a greater incidence angle, there are larger components of horizontal displacements being monitored than vertical displacements. If available, the analysis might benefit by using the ascending pass and descending pass of the satellite over a given target. The image stack being processed for the MT-InSAR analysis should all be the same pass. Moreover, the acquisitions period being studied must consider the launch date of the satellite to ensure coverage over the area during the period.

Some other useful terminology for processing should be introduced before proceeding with some applications and examples. Images are usually processed in the slant range, meaning that the image has some degree of change in elevation and distance between the antenna of the sensor and any point in the image. This can be observed in Figure 4.2 where the slant range is the centroidal distance (distance to the center of the AOI) between the antenna and the target. The temporal baseline of an image pair is the time between the master and slave image acquisitions. The normal baseline is the perpendicular distance between the satellite images taken at two different times over the same area. This is denoted as ‘Interferometric baseline’ in Figure 4.2. The shorter this is, the smaller the effect of decorrelation in the interferogram. The doppler centroid is a measure of antenna direction to the target. If the doppler centroid is zero, the antenna is pointed perpendicular to the flight direction towards the target. The azimuth direction and ground range direction of an SM image are also presented in Figure 4.2. The processed images will be in reference to these coordinates.

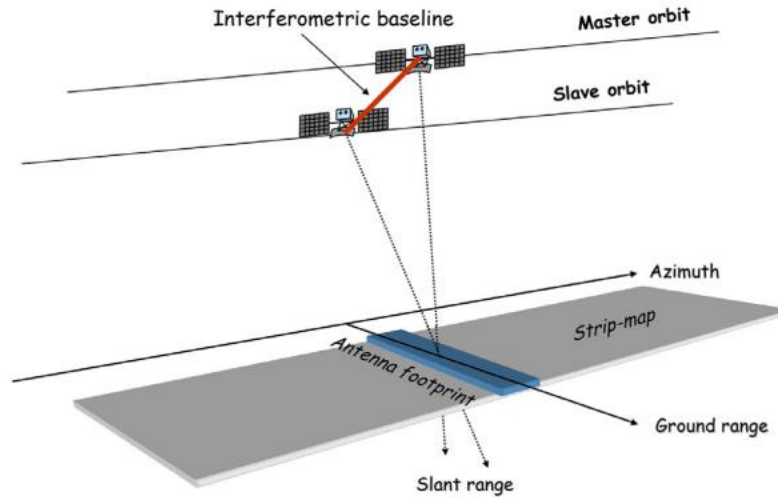


Figure 4.2. Acquisition geometry of an image pair (acquired from Perissin (2018))

It is important to note that using a small number of images for the analysis will bias the temporal coherence to higher values, making it difficult to distinguish between stable and noisy points (Perissin, 2018). Hence, the temporal coherence of points and their corresponding displacement magnitudes are less reliable with a decrease in the image stack. This is useful to understand for bridge monitoring applications when considering the number of images for the analysis. A larger stack of images is always better for more accurate and reliable results.

4.1.3. Study of Publications Applying InSAR to Bridge Monitoring

Milillo et al. (2019) presented a method for monitoring and assessing bridge deformations using InSAR through a comprehensive analysis of the Morandi Bridge collapse in Genoa, Italy. The method used for this analysis was a combination of MT-InSAR to determine LOS displacements and Markov Chain Monte Carlo (MCMC) 3D inversion to solve for the 3D displacements and obtain relative errors. In this study, over 500 images were processed over a period of 9 years prior to the collapse of the bridge on August 14, 2018. These included ascending and descending paths from the Italian constellation COSMOS-SkyMed (CSK) and European

constellation Sentinel-1A/B satellite missions. By using such a high quantity of satellite images, precise data were obtained suggesting that the bridge was experiencing abnormally large displacements in one of the bridge decks several days prior to its collapse.

For the analysis, the satellite data from each path were used to capture the LOS deformations from several different perspectives of the bridge. This gave the research team the ability to obtain time-series bridge deformations in 1D and convert them to 3D bridge displacements. The satellite data were processed in SARPROZ software, a tool developed for implementing various InSAR and MT-InSAR techniques (Perissin, 2021). The relative displacement observed in deck 9 compared to the other two decks illustrates the extraordinary deformation trend leading to the collapse of the bridge, as depicted in Figure 4.3.

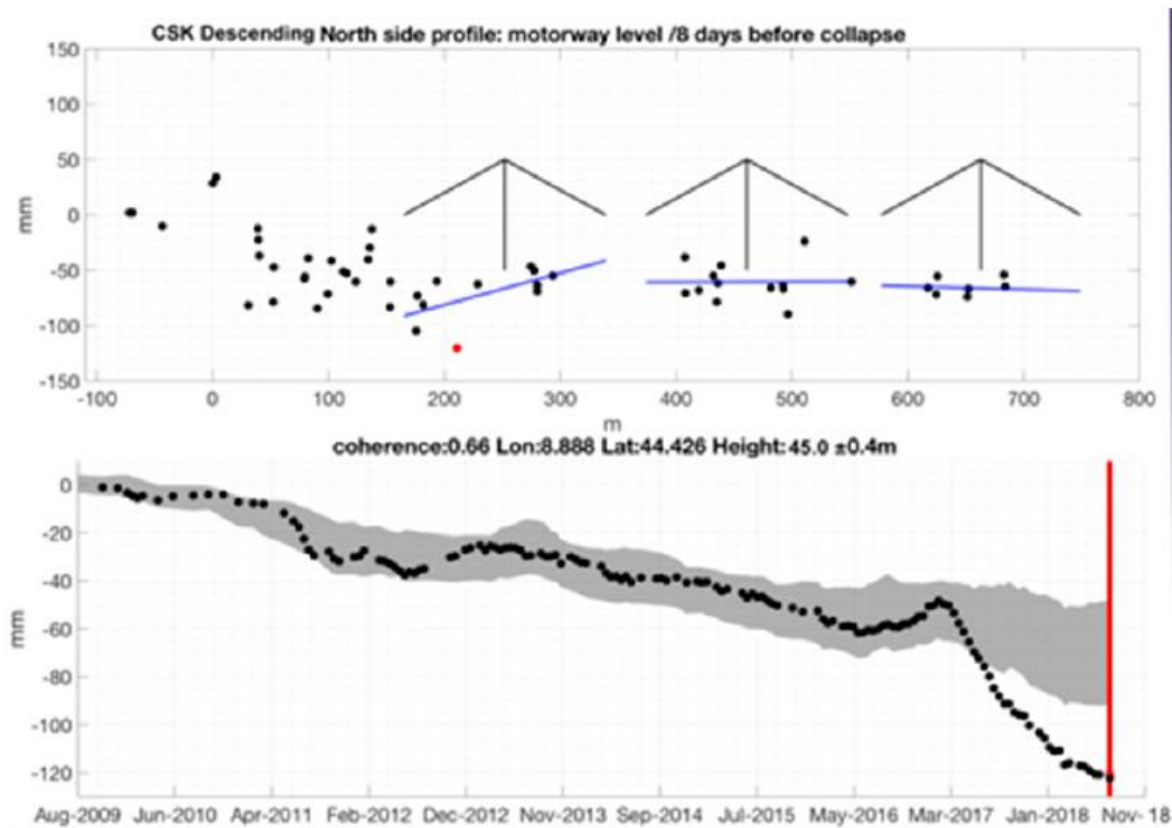


Figure 4.3. Time-series deformation for descending CSK satellite with roadway level shown on the top falling 8 days before the collapse (adapted from Milillo et al., 2019)

The evaluation of the Morandi Bridge using InSAR data with the MCMC technique resulted in displacement rates with millimetric accuracy. Evaluating the displacement trends overtime proved to be important in identifying the onset of nonlinear behavior days in advance of the collapse. The satellite data used for their evaluation had spatial resolutions of 4x22 meters and 3x3 meters and revisit times of 16 and 6 days for the Sentinel-1A/B and CSK satellite missions, respectively (Milillo et al., 2019). This proved to be sufficient for identifying the critically damaged area of the bridge prior to the collapse.

Another study presenting the potential of InSAR for bridge monitoring is discussed in Sousa and Bastos (2013). The combined SBAS and PSI approach was used in this study to show the increased rate of deformation of the Hintze Ribeiro Centennial Bridge prior to its collapse in March 2001. A stack of 52 European Remote-Sensing Satellite (ERS) SAR images acquired over a period of 5.5 years was used for their analysis. The analysis method was successful in identifying hundreds of stable points in the AOI. Only four of these points were used to capture the significant deformation occurring at the location of the collapsed section of the bridge. The LOS velocities were used in their analysis to capture this trend. Three-dimensional displacements were not obtained in their study, and yet, the LOS displacements trends provided enough information to know that the rate of deformation for the identified points near the damaged pier was anomalous. Monitoring the bridge deformations using this satellite data prior to the collapse could have prevented the collapse of the structure, and moreover, the loss of numerous lives. Figure 4.4 illustrates the identified PS from the analysis and their relative movement approaching -20 mm/year before the time of collapse at pier 4 (P4).

