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Fatigue of Older Bridges in Northern Indiana due to Overweight and Oversized Loads

Volume 1: Bridge and Weigh-In-Motion Measurements

by

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Introduction

An important part of the economy of northwestern Indiana is the shipping of steel and other various products to Michigan for the manufacturing of automobiles and other commodities. The extra heavy-duty corridor is composed of segments of roads totaling 94 miles in northwest Indiana. It was put into place to facilitate the shipping of large truck loads, such as coils of sheet steel. The extra heavy-duty corridor highway permits truck loads of up to 134,000 lbs. transported by multiple trailer, multiple axle "Michigan Train" trucks. The purpose of this study is to examine and evaluate the fatigue

strength of the steel bridges along the extra heavy duty corridor.

The work in this study consisted of two portions: field measurements (Vol. 1) to determine the spectrum of the truck axle loads on the heavy-weight corridor and the influence of those loads on the response of one steel bridge located relatively close to the WIM, and fatigue analysis and evaluation (Volume 2) to estimate the response and remaining fatigue life of steel bridges along the heavy weight corridor.

Findings

To evaluate the fatigue strength of the steel bridges along the extra heavy weight corridor, an accurate evaluation of the types and weights of the trucks that travel the corridor has to be collected. Once such a load history had been accumulated, then the fatigue life can be reasonably evaluated by predicting the stress ranges generated by those loads. The truck weights were evaluated by using a weigh-in-motion sensor installed in the roadway to measure the truck gross vehicle weights, the individual axle weights, and the class (type) of vehicle. To provide an additional check on the actual live-load stress ranges generated in the bridge superstructure versus those predicted by using the measured truck weights and standard load distribution factors, the strain range values were measured in one bridge structure on the corridor at a location relatively close to the weigh-in-motion system.

A number of observations were made as a result of the weigh-in-motion field measurements. First, data on truck information were gathered over a four month period to provide a breakdown of the types of trucks using the heavy weight corridor. Class 9 trucks are typical five axle trucks that are commonly used by the trucking industry, and Class 13 trucks have seven or more axles and are generally used for the Michigan Train configuration. It was found that 44% are Class 9 trucks and 14% are Class 13 trucks. Second, it was found that the average gross vehicle weight (GVW) for Class 9 trucks is 54,400 lbs and 119,500 lbs for Class 13. The average GVW on the extra heavy duty highway is 52,370 lbs for all trucks in all directions, with 56,560 lbs and 47,780 lbs in the eastbound and westbound directions, respectively. Third, it was observed that some trucks travel overweight while most travel with their legal limits. The WIM data indicated that

15% of the Class 9 trucks traveled over the 80,000 lbs limit, while 26% of the Class 13 trucks travel over the 134,000 lbs limit.

A number of strain gages were installed in the last two spans of a ten span continuous bridge on US 20 over Chandler Ave. and an Amtrak line in the Town of Pines near Michigan City, IN. The bridge carries two lanes in both the east bound and westbound directions; only the eastbound spans were instrumented. The structure, which is 401-ft long and 38-ft wide, is composed of six 27-in deep rolled beam members of various sizes with a 9 3/4-in thick concrete deck. The instrumented spans have no skew, although a number of the spans have a significant 45° skew. The bridge is situated about one mile east of the weigh-in-motion system that was used to evaluate the truck loads. The strain gages were monitored over the same four month period as the WIM. The absolute maximum strain caused by a truck during the four month period was 195 µε (microstrain) while the absolute maximum strain range was measured to be 227 µε. It was found that the strain pattern caused by Class 9 and 13 trucks are quite different, with a bimodal pattern for the Class 9 trucks and a single peak for the Class 13

trucks. Although the bimodal pattern will result in more loading cycles per truck, the strain ranges for the Class 9 trucks are less than those caused by the Class 13 trucks. The average strain range values induced by Class 13 trucks were found to be about 20 µε higher than those induced by Class 9 trucks.

Measured strains were compared to strains predicted using two-dimensional and three-dimensional models of the bridge structure. Several factors provide possible discrepancies between predicted and measured strains: varying degrees of composite behavior throughout the bridge, truck location within the lane, truck impact effects, and WIM measurement error. Nevertheless, it was found that reasonably good comparisons between predicted and measured strains could be obtained using a three-dimensional model. Lastly, based upon the measured strain data, it was found that the bridge structure was not susceptible to fatigue damage at the Category C and D details used in the bridge. It was found that less than one percent of the trucks produce a stress range that exceeds the variable amplitude fatigue limit.

Implementation

Based upon the measured truck gross vehicle weights, experimental strain measurements, and analytical modeling for one bridge structure on the extra heavy-weight corridor, it does not appear that fatigue is a serious problem. However, a second stage of the study will develop a more thorough analytical model than

was used in this phase of the study that will be applied to other steel bridges along the extra heavy-weight corridor. The fatigue response of the steel bridges should still be periodically monitored through routine biennial inspections, especially the response at the most fatigue-critical structural details.

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16. Abstract <p>This report is the first of a two-volume final report presenting the findings of the research work that was undertaken to evaluate the fatigue behavior of steel highway bridges on the extra heavy weight truck corridor in Northwest Indiana. The purpose of the study was to evaluate the type and magnitude of the loads that travel along the corridor and then assess the effect of those loads on the fatigue strength of the steel bridge structures on the corridor. This volume presents the results of the experimental field study conducted to evaluate the load and load effects on one steel bridge structure on the corridor. A weigh-in-motion (WIM) system was installed near the bridge structure to evaluate the loads that would cross over the bridge being monitored. Strain values were monitored on two spans of the ten-span continuous bridge being evaluated. Comparisons were then made between strain measurements in particular girders and strain values predicted using the measured truck axle weights. The WIM data indicated that 15% of the Class 9 trucks and 26% of the Class 13 trucks travel heavier than their respective legal limits. Extreme weights of more than 200,000 lbs were observed. In spite of the heavy truck loads being carried, it was found that less than 1 percent of the trucks induce a strain range that exceeds the variable amplitude fatigue limit of the fatigue critical details in the structure being monitored. Lastly, it was found that three-dimensional analytical models provide the best agreement between predicted and measured strain values in the bridge.</p> <p>The titles of the two volumes (Report Number in parentheses) are listed below:</p> <p>Volume 1: Bridge and Weigh-In-Motion Measurements (FHWA/IN/JTRP-2005/16-1)</p> <p>Volume 2: Analysis Methods and Fatigue Evaluation (FHWA/IN/JTRP-2005/16-2)</p>			
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CHAPTER 1 – INTRODUCTION

1.1 Reason for Study

The economy of Northwest Indiana is very dependent upon the steel industry. The steel industry faces severe competition in both local production and global production. The steel producers and the trucking industry have continually lobbied for increased legal truck weights. As a step to ensure the continued success of Northwest Indiana's economy, the Indiana General Assembly has designated several sections of highway in Northwest Indiana as "extra heavy duty highways." The extra heavy duty highway permits truck loads of up to 134,000 lbs transported mostly by "Michigan Trains." "Michigan Trains" are generally multiple trailer, multiple axle trucks designed for the higher legal weight limit of 164,000 lbs in Michigan.

It is absolutely essential for the Indiana Department of Transportation (INDOT) to have an accurate understanding of the effects of the "Michigan Train" truck traffic on the fatigue life of bridge structures. To evaluate this effect, accurate information on the configuration and weights of the trucks must be acquired. In addition to truck information, bridge measurements must be performed to determine their effective load patterns and overall effect on the bridge structures. The measurements from this study will be used to develop a fatigue load model for the bridge structures on the roadways designated as extra heavy duty highways.

1.2 Project Objectives and Scope

The objectives of this study are to evaluate traffic loadings and their effect on bridge structures along the extra heavy duty highway corridor. The results of the project

will be used for the purposes of evaluating the safety of these bridge structures through the development of a fatigue load model.

This study will provide information on truck loading patterns and histories and bridge strain response. Truck measurements are determined through the use of Weigh-In-Motion (WIM) technology. Bridge strain response is evaluated by comparing field strain measurements with values predicted using an analytical approach.

A summary of similar studies and measurement approaches are provided in Chapter 2 (“Literature Review”). Other studies performed on the extra heavy duty highway are included and provide information deemed important by the INDOT. The literature review also details WIM and bridge strain measurements methods and results.

Chapter 3 (“Location and Structure Description”) provides details on the extra heavy duty highway, “Michigan Train” trucks, and the bridge chosen for field study.

Instrumentation utilized in the field is described in Chapter 4 (“Instrumentation”) and provides details on the methods used to collect and perform measurements. This section provides details in determining the instrumentation locations and includes the sensor and gage layouts. This section also includes details on the WIM calibration.

Chapter 5 (“Measurements”) provides details on the selected equipment and programming specific to the project. Also included in this section are details on the instrumentation output and calibration.

The results of the WIM measurements and bridge strain measurements are presented in Chapters 6 and 7. Evaluation and comparisons of these measurements are included in these chapters.

The development of an analytical model for the bridge structure is described in Chapter 8 (“Analysis and Prediction of Bridge Response”). These results are compared with the experimental results.

Chapter 9 (“Conclusions”) provides conclusions based on the evaluation of the experimental and analytical results.

CHAPTER 2 – LITERATURE REVIEW

2.1 Background

Several studies related to heavy truck loads and their effects on bridge structures have been documented. These studies have been sponsored by organizations such as the Federal Highway Administration, U.S. Department of Transportation, and many businesses with transportation concerns.

To help reduce transportation costs, the trucking industry has often lobbied for heavier legal truck weight limits. In their view, it is generally more economical for trucks to transport heavier loads reducing the number of loads to transport. Northwest Indiana is economically dependent upon the steel industry, an industry that requires the transport of material that is very heavy proportional to size. To cope with the demands of these businesses, many Department of Transportations have designed their roadways or designated specific routes to allow extra heavy trucks. To allow these extra heavy loads and maintain highway safety, some Department of Transportations require special permitting and have developed standard allowable trucks with specified configurations. Indiana's neighboring state, Michigan, has some of the most lenient laws pertaining to truck loads. Michigan law allows vehicles based on a formula of axle loads and axle spacing (Williams and Associates, 1986). In 1986 the Indiana General Assembly passed similar legislation allowing trucks of configuration similar to that allowed in the state of Michigan. This newly allowable truck is justifiably referred to as the "Michigan Train" truck and is allowed with special permit on sections of highway designated as the extra heavy duty highway (Poe, 2000).

The increase in truck traffic and loadings along highways and their effects on the pavement and bridge structures are of major concern. Consequently most states operate static weigh stations to monitor and enforce loads. Nowak, et al (1994) found that many

truck weight studies appear to be biased due to the motivation to avoid static weigh station scales. To better understand the actual loads imposed upon the highway many states have introduced Weigh-In-Motion (WIM) systems. These systems are not easily visible to truck drivers and allow the measurement of a truck's weight while at highway speeds. WIM data and citation data shows that even with heavier allowable weights many trucks are still overloaded (Nowak, et al, 1994). These heavier weights increase the chances of structural overstressing and fatigue related failures.

Initial volume estimates of the use of the "Michigan Train" truck along the extra heavy duty highway were very small with a volume of a few hundred per week (Williams and Associates, 1986). However, since the passage of the extra heavy duty highway "Michigan Train" traffic has increased significantly to more than one hundred per day. The increase in truck traffic has greatly increased the need for accurate truck data to ensure the continued safety of our highways. The collection of truck data is an important part in determining maximum load effects and frequency distribution of heavy traffic.

Many bridges along the extra heavy duty highway are approaching 50 years in service. Due to this length of service it is important to determine the remaining fatigue life of these structures and to ensure that structures are not being overstressed. The effect of the GVW, axle weight, and axle spacing can be determined by the resulting moments and stress ranges (Nowak, et al, 1994).

Examples of these studies and their relevant sections and conclusions are described in this chapter.

2.2 "Michigan Train" Truck Route Studies

An initial study along the extra heavy highway was performed by Clyde E. Williams and Associates, Inc (1986). This study was performed to determine the additional damage and effect "Michigan Train" truck traffic would create along the highway.

In determining the structural capacity of bridge structures along the extra-heavy-duty highway, the "Michigan Train" truck No.5 and No. 8 were considered. The axle

weights, axle spacing, and total truck weight affect the stresses in highway bridges.

There are two rating levels for a bridge structure when using allowable stresses: inventory level and operating level. The inventory level is the stress level below which a structure can be utilized for an indefinite period of time. The operating level is the maximum stress level to which the structure may be subjected. Trucks inducing stress levels higher than the operating level should not be allowed to travel across the structure.

An analysis of the flexural capacity of 23 bridges along 62 miles of extra heavy duty highway indicated that eleven bridges exceed inventory stress levels and one bridge exceeds operating stress levels. The fatigue life of the structures was also investigated. The primary factor in determining the fatigue life of bridge structures is the stress range. This factor is calculated as the difference between the minimum stress and the maximum stress experienced during a loading cycle. To estimate the accumulated fatigue damage caused by past traffic, historical traffic count data was obtained. Truck distribution was estimated using recommended AASHTO (1984) factors. A constant truck distribution and weight were assumed throughout the life of each structure. An estimate for “Michigan Train” truck traffic was provided by area steel producers. The fatigue life for each structure was estimated using Miner’s Method. It was found that the “Michigan Train” truck traffic did not significantly affect the fatigue life of the steel structures along the extra heavy duty highway (Williams and Associates, 1986).

Two very similar studies conducted by Cole and Associates analyze the effect of “Michigan Train” truck loading on the structural integrity and fatigue life of several additional bridges along the extra heavy duty highway.

Analyzing the flexural capacity of structures along the corridor using typical AASHTO (1984) factors, including distribution and impact, it was determined that the stress levels induced by the Michigan Train truck No.5 and No. 8 were above the bridge’s inventory level but below its operating level.

To estimate the damage from “Michigan Train” trucks and the remaining fatigue life of the structures, Cole and Associates used Miner’s Method and historical traffic data to estimate the total accumulated fatigue damage. To estimate the additional impact of “Michigan Train” traffic upon the fatigue life of the structures, future truck traffic was

estimated with a constant 1.25% growth rate. The excess impact of the “Michigan Train” truck was found to be minimal versus the standard HS20 truck (Farrand, 1991, 1992).