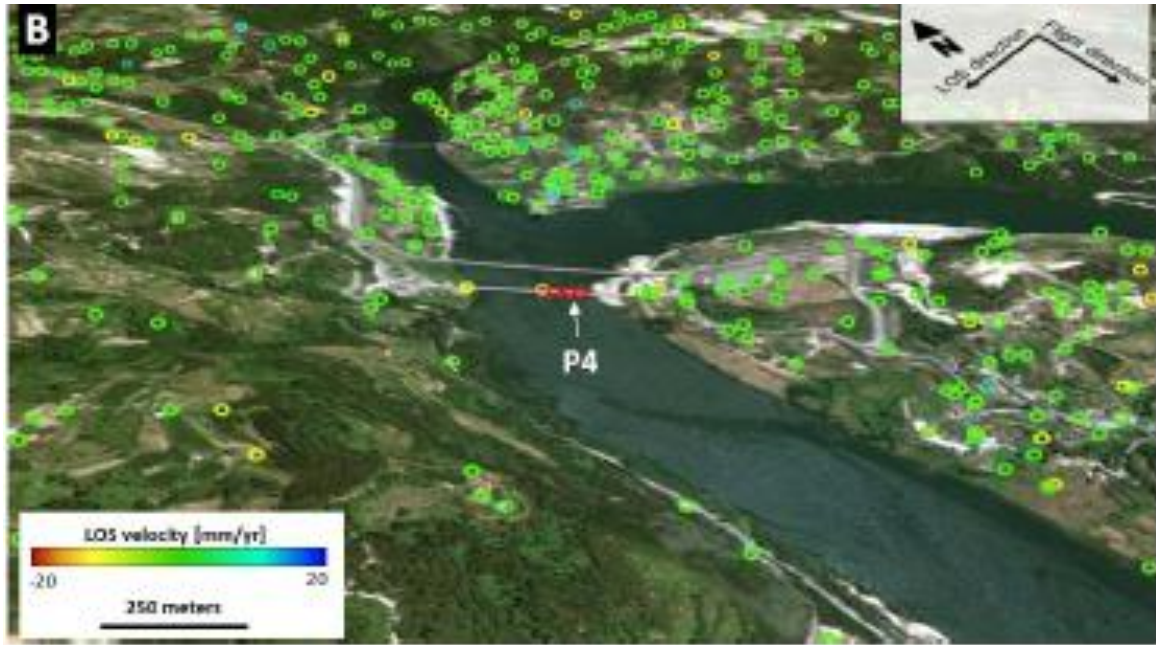


Figure 4.4. Average linear deformation rate with red points denoting the collapsed pillar labeled P4. The figure was generated by viStaMPS (adapted by Sousa & Bastos, 2013)

Another case study presented by Selvakumaran et al. (2018) introduced an InSAR method that was used to formulate an early warning system for bridges to monitor the status and risk of a bridge failing. The study used the SBAS technique and applied it to the Tadcaster Bridge which collapsed on the 29th of December 2015. A data stack of 48 TSX images acquired over a period of 1.5 years was used to evaluate the LOS movement of the identified PS on the bridge. Because of the tree foliage blocking the LOS to the bridge deck, the PSI technique returned an insufficient density of points on the bridge for the analysis (Selvakumaran et al., 2018). For this reason, the SBAS approach was more appropriate and returned eight points on the bridge for the analysis. The movement of the eight scatterers was monitored, and the results of the analysis are shown in Figure 4.5. The PS labeled 'b' in the figure was located near the location of the failed pier. This point shows the estimated displacement trend leading to the collapse of the bridge. The green lines illustrate some predefined boundaries for detecting potential outliers in the analyzed dataset. The

final two acquired dates show that scatterer b exceeded the limits, meaning that the bridge should be inspected, and traffic ceased to identify the cause of this trend.

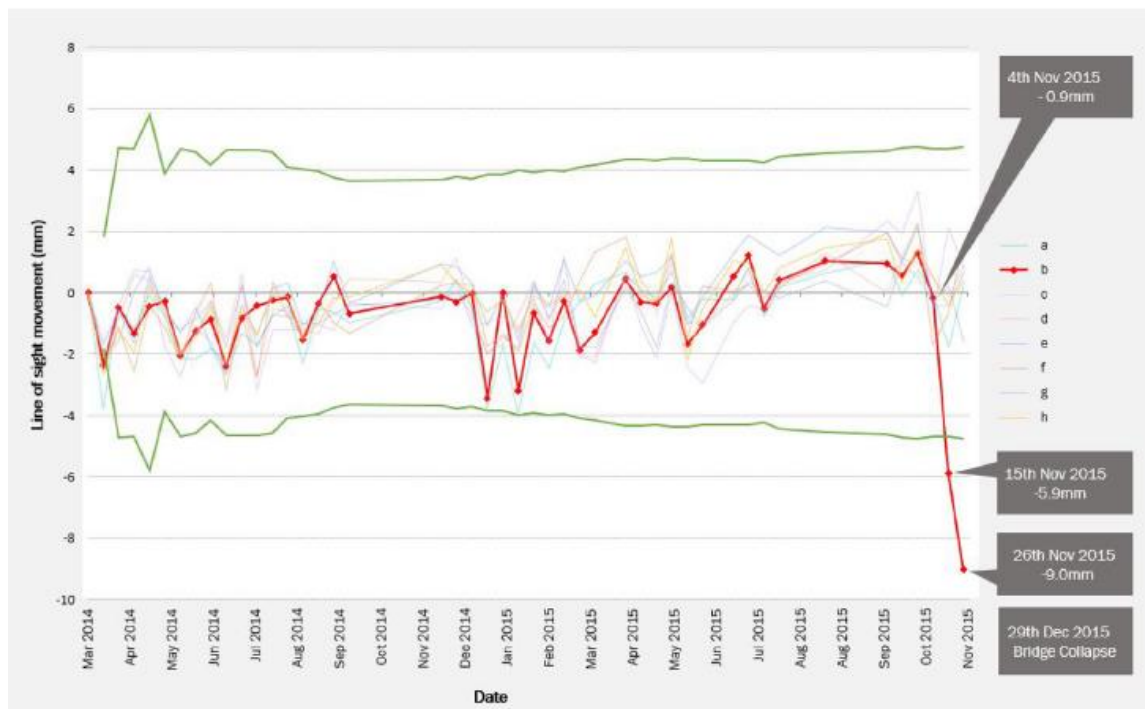


Figure 4.5. Movement of the 8 scatterers identified on the bridge over a 20-month period (adapted by Selvakumaran et al., 2018)

InSAR can also be used to study thermal deformation trends. A case study of the thermal displacement of Lion Gates Bridge was conducted to examine the effectiveness of InSAR for thermal displacement monitoring of bridges (Cusson et al., 2012). The researchers made use of the PSI method for obtaining displacements in the structure. The images were processed over a period of two years. Thermal correlation coefficients for the temperature change were obtained and validated against a similar suspension bridge called the Tamar River Bridge to compare the results. A correlation coefficient of 0.93 was obtained from the InSAR analysis that compared well with Tamar River Bridge which yielded an absolute correlation coefficient of 0.92 (Cusson et al., 2012). Overall, the results demonstrated the sensitivity of these sensors for obtaining thermal displacements on a bridge.

4.2. Bridge Deformation Studies using MT-InSAR Analysis

4.2.1. Study of the Memphis Bridge Damage

4.2.1.1. Bridge Description

To confirm the potential of this technology and to explore the simplicity of applying InSAR to bridge deformation monitoring, an MT-InSAR analysis was performed on a multi-span double through-arch steel truss bridge located in Memphis, TN. The bridge is called the Hernando De Soto Bridge. It was closed indefinitely on May 11, 2021, after inspectors identified a large fracture (shown in Figure 4.6) in a critical bottom steel tie girder. The bridge is heavily trafficked, having average daily truck traffic (ADTT) of 35,000. The bridge also has several decades of service being opened to the public in 1973. The superstructure has a total length of 9,431 ft with two maximum span lengths of 900 ft shown in Figure 4.6. The fracture occurred in the west span close to the intermediate pier. The bridge fracture was evident as early as 2016 based on some drone footage captured by a bystander during that year.

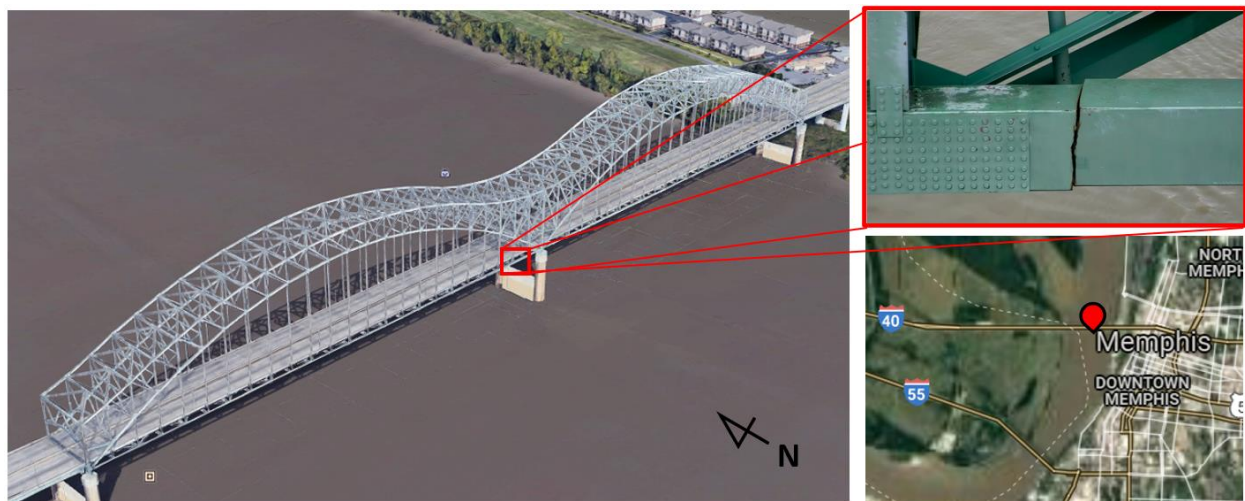


Figure 4.6. Image of the Hernando De Soto Bridge located in Memphis, TN showing fracture of bottom steel tie girder (image captured from Google Earth)

The missed fracture during previous inspections resulted in the termination of the inspector responsible for inspecting that portion of the bridge during the previous two years (ARDOT Report, 2021). Prior inspection data for the bridge suggested that the bridge had been rated in poor condition several consecutive years in advance of the complete fracture of the tie girder (U.S. Department of Transportation, 2020). Overall, the results of this incident helped drive the state Departments of Transportation (DOT) to improve inspection procedures and reorganize these procedures to prevent this from occurring on other bridges.

The bridge was selected for this study to determine whether signs of damage could be detected through an MT-InSAR analysis of the bridge. The steady, long-term damage which had accrued over the years made this bridge a suitable candidate for testing this deformation monitoring method. Moreover, the bridge is made of steel, which allows for a great density of PS to be identified on the bridge. This is particularly advantageous when applying the PSI MT-InSAR analysis method, which relies on a good density of points to make reliable estimates on the deformation of the bridge.

4.2.1.2. Image Processing and Software

The satellite coverage was very limited for the study region. Only seven StripMap (SM) TSX/TDX images (acquired over a six-month period) which qualified as “Historic Data” (over 18 months old) were available through proposal MTH3776 of the DLR free of charge. A photograph of the TSX imaged area is shown in Figure 4.7. The images were ordered through the EOWEB GeoPortal (EGP <https://geoservice.dlr.de/egp/>) and made available for download via FTPS using the FTP client ‘FileZilla’. After downloading and unzipping the images into a folder on the computer, they were ready to be processed.



Figure 4.7. TSX SM image over Memphis used for Memphis Bridge analysis

The TSX/TDX satellites were launched in June 2007 and June 2010, respectively, by the DLR and are still in orbit as of the year 2021. The images were acquired during the ascending pass with an incidence angle of approximately 45 degrees. The images used for the analysis covered the period of June 2019 to January 2020. They were acquired at different intervals of time, with a time gap between some images as great as 3 months and others as few as 11 days.

The SARPROZ software was used to process the images. The processing steps used to perform the analysis in SARPROZ are made available on <https://www.sarproz.com/>. The images were processed using the Small Area Processing (S.A.P) feature of SARPROZ which allowed for a more focused region of the bridge to be analyzed quickly with few images. With the limited quantity of images, it was difficult to accurately estimate the necessary parameters using the general “Site Processing” feature. This made the S.A.P the best option for measuring the bridge displacements.

The bridge was analyzed in interferometric pairs, grouping the image with the earlier acquisition date with the subsequent image in the stack. The image with the earlier date was always chosen as the master image. There was a total of six pairs for the analysis. Each processed pair would result in a cumulative displacement over the time between the acquired images. The total cumulative displacement over the 6-month period was determined by summing up the cumulative displacements over each processed pair of images in the stack.

The cumulative displacements are estimated through the measured deformation trends by the software. The following equation is used for converting the deformation trends to cumulative displacements

$$d = \frac{X}{12} \times (T) \quad (\text{Eq. 4-1})$$

where d is the cumulative displacement, X is the amount of time (in months) between the acquisition dates of the image pair, and T is the measured displacement trend in millimeters per year.

4.2.1.3. Results

The plan view dimensions, showing the damaged region of the bridge and corresponding location on the reflectivity map generated by the software are shown in Figure 4.8. These images were used to help evaluate the results after the analysis by ensuring that the points defined by the software in the reflectivity map correspond to the points on the bridge in plan view. The figure also helps illustrate where the damage occurred on the bridge for both the Google Earth image and the reflectivity map.

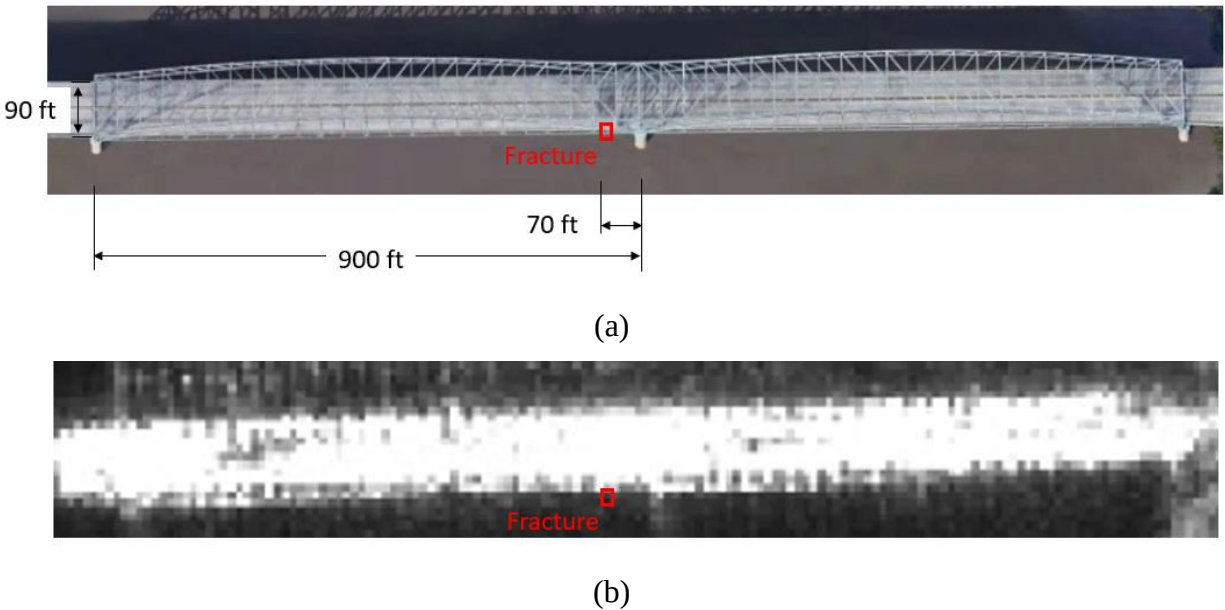


Figure 4.8. Plan view of the bridge (a) Google Earth image showing the location of fractured girder and (b) corresponding reflectivity map

The displacements were measured by averaging the displacement of three or more relatively consistent PS identified on the bridge within an area of the bridge. The scatterers that

were most consistent with each other were combined in the average to obtain a single value of displacement for different zones on the bridge. The density of the PS identified on the bridge with their corresponding magnitude is presented in Figure 4.9. When geocoding the PS to Google Earth, the points shifted towards the east slightly. The pier locations indicated in Figure 4.9 reflect this shift in points over the reflectivity map.

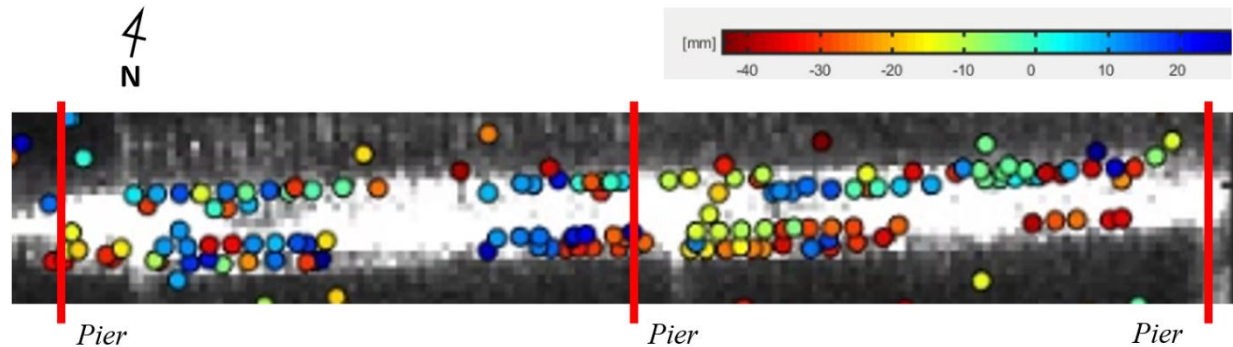


Figure 4.9. Density of PS on Hernando De Soto Bridge

The averaged cumulative displacements over the entire acquisition period are presented in Figure 4.10. The figure shows the average displacements in millimeters and their relative directions (towards (+) or away (-) from the satellite) with respect to the satellite-looking direction. The bridge has been split into eight regions to better compare the maximum absolute deformation of any region of the bridge. The fracture location is indicated on the structure by the triangle in Figure 4.10. The table is aligned with the piers of the bridge to show where the averaged points are located relative to the region of the bridge in the table. As observed from the data, the largest movement was found in the region of the bridge where the fracture was located. The magnitude of the displacement in this region is at least 63% greater than the maximum deformation of any other region on the bridge. Furthermore, the south side of the bridge shows more movement than the north side of the bridge. This information suggests that there may be damage in the south-west portion of the bridge that should be inspected.