2.3 Load Measurement Studies

The service life of highway bridge superstructures are directly affected by the gross weight, axle weight, and axle configuration of heavy trucks. Reliable truck weight data would permit the evaluation of load capacity, remaining life estimates, and deterioration rate. It is essential to obtain information pertaining to heavy truck traffic to ensure the operational safety and fatigue life of bridge structures. Analyses of some structures with heavy truck traffic indicate that heavy traffic will not cause severe fatigue problems on fatigue categories A, B, and C. Studies indicate the majority of fatigue damage is caused by trucks of 4 and 5 axles. In short spans less than 30 feet, Class 9 trucks induce a higher number of stress cycles than specified in the AASHTO (1996) code (Wang, 2000).

There are several studies detailing the traffic and loadings of the “Michigan Train” or heavy truck. These studies have been performed by universities and consulting teams and researched traffic patterns, loads, and fatigue damage.

Girder distribution factors are very important when analyzing a bridge structure. Girder distribution factors are essential in determining the load each girder must carry as vehicles drive across the structure. As the girder carries more and more of the vehicle load the distribution factor increases. The girder distribution is very dependent upon the girder configuration. A field test of six bridges, using strain measurements, loaded with heavy 11 axle trucks confirm that the girder distribution factors provided in the AASHTO (1996) code are conservative even when loaded simultaneously with two 11 axle trucks. The girder distribution factors specified by AASHTO (1996) are often inaccurate and in some cases overly conservative. (Nowak and Eom, 2001)

A Michigan study by Nowak, et al. (1994) examines the effect of truck loadings on bridges. This study utilized both an experimental and analytical approach to determine loadings and their effect on bridges. The experimental approach examines

historical traffic information, takes advantage of WIM technology, and examines the fatigue damage caused by truck loading. The analytical section calculates the expected moments caused by measured truck data and compares the results to actual measured moments from bridge instrumentation.

Truck weights, axle loads, and axle spacing are important parameters of the live load. Moreover, Nowak, et al. (1994) report that most of the truck weight studies appear to be biased due to the motivation to avoid static weigh station scales. Citation data shows some “Michigan Train” trucks to weigh over 200 kips with heavy axles around 40 kips. Although truck information has been gathered for years at static weigh stations, WIM systems are important because drivers of overloaded trucks tend to avoid weigh scales. Using new WIM technology trucks can now be weighed at normal highway speeds without bias as drivers are not aware of the measurements and therefore do not attempt to avoid the scales. During the study the heaviest truck measured by the WIM system was about 230 kips and the heaviest axle measure was about 50 kips. Comparing these results with static weigh station scale data show an unbiased heavy weight 30-50% larger than stationary scales. When a static scale was closed for repairs heavy weights increased 30-40%.

Actual load information is very important in predicting the remaining life and load capacity of existing bridges. This information can be used for future predictions and the development of fatigue load models. Field measurements indicate that moments and shears are considerably larger than calculated maximum values resulting from static weigh station measurements, another indication that static weigh station data are biased due to heavy trucks avoiding the scales. The collection of truck data is an important part in determining maximum load effects and the frequency distribution of heavy traffic. The average gross vehicle weight (GVW) for highways that carry 11 axle trucks (Class 13) is considerably higher than highways that typically carry 5 axle trucks (Class 9).

Of particular importance for fatigue cycles and moment distributions are the axle weights and spacing. The effect of the GVW, axle weight, and axle spacing can be assessed by examining the lane moment caused by those factors. An important measure in fatigue is the correlation of GVW to the moment effect. Shorter spans show little

GVW to moment effect correlation. However, the correlation improves as the span length increases. When comparing truck induced moment on spans shorter than 60 feet there is very little difference in the moments induced by 5 axle and 11 axle trucks. However, for spans greater than 60 feet, as the span length increases the moments induced by 11 axle trucks are higher than those induced by 5 axle trucks.

Various models exist for the fatigue analysis of members subject to repetitive loading. In most of these models, the same vehicle is used for both strength design and fatigue design. The fatigue damage, however, is caused by the passage of many types of vehicles with some causing multiple strain cycles. Fatigue failure is the result of accumulated damage created by a variety of vehicles over an extended period of time. Development of a fatigue load model requires the collection of actual dynamic strain time histories. In shorter spans, a greater number of axles tend to produce a higher number of significant average stress cycles per vehicle. (Nowak, et. al., 1994).

2.4 Weigh-In-Motion Studies

Within the last few years the use and availability of various WIM systems has become increasingly important. Numerous WIM studies have been performed in Europe, Australia, the United States, and throughout the world. WIM systems are used in a variety of situations, including research, permitting and enforcement of legal truck loading, road design, and to ensure the overall safety and life of road structures.

Typically there are three main types of WIM sensors. These are piezoelectric sensors, bending plate, and load cells. Within each of these types there are several manufacturers and different technologies. Each of these sensors has a distinct cost, installation procedure, and associated accuracy.

Installation for each WIM is quite different. Piezoelectric sensors require only a small cut in the road of about 1-2" deep by 1-2" wide. They are then set in place with a quick set epoxy grout compound. This installation can be accomplished in one (1) day.

There are two installation options that are very different in cost which may be used or are necessary for bending plate sensors. If the sensor is installed in an adequately

thick (that being thick enough to house the sensors while maintaining functional road conditions) concrete pavement, road installation requires a small excavation to be performed. The scale frame is then set in place and anchored with anchor bars and epoxy. This installation can be accomplished in one (1) day. If the sensor is to be installed in a thin concrete pavement or a bituminous mix pavement, then a concrete vault is necessary. The pit must be 30" deep by 4'10" wide by 13'10" long. The frame is then placed and cast into the concrete foundation. This installation can be accomplished in three (3) days.

For the installation of a load cell, a concrete vault as described previously must be installed. The load cell is then placed with weighing platforms. This installation can be accomplished in three (3) days.

The accuracy of WIM systems are provided by their respective manufacturers and other contracted studies. These performance measurements are developed using near lab conditions and provide a higher estimated accuracy than what has been observed in the field. However, performance measurements are based on ASTM standards at the 95% confidence interval. Load cell systems are generally the most accurate, with GVW measurements within $\pm 3\%$. It is also the most expensive to install and maintain. Bending plate and quartz sensor WIM systems have a GVW accuracy of about $\pm 5\%$, while Piezoelectric sensors have a GVW accuracy of about $\pm 10\%$. In general as the cost of equipment, installation, and maintenance increases so does the accuracy. Each of these technologies have their own advantages and disadvantages. WIM system choice is very sensitive to site and funding conditions (Pratt and Bushman, 1998; McCall and Vodrazka, 1997).

CHAPTER 3 – LOCATION AND STRUCTURE DESCRIPTION

3.1 Overview

For this study, an appropriate location had to be determined to provide a steel bridge structure along the extra heavy duty highway. Moreover, adequate access was needed to allow for instrumentation installation and data collection. In addition the structure needed to be in such a location as to provide for adequate traffic volume to provide a large enough sample for reliable results. Based on these criteria and consultation with INDOT, a single representative structure was chosen from among the several structures along the segments of the extra heavy duty highway.

3.2 Heavy Duty Corridor

The extra heavy duty highway corridor located in northwest Indiana is of particular interest to this study. This corridor is composed of segments of roads, totaling 94 miles leading from various manufacturers in northwest Indiana to the state of Michigan (Poe, 2000). Figure 3.1 shows an overview of all highways in Northwest Indiana designated as an extra heavy duty highway as of 2002. The legislation designating various segments of the extra heavy duty highway is provided in Appendix A. To remain competitive the trucking industry is continually being pressured to transport larger and heavier loads (Williams and Associates, 1986). The corridor was developed to provide a route that steel producers and other manufacturers could use to transport cargo heavier than the typical legal limit of 80,000 lbs. Along this corridor trucks with special permits are allowed to travel on the extra heavy duty highway at a legal limit of up to 134,000lbs.

The typical trucks used to transport such heavy loads are commonly referred to as “Michigan Trains.” These trucks are generally multiple trailer, multiple axle trucks designed to carry a significant portion of Michigan’s heavier legal weight limit of 164,000 lbs (Schermerhorn, 1998). There are several different heavy truck configurations that have been developed by the American Association of State Highway Transportation Officials (AASHTO) to provide a comprehensive tool to be used in the design of bridge structures. However, two configurations which are used in Indiana are designated as Michigan Train Truck Number 5 and 8 (see Figure 3.2 and Figure 3.3, respectively). Appendix B contains photographs of typical trucks that travel on the extra heavy duty highway.

3.3 Location Description

The bridge structure chosen for study is located on U.S. 20 and spans over both Railroad/Chandler Ave. and an Amtrak rail line in the Town of Pines, IN near Michigan City, IN (Figure 3.4). It is located approximately four (4) miles west of U.S. 421 and one (1) mile east of S.R. 520 between Mile Marker 37 to the west and 38 to the east. The center of the bridge is located at Mile Marker 37+37.

This structure was chosen for several reasons. First and foremost the bridge structure is a steel non-composite structure. It is also located east of most of the steel production facilities, along a section most traveled by “Michigan Trains” before proceeding north into Michigan, guaranteeing significant traffic volume. The bridge is also easily accessible from underneath with the aide of a bucket truck via a low traffic volume county road or INDOT right-of-way.

3.4 Structure Description

The bridge is a ten (10) span non-composite continuous steel bridge supporting four lanes of traffic, two each in the westbound and eastbound directions. The bridge has two separate superstructures, one for each direction of traffic, that share a single substructure. The cross-section for each structure is composed of six (6) continuous beam members with a 9 ¾" concrete deck (Figure 3.5). The overall structure is 401' long and 38' wide in each direction. A plan view of the eastbound structure is provided in Figure 3.6.

The span lengths vary throughout the structure from 8' to 60'-11". The structure is composed of both straight and skew (45°) spans to eliminate the need for any horizontal curvature on U.S. 20, Railroad/Chandler Rd., and the Amtrak rail line. This is achieved with a triangular type substructure support system in spans "C" and "H." The spans from west to east are 42'-3", 43'-0" (varies), 60'-11" (varies), 44'-3", 44'-3", 50'-11", 44'-3", 60'-11" (varies), 43'-0" (varies), 42'-3". Spans "B" and "I" vary from 28'11" to 43'0." Spans "C" and "H" vary from 8'0" to 60'11." A substantial bolted plate splice at each support provides structural continuity (Figure 3.7).

The longitudinal beams of the structure are of four different size rolled sections; WF27x84, WF27x94, WF27x102, and WF27x114. Note that there are two different types of steel strengths; A36 and A441. These sections are distributed throughout the bridge as shown in Table 3.1. Each structure is composed of six beams spaced at 6'8" on center. To provide lateral stiffness, diaphragms are located perpendicular to the beams at midspan and at each support location. Diaphragms are of Type 16B26. The diaphragms are connected to the web of the beams through either an intermittent weld detail or a welded/bolted plate detail (Figure 3.8 and Figure 3.9). The diaphragms with the intermittent weld detail are attached to the beam with a continuous fillet weld located on top of both flanges of the diaphragms and intermittent welds along both sides of the web. The diaphragms with the welded/bolted plate detail are attached to a shear plate with bolts through the web of the diaphragm, and the plate then is attached to the beam with a continuous fillet weld.

Due to extreme corrosion caused by an open tooth joint at the ends of the structure, rehabilitation was deemed necessary and performed in 1998. During this rehabilitation, 1/4" of the original 8" bridge deck was removed and replaced with a 2" wearing surface. The approach to the structure was modified to allow for the expansion of the deck beyond the end of the structure. This repair also included the removal of the bridge deck in the end spans and the attachment of 4 rows of 3 shear studs on the beam end closest to the approach. The end joint was then also replaced and located beyond the end of the structure to allow for proper drainage.

Table 3.1 – Summary of Bridge Beam Structural Members

Span(s)	Span Length	Section (eastbound only)	Steel Strength (eastbound only)
“A”, “J”	42’ 3”	WF27x84	A441
“B”	28’ 11” – 43’ 0”	WF27x84	A36
“C”, “H”	8’ 0” – 60’ 11”	WF27x114	A441
“D”, “E”	44’ 3”	WF27x94	A36
“F”	50’ 11”	WF27x94	A441
“G”	44’ 3”	WF27x102	A36
“I”	28’ 11” - 43’ 0”	WF27x102	A36

INDIANA EXTRA HEAVY DUTY HIGHWAYS

Figure 3.1 – Designated Extra Heavy Duty Highway (2002)

MICHIGAN TRAIN TRUCK NUMBER 5

GROSS WEIGHT 134,000lbs.

Figure 3.2 – Michigan Train Number 5

MICHIGAN TRAIN
MODIFIED TRUCK NUMBER 8
GROSS WEIGHT 134,200lbs.

Figure 3.3 – Michigan Train Number 8

(a) Bridge Location in the Town of Pines

(b) Close-up View of Bridge Location

Figure 3.4 – Maps of Bridge Location

Figure 3.5 – Cross Section of Eastbound Structure

Figure 3.6 – Plan View of Eastbound Structure

Figure 3.7 – Typical Beam Splice at Pier Support

Figure 3.8 – Intermittent Weld Diaphragm Connection

Figure 3.9 – Bolted Diaphragm Connection

CHAPTER 4 – INSTRUMENTATION

4.1 Overview

A system of instrumentation was designed to collect bridge strain data in fatigue critical areas, as well as data on truck axle weights and truck configurations. Two separate data acquisition systems were used to collect the truck axle and bridge strain information.

Truck traffic is estimated, from previous traffic data, to occur primarily in the eastbound direction. Strain data, therefore, was collected solely in the eastbound structure. Both the eastbound and westbound structures are identical. The bridge structure discussed throughout this report will detail the eastbound structure.

4.2 Weigh-In-Motion Instrumentation

The use of a Weigh-In-Motion (WIM) system is crucial to this project. It allows direct correlation of strain patterns and magnitudes to truck weights and configurations. The WIM information will be used along with all data received from the instrumentation and analysis, to develop an accurate representation of the load effects of the Michigan train trucks on the bridge response.