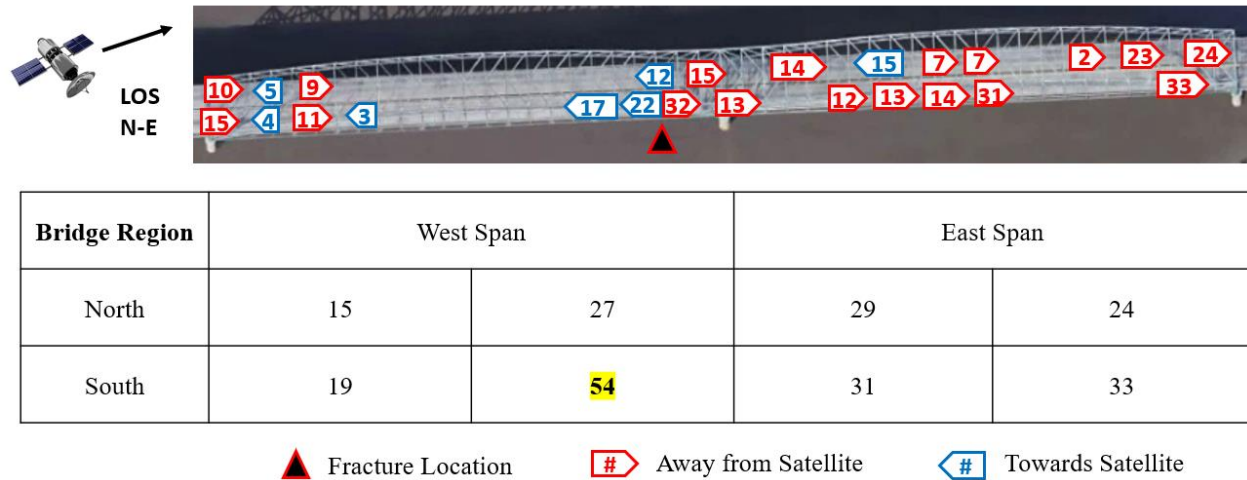


Figure 4.10. Final averaged cumulative bridge displacement values throughout the entire studied duration in millimeters

4.2.2. Study of the Chicago Outer Shore Drive Bridge Damage

4.2.2.1. Bridge Description and Satellite Data Processing

The Outer Shore Drive Bridge of Chicago, IL was studied to see if the MT-InSAR PSI analysis of the data can give an indication of damage in the bridge prior to its damage. The main span of the bridge is a movable double-deck bascule bridge with several approach spans that are steel multi-girder type. The original construction of the bridge was completed in 1937. However, the approach spans were constructed in 1986. The bridge is consistently ranked among the topmost traveled structurally deficient bridges in Illinois (American Road & Transportation Builders, 2021). The bridge was closed on February 11, 2019, due to a large crack at the expansion joint of one of the approach spans. The location of the crack and an image of the fully developed crack on the day of the closure is shown in Figure 4.11. The full crack may have been caused by the extreme temperature fluctuations observed during this period preceding the damage. The temperature increased by as much as 68 degrees Fahrenheit in just a few days.

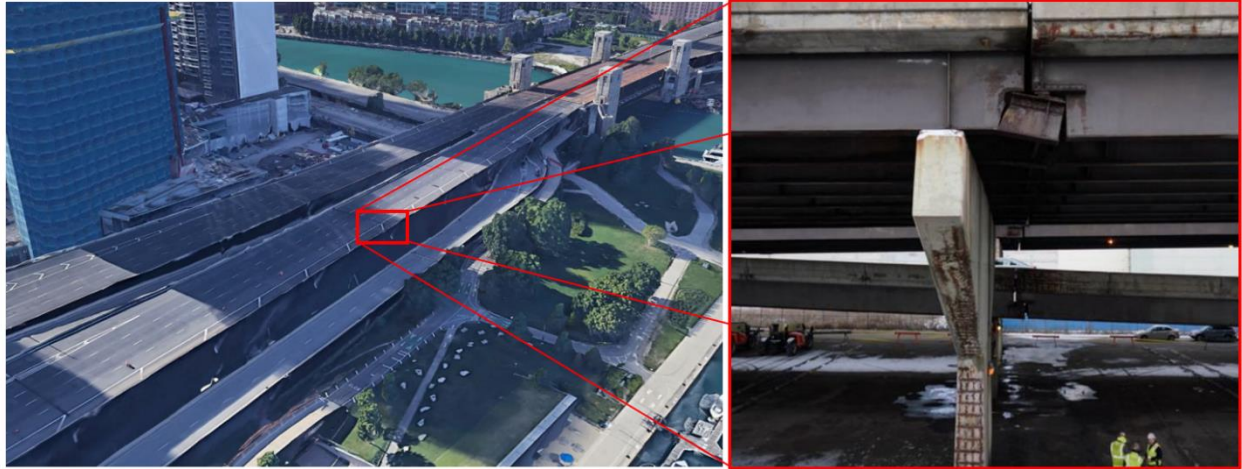


Figure 4.11. Outer Shore Drive Bridge Chicago, IL and crack at the expansion joint of the approach span

There were 13 TSX images available in the SM mode through the DLR free of charge that was used for this analysis. One of the images is presented in Figure 4.12. The images covered the period of September 2018 to February 2019. The images were acquired during the ascending pass with an incidence angle of approximately 45 degrees. Although 13 images is a low quantity of images for the analysis, the images were taken at 11-day intervals. With increased revisit time, the coherence of the points tends to be greater. Thus, having more reliable points to use for the analysis. It was decided to use the site processing module in SARPROZ for the analysis.



Figure 4.12. TSX SM image over Chicago used for Chicago Bridge analysis

4.2.2.2. Results

The density of PS over the approach spans of the bridge was very low. The PS geocoded over the Google Earth plan view of the bridge is shown in Figure 4.13. There are three points on the bridge labeled 1 through 3 in Figure 4.13 that have similar structural characteristics that will be tracked over the acquisition period. These points are all located along different expansion joints of the bridge. The damage occurred as indicated in the figure. There was no PS identified on the east side of the expansion joint where the crack occurred. However, point 1 was the closest to the damage and was used to compare with the other PS to identify if there was more movement in this region than the other points.

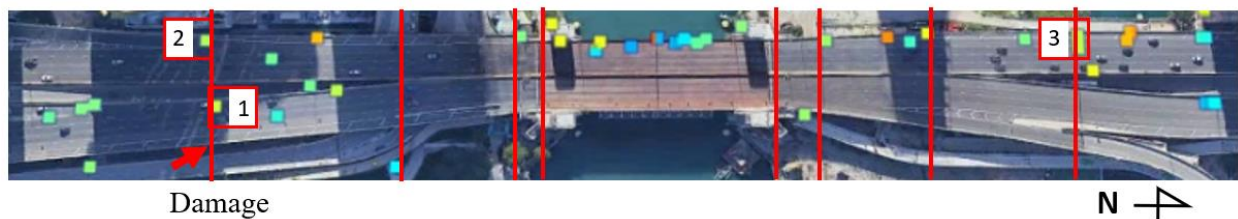


Figure 4.13. Outer Shore Drive Bridge showing PS used for the analysis and expansion joint locations

The displacements were determined by evaluating the image pairs in reference to the second image in the stack. The second image was chosen as the reference image because it was found to have the greatest temperature associated with its acquisition date. This way, the largest temperature differential could be captured in the comparison. After running the analysis for each of the eleven image pairs, the cumulative displacements were obtained for each pair. To best compare the correlation of temperature with the measured displacements, the temperature difference and displacement plots for each pair of images were created and presented in Figure 4.14.

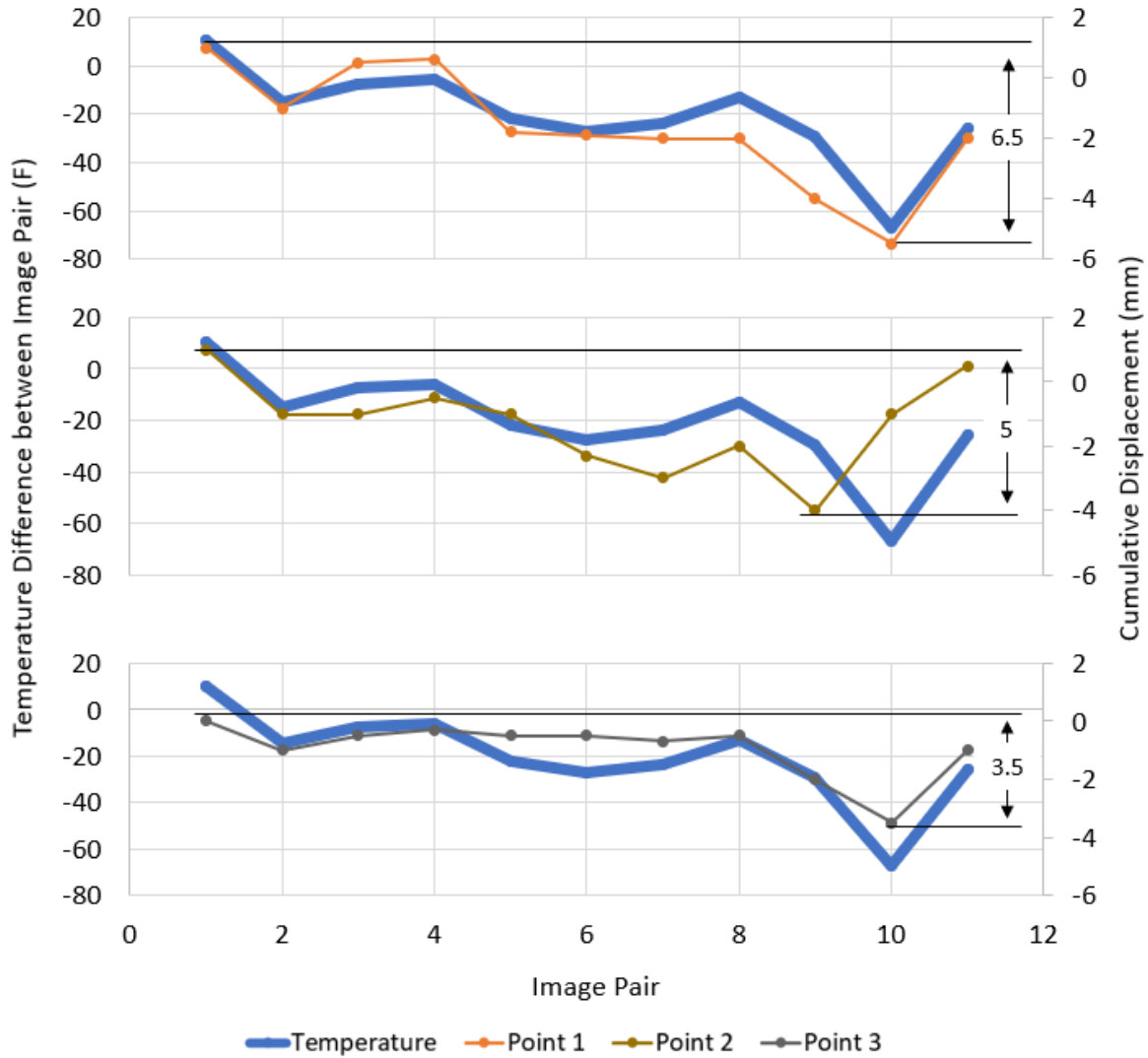


Figure 4.14. Temperature differential and cumulative displacement plots for 3 scatterers over the acquisition period

The plots give the temperature differential in the left vertical axis and the displacement in the right vertical axis. The magnitude of the displacements over this period of time was at a maximum of about 6.5 mm which occurred at point 1. This is compared to 5 mm and 3.5 mm for points 2 and 3, respectively. Moreover, point 1 does appear to be more sensitive to large fluctuations in temperature between the image pairs than the other two points. This can be an indication of larger thermal expansion of the steel components which can be attributed to the

damage at the bridge expansion joint. While these few observations could be made from the available data, this is no indication of any definitive conclusions about the damage to the bridge.

The deformation trend of the points shows that the MT-InSAR analysis can capture millimetric displacements that are correlated to the change in temperature over time. These trends were observed on a point-by-point basis of the PS over the bridge. Greater density of points over the length of the bridge can give a better depiction of the overall bridge response. The primary limitation with the analysis of the damage was that an insufficient quantity of PS was available to conduct a good comparison with other bridge spans. One way that may assist to better justifying that there is damage in the bridge is to use a higher resolution imaging mode to increase PS density on the bridge. The Staring Spotlight mode of the TSX satellite for example is a greater resolution imaging mode which can help improve the density of the points. This has been successfully demonstrated in several studies proving the increase in density of the feature points after the analysis (Lazecky et al., 2016; Hoppe et al., 2019; Bamler et al., 2009).

4.2.3. Thermal Deformation Trends in IL-251 Bridge

During the early stages of this research, an investigation of the potential of InSAR to be used to monitor thermal deformation trends in bridges was being considered. For this analysis, 72 Sentinel-1A satellite images acquired in the SM mode were available covering the period of January 2015 to July 2017. An image of the Sentinel-1 coverage over Illinois is presented in Figure 4.15. The data was acquired from the European Space Agency (ESA) open-source satellite data. The large spatial coverage of the SM image made it possible to search for regions with bridges which would provide sufficient point density using this satellite in this imaging mode.

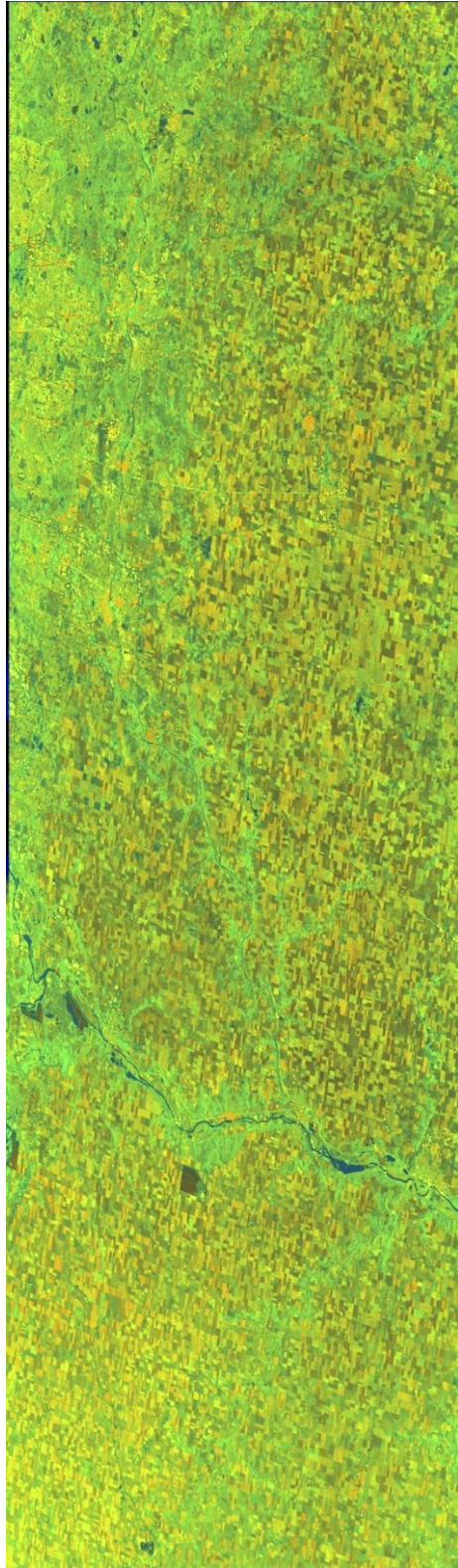


Figure 4.15. Sentinel-1A image over Illinois used for IL-251 Bridge analysis

The bridge that was chosen for the study was the IL-251 bridge over the Illinois River shown in Figure 4.16. The bridge is a steel truss through deck bridge that was built in 1958. It has a main deck span of 477 ft and carries two lanes of traffic. The bridge was a good candidate for this study because of the reflectivity of the steel components to the satellite and the relatively remote location making the bridge more distinguishing for the PSI processing. This bridge had no recorded signs of damage and was in good condition during the entirety of the analyzed period.



Figure 4.16. Image of the IL-251 bridge over Illinois River

The bridge had a good density of PS coverage over the main span of the bridge. The PS were inspected for their displacement histories throughout the acquisition period. An example of one of the points that was chosen towards the middle span of the bridge is shown in Figure 4.17. The image shows the displacement history of the point over time.

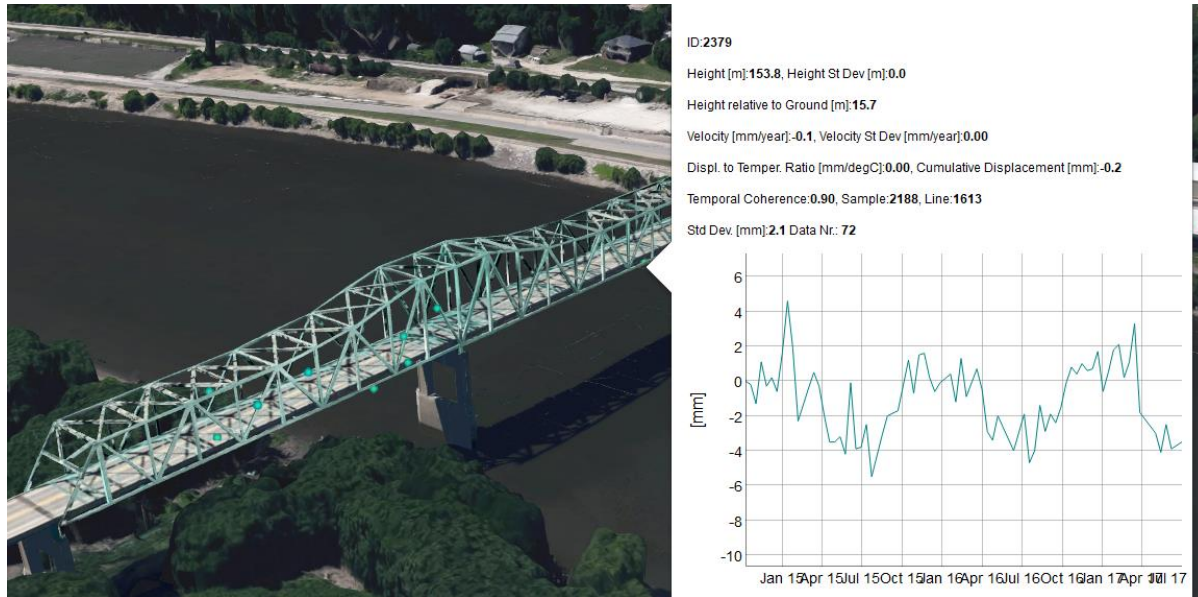


Figure 4.17. Thermal deformation trend of scatterer on midspan of IL-251 bridge

A lot of data is made available upon clicking on one of the points in the visualization toolbox of SARPROZ. However, the focus of this study was to determine if the fluctuation in displacement for the seemingly healthy steel truss bridge was consistent with the changes in temperature from season to season. The plot of the PS shows the entire acquisition period with the corresponding recorded displacements relative to the master image. A figure correlating the cumulative displacements of the point with the temperature over the acquisition period is shown in Figure 4.18. The horizontal axis displays the month and year each image was taken separated by 3 months up until the final acquisition date in July 2017.