4.2.1 Site Selection

A very critical aspect of a WIM system is site selection. There are several criteria necessary to ensure an accurate and reliable WIM system. Several studies and publications, such as the ASTM “Standard Specification for Highway Weigh-In-Motion Systems with User Requirements and Test Methods,” (2002) are available providing

detail on the criteria for WIM systems and how they affect the accuracy and reliability of the system. “Standard Specification for Highway Weigh-in-Motion Systems with User Requirements and Test Methods” (ASTM, 2002) provides a typical standard for WIM sites, installations, and accuracy requirements. Of particular importance are road cracking, horizontal curvature, and vertical curvature. The dynamics created by typical road geometries greatly affect the accuracy of a WIM system. Road cracking around the desired site location should be minimal. If road cracking does exist the vertical humping around the crack should be removed through a milling process. The system should also be located in such a way that no cracking will cross any part of the system sensors. If necessary a new resurface should be performed to ensure the quality of the road. The road grade needs to be less than 1% for at least 1000 ft before the sensors. No vertical or horizontal curvature should exist for at least 1000 ft before the system. Figure 4.1 provides an overview of the final site selected for the project’s WIM system.

Due to the road geometry and utility access, the WIM site is located one mile from the structure. It is desired to install the WIM as close to the structure as possible to capture the true traffic impacting the bridge and to eliminate any bias as to the exact vehicle being measured. The bridge approach grade and horizontal curvature on either side of the structure prohibited locating the WIM immediately adjacent to the structure. A site located one mile west of the structure was chosen as the most ideal location meeting the desired requirements for WIM accuracy. This site was capable of providing access to utilities and the appropriate road geometry. Figure 4.2 provides a map showing the site location an overview of the road geometry necessitating a site location that was not immediately adjacent to the structure.

4.2.2 Types of WIM Systems

The use of a piezoelectric WIM system was chosen over other systems for this project. These systems have been used extensively throughout the world and have proven to be very economical and accurate. Road conditions would not permit quartz sensors and the slightly greater accuracy of other systems did not warrant additional cost.

4.2.3 WIM Sensors and Installation

The installation of the WIM system was coordinated through Purdue University and the Indiana Department of Transportation (INDOT). Hawk Enterprises installed the WIM system on October 29, 2001 with underground work being performed a couple of weeks prior. International Road Dynamics (IRD) provided all WIM accessories and WIM piezoelectric sensors.

All loop detectors, controller cabinets, and piping are typical and were performed to INDOT Standards, Section 805. IRD sensors of type Class I Measurement Specialties Corporation RoadTrax BL Series Piezo Sensors were used on this project. In addition to the road sensors, IRD supplied all electrical equipment as detailed in INDOT Contract No. T-25097-A. Figure 4.3 presents the WIM layout.

Installation began by determining the most appropriate location for the monitoring cabinet. Once the cabinet location was marked, all underground and off-road work could be performed. The underground work involves trenching and boring for all piping required to house wires. This work was completed a couple of weeks in advance of the scheduled roadwork. On October 29, 2001 all roadwork was performed (Figure 4.4). The first course of action was determining the exact and most appropriate sensor locations within site boundaries. The locations of all inductive loops, and piezoelectric sensors were then measured and marked with marking paint. The marked inductive loops and piezoelectric locations were then sawcut to the appropriate depth and width as determined by IRD. After sawcutting, inductive loops and piezoelectric sensors could be placed. After placing the sensors, grout was mixed to seal all sawcuts. After installation it is necessary to connect all wires within the monitoring cabinet and to perform a calibration of the system. Calibration of the WIM system involves multiple passes of a fully loaded 5-axle semi trailer (Figure 4.5). Adjustment factors are then applied to the WIM algorithm to ensure accurate measurements.

4.3 Bridge Instrumentation

The first decision in bridge instrumentation is to decide on the appropriate gage type(s) and gage locations. Due to a necessary limitation upon the quantity of gages connected to the structure, the structure configuration, impact considerations, and the accessibility of certain spans, it was decided that only the two easternmost spans would be instrumented. Moreover, since most loaded “Michigan Trains” were estimated to travel eastbound it was also decided to instrument only the eastbound structure. Strain measurements were used in correlation with the WIM measurements to determine the bridge response and typical patterns.

Electrical resistance gages are used in a variety of experimental testing environments and have proven accurate and reasonably simple to use. Consequently, electrical resistance strain gages were used to determine the strains being distributed through the steel bridge members.

4.3.1 Strain Gage Selection

Numerous strain gages are available for various testing environments. Several criteria must be considered when determining which particular type of strain gage to use. Some of these criteria are the testing material, environmental exposure, strain level, and the desired testing results. Using Vishay’s Measurements Group technical reports and Mico-Measurements references, a CEA-06-250UN-350 type strain gage was selected. This gage is ¼” long with enlarged soldering tabs and a gage resistance of 350 Ohms.

4.3.2 Gage Location Selection

The gage locations to be used for this project will serve two purposes. The gages must be capable of providing strain information as it relates to fatigue critical details and provide loading patterns. With this in mind two typical locations were determined necessary. These locations are at fatigue critical details and in the maximum moment regions. In each of these locations strain gages were placed as to provide the strain

distribution at the critical beam sections. These locations were determined to provide the most critical information for this project.

4.3.2.1 Determination of Fatigue Critical Details

The use of the fatigue detail categories in the LRFD Standard Specifications (AASHTO, 1998) enables one to determine the critical details. Detail categories C through E are generally considered critical or fatigue governing details. These connections are considered critical because they have lower cyclic lives than details in categories A through B'. Strain information in locations where the critical details are positioned can be used in developing a random loading model in further studies. Areas of fatigue concern for the U.S. 20 bridge were at the diaphragm connections.

As previously discussed, there are two typical connection details for the diaphragms: the first being intermittently welded to the beam web and the second being bolted to an attachment plate that is welded to the beam web (Figures 4.6 and 4.7).

To provide an understanding of the magnitude and distribution of strain in the diaphragm area, six (6) gages were connected to the beam, three (3) on each side of the web. A gage was attached to the bottom of the beam top flange, slightly below the diaphragm, and on the top of the beam bottom flange (Figures 4.8 and 4.9). For complete details on the cross sectional gage location, refer to Appendix C. The gages located near the diaphragm will be referred to as diaphragm gages hereafter.

A second fatigue critical detail on the U.S. 20 bridge involves an improperly located attachment plate (Figure 4.10). It was intended to use this plate as a diaphragm attachment. However, the plate was improperly located and thus unsuitable for use. Rather than removing the shear plate, it was left intact.

4.3.2.2 Other Gage Locations

For the study it is also desirable to assess the maximum strain behavior of the structure. This information will provide the maximum positive moment strain ranges the

bridge structure undergoes. This is accomplished by locating gages at the maximum strain or moment locations. The maximum strain locations were determined from an analysis of the structure. In the maximum moment locations, near midspan, four (4) gages were connected with one gage located at the bottom of the top flange, one gage located in line with the top of the diaphragm, one gage located in line with the bottom of the diaphragm, and one on the top of the bottom flange (Figures 4.11 and 4.12). For complete details on the cross sectional gage location, refer to Appendix C. By monitoring both the diaphragm and maximum moment locations, then relevant information can be determined from strain changes as vehicles pass over the bridge.

Another area of possible scrutiny is over the negative moment /support region. Long bolted splice plates were used at each support to join together beam members from adjacent spans. It was determined that gages located in this region, although providing interesting information, would be difficult to instrument and may provide unpredictable information. The size and extent of the bolted splice plates in the negative moment location would prohibit the collection of any pertinent information. Moreover, the bolted splice is a category B fatigue detail, which should not experience significant decrease in cyclic life due to the excellent fatigue resistance of the detail.

4.3.2.3 Gage Summary

In total forty-three strain gages were connected to the bridge structure providing four (4) moment locations and five (5) diaphragm locations. A summary of the instrumented locations and gage numbering is provided in Table 4.1 and Figure 4.13.

4.3.3 Gage Installation Procedure

The next step in instrumentation was the installation of gages on the bridge structure in the field. The installation procedure required approximately two weeks and the use of INDOT bucket trucks. The installation of a P-Type traffic cabinet to house the data acquisition units was completed prior to any gage installation. The cabinet was

located in a way that would require the minimum cable lead length from the strain gage to the data acquisition equipment.

Gage locations were first determined and their position then marked. An electric disc grinder was used to remove the paint layer and any steel pitting and provide a rough steel surface. The area was then prepped to provide a smooth, contaminant free surface as per instructions provided by Micro-Measurements Instruction Bulletin B-137-16 (1995). The electrical resistance strain gages of type CEA-06-250UN-350 and solder terminals were then bonded to the steel surface with M-Bond AE-10 adhesive as per the aforementioned instruction bulletin.

Conduit was then brought from the cabinet, attached vertically to the support column, ran lengthwise along the beams, and terminated in the appropriate locations. Gage 22, twisted, three conductor, shielded cable was then pulled through the conduit using an electrical fishtape from the cabinet to its appropriate location. The shielded cable was then soldered to the solder terminals and strain gages.

To protect the gages from the harsh exterior environment a combination of a microsilicon wax and Micro-Measurements M-Coat F was utilized. The application of M-Coat F is detailed in Micro-Measurements Instruction Bulletin B-134-4 (1996).

4.3.4 Data Acquisition

Following installation, all gages were then connected to the data acquisition equipment. The data acquisition equipment was supplied by Campbell Scientific.

There are several types of data acquisition equipment available by several manufacturers. To determine which system would provide the project with the most benefit several criteria were determined. Some of these requirements are the following, not necessarily listed in order of importance:

- 1) Ability to scan electrical resistance strain gages over several channels.
- 2) Must be nearly dynamic with a scan rate of at least 50 Hz over 20-30 channels.
- 3) Adequate data storage space without the use of a computer.
- 4) Ability to withstand an exterior environment with large temperature changes.

- 5) Ability to communicate with telephone, cable, or cellular modem and provide remote access.

Based upon these criteria the CR5000 Measurement and Control System supplied by Campbell Scientific was determined to be the most appropriate system for the project (Figure 4.14). The CR5000 met all criteria was simple to use and provided large flexibility to the measurement decisions.

Table 4.1 – Summary of Strain Gage Location and Number

Member/Gage Location	Gage #	Location
Beam #8/Moment-Interior Span	1-8-M-N-1	Bottom of Top Flange
	1-8-M-N-2	Beam Web In-Line w/ Top of Diaphragm
	1-8-M-N-3	Beam Web In-Line w/Bottom of Diaphragm
	1-8-M-N-4	Top of Bottom Flange
Beam #8/Diaphragm-Interior Span	1-8-D-N-1	Bottom of Top Flange
	1-8-D-N-2	Beam Web at Bottom of Diaphragm
	1-8-D-N-3	Top of Bottom Flange
	1-8-D-S-1	Bottom of Top Flange
	1-8-D-S-2	Beam Web at Bottom of Diaphragm
	1-8-D-S-3	Top of Bottom Flange
Beam #8/Moment-End Span	2-8-M-N-1	Bottom of Top Flange
	2-8-M-N-2	Beam Web In-Line w/ Top of Diaphragm
	2-8-M-N-3	Beam Web In-Line w/Bottom of Diaphragm
	2-8-M-N-4	Top of Bottom Flange
Beam #8/Diaphragm-End Span	2-8-D-N-1	Bottom of Top Flange
	2-8-D-N-2	Beam Web at Bottom of Diaphragm
	2-8-D-N-3	Top of Bottom Flange
	2-8-D-S-1	Bottom of Top Flange
	2-8-D-S-2	Beam Web at Bottom of Diaphragm
	2-8-D-S-3	Top of Bottom Flange
Beam #10/Moment-Interior Span	3-10-M-N-1	Bottom of Top Flange
	3-10-M-N-2	Beam Web In-Line w/ Top of Diaphragm
	3-10-M-N-3	Beam Web In-Line w/Bottom of Diaphragm
	3-10-M-N-4	Top of Bottom Flange
Beam #10/Diaphragm-Interior Span	3-10-D-N-1	Bottom of Top Flange
	3-10-D-N-2	Beam Web at Bottom of Diaphragm
	3-10-D-N-3	Top of Bottom Flange
	3-10-D-S-1	Bottom of Top Flange
	3-10-D-S-2	Beam Web at Bottom of Diaphragm
	3-10-D-S-3	Top of Bottom Flange
Beam #10/Moment-End Span	4-10-M-N-1	Bottom of Top Flange
	4-10-M-N-2	Beam Web In-Line w/ Top of Diaphragm
	4-10-M-N-3	Beam Web In-Line w/Bottom of Diaphragm
	4-10-M-N-4	Top of Bottom Flange
Beam #10/Diaphragm-End Span	4-10-D-N-1	Bottom of Top Flange
	4-10-D-N-2	Beam Web at Bottom of Diaphragm
	4-10-D-N-3	Top of Bottom Flange
	4-10-D-S-1	Bottom of Top Flange
	4-10-D-S-2	Beam Web at Bottom of Diaphragm
	4-10-D-S-3	Top of Bottom Flange
Beam #9/Attachment Plate- End Span	5-9-S-S-1	Bottom of Top Flange
	5-9-S-S-2	Beam Web In-Line w/Bottom of Diaphragm
	5-9-S-S-3	Top of Bottom Flange

Figure 4.1 – WIM Site Overview

Figure 4.2 – WIM Location

* Distance/Placement to be determined in the Field

Figure 4.3 – WIM Layout

Figure 4.4 – WIM Installation

Figure 4.5 – WIM Calibration Truck

Figure 4.6 – Typical Intermittent Weld Diaphragm-to-Beam Connection

Figure 4.7 – Typical Bolted/Welded Plate Diaphragm-to-Beam Connection

Figure 4.8 – Midspan Diaphragm Gage Locations and Numbering Scheme

Figure 4.9 – View of Diaphragm Gages on Beam 10

Figure 4.10 – Improperly Located Attachment Plate

Figure 4.11 – Near Midspan Gage Locations and Numbering Scheme

Figure 4.12 – View of Near Midspan Gages on Beam 10

Figure 4.14 – Campbell Scientific Model CR5000 Data Acquisition System

CHAPTER 5 –MEASUREMENTS

5.1 Overview

The proper programming of the data acquisition equipment is essential in collecting reliable and useful data. This programming determines the format and storage of information available during the measurements stage of the experiment. The WIM system is largely preprogrammed according to contract specifications and limited in its ability to be customized. However, the strain system is very flexible. The strain system is completely customizable making it necessary to determine the desired results before installation.