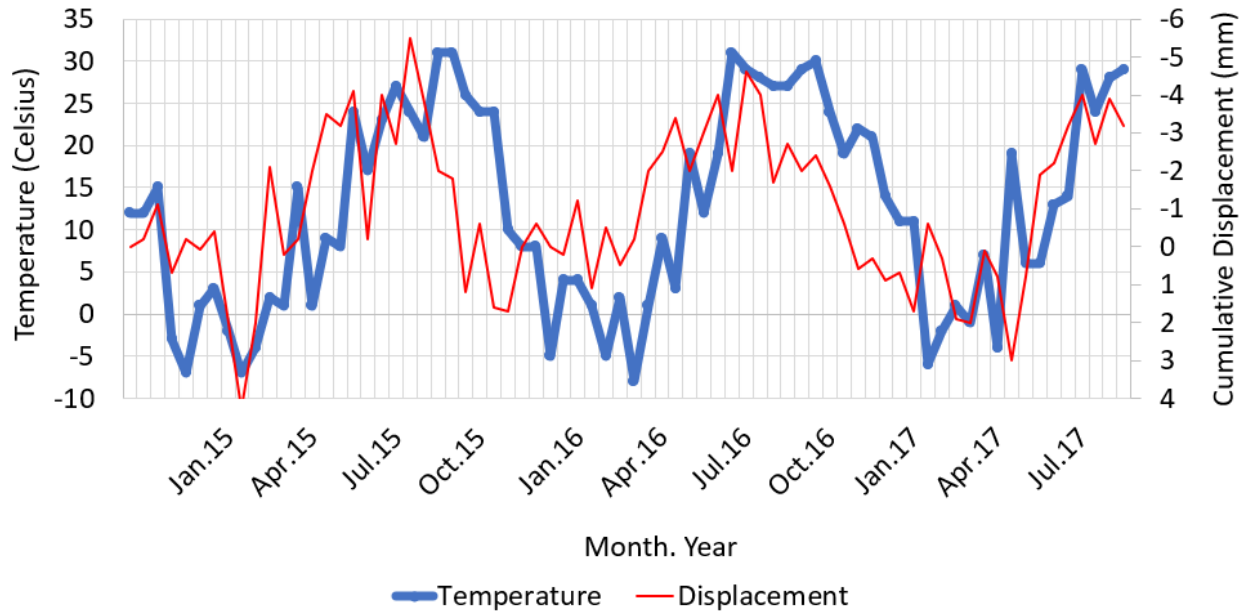


Figure 4.18. Plot showing the correlation of displacement with the temperature of a scatterer on the bridge over the entire acquisition period

Figure 4.18 shows that the point consistently trends towards the negative direction (away from the satellite) during the warmer months and in the positive direction during the colder months. This is a good indication that the software is capturing the thermal deformation trends of the structure. Because the images covered a period of approximately 2.5 years, the deformation behavior could be better understood and validated using this technique.

4.3. Conclusions

This chapter presented the novel MT-InSAR technique for bridge deformation monitoring applications. The chapter commenced with a literature review of the InSAR measurement procedures and a brief background of working with the satellite data. A summary of a few previous publications applying InSAR to bridge monitoring follows. Finally, the application of this technology to three different bridges in the United States is presented. Some of the key takeaways from this chapter are as follows:

- A small number of satellite images can be used to detect movement patterns of a bridge and identify critically damaged areas of a bridge.
- The PSI MT-InSAR method shows great potential of capturing millimetric displacements in bridges which can serve as an early warning to engineers that there is damage in a bridge.
- Thermal deformation trends were captured using InSAR over the scope of 2.5 years. These trends can be used to monitor the health of the structure over time and identify irregular deformation behavior in the future.
- Satellite imaging does not require the application of high loads on the bridge to determine the deformation behavior. Instead, the bridge can be monitored continuously under service load conditions avoiding the need for frequent field testing.
- This approach can discover historical information about bridges that otherwise would be difficult to determine using conventional testing methods.
- Satellite monitoring methods can be a much cheaper and simpler alternative to load testing of bridges.

4.4. Future Outlook

Satellite imaging using InSAR techniques addressed in this research are still under development and require more research for bridge monitoring applications. Nevertheless, the latest capabilities in X-band InSAR imagery are well equipped for bridge monitoring. InSAR will continue to enhance its resolution and revisit times. The PAZ radar satellites, for example, launched in 2018, will double the acquisition capacity with half the revisit time for interferometric applications further increasing the quality and quantity of selected PS for deformation analysis

(Selvakumaran et al., 2018). Capturing slow deformations at greater spatial resolutions with high-density PS will be ideal for bridge monitoring. When considering localized displacements, the use of reflectors may be necessary unless the methods have been improved to provide localized PS at points of interest. InSAR will not be able to capture the dynamic responses of a bridge, which makes this technique better suited to long-term monitoring applications. Although, near real-time data can be obtained with increased satellite revisit times. As InSAR analysis methods and algorithms for deformation monitoring continue to improve, monitoring bridges using this technology will become the most efficient method for bridge monitoring. With improved resolution capabilities and revisit times, InSAR applications for monitoring bridge deformations is even more promising. Implementing this to more bridges can be of substantial benefit to structural monitoring applications in the future.

CHAPTER 5: SUMMARY AND CONCLUSIONS

5.1. Summary

This research aimed at studying the practicality and constraints of using bridge diagnostic and proof load testing as load rating methods for reinforced concrete (RC) slab bridges. These two field testing methods were employed on the Piatt County and McLean County RC slab bridges for computing modified rating factors (RFs) to compare with the AASHTO analytical load rating results. Two approaches, namely AASHTO's MBE approach and the Effective Strip Width (EW) approach, were used and discussed to investigate the impact of customary load testing procedures on the results. The costs and challenges associated with the conventional field instrumentation setup prompted the study of a cheaper and more convenient feature-based image registration (FBIR) method to be used during these tests. The FBIR method was applied during the two field tests to study the potential of using this technology for field testing. Laboratory tests were also performed to study the intrinsic parameters of the approach for improving performance during field tests. Finally, an even more advanced approach using Multi-Temporal Interferometric Synthetic Aperture Radar (MT-InSAR) for structural health monitoring of bridges was investigated to overcome the problems associated with field testing and study the potential of this method for structural health monitoring of bridges. This approach is based on satellite images and was applied to three different bridge studies to demonstrate the capabilities of this technology to monitor millimetric displacement trends in bridges.

5.2. Conclusions

The following main conclusions can be drawn from this research:

1. The load testing results from the field tests gave more reliable estimates of the bridge load capacities. The results from the Piatt County bridge study showed that load testing may reduce the load ratings by as much as 26% for RC slab bridges that have noticeable damage and unique geometric properties. In such cases, conservative assumptions may be necessary when estimating the material properties and section properties of the concrete slab for load rating similar bridges. The McLean County bridge study showed that the capacity may have improved by as much as 44% through the load testing evaluations. The results demonstrated that the bridge was in good condition.
2. The FBIR displacement measurement approach demonstrated great potential during the field and laboratory tests. The FBIR returned displacement values on average within 5% of conventional LVDT displacement sensors during the Piatt County bridge test, 1.6% during the McLean County bridge study, and 3.3% during the laboratory tests. The approach has the advantage of simplifying the load testing strategy by replacing numerous LVDTs which rely on a fixture with one or a few cameras. The laboratory tests show that multiple displacements can be measured simultaneously with a single camera with an average accuracy of 0.78% compared to LVDT sensors from a distance as far as 33 ft from the bridge. Furthermore, the proposed FBIR approach can reduce costs significantly by requiring less setup time and less costs for the equipment.
3. The potential of the MT-InSAR Persistent Scatterer Interferometry (PSI) technique to detect damage in a structure through millimeter-scale displacements has been demonstrated in the literature review and also through the several bridge studies conducted in this thesis. This technology has been shown to detect the location of damage in the Hernando De Soto Bridge years in advance of the closure of the bridge, showing that displacements in this region were

as much as 63% greater than any other region of the bridge. The study of the Outer Shore Drive Bridge in Chicago and the IL-251 Bridge in Jonesville, IL demonstrated that InSAR is sensitive to thermal deformations in bridges. Thermal displacements of point 1 located near the damaged joint on the Outer Shore Drive Bridge moved at least 30% more than similar points detected on the bridge suggesting that there may be some damage in this portion of the bridge.

5.3. Proposed Future Work

The conclusions made for the bridge load testing studies are solely based on the results of two bridges. A greater inventory of RC slab bridges should be considered in future work to further verify the outcomes of these investigations. Developing methods to save time during field tests can improve the efficiency of using load tests for load rating. Methods for reducing the number of necessary field workers, the time associated with loading the trailers and the bridge, and on-site computer processing time can save costs for conducting field tests. Incorporating well-equipped image-based technology to monitor bridge displacements during the field tests in place of conventional LVDTs can substantially reduce costs and complications with the field setup.

The effects of weather and humidity on the FBIR displacement measurement results should be addressed to improve the practicality of FBIR for field testing. The intrinsic parameters of the cameras used for the FBIR measurements should be studied to improve the image quality for better performance. Research should be performed to improve the feature detection and matching algorithm in the code to be more versatile when applied to field tests. These considerations will help improve the accuracy and reliability of the FBIR displacement measurement approach.

InSAR techniques addressed in this thesis are still under development and require more research for bridge monitoring applications. Implementing this technology on more bridges can be of substantial benefit to structural monitoring applications in the future. Attempts at making this

method more tailored to the local displacement behavior of bridges can be done by incorporating reflectors on the bridges being studied. Using the latest high-resolution imaging modes can improve feature point density and reliability in the measured responses. Increasing the number of images used in the MT-InSAR analysis will improve the accuracy of the results. An effort should be made at studying various types of bridges in different locations to test the resourcefulness of this method under different conditions.

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APPENDIX A: BRIDGE DIAGNOSTICS INC. SOFTWARE

BDI STS-LIVE Software

The STS-LIVE software is provided with the STS4 package for visualizing the sensor data during the field tests. The software was used for both diagnostic and proof load tests to monitor the bridge responses during the field tests. The software gives control over all the connected devices, sensors, calibration files, and more, for setting up the load tests and configuring the devices. Important information with regards to the setup, features, and load test procedure utilizing the STS-LIVE will be discussed in this section.

Upon opening the STS-LIVE software, the user will be confronted with the Settings tab where the project name, description, file directory, and sampling rate can be selected (see Figure A.1). Recommended sampling rates depending on the type of test being conducted. For typical diagnostic tests where the truck moves at a crawl speed along the bridge span, the sampling rate should be among 50 and 20 S/s. For static load tests common in proof load testing, the 10 S/s rate is recommended. Under this same tab, there is a sub-tab where the user can set autozero on or off. This feature was set to ‘on’ for all field tests in this study. However, this feature can be useful when running multiple tests to monitor residual displacements between each loading stage. The final sub-tab allows for the user to lock access to the calibration files for the sensors. The password to unlock or lock the sensor edit feature is ‘BDI’.

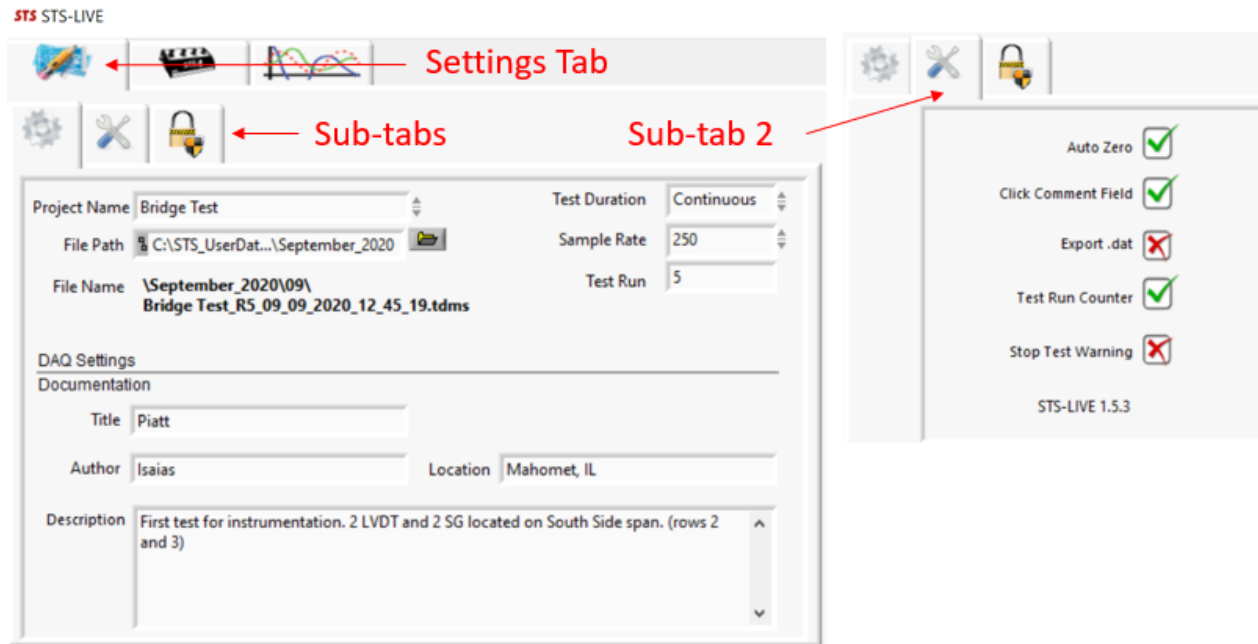


Figure A.1. Settings tab

The second tab is called the Hardware tab shown in Figure A.2 where the WBS, STS4 nodes, and sensors can be modified for the test. The STS4 nodes will appear in the window showing their battery power and signal strength along with some other information. The signal strength is given by the RSSI in decibels (dB) which should be no greater than ± 80 dB to provide a good connection. If the RSSI is high, then the WBS can be moved to help resolve this. The signal strength will update automatically in real-time in STS-LIVE as the WBS is moved closer or further from the computer. Every node will show the sensors connected to each corresponding data channel in the drop-down list. When the sensors view option is selected, the window will display the maximum and minimum readings for each sensor, units of the measurements, and other information which are relevant for the test. The Hardware tab is also where the user has access to the calibration factors and sensor limits. These parameters can be adjusted, but it is not recommended for most cases since the sensors are already calibrated and any changes can result in misleading sensor values.

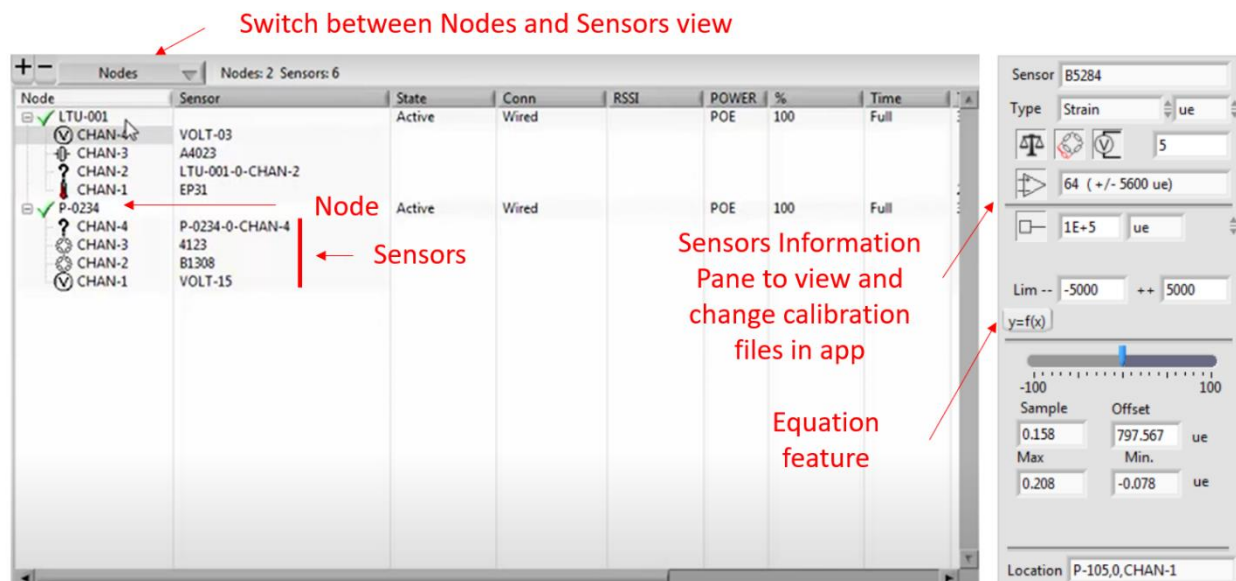


Figure A.2. Hardware Tab

One useful feature in the sensor information pane is the equation feature. STS-LIVE allows the user to specify a closed-form equation to convert the sensor readings during the test to some other values driven by the equation. For example, this was useful when converting the readings for the ST350s when they have extensions on them to the appropriate values using Eq. 2-10. An important consideration when using this feature is that the values which are recorded and saved into the TDMS file are the value after the operation is performed. This means that the raw readings from the sensors for which this equation is applied are not saved into the file for post-processing.

When setting up the hardware for the test, the computer must be connected to the appropriate IPN to connect to the WBS. Video tutorials on how to setup and use the STS-LIVE can be found on <https://bditest.com/applications-tutorials/>. Once the settings for the IPN are changed, the WBS can be turned on and the STS network will be available within minutes. The WBS will appear first in the Hardware tab with a checkmark to indicate that the connection has been established. It is important to first check the signal strength under the RSSI column to ensure that the signal strength is sufficient. This number should be no more than ± 80 . The closer the

number is to zero, the better the connection. After connecting the sensors to the nodes for the test, the nodes can be turned on. These should connect to the WBS and appear in the window when they have been successfully connected to the system.

The final tab in STS-LIVE is the Data Display tab where the data can be visualized during the load tests. The display window shows the sensor values for any of the sensors used for the test. This window allows for multiple different sensors to be shown on one plot. The sensors can also be grouped to visualize only specific sensors during the load test. The display can be customized by the user with different visualization tools. Note that the more sensors being visualized at once, the more CPU is required. This window also displays the comment feature in the bottom left-hand corner of the screen where the start button is located. This comment feature can be used to place ticker marks (or time stamps) in the data during the load test. For example, this is useful for marking the truck load position during the load tests. Each tick mark will correspond to a different truck position. When the test is completed, the clicker data with the time stamp will be saved in the TDMS file for post-processing.

Data is only recorded once the start button is pressed (shown in Figure A.3). A preview data button can be used to preview the measurements without storing the data on the hard drive. There are controls for setting the nodes into sleep and standby modes to save battery during long field tests.