5.2 Weigh-in-Motion Measurements

The WIM system developed by IRD provides the truck axle weight information required by the project. The WIM system continuously monitors traffic and provides a timestamp, Federal Highway Administration vehicle classifications (Appendix D), gross vehicle weights (GVW), axle weights, axle spacing, speed, driving lane, 18-kip equivalent single axle load (ESAL), total vehicle length, and traffic counts. The pertinent information can be viewed in real-time or stored for later data viewing. The WIM system is capable of storing at least ten years of traffic information based on current traffic volumes.

5.2.1 WIM Data Formats

Communications with the WIM station is accomplished onsite or through a telephone modem allowing for both real-time monitoring and data collection. Real-time

monitoring can present data in two basic formats, graphical and text. Both formats provide the same information for every vehicle passing over the system. The graphical format, as its name implies, presents the vehicle information graphically with each axle represented by a small circle with summarizing data above the graphical display. For each of the circles/axles the format provides the axle load below and the axle spacing above (Figure 5.1). The text format provides the same information without the graphical display (Figure 5.2).

Data can also be stored for later viewing. There are several options available with later data viewing. Vehicle information can be viewed in a similar format to the real-time viewing with the ability to scroll through vehicles recorded at different times. In addition, the data can be used to create several different reports that summarize data based on different preset requirements. All data can also be transferred to a text file to view all information stored on the WIM system. This data transfer ability is used to transfer WIM data to the Microsoft SQL Server 7.0 database created for data storage and manipulation.

5.2.2 Uses of WIM Data

The WIM system provides traffic patterns that are necessary in determining loading histories to be used in the development of a fatigue reliability model. The traffic patterns are used to develop approximate strain patterns in relation to vehicle loadings or configurations. These approximated strain patterns are compared with the actual strain patterns developed through the bridge instrumentation in coordination with the vehicle data as captured by the WIM system. As with all WIM systems, some error is inherent. In spite of this error, data provided by the WIM is used as an approximation of actual traffic patterns and is not viewed as exact.

5.3 Strain Measurements

Strain measurements are an integral part of the project and will provide information pertaining to the bridge behavior under different vehicle loadings and configurations. Strain patterns from different vehicles will be used in coordination with the WIM measurements to determine approximate strain patterns as related to their corresponding loadings and configurations.

5.3.1 Strain Measurements Procedure

As previously discussed in Chapter 3, a total of forty-three strain gages were attached to the bridge structure to provide the most beneficial information. Two Campbell Scientific CR5000 dataloggers were purchased to perform the strain data measurements. Each gage was attached to a unique channel on the CR5000 (Table 5.1 and Figure 5.3). However, each CR5000 unit can measure across only twenty channels. It was thus necessary to determine which gages would provide the most relevant information and rely on the three remaining gages for spares in the event that any other gages failed. It was determined that the three gages attached to beam #9 would serve as spares.

5.3.1.1 Datalogger Programming

Once all gages were attached to the system, a CR5000 program was developed to scan the gages and retrieve the bridge strains. The CR5000 program determines the operation of the dataloggers: the scan rate, conditions to determine the data to be collected, and the data storage format.

To enable the determination of dynamic effects, the CR5000 was set to scan the strain gages at a rate of 100 Hz. This scan rate was determined to be appropriate based upon a rough estimate of the natural period of the bridge structure. It is generally recommended to scan strain gages at a rate at least 10 times the natural frequency of the structure. The natural period of the bridge structure was determined analytically as

approximately 0.213 seconds corresponding to a frequency of 5.7 Hz. Thus a minimum of 50 Hz was desired.

The system was designed to continually monitor strain data. Due to the scan rate and number of channels recording data it would be impractical to continuously store all data. To capture data during significant truck loading periods only, a trigger variable was established within the datalogger programming. The trigger was set on a bottom flange gage at $30\ \mu\epsilon$ correlating to a vehicle with GVW of approximately 40 kips. It was decided any vehicles inducing smaller strains would be considered negligible, since it was estimated that they caused a stress excursion below the fatigue limit state thus inducing no fatigue damage.

A trigger was set on the bottom flange gage positioned near the midspan in all four locations: interior span beam #8, interior span beam #10, end span beam #8, and end span beam #10 for a total of four triggers. Each trigger was used to activate the recording of 10 gages in its respective span and beam line. Using the trigger event, anytime the declared bottom flange gage reached a strain of $30\ \mu\epsilon$ the dataloggers would record data for approximately 2.5 seconds before and after the event. This time interval and strain level was determined by monitoring vehicle information as provided by the WIM and the resulting strain event. A total interval of 5 seconds was determined adequate in providing a sample of strain readings before and after all dynamic loading effects.

5.3.1.2 Data Collection

Data is stored in the CR5000 on an 85 MB flash disk memory card. Data can be retrieved from this PC Card by removing the card and reading with a PC Card reader or directly through the data communications port on the CR5000. Data is initially stored on the PC Card in binary format. This data can then be collected using software provided by Campbell Scientific and converted to ASCII format. All data were stored in each datalogger in a series of 20 tables, 10 for each span. The use of multiple tables allows for smaller data sets and easier downloads.

Communication with the dataloggers is essential. As with the WIM, the dataloggers can be controlled either onsite or remotely. Onsite communication is established simply by connecting a serial cable from a laptop/computer to the CR 5000's RS-232 communications port. Due to the location of the site relative to Purdue University, it was also desirable to establish a form of remote communication. Several forms of communication were discussed and evaluated: cellular phone link, cable, telephone, and ethernet. Due to site restrictions and accessibility, the only practical form of communication was through a telephone line provided by SBC Ameritech. Communication with the CR5000 is accomplished in much the same manner as the direct connection. Using a serial cable and a null modem cable, a U.S. Robotics 56K external modem was connected to the datalogger's RS-232 communications port. Using software provided by Campbell Scientific, a remote connection established from the office computer to the site modem could be achieved. This connection provides real-time monitoring and data collection abilities.

All data from strain measurements will be used to determine strain patterns and events. These data will then be imported into the SQL database and correlated to the WIM data.

5.3.1.3 Strain Measurements Calibration

Calibration of strain measurements was performed using a loaded two axle dump truck provided by INDOT (Figure 5.4). The truck was loaded with gravel and weighed at a nearby weigh-station on Interstate 94. The empty weight (24,360 lbs), loaded weight (30,900 lbs), and individual axle weights were measured to provide known weights for measurements and later analysis. In addition the axle spacing, axle width, and tire contact area were measured see Figure 5.5.

The calibration was composed of both a static and dynamic loading situation. Several static loading locations were used. The static loading contains four main locations, one in each of the two instrumented spans and each eastbound lane. These loading positions correspond to locations that provide maximum strains in the gage

locations (Figure 5.6). Within each main location, fifteen secondary stations were used: a grid with 5 longitudinally spaced at 2' and 3 transversely spaced at 1'6." These stations were marked on the deck to determine the effects of loading off the center of the driving lane. Each marked location corresponded to the middle of the rear axle tire (Figure 5.7). The gages were zeroed with no loading on the bridge structure. The truck was then moved into each position and measurements were taken with no other vehicles on the bridge.

After static loading, a dynamic loading phase was then executed. To perform dynamic loading the calibration truck was driven over the structure at both a crawl (≈ 5 mph) and highway speed (≈ 55 mph). The truck made one pass at each speed and in each lane. Measurements were performed as the truck drove over the bridge.

While reviewing the data collected during dynamic calibration several unusual readings and data truncation were observed. These truncated readings were later found to be related to early datalogger operating system flaws and overloading of the equipment processor. Future calibrations were not possible due to costs and scheduling conflicts. Due to the number of obscure and truncated readings, the data from dynamic calibration was very unreliable.

During the WIM system calibration minimal dynamic testing was available. To perform the dynamic testing, the loaded WIM calibration truck was driven over the bridge at highway speed in both lanes and measurements performed. Measurements were also taken with the WIM calibration truck driving in the right/outside lane at crawling speed. Due to time constraints and truck availability left/inside lane measurements at crawling speed was not available. Nevertheless, these measurements were very useful in evaluating system response under a known loading

Table 5.1 – Gage Wiring Schematic

Gage #	Location	CR5000 Serial #	Channel #
1-8-M-N-1	Bottom of Top Flange	1101	1
1-8-M-N-2	Beam Web In-Line w/ Top	1101	2
1-8-M-N-3	Beam Web In-Line	1101	3
1-8-M-N-4	Top of Bottom Flange	1101	4
1-8-D-N-1	Bottom of Top Flange	1101	5
1-8-D-N-2	Beam Web at Bottom of	1101	6
1-8-D-N-3	Top of Bottom Flange	1101	7
1-8-D-S-1	Bottom of Top Flange	1101	8
1-8-D-S-2	Beam Web at Bottom of	1101	9
1-8-D-S-3	Top of Bottom Flange	1101	10
2-8-M-N-1	Bottom of Top Flange	1101	11
2-8-M-N-2	Beam Web In-Line w/ Top	1101	12
2-8-M-N-3	Beam Web In-Line	1101	13
2-8-M-N-4	Top of Bottom Flange	1101	14
2-8-D-N-1	Bottom of Top Flange	1101	15
2-8-D-N-2	Beam Web at Bottom of	1101	16
2-8-D-N-3	Top of Bottom Flange	1101	17
2-8-D-S-1	Bottom of Top Flange	1101	18
2-8-D-S-2	Beam Web at Bottom of	1101	19
2-8-D-S-3	Top of Bottom Flange	1101	20
3-10-M-N-1	Bottom of Top Flange	1189	1
3-10-M-N-2	Beam Web In-Line w/ Top	1189	2
3-10-M-N-3	Beam Web In-Line	1189	3
3-10-M-N-4	Top of Bottom Flange	1189	4
3-10-D-N-1	Bottom of Top Flange	1189	5
3-10-D-N-2	Beam Web at Bottom of	1189	6
3-10-D-N-3	Top of Bottom Flange	1189	7
3-10-D-S-1	Bottom of Top Flange	1189	8
3-10-D-S-2	Beam Web at Bottom of	1189	9
3-10-D-S-3	Top of Bottom Flange	1189	10
4-10-M-N-1	Bottom of Top Flange	1189	11
4-10-M-N-2	Beam Web In-Line w/ Top	1189	12
4-10-M-N-3	Beam Web In-Line	1189	13
4-10-M-N-4	Top of Bottom Flange	1189	14
4-10-D-N-1	Bottom of Top Flange	1189	15
4-10-D-N-2	Beam Web at Bottom of	1189	16
4-10-D-N-3	Top of Bottom Flange	1189	17
4-10-D-S-1	Bottom of Top Flange	1189	18
4-10-D-S-2	Beam Web at Bottom of	1189	19
4-10-D-S-3	Top of Bottom Flange	1189	20
5-9-S-S-1	Bottom of Top Flange	Varies	Spare
5-9-S-S-2	Beam Web In-Line	Varies	Spare
5-9-S-S-3	Top of Bottom Flange	Varies	Spare

(105) LANE #3 TYPE 9 GVW 76.1 kips LENGTH 68 ft
 18-K ESAL 2.099 SPEED 47 mph MAX GVW 80.0 kips Wed Jan 02 00:35:23.31 2002

Figure 5.1 – WIM Graphical Format

(105) LANE #3 TYPE 9 GVW 76.1 kips LENGTH 68 ft
 18-K ESAL 2.099 SPEED 47 mph MAX GVW 80.0 kips Wed Jan 02 00:35:23.31 2002

UNIT	SEPARATION	WEIGHT	ALLOWABLE
	(ft)	(kips)	(kips)
1		10.3	20.0
2	18.3	17.2	17.0
3	4.5	15.4	17.0
4	34.1	15.7	17.0
5	4.1	17.5	17.0

Figure 5.2 – WIM Text Format

Figure 5.3 – Datalogger Wiring

Figure 5.4 – Strain Calibration Truck

Strain Calibration Truck - GVW 30.9k

Figure 5.5 – Strain Calibration Truck Schematic

BLOWUP OF FRAMING PLAN FROM P9 TO P11
 ° DENOTES REAR TIRE LOCATION

Figure 5.6 – Static Loading Locations

Figure 5.7 – Static Loading Wheel Location

CHAPTER 6 – WEIGH-IN-MOTION RESULTS

6.1 Overview

Weigh-In-Motion (WIM) systems are widely used throughout the world. Their purpose is to aide in the collection of vehicle data. WIM's have the ability to weigh vehicles and gather vehicle data at highway speeds and without bias. Most WIM systems are used by Department of Transportations for weight enforcement and road and bridge design. A WIM system was installed and used in this project to gather truck data and to aide in the determination of the correlation of truck weight and configuration to bridge strains and fatigue damage. The performance and details of the WIM measurements are described in Chapter 3.

6.2 Truck Traffic

Of primary importance for the project is truck traffic. Most other vehicles do not induce large enough strain levels to reach the variable-amplitude fatigue endurance limit of various steel details and, thus, they create negligible fatigue damage. To aide in the development of a fatigue reliability model to be used for bridges along the corridor, adequate truck information must be gathered to enable the estimation of truck loading histories and to predict future loadings

The WIM system collects several very important factors needed to determine the fatigue damage trucks create. These factors include:

- 1) Average daily truck traffic detailing driving lane, travel direction, and travel time.
- 2) FHWA Truck Classification including the number of axles and axle spacing.
- 3) Gross vehicle weight (GVW), and axle weight.

Other factors such as truck speed and the 18 kip equivalent single axle load (ESAL) are also available.

The AASHTO 1998 LRFD code determines the fatigue life of a structure dependent upon the type of detail, stress ranges, and the number of loading cycles. There are several methods in determining the number of loading cycles. However, details of these methods are beyond the scope of this report. When evaluating the fatigue life of a bridge structure using the AASHTO code, it is necessary to know the traffic volume and axle loads the bridge will experience to determine the stress range at the critical details and number of loading cycles. The traffic volume can be determined using the expected average daily truck traffic (ADTT).