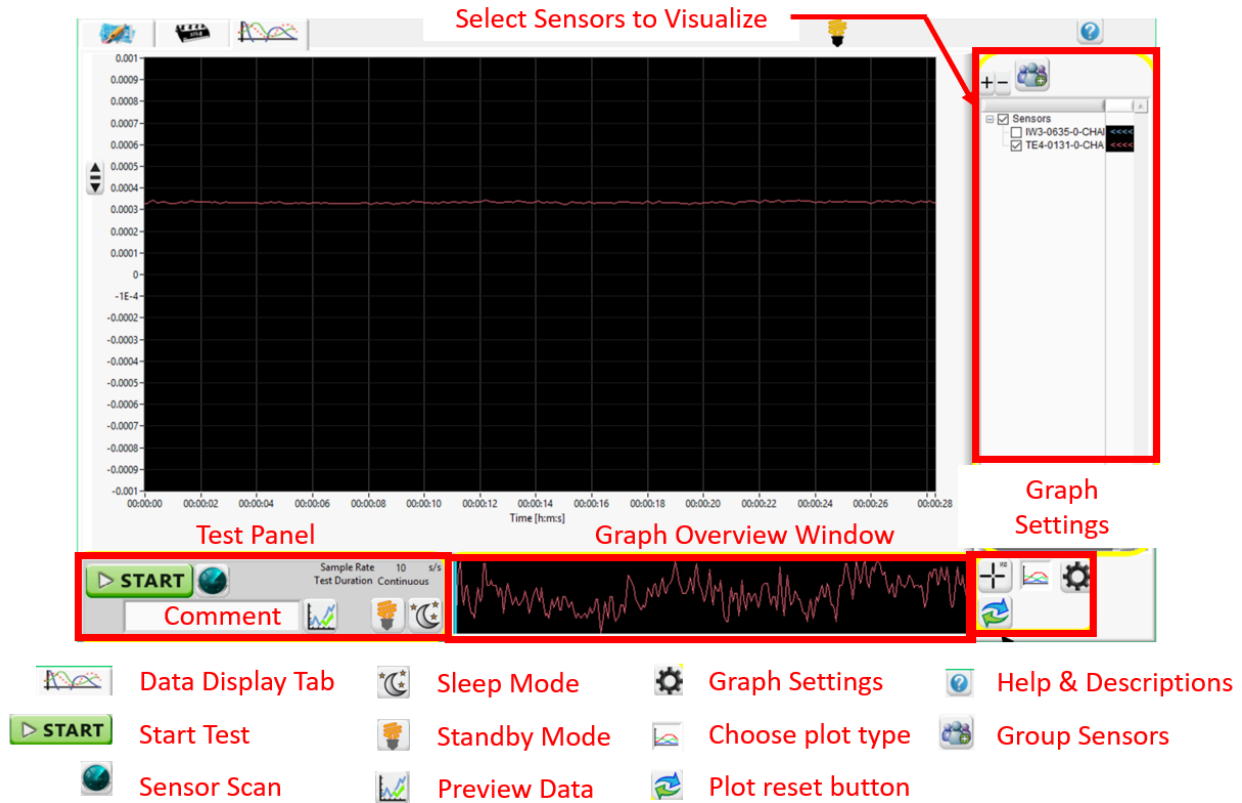


Figure A.3. Data display tab

It is suggested that the instrumentation be turned on and connected at least 30 minutes before commencing the load test. For the load tests, the start button is pressed before any loading is placed on the bridge to have a reference zero from all the sensors. The sensors will be automatically zeroed when starting the test if the Autozero was checked in the Settings tab prior to starting the test. The loading commences by driving the trucks over the bridge. Markings can be placed on the roadway to represent the quarter spans of the bridge so that the comment feature can be used to track the position of the truck as it crosses each of the markings. Once the truck(s) have completely crossed the bridge, the test can be stopped after a few seconds to save the data to the TDMS file. The process is repeated for every load path. The results can then be visualized in Microsoft Excel by opening the TDMS file or processed in BDI's STS-VIEW software.

BDI STS-VIEW Software

The software used for post-processing the results was BDI's STS-VIEW software. The software runs on MATLAB and allows the user to easily visualize the TDMS file data (MathWorks 2021). The data can be directly imported from the raw TDMS files from the test. STS-VIEW is not typically included with the system and comes at an additional cost. Several features are available in the software as visualization and post-processing tools. However, only some of the features in the software used for the field tests will be discussed in this section.

The data ingested in the STS-VIEW software is in the TDMS file format and automatically saved from the STS-LIVE test. The data can be uploaded from the files menu tab where the user can upload multiple TDMS files at once. The clicker data taken during the test was automatically saved into this file with the time stamp corresponding to when the comment was clicked during the test. In the event that these markings do not give the maximum or minimum value that the user wants to process, new time stamp data point or modifications to the clicker data can be made to manually change where to inspect the data. This can be found under the clicker overrides option in the Edit Data tab shown in Figure A.4. If position data is of interest, the positions of the truck between clicks can be input into the data to get a measurement versus truck position plot. The Sample column is where the user can modify the position of the click or add new clicks to the data. The desired sample number can be calculated by finding the time where the desired point is to be taken and multiplying this time by the sample rate. The clicker override feature was used frequently in this study to obtain the peak magnitudes from the data and plot the distribution of the peak values across the slab after the field tests. It will often occur that the critical section of the bridge is not exactly at the position predicted from the analysis. For this reason, the clicker modification window is important for the analysis and post-processing.

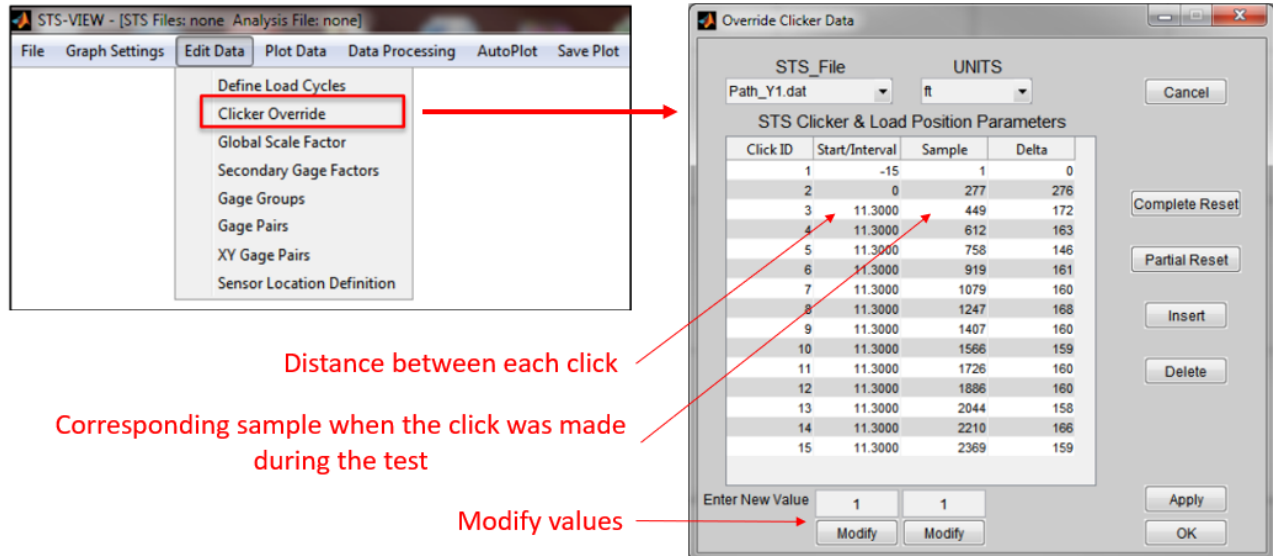


Figure A.4. Clicker override feature

The software is useful for quickly obtaining the cross-sectional distribution of the sensor readings on the bridge from the load tests. This can be done by using the gauge groups window under the edit data tab as shown in Figure A.5. The sensors should be organized such that they are in the order they appear on the cross section of the bridge. The group can then be saved and plotted based on the truck position specified in the clicker override window. The procedure for obtaining the group sensor distribution is shown in Figure A.5.

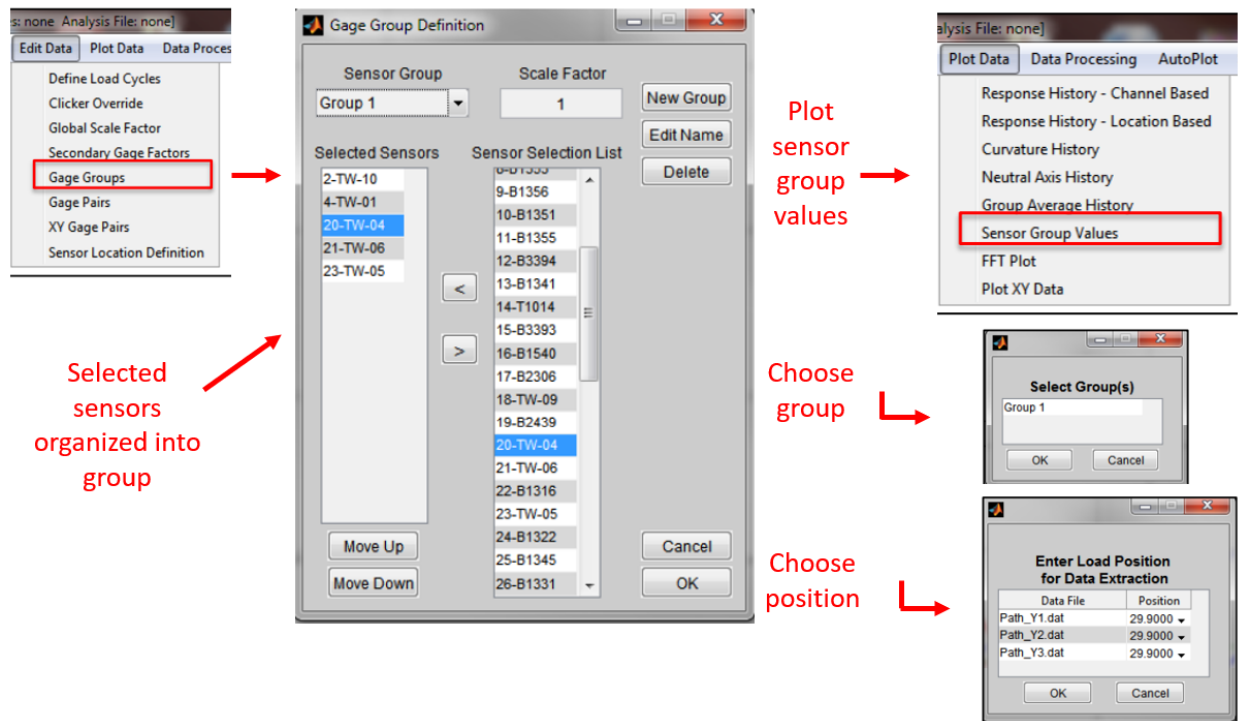


Figure A.5. Gauge groups window and sensor group values plot for displaying the cross-sectional data for the bridge

The sensors results can be plotted versus time simply by using the response history channel-based option in the Plot Data drop-down menu. When the user has input the position data in the clicker override window, a plot of the sensor response versus truck position can also be generated. This is useful for model calibration and for understanding the position of the axle loads that produced the responses. Note that multiple datasets can be displayed on a single plot for either option which makes it easy to compare responses based on multiple load paths or compare repeat passes of the truck on the bridge.