Table 6.1 provides the ADTT of all FHWA truck classes (FHWA truck classes are provided in Appendix D) along the highway segment including the bridge structure in the study. The most common trucks are Class 5 trucks with a total ADTT of 385, Class 9 trucks with a total ADTT of 587, and Class 13 trucks with a total ADTT of 192. Class 5 trucks are two axle trucks with dual rear wheels and would include trucks used for general purpose delivery of small and low quantity goods. Class 9 trucks are the typical 5-axle truck used for most truck deliveries and are the most commonly used truck on U.S. highways. Class 13 trucks are 9 and 11 axle trucks that are referred to as “Michigan Trains.” The table also verifies the previous assumption that a larger number of trucks travel in the eastbound direction with a total eastbound and westbound ADTT of 785 and 550, respectively. This direction preference is more apparent for “Michigan Train” truck traffic with 64% of the Class 13 trucks traveling in the eastbound direction. Also shown is the breakdown of the ADTT for each lane of traffic. This breakdown shows very little truck traffic in the left/passing lane. It is also believed that a majority of truck traffic that travels over the WIM in the passing lane switches to the driving lane before it reaches the study bridge.

Figure 6.1 details the times that all FHWA truck classes travel. The figure shows that most loads lighter than the 80,000 lb weight limit travel during the middle two quarters of the day, from 6 am – 6 pm, while heavier trucks travel during the latter two quarters of the day, from 12 pm – 12 am. Figure 6.2 details the times Class 9 trucks

travel. Most Class 9 trucks show a preference to travel during the middle two quarters of the day. Figure 6.3 details the times traveled by Class 13 trucks. Most Class 13 trucks show a preference to travel during the latter two quarters of the day.

6.3 Truck Weight

In addition to the loading cycles the stress range is also very important when determining the fatigue life of the structure. Truck GVW and axle spacing primarily determine the stress range. A heavier truck will usually correlate to higher bridge member stress ranges. However, due to the “Michigan Train’s” configuration and number of axles this may not be completely accurate. (The stress ranges induced by these trucks will be discussed in further detail in Chapter 7.)

As discussed previously, the legal weight limit of “Michigan Train” trucks on the extra heavy duty highway is 134,000 lbs, which is considerably greater than the 80,000 lbs legal weight limit on typical interstate and state highways. Table 6.2 provides the average truck weights of the various FHWA truck classes. The average GVW of all trucks is 52,368 lbs showing that most trucks are below the 80,000 lbs legal limit. The most common truck, the Class 9 truck, has an average GVW of 54,356 lbs. The average GVW of the “Michigan Train” truck, Class 13, is considerably higher than other truck classes with an average GVW of 119,459 lbs. There is a small difference in the GVWs of eastbound and westbound “Michigan Trains,” an indication that very few travel unloaded. On interstate and state highways there is a tendency for overweight trucks to travel during off-peak hours. However, overweight trucks along the extra heavy duty highway do not show any strong tendency to this time preference.

Figure 6.4 displays the distribution of GVWs of all trucks. The distribution shows a large variation in GVW with three peaks. The first peak shows that a number of trucks travel unloaded. The second peak around 70,000 lbs is near the 80,000 lbs legal weight limit associated with Class 9 trucks. The third peak is around 115,000 lbs and is close to the extra heavy duty 134,000 lbs legal weight limit.

Figure 6.5 is the GVW distribution of Class 9 trucks. Notice the two peak distribution. The first peak shows a large number of empty or very lightly loaded trucks and the second peak, near the legal weight limit, shows that most Class 9 trucks travel heavily loaded. The average GVW of Class 9 trucks plus 1 standard deviation is still under the legal limit and about 78,000 lbs. Although most trucks travel under the legal weight limit, 15% of Class 9 trucks still travel overweight.

Figure 6.6 is the GVW distribution of Class 13 trucks. Notice the single peak of the GVW distribution indicating that most Class 13 trucks travel loaded and rarely travel unloaded or lightly loaded. The average GVW plus 1 standard deviation of Class 13 trucks is about 150,000 lbs, well above the 134,000 lbs legal “Michigan Train” limit. Of some interest are some of the extreme weights that “Michigan Train” trucks travel. During the study several trucks of more than 200,000 lbs were observed. In addition to these extreme weights, 26% of “Michigan Train” trucks were observed to travel overweight.

Table 6.1 – ADTT Summary

Truck Class	ADTT All Directions	ADTT Westbound		ADTT Eastbound	
		Right Lane	Left Lane	Right Lane	Left Lane
Class 5	385	113	32	180	60
Class 6	71	32	5	23	11
Class 7	12	6	1	4	1
Class 8	41	20	2	15	4
Class 9	587	218	30	269	70
Class 10	35	16	2	14	3
Class 11	2	1	0	0	1
Class 12	10	3	0	6	1
Class 13	192	62	7	119	4
All Trucks	1335	471	79	630	155

Table 6.2 – Average Truck Weight

Truck Class	GVW All Directions	GVW Westbound	GVW Eastbound
Class 5	10,741 lbs	11,046 lbs	10,436 lbs
Class 6	31,599 lbs	31,429 lbs	31,769 lbs
Class 7	76,812 lbs	75,676 lbs	77,948 lbs
Class 8	28,936 lbs	28,910 lbs	28,961 lbs
Class 9	54,356 lbs	51,163 lbs	57,544 lbs
Class 10	65,842 lbs	60,422 lbs	71,261 lbs
Class 11	46,664 lbs	48,732 lbs	44,596 lbs
Class 12	83,991 lbs	75,289 lbs	92,692 lbs
Class 13	119,459 lbs	118,384 lbs	120,534 lbs
All Trucks	52,368 lbs	47,776 lbs	56,959 lbs

Count of GVW by Quarter - All Classes -- All Lanes
01/01/02 - 04/30/02

Figure 6.1 – Truck Count by Quarter of the Day (6 hours)

Count of GVW by Quarter for Class 9 -- All Lanes
01/01/02 - 04/30/02

Figure 6.2 – FHWA Class 9 Truck Count by Quarter of the Day (6 hours)

Count of GVW by Quarter for Class 13 -- All Lanes
01/01/02 - 04/30/02

Figure 6.3 – FHWA Class 13 Truck Count by Quarter of the Day (6 hours)

Figure 6.4 – GVW Distribution of All Trucks From January Through April 2002

**GVW of Class #9 - All Lanes
01/01/02 To 04/30/02**

Figure 6.5 – GVW Distribution for FHWA Class 9; January Through April 2002

GVW of Class #13 - All Lanes
01/01/02 To 04/30/02

Figure 6.6 – GVW Distribution for FHWA Class 13; January Through April 2002

CHAPTER 7 – STRAIN RESULTS

7.1 Overview

Strain gages are very versatile instruments used to aide in the measurement of material response to various loading and environmental conditions. Consequently, strain gages were installed and monitored to provide information pertaining to the response of the test bridge under vehicle loadings. Using the strain gage data in conjunction with the Weigh-In-Motion (WIM) data, it is possible to determine the impact certain vehicles create on the superstructure of the bridge in this study. Details of the strain gage instrumentation are provided in Chapter 4 (“Instrumentation”). This chapter will describe the information gathered using the strain gage instrumentation.

7.2 Strain Measurements

The strain gage measurements are gathered, stored, and retrieved from the Campbell Scientific data acquisition equipment installed at the bridge location. The strain readings collected were used to evaluate the bridge response and resulting fatigue damage due to truck loadings.

In the LRFD Standard Specification (AASHTO, 1998), the design fatigue life of a structure is dependent upon the type of detail, stress range, and the number of stress range cycles. Therefore, to estimate the fatigue life, the true measured maximum and minimum stress and the number of loading cycles incurred throughout the life of the structure are of interest. The stress range is obtained simply by taking the difference between the maximum stress and minimum stress induced during a loading cycle. There are several methods in determining the number of stress range cycles; including the rainflow method, the reservoir method, the peak count method, and the mean-crossing peak count method.

Complete details of these methods are beyond the scope of this report. In several design codes the fatigue life of a detail is characterized by load induced stress range cycles. However, since bridge strain measurements were collected during the study, all code specified stresses will be converted to their equivalent strains. Most structures, as well as the study bridge, are designed to operate in the elastic region. Within the elastic region stress and strain are simply correlated through the material's Modulus of Elasticity.

A summary of the maximum, minimum, and average strains observed in some of the bottom flange gages of beam #10, during a study period of four months, is provided in Table 7.1. Information detailing the instrumentation and gage locations is provided in Chapter 4. The slight differences observed in the gages on opposite sides of the diaphragm are due to measurement noise. The strain results shown in Table 7.1 reveal a large range of strains in both tension and compression induced in the bottom flanges of the structure. The negative/compressive strains result from strains measured while the truck is in the preceding span and demonstrates the continuity of the structure. The average maximum strain values for all trucks are considerably smaller than the absolute maximum strain values, illustrating that many vehicles are lightly loaded or configured in a way that does not induce high strains. Also notice that the maximum strains in the end span are considerably higher than the interior span. Most of this difference is due to the beam size within the end span, however, a large portion of the difference is caused by impact as the truck contacts the uneven end joint.

The strains caused by vehicles driving over the structure are generally higher than that which could be caused by their static weights. Road dynamics, uneven pavement, shifting loads, centrifugal tire forces, and several other factors act to create an impact on the structure as it is loaded. The impact on the structure will vary tremendously depending on the loading location, amount of joint unevenness, and several other factors. The impact however can be generally determined using static and dynamic loading.

A calibration truck with known axle loads was driven over the structure at about 5 mph, simulating a static loading, and also at highway speed, simulating a dynamic loading. Figure 7.1 shows the strain data gathered from the end span for the WIM calibration truck (Figure 8.8) during the simulated static and dynamic loading on a time adjusted scale. The

strain difference between the two strain measurements is caused by the dynamic impact. The maximum strain values in the strain gages for each loading scenario were then compared. For the given truck it was found that within the interior span little to no impact occurred since no difference in strain was present. However for the end span, a 13% difference in strain was found in the bottom flange gage near midspan. As noted earlier, the impact (strain difference) is believed to be due to the truck wheels hitting the joint at the end of the bridge.

Tables 7.2 and 7.3 detail the maximum, minimum, and average strains observed for the same bottom flange gages of Table 7.1 separated by Class 9 and Class 13 trucks, respectively. The maximum and minimum strains of both truck classes reveal that heavy Class 9 and Class 13 trucks induce similar strains. However, the average strain of the Class 13 truck is considerably higher, approximately $15\mu\epsilon$ in the interior span and $20\mu\epsilon$ in the end span, than the Class 9 truck. This is expected given the heavier average GVW of Class 13 trucks as shown in Chapter 6, Table 6.2.

Histograms of the maximum strains observed throughout the study for the same gages as detailed in Tables 7.1 through 7.3 are provided in Figures 7.2 through 7.5. Figures 7.2 through 7.4 show the histograms of the maximum strains in the bottom flange interior span gages. The strain histograms show that all three gages provide very similar results and strain distributions. The close proximity of these three gages justifies the similarity between the gages. Within each figure the histograms are provided for all trucks and then separated to show the distribution differences between truck Class 9 and Class 13. Due to the high traffic volumes of Class 9 trucks it can be seen in Figures 7.2 and 7.3 that the distribution for all trucks is controlled by the strain distribution of Class 9 trucks. The peak frequency for all trucks and Class 9 trucks are equivalent and is around $45\mu\epsilon$. Figure 7.4 shows that Class 13 trucks exhibit a fairly flat distribution with an almost indistinguishable peak frequency. It can be seen from the Class 13 distribution that Class 13 trucks generally cause much higher strains than the Class 9 truck.

Figure 7.5 shows the strain distribution for the beam #10 end span bottom flange gage. The strain distribution in this span is considerably different than seen in the interior span (Figures 7.2-7.4). Again due to the high Class 9 truck volumes the distribution for

all trucks and Class 9 trucks are very similar. Unlike the single peak exposed in the interior span the end span shows two very distinct peaks; one at low strains near $33\mu\epsilon$ and another at higher strains near $67\mu\epsilon$. Since the distributions are created using the same sample of trucks, the two peak observation is very interesting and might be a result of additional restraint from web stiffeners and the end span supports and varying impacts as the truck travels over the end joint. The strain distribution for Class 13 trucks, however, is very similar to the interior span, with the exception of a shift from the mean, and still exhibits a very flat distribution.

7.3 Strain Patterns Due To Loading

Variations in the vehicle configuration can correspondingly induce a different number of stress ranges and loading cycles as a truck passes over a bridge, and thus ultimately affect the fatigue life of the structure in different ways. The strain patterns created by the vehicle loadings can be used as a tool to determine the number of loading cycles. For example, Figure 7.6 shows a typical Class 9 and Class 13 truck as captured by the WIM system. Their resulting strain patterns developed in the bottom flange gage in the end span of beam #10 as they drive across the bridge are shown in Figure 7.7. Note the obvious bimodal pattern shown in Figure 7.7 exhibited by the Class 9 truck passage. This bimodal pattern is repeatedly exhibited for Class 9 trucks, while Class 13 trucks shown in Figure 7.7 repeatedly exhibit a nearly single peak pattern. This observation demonstrates the possibility of multiple stress range cycles for a single loading.

7.4 Strain Distribution

Bridge design codes such as the LRFD Standard Specifications (AASHTO, 1998) establish certain standards for the design of bridge structures. The design codes generally establish factors that ensure elastic behavior in a structure. In a purely elastic structure, the load induced strain distribution will be a linear line versus depth. A steel beam that

resists loading without the assistance of composite action has a neutral axis located at middepth. Figure 7.8 shows the strain distribution in beam #10. The linearity of the strain measurements versus beam depth show the structure behaves elastically. The figure also shows that some amount of composite action exists since the point of zero strain is not at the middepth of 13.6 inches.

7.5 Strain and Gross Vehicle Weights

Generally, as the GVW of a truck increases, the induced strain would also be expected to increase. Thus, an expected observation would be to have strains due to the higher GVWs of “Michigan Train” trucks higher than that of Class 9 trucks. However, when reviewing the strains induced by truck loadings it is not obvious that higher GVWs induce higher strains. Figures 7.9 through 7.11 provide a scatter plot showing strains induced by trucks of particular GVWs.

Notice only a slight positive trend in Figure 7.9 as the GVW increases. Also shown in Figure 7.9 is a plot of the average strain as the GVW increases and a corresponding +2 and -2 standard deviations curve. Although there is a lot of scatter in the data and a resulting high standard deviation most data is located within 2 standard deviations and in the vicinity of the expected mean value.

Figure 7.10 details the strains induced by the GVW of Class 9 trucks. Results similar to Figure 7.9 are shown with a large percentage of trucks near the 80,000 lbs legal limit and most of their corresponding strains near the mean and within 2 standard deviations. Several lighter trucks are also shown and also provide similar results. Again, only a slight upward trend in strain is observed as the GVW increases.

Figure 7.11 details the strains induced by the GVW of Class 13 trucks. The Class 13 trucks on average tend to induce higher strain levels than Class 9 trucks. Most of the data is also within 2 standard deviations but tends to show less grouping about the mean. Although still a slight trend of increasing strain corresponding to an increasing GVW it is even less visible for the Class 13 truck than the Class 9 truck.

These observations are not totally unexpected as there are a large number of unknown variables in addition to the GVW, such as axle configuration, impact, truck suspension, etc., that affect the measurements.

7.6 Strain Ranges

The strain ranges induced by trucks are of particular importance in determining the fatigue damage created in the structure. The strain range is determined by finding the difference between the maximum strain and the minimum strain induced by a particular vehicle. A summary of the strain ranges at the bottom flange gages is provided in Table 7.4 for all trucks. Since the bridge is a continuous structure an individual truck will induce both negative and positive strains. This observation is seen as the maximum and average strain ranges shown in Table 7.4 are considerably higher than the maximum and average strain values shown in Table 7.1. The standard deviations of the strain ranges in the interior span are very nearly equal. However, the standard deviation in the end span is much higher than the interior spans. This is expected as there are dynamic factors induced by the end joint.

Table 7.5 presents the maximum strain ranges separated by Class 9 and Class 13 trucks. With the exception of the end span, the ratio of Class 13 truck to Class 9 truck strain values is nearly equal to unity, showing there is very little difference in the maximum strain ranges induced by the Class 9 and Class 13 trucks. It is also seen that throughout the four month period from January 1, 2002 through April 30, 2002, the Class 9 truck has produced the largest maximum strain range. The smaller ratio for the maximum end span strain range indicates the influence of impact on the different truck configurations. Table 7.6 presents the average strain ranges separated by Class 9 and Class 13 trucks. The ratio of Class 13 truck average strain range versus Class 9 truck average strain range shows the Class 13 truck induces average strain ranges 30% higher than the Class 9 truck. This result is expected as “Michigan Train” trucks rarely travel unloaded thus rarely induce low strains ranges.

There are two limit states that must be considered when estimating the fatigue life of a structure; the constant amplitude fatigue limit and the variable amplitude fatigue limit. The constant amplitude fatigue limit (CAFL) is a limit state where a constant stress range below the limit state will not create fatigue damage. However, in real structures a constant stress range is rarely achieved. This is especially true for bridge structures. Therefore, the variable amplitude fatigue limit (VAFL) has been estimated to be half the CAFL thus taking into account the existing variable loading inflicted upon bridge structures. It is thought that once a structure has been subjected to the CAFL any induced stress range between the CAFL and VAFL also contributes to reducing the fatigue life of a structure.

The fatigue resistance of a structure can be determined using the LRFD Standard Specification (AASHTO, 1998). The most fatigue critical connection detail on the study bridge are of AASHTO category C. A study on intermittent weld diaphragm details, similar to the fatigue critical detail on the bridge structure, has shown that the intermittent weld diaphragm detail behaves somewhere between AASHTO category C and category D (Barth and Bowman, 2001). Refer to Barth and Bowman (2002) for complete details on the intermittent weld detail behavior and findings. Using this information and the current ADTT shown in Chapter 5 the fatigue resistance for the structure can be determined using the following equation defined in the LRFD Standard Specification (AASHTO, 1998):

$$(\Delta F) = \left(\frac{A}{N}\right)^{\frac{1}{3}} \geq \frac{1}{2} * (\Delta F)_{TH}$$

Where A is a constant based upon the detail category and N is an estimate of the number of cycles induced through the design life of the structure and is estimated using the following:

$$N = (365) * (75) * n * (ADTT)_{SL}$$

Estimating the single lane ADTT to be 85% of the current 2 lane ADTT the strain range fatigue resistance for category C and D is approximately 214µε and 170µε, respectively. Under current traffic conditions the critical strain ranges experienced by the bridge are in the end span. The maximum end span strain range adjusted, using interpolation along the

linear strain distribution, to the fatigue detail is approximately $215\mu\epsilon$. Thus using current loading conditions the structure does not meet code requirements.

The approximate fatigue life of a structure under variable amplitude loading can be determined using Miner's Rule. This linear method is commonly used in civil engineering and determines the fatigue life of a structure by assuming that any stress range induces a damage fraction that is a linear function of the number of cycles that takes place at that stress range.

Thus using Miner's Rule the fatigue life of a structure can be calculated using the following:

$$\sum \frac{n_i}{N_i} = 1$$

Where n_i is the number of cycles that takes place at stress range level i and N_i is the number of cycles that would cause failure at stress range level i (Fisher, et. al. 1998).

The VAFL is estimated to be at $\frac{1}{2}$ the CAFL as determined by the LRFD Standard Specifications (AASHTO, 1998) and can be used to determine a strain range level above which is causing fatigue damage. The VAFL for category C and category D details is thus estimated to be $172.5\mu\epsilon$ and $120.8\mu\epsilon$, respectively. Using strain range data from the most fatigue critical location, the end span diaphragm, the number and level of strain ranges above the VAFL can be determined. The relevant strain range is found by adjusting the measured strain range to the detail location by interpolation to the bottom depth of the detail along the linear elastic strain distribution. Figure 7.12 shows a graphical representation of a variable loading spectrum with the AASHTO fatigue life curves. The figure shows the stress range on the vertical axis and the number of cycles on the horizontal axis for different AASHTO detail categories. The variable loading spectrum is representative of actual loadings and shows that the amplitude of the loadings is not constant and that some loadings can induce stresses above or below the VAFL. This information, with the assistance of Miner's Rule, can be used to determine the fatigue life of the structure. Using the linear Miner's Rule equation with the percentage of ADTT within each strain range bin above the VAFL and the cycles to failure within

each strain range bin, the equivalent days to failure can be estimated. The fatigue life of the structure using current traffic patterns is thus estimated to be nearly infinite. This estimated infinite fatigue life is due to the low volume, less than 1% of the ADTT of 785, of trucks causing strain ranges above the VAFL resulting in very little fatigue damage induced by the current truck traffic. Using Miner's Rule the structure is shown not to be susceptible to fatigue failure at the details studied.

Table 7.1 – Summary of Strains at Beam #10 Bottom Flange Gages for All Trucks

Gage Location	Absolute Max. Strain ($\mu\epsilon$)	Absolute Min. Strain ($\mu\epsilon$)	Ave. Max. Strain ($\mu\epsilon$)
Interior Span Near Midspan	143	-31	51.4
Interior Span South Diaphragm	136	-36	53.6
Interior Span North Diaphragm	130	-36	51.2
End Span South Diaphragm	195	-35	71.0

Table 7.2 – Summary of Strains at Beam #10 Bottom Flange Gages for Class 9 Trucks

Gage Location	Absolute Max. Strain ($\mu\epsilon$)	Absolute Min. Strain ($\mu\epsilon$)	Ave. Max. Strain ($\mu\epsilon$)
Interior Span Near Midspan	143	-30	47.1
Interior Span South Diaphragm	136	-34	49.3
Interior Span North Diaphragm	130	-35	47.1
End Span South Diaphragm	195	-35	66.2

Table 7.3 – Summary of Strains at Beam #10 Bottom Flange Gages for Class 13 Trucks

Gage Location	Absolute Max. Strain ($\mu\epsilon$)	Absolute Min. Strain ($\mu\epsilon$)	Ave. Max. Strain ($\mu\epsilon$)
Interior Span Near Midspan	141	-31	61.7
Interior Span South Diaphragm	131	-36	61.7
Interior Span North Diaphragm	125	-36	64.4
End Span South Diaphragm	185	-31	86.8

Table 7.4 – Summary of Strain Ranges at Beam #10 Bottom Flange Gages for All Trucks

Gage Location	Absolute Max. Strain Range ($\mu\epsilon$)	Avg. Strain Range ($\mu\epsilon$)	Strain Range Standard Deviation ($\mu\epsilon$)
Interior Span Near Midspan	172	64.5	21.9
Interior Span South Diaphragm	170	68.7	22.2
Interior Span North Diaphragm	164	66.8	21.7
End Span South Diaphragm	227	83.0	32.4

Table 7.5 – Maximum Strain Ranges at Beam #10 Bottom Flange Gages

Gage Location	Class 9 Truck Max. Strain Range ($\mu\epsilon$)	Class 13 Truck Max. Strain Range ($\mu\epsilon$)	Max. ϵ_{r13} / Max. ϵ_{r9}
Interior Span Near Midspan	172	170	0.99
Interior Span South Diaphragm	155	154	0.99
Interior Span North Diaphragm	161	159	0.99
End Span South Diaphragm	227	207	0.91

Table 7.6 – Average Strain Range at Beam #10 Bottom Flange Gages

Gage Location	Avg. Class 9 Truck Strain Range ($\mu\epsilon$)	Avg. Class 13 Truck Strain Range ($\mu\epsilon$)	Avg. ϵ_{r13} / Avg. ϵ_{r9}
Interior Span Near Midspan	59.3	77.3	1.30
Interior Span South Diaphragm	61.3	80.8	1.32
Interior Span North Diaphragm	63.1	82.9	1.31
End Span South Diaphragm	77.9	99.8	1.28

Static vs. Dynamic Strain Measurement in End Span - Beam #10

Figure 7.1 – Static and Dynamic Strain Readings on Beam #10

Maximum Strain Histogram for Bottom Gage Interior Span Near Midspan

Figure 7.2 – Maximum Strain Frequency: Bottom Flange Gage Near Midspan of Beam #10 Interior Span

Maximum Strain Histogram for Bottom Gage Interior Span North Diaphragm

Figure 7.3 – Maximum Strain Frequency: Bottom Flange Gage North Diaphragm of Beam #10 Interior Span

Maximum Strain Histogram for Bottom Gage Interior Span South Diaphragm

Figure 7.4 – Maximum Strain Frequency: Bottom Flange Gage South Diaphragm of Beam #10 Interior Span

Maximum Strain Histogram for Bottom Gage End Span South Diaphragm

Figure 7.5 – Maximum Strain Frequency: Bottom Flange Gage South Diaphragm of Beam #10 End Span

Figure 7.6 – Typical Trucks for Comparison of Strain Patterns

Figure 7.7 – Typical Strain Pattern for Class 9 and 13 Trucks

Figure 7.8 – Measured Strain Distribution in Interior Span Near Midspan Gages

Maximum Strain for All Trucks in End Span Bottom Flange Gage vs. GVW

Figure 7.9 – Maximum Strain for All Trucks vs. Gross Vehicle Weight in End Span Bottom Flange Gage

Maximum Strain for Class 9 Trucks in End Span Bottom Flange Gage vs. GVW

Figure 7.10 – Maximum Strain for Class 9 Trucks vs. Gross Vehicle Weight in End Span Bottom Flange Gage

Maximum Strain for Class 13 Trucks in End Span Bottom Flange Gage vs. GVW

Figure 7.11 – Maximum Strain for Class 13 Trucks vs. Gross Vehicle Weight in End Span Bottom Flange Gage

Figure 7.12 – Fatigue Life According to AASHTO Specification

CHAPTER 8 – ANALYSIS AND PREDICTION OF BRIDGE RESPONSE

8.1 Overview

An analytical model was developed to predict the structural behavior of the test bridge. Structural analysis was used as a tool to assist in determining instrumentation locations and to develop strain predictions. The instrumentation locations were positioned near the maximum strain locations. These locations were then used as points of correlation between bridge strain measurements and the analytical predictions of known truck loadings. The development of the analysis model and comparisons with the bridge measurements are presented in this chapter.

8.2 Analysis Development

Several methods of analyzing the structure were developed. These methods range from simple 2-dimensional models to a more detailed 3-dimensional model. The 2-dimensional models rely upon a separate transverse analysis to determine the beam distribution factors, as well as a longitudinal analysis using an influence line method.

The first 2-dimensional analysis developed assumed a rigid unyielding support system and will be referred to hereafter as the rigid model. The second 2-dimensional analysis developed attempted to more closely represent load sharing and used flexible, linear spring supports, hereafter referred to as the spring model. Two analyses, lateral and longitudinal, were necessary for both 2-dimensional models. The lateral analysis was performed in the transverse direction to determine the wheel loads that act on each beam in the structure.

Two methods were considered in determining the wheel loads: the LRFD Standard Specification (AASHTO, 1998) and a structural analysis. Using AASHTO, the beam distribution factor is calculated with the following formula:

$$\text{Distribution Factor} = 0.075 + \left(\frac{S}{2900}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2}$$

Where S = beam spacing and L = span length

This results in a beam distribution factor on all beams equal to 0.64. Due to the lane to beam configuration it is believed that an analysis would provide a beam distribution factor that is better suited for the lane loading. The analysis was preformed using SAP 2000, a linear finite element program.

To perform the lateral analysis a suitable model must be developed. To represent the deck for the lateral analysis, a model concrete member of 9 3/4" depth x 12" width was used. The model member depth is representative of the true 9 3/4" deck depth, and the model member width of 12" is representative of the tire pressure area of a truck passing over the bridge. Differing deck widths of 8", 12", and 16" were also examined in the analysis.

The model member was extended over the entire width, with supports at the beam locations (Figure 8.1). In the rigid analysis these supports were assumed to act as simple pinned restraints. In the spring model, however, the supports were modified to spring supports to more accurately represent the load sharing occurring in the bridge structure. In the spring model the spring stiffness was modeled as the beam stiffness obtained using the analytical beam deflection at midspan under a known load.

The AASHTO HS20-44 axle loading of 32 kips with a wheel spacing of 6' was used for the analysis (Figure 8.1). Note that the girder distribution factor is a percentage of a load, making the actual load irrelevant. The loading was then placed in the appropriate lane and positioned at every 1' within the lane. The reactions from each loading scenario were then recorded for use in determining the girder distribution factors. The girder distribution factor is determined by dividing the maximum reaction by the wheel load.

The third analysis method used a 3-dimensional finite element model, hereafter to as the 3-D model. This model was developed and analyzed using SAP 2000.

8.2.1 Impact Factor

Due to the small static sample size (only one calibration truck) to determine the approximate average impact in each span, the impact factor in the LRFD Standard Specification (AASHTO, 1998) was used. For all interior bridge components AASHTO defines the impact factor as 33% for all limit states, excluding fatigue and fracture where 15% is utilized.

8.2.2 Rigid Model

The results of the rigid model lateral analysis provided reactions at each support. For example Figure 8.2 shows the reactions at each support of the structure when the HS20-44 axle load of 32 kips is applied in the center of the left lane. Notice the large discrepancy from support to support showing that only a few beams carry a majority of the loading. Additionally beam #8 is shown to carry more than one 16 kip wheel load.

These reactions were then used to determine the beam distribution factors. From the analysis of the structure with a truck located within either lane it was determined that beam #8 and beam #10 are the beams most heavily loaded (Figure 3.5 and 3.6). The resulting beam distribution factors are 1.28 and 0.99 for beams #8 and #10, respectively. If the impact factor of 33% is considered the resulting beam distribution factors for beam #8 and beam #10 are 1.70 and 1.32, respectively.

8.2.3 Spring Model

In the spring model, the supports were modified to spring supports to more accurately represent the load sharing occurring in the bridge structure. In the spring model the spring stiffness was modeled as the beam stiffness obtained using the analytical beam deflection at midspan under a known load. The results of the spring model lateral analysis provided reactions at each support. For example, Figure 8.3 shows the reactions at each support of the structure when the HS20-44 axle load of 32 kips is applied in the right lane. Notice the evidence of load sharing as more load is distributed

amongst each support. Additionally all beams carry less than a single wheel load. As before, the value of the reactions for the axle loads in both the left and right lane positions were used to determine the beam distribution factors. The resulting beam distribution factors are 0.72 and 0.67 for beams #8 and beam #10, respectively. With the impact factor of 33% considered the resulting beam distribution factors for beams #8 and #10 are 0.93 and 0.87, respectively.

8.2.4 2-D Longitudinal Analysis

After the beam distribution factors were determined, a longitudinal structural analysis was performed on each beam for both the rigid and spring model. The longitudinal analysis relies on the individual beam influence lines. The influence lines for beams #8 and #10 were developed with the aid of SAP 2000. To model the structure, frame elements were developed using the cross sectional properties of the actual beams in the structure. The structure was modeled in the same configuration as the actual structure with the appropriate spans and beam sizes throughout. A simple pin or roller type of support is placed at every pier support location. The structure is divided into nodes at every support and roughly every 5' to provide an adequate amount of information for development of the influence lines. The span lengths of the model for the last three (easternmost) spans for beams #8 and #10 are shown in Table 8.1. Note that the span length from pier 8 to pier 9 is different for the two models reflecting the influence of the skew on the resulting span lengths.

To develop the influence lines for beams #8 and #10, a unit load was applied to the model, and the frame element forces were determined accordingly at 2.5 feet increments along the length. To develop the “full” influence line, this procedure was repeated by moving the unit load along the structure in 5-foot intervals and recording the corresponding results. Once the influence lines were determined, a Visual Basic program was written in Microsoft Excel to incorporate the beam distribution factors and different loading configurations.

The Visual Basic program was then used to move vehicle loads to a position every 6 inches along the entire length of the structure and analyzed by interpolating between the influence line coefficients developed in SAP 2000. The result is a moment envelope for the structure based upon the beam distribution factor with impact. The envelopes represent the maximum and minimum moment values that will occur at specific locations in the beam as a given loading travels across the bridge.

The envelope results were checked by hand with a simple moment distribution calculation. The calculation checks were performed with the AASHTO HS20-44 wheel loads, factored by the appropriate beam distribution factor. To perform the moment distribution calculations, the stiffness for each beam size along the entire structure and their appropriate moment distribution factors were determined. Once these factors were determined and the moment distribution factors distributed along the structure, the moments for several different loading conditions were calculated and compared to the analysis. These same loading scenarios were then repeated using the analysis program. The results were compared and found to be in agreement.

The resulting moment envelopes for the two critical beams, #8 and #10, are illustrated, respectively, in Figures 8.4 and 8.5 for the rigid model, and Figures 8.6 and 8.7 for the spring model.

Figure 8.4 illustrates the positive and negative moment envelopes induced by the AASHTO HS20-44 truck and H15-44, the Michigan 5 truck (Figure 3.2), and the Michigan 8 truck (Figure 3.3) on the three easternmost spans of beam #8. Note that the supports are located at the minimum negative moment and the results are calculated without impact. With the exception of the light H15-44 truck the resulting moments for all truck types are very close to each other. In the two easternmost spans the Michigan 8 induces slightly larger moments than the other trucks. However, as the span length decreases the effect of the truck axle configuration is evident with the Michigan 5 and HS20-44 inducing the largest maximum moments. The rigid analysis also indicates that the heavy trucks will all induce moments of very high magnitude. The short span in beam #8 results in the smallest moments due to the relatively higher stiffness of the steel section used in that location. Refer to Figure 3.6 for bridge framing information. The

endspan results in the highest moments because of the small steel section used in that location.

The same information, as provided in Figure 8.4, for beam #10 is provided in Figure 8.5. The results are nearly identical with the Michigan 8 truck inducing slightly larger moments than the other trucks. However, due to the lower load distribution on beam #10 the moment magnitudes are lower. Although, the same relatively stiffer steel section used in the short span of beam #8 is used in the corresponding span of beam #10, the moments are higher because of the increased span length.

Figures 8.6 and 8.7 illustrate the moment envelopes induced by the different trucks when analyzed using the spring analysis distribution factors. The results are identical to the results described in Figures 8.4 and 8.5 except for the much smaller moment magnitude.

It was found that the analysis model resulted in considerably higher maximum strains than measured on the structure. Due to friction and the addition of shear studs at the end of the beams some of the discrepancy was thought to have been caused by composite action. Figure 8.9 shows the measured and predicted strain distribution for the structure acting compositely and non-compositely under the WIM calibration truck loading shown in Figure 8.8. Notice the decrease in predicted strain from the rigid model to the spring model. The measured strain distribution clearly shows the bridge behaving compositely. Each successive model more closely predicted the measured bridge strains. However, even after determining the analytical fully composite strains a large discrepancy was still evident. Another explanation of the discrepancy was thought to have been created through much better load sharing in the bridge than could be represented by using the beam distribution factors in the linear model. Consequently, a 3-dimensional model was thus developed to examine the load distribution behavior.

8.2.5 3-D Model

The 3-D model was developed using the SAP 2000 structural analysis software. Due to the large size of the structure and the resulting computational needs, only the last

three (easternmost) spans were modeled. It is believed that this will still provide for an accurate representation of the structural behavior for the instrumented spans.

The model was developed using a combination of frame and shell elements. The shell elements are used to model the bridge deck and are of 9 ¾" in thickness to represent the actual deck thickness. To make the shell elements as uniform as possible an attempt was made to make all shell elements 12" x 12", resulting in an aspect ratio of 1. In the locations where it was not possible to form these uniform shell elements, rectangular and triangular elements were formed all with aspect ratios of less than 2. These non-uniform elements are used near the skew, at loading locations, and near the beam locations.

Frame elements were used to model the steel beam members. Similar to the previous longitudinal model, all elements are formed using the true beam sections. The bridge deck and the beams are attached together at their associated nodes. A limitation in the model attaches the shell elements and the frame elements at same elevation, their middepth. This limitation could result in an underestimation of the frame element internal forces as the stiffness of the structure is not exactly the same as that of the real structure with the deck located on top of the beam. A perspective and plan view of the 3-D model is shown in Figures 8.10 and 8.11, respectively.

To load the structure, a truck is assumed to be located in the center of one of the two lanes. The contact area of each tire is approximately 8"x 12". A pressure load is placed based on the axle spacing and loading as gathered by the WIM measurements. The appropriate pressure load is the half axle load (represents a tire load) divided by the tire contact area of 96 in² (8"*12"). Figure 8.12 provides the typical deformed shape of the three-span portion of the structure under a left lane truck loading.

After running the analysis the resulting moments can be found. In SAP 2000 the appropriate elements can be chosen and the moment read from the results at any location on the structure. The 3-D model can then be used to predict moments and strains at any location in the structure due to any type of truck loading.

8.3 Analysis Comparison

Tables 8.2 and 8.3 provide a summary and comparison of the different analytical models to actual bridge data for Beams #8 and #10, respectively. Six different truck samples were collected from the WIM system and entered into the analysis models. The analysis was run and the maximum moments at the cross sections where strain gages were attached to the beams were determined. For this comparison the analysis was run without an impact factor. From these moments the non-composite and fully composite strains at the gage locations were calculated. The fully composite strains were calculated due to the repair previously mentioned in section 3.4 and the addition of shear studs.

As can be seen in Tables 8.2 and 8.3, each model is successively better at predicting the actual strains occurring in the bridge with a given truck. This comparison is graphically illustrated in Figure 8.13 for the WIM calibration truck loading shown in Figure 8.8, with the x-axis showing strain values and the y-axis showing the two instrumented spans. As shown in Figure 8.13 the rigid model provides much less load sharing between beams and, thus, it predicts much higher strains in the instrumented beams. The spring model, although much better at estimating the beam load sharing, still predicts considerably higher strains than those measured in the instrumented beams. The 3-D model provides a very close prediction to the actual measured strains.

It is not possible to develop a model that will precisely predict the actual bridge strains due to the varying dynamics of the trucks and possible errors in the WIM measurements. Other factors leading to possible discrepancies are the varying degrees of composite behavior throughout the bridge, the true position of the truck within the lane, the truck's impact, and WIM measurement error. However, it can be seen that the 3-D model predicts the measured strains with fairly good accuracy.

Table 8.1 – Span Length for Longitudinal Model of Beams #10 and #8
Last (Easternmost) Three Spans.

	Span Location	Span Length
Beam #10	Pier 8-9	46.833'
	Pier 9-10	43.0'
	Pier 10-11	42.25'
Beam #8	Pier 8-9	32.7'
	Pier 9-10	43.0'
	Pier 10-11	42.25'

Table 8.3 – Actual vs. Analytical Comparison on Beam #10

Truck #1 WIM Calibration Truck										
Beam #10	GVW = 79.7		Actual Strains (µε)		Predicted Strains - 2D Rigid		Predicted Strains - 2D Spring		Predicted Strains - 3D	
	Axle Load (k)	19.65	19.65	14.75	14.75	14.75	14.75	14.75	14.75	10.9
	Wheel Load (k)	9.83	9.83	7.38	7.38	7.38	7.38	7.38	7.38	5.45
	Axle Spa. (ft)	4.2	35.0	4.4	17.5	17.5	17.5	17.5	17.5	0
Beam #10	GVW = 98.4		Actual Strains (µε)		Predicted Strains - 2D Rigid		Predicted Strains - 2D Spring		Predicted Strains - 3D	
	Interior Span Moment	48	148	215.3	146.0	146.0	145.5	98.7	71.3	48.4
	End Span Moment	72	192	355.1	233.7	233.7	240.4	158.2	103.6	68.2
	GVW = 75.1		Actual Strains (µε)		Predicted Strains - 2D Rigid		Predicted Strains - 2D Spring		Predicted Strains - 3D	
Beam #10	Axle Load (k)	13.7	13.6	16	17	17	16.5	109.5	86.9	57.2
	Wheel Load (k)	6.85	6.8	8	8.5	8.5	109.1	74.0	61.1	41.4
	Axle Spa. (ft)	10.9	31.5	4.5	21.9	21.9	166.5	109.5	47	57.2
	GVW = 98.4		Actual Strains (µε)		Predicted Strains - 2D Rigid		Predicted Strains - 2D Spring		Predicted Strains - 3D	
Beam #10	Interior Span Moment	48	148	215.3	146.0	146.0	145.5	98.7	71.3	48.4
	End Span Moment	72	192	355.1	233.7	233.7	240.4	158.2	103.6	68.2
	GVW = 75.1		Actual Strains (µε)		Predicted Strains - 2D Rigid		Predicted Strains - 2D Spring		Predicted Strains - 3D	
	Axle Load (k)	13.7	13.6	16	17	17	16.5	109.5	86.9	57.2
	Wheel Load (k)	6.85	6.8	8	8.5	8.5	109.1	74.0	61.1	41.4
	Axle Spa. (ft)	10.9	31.5	4.5	21.9	21.9	166.5	109.5	47	57.2

Table 8.3 (Cont.) – Actual vs. Analytical Comparison on Beam #10

Truck #4													
Vehicle Captured by WIM													
Beam #10	Axle Load (k)	13.1	12.9	22.8	22.6	11.1							
	Wheel Load (k)	6.55	6.45	11.4	11.3	5.55							
	Axle Spa. (ft)	4.3	34.9	4.4	17.5	0							
Truck #5	GVW = 82.5	Actual Strains (μ _ε)	Predicted Strains - 2D Rigid			Predicted Strains - 2D Spring			Predicted Strains - 3D				
			Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)		
		Interior Span Moment	50	168	244.4	165.8	114	129	165.8	112.5	62	90.2	61.2
		End Span Moment	65	190	351.4	231.2	129	129	238.6	157.0	64	118.4	77.9
Truck #5	GVW = 82.5	Actual Strains (μ _ε)	Predicted Strains - 2D Rigid			Predicted Strains - 2D Spring			Predicted Strains - 3D				
			Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)		
		Interior Span Moment	59	141	205.1	139.1	95	95	138.2	93.7	49	71.3	48.4
		End Span Moment	88	193	357.0	234.9	130	130	240.4	158.2	57	105.4	69.4
Truck #6	GVW = 122.4	Actual Strains (μ _ε)	Predicted Strains - 2D Rigid			Predicted Strains - 2D Spring			Predicted Strains - 3D				
			Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)		
		Interior Span Moment	59	141	205.1	139.1	95	95	138.2	93.7	49	71.3	48.4
		End Span Moment	88	193	357.0	234.9	130	130	240.4	158.2	57	105.4	69.4
Truck #6	GVW = 122.4	Actual Strains (μ _ε)	Predicted Strains - 2D Rigid			Predicted Strains - 2D Spring			Predicted Strains - 3D				
			Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)		
		Interior Span Moment	59	141	205.1	139.1	95	95	138.2	93.7	49	71.3	48.4
		End Span Moment	88	193	357.0	234.9	130	130	240.4	158.2	57	105.4	69.4
Truck #6	GVW = 109.8	Actual Strains (μ _ε)	Predicted Strains - 2D Rigid			Predicted Strains - 2D Spring			Predicted Strains - 3D				
			Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)	Moment (ft-k)	Non-Composite (μ _ε)	Composite (μ _ε)		
		Interior Span Moment	46	102	148.4	100.7	69	69	100.4	68.1	35	50.9	34.5
		End Span Moment	78	134	247.8	163.1	91	91	168.3	110.7	39	72.1	47.5

Figure 8.1 – Transverse Model Showing Left Lane Loading of HS20-44.

Figure 8.2 – Showing Left Lane Reactions From Transverse Rigid Analysis

Figure 8.3 – Showing Right Lane Reactions From Transverse Spring Analysis

Figure 8.4 – Rigid Model, Beam #8 Moment Envelopes, Last 3 Spans

Figure 8.5 – Rigid Model, Beam #10 Moment Envelopes, Last 3 Spans

Figure 8.6 – Spring Model, Beam #8 Moment Envelopes, Last 3 Spans

Figure 8.7 – Spring Model, Beam #10 Moment Envelopes, Last 3 Spans

Figure 8.8 – WIM Calibration Truck

Measured and Predicted Strain Distribution for Composite and Non-Composite

Figure 8.9 – Measured, Composite and Non-Composite Strains Distribution for Beam #10

Figure 8.10 – Perspective View of 3-D Model

Figure 8.11 – Plan View of 3-D Model

Figure 8.12 – Perspective View 3-D Model Deformed Shape

Figure 8.13 – Beam #10 Near Midspan Maximum Measured Strain vs. Maximum Analytically Predicted Strains

CHAPTER 9 – SUMMARY AND CONCLUSIONS

9.1 Overview

This study has evaluated the influence of heavy-weight truck traffic on the steel bridges along the extra heavy duty highway corridor in Northern Indiana by monitoring one typical bridge structure. This corridor enables the steel industry in Northwest Indiana to transport multiple, heavy steel coils into the state of Michigan. Data collected during the study was measured using a combination of a Weigh-In-Motion (WIM) system and strain gage instrumentation. Different analysis methods were examined and evaluated by comparing the analysis predictions to the measurement data.

9.2 Conclusions and Observations

The information gathered during the experimental test conducted in the field on a bridge structure located on the extra heavy duty highway provided a number of observations. From these observations several conclusions were drawn and are noted in the following:

- (1) Of main concern when evaluating the bridge fatigue damage due to loading are the frequency and loading of trucks. Truck information was gathered using a Weigh-In-Motion system installed for this study. On most highways the most common trucks are of Class 9, however, due to the designation as an extra heavy duty highway, trucks of Class 13 are also common. Trucks of Class 13 have 7 or more axles and are generally used for loads greater than the typical 80,000 lbs and referred to as “Michigan Trains.” The extra heavy duty corridor has an ADTT of 1,335 in all directions with 785 and 550 trucks traveling in the

eastbound and westbound direction, respectively. Of this truck traffic most are of Class 9 and Class 13: roughly 44% are Class 9 and 14% are Class 13.

- (2) The legal weight limit on the extra heavy duty highway is 80,000 lbs for truck type 9 and 134,000 lbs for truck types 10 and 13. It is generally recognized that some trucks travel overweight while most travel within their legal limits. During the study it was observed from the WIM data that 15% of truck type 9 and 26% of truck type 13 travel over their respective typical legal limits; although not studied specifically, some of the heavier loads undoubtedly obtained special permits to carry heavier loads. Extreme weights of more than 150,000 lbs and 200,000 lbs from truck types 9 and 13, respectively, were observed during the 4 months that the WIM data was monitored. The average gross vehicle weight (GVW) on the extra heavy duty highway is 52,368 lbs of all trucks in all directions with 56,959 lbs and 47,776 lbs in the eastbound and westbound directions, respectively. The average GVW of truck type 9 is 54,356 lbs. The average GVW of truck type 13 is 119,459 lbs.
- (3) It was observed that about 66% of the Class 9 trucks travel between 6 am – 6 pm while about 64% of the Class 13 trucks travel between 12 pm – 12 am. Overweight trucks generally exhibit a time preference based on the operating hours of static weigh stations. However, since no weigh station is located along U.S. 20, overweight trucks were observed to travel throughout the day and show no time preference.
- (4) Strain gages were installed on a bridge structure located on the extra heavy duty highway to monitor the strains induced by truck loading. The strain readings provide valuable information in evaluating the bridge response and resulting fatigue damage due to truck loadings. Even with the much heavier GVW of “Michigan Train” trucks the maximum strains and strain ranges induced by the different truck types are nearly equal. Due to the axle configurations of “Michigan Train” trucks and the short spans of the particular structure, there is only a slight upward trend observed in the strain values as the GVW increases.

- (5) The average maximum strains and the average strain range values induced by truck Class 13 trucks are about 20 $\mu\epsilon$ higher than those induced by Class 9 trucks. The absolute maximum strain found during the 4 month observation period induced by truck traffic was 195 $\mu\epsilon$ while the absolute maximum strain range was 227 $\mu\epsilon$.
- (6) The strain data created by the vehicle loadings can be used to determine the number of loading cycles per truck. The typical strain patterns of Class 9 and 13 trucks are quite different. As trucks of Class 9 travel across the bridge they exhibit an obvious bimodal pattern. This bimodal pattern is repeatedly exhibited for Class 9 trucks while Class 13 trucks repeatedly exhibit a nearly single peak pattern.
- (7) Using Miner's Rule it was found that the bridge structure was not susceptible to fatigue failure. This is mostly due to the small volume of trucks, less than 1%, that induce strain above the variable amplitude fatigue limit thus causing very little fatigue damage.
- (8) Both two-dimensional and three-dimensional analytical bridge models were evaluated. The two-dimensional models were developed using rigid supports and spring supports. The two dimensional models were unable to closely predict the load sharing of the bridge structure and greatly exaggerated the predicted strains when compared to the measured strains. It was found that a three-dimensional finite element model was necessary to accurately predict the bridge response.

9.3 Implementation Recommendations

Based upon the measured truck gross vehicle and axle weights, experimental strain measurements, and analytical modeling reported herein for one bridge structure on the extra heavy-weight corridor, it does not appear that fatigue is a serious problem. However, a second stage of the study will develop a more thorough analytical model than was used in this phase of the study that will be applied to other steel bridges along the extra heavy-weight corridor. The fatigue critical structural details should still be inspected and monitored through routine biennial inspections.

LIST OF REFERENCES

- AASHTO LRFD Bridge Design Specifications* (1998), Second Edition, American Association of State Highway and Transportation Officials, Washington D.C., 1998.
- ASTM Standard E1318-94 (1994), *Standard Specification for Highway Weigh-in-Motion Systems with User Requirements and Test Methods*, 1994.
- Barth, A.S. and Bowman, M.D. (2001), *Fatigue Behavior of Welded Diaphragm-to-Beam Connections*, ASCE, 127(10), 1145-1152.
- Barth, A.S. and Bowman, M.D. (2002), *Fatigue Behavior of Beam Diaphragm Connections with Intermittent Fillet Welds: Part I, Volume 2, Laboratory Fatigue Evaluation*, Final Report FHWA/IN/JTRP-2001/10-I-2, Indiana Department of Transportation, pp. 291.
- Farrand, R.E. (1991), *MICHIGAN TRAIN TRUCK ROUTE STUDY – I/N Tek – I/N Kote Plant Access Road East on SR 2 to US 31 and North to Michigan State Line*, Cole Associates, Inc., prepared for St. Joseph County, Indiana, February 1991.
- Farrand, R.E. (1992), *MICHIGAN TRAIN TRUCK ROUTE STUDY – Steel Warehouse Company, Inc. Access Road South on Olive Street to SR23 West on SR23 to US31 and North to SR2 Interchange*, Cole Associates, Inc., prepared for Steel Warehouse, Inc., Indiana, September 1992.
- McCall, Bill and Vodrazka Jr, Walter C. (1997), *States Successful Practices Weigh-in-Motion Handbook*, U.S. Department of Transportation, Federal Highway Administration, Center for Transportation Research and Education, Iowa State University, December 1997.
- Nowak, A. S. and Eom, J. (2001), *Verification of Girder Distribution Factors for Steel Girder Bridges*, Research Report UMCE 00-10, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, report submitted to Michigan Department of Transportation, April 2001.

- Nowak, A. S., Laman, J.A., and Nassif, H. (1994), *Effect of Truck Loading on Bridges*, Research Report UMCE 94-22, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, report submitted to Michigan Department of Transportation, December 1994.
- Oversize/Overweight Vehicle Permitting Handbook* (1998), Indiana Department of Revenue, Motor Carrier Services Division-Permit Section, Publication 101, Indianapolis, Indiana, January 1998.
- Poe, Jim (2000). *Final Report of the Northwest Indiana Transportation Study Committee*, Indiana Legislative Services Agency, Indianapolis, IN, 2000.
- Pratt, Andrew J. and Bushman, Rob (1998), *Weigh-in-Motion Technology – Economics and Performance*, Presented at NATMEC '98, Charlotte, N.C., 1998.
- SAP2000 (2001). Computers and Structures, Inc., Structural Analysis Program, Berkeley, CA.
- Schermerhorn, Phil (1998). *Legislative Evaluation and Oversight Policy Subcommittee, Meeting Minutes*, Indiana Legislative Services Agency, Indianapolis, IN, 1998.
- Wang, Ton-Lo (2000), *Influence of Heavy Trucks on Highway Bridges*, Summary of Final Report, BC-379, Florida Department of Transportation, October 2000.
- Williams and Associates (1986), *Cost Allocation for Heavy Trucks – A Pavement and Bridge Evaluation*, Clyde E. Williams and Associates, Inc., Indianapolis, prepared for Indiana Department of Highways, August 1986.

Appendix A – INDOT Extra Heavy Duty Highway Legislation

A.1 Introduction

This section contains the 2002 version of the Indiana Department of Transportation Extra Heavy Duty Highway Legislation. The legislation provides the written law pertaining to the extra heavy duty truck route, permitting, weight controls, and allowable truck types.

A.2 Legislation

IC 9-20-5

Chapter 5. Heavy Duty Highways and Extra Heavy Duty Highways

IC 9-20-5-1

Establishment and designation of heavy duty highways; removal of designation; publication of map

Sec. 1. (a) The Indiana department of transportation may adopt rules under IC 4-22-2 to do the following:

- (1) Establish and designate a highway as a heavy duty highway.
- (2) Remove the designation of a highway or part of a highway as a heavy duty highway.

(b) The Indiana department of transportation shall periodically publish a map showing all highways designated by the department at the time as heavy duty highways.

As added by P.L.2-1991, SEC.8.

IC 9-20-5-2

Maximum weight limitations; heavy duty highways

Sec. 2. Whenever the Indiana department of transportation designates a heavy duty highway, the department shall also fix the maximum weights of vehicles that may be transported on the highway. The maximum weights may not exceed the following limitations:

- (1) A vehicle may not have a maximum wheel weight, unladen or with load, in excess of eight hundred (800) pounds per inch width of tire, measured between the flanges of the rim, or an axle weight in excess of twenty-two thousand four hundred (22,400) pounds.
- (2) The total weight concentrated on the roadway surface from any tandem axle group may not exceed eighteen thousand (18,000) pounds for each axle of the assembly.
- (3) The total gross weight, with load, in pounds of a vehicle or combination of

vehicles may not exceed eighty thousand (80,000) pounds.

As added by P.L.2-1991, SEC.8.

IC 9-20-5-3

Designation of heavy duty highways; conditions

Sec. 3. The Indiana department of transportation may not designate an Indiana highway as a heavy duty highway unless the department finds that the highway is:

- (1) so constructed and can be so maintained; or
- (2) in such condition;

that the use of the highway as a heavy duty highway will not materially decrease or contribute materially to the decrease of the ordinary useful life of the highway.

As added by P.L.2-1991, SEC.8.

IC 9-20-5-4

Extra heavy duty highways; listing

Sec. 4. In addition to the highways established and designated as

heavy duty highways under section 1 of this chapter, the following highways are designated as extra heavy duty highways:

- (1) Highway 41, from 129th Street in Hammond to Highway 312.
- (2) Highway 312, from Highway 41 to State Road 912.
- (3) Highway 912, from Michigan Avenue in East Chicago to the U.S. 20 interchange.
- (4) Highway 20, from Clark Road in Gary to Highway 39.
- (5) Highway 12, from one-fourth (1/4) mile west of the Midwest Steel entrance to Highway 249.
- (6) Highway 249, from Highway 12 to Highway 20.
- (7) Highway 12, from one and one-half (1 1/2) miles east of the Bethlehem Steel entrance to Highway 149.
- (8) Highway 149, from Highway 12 to a point thirty-six hundredths (.36) of a mile south of Highway 20.
- (9) Highway 39, from Highway 20 to the Michigan state line.
- (10) Highway 20, from Highway 39 to Highway 2.
- (11) Highway 2, from Highway 20 to Highway 31.
- (12) Highway 31, from the Michigan state line to Highway 23.
- (13) Highway 23, from Highway 31 to Olive Street in South Bend.
- (14) Highway 35, from South Motts Parkway thirty-four hundredths (.34) of a mile southeast to the point where Highway 35 intersects with the overpass for Highway 20/Highway 212.
- (15) State Road 249 from U.S. 12 to the point where State Road 249 intersects with Nelson Drive at the Port of Indiana.
- (16) State Road 912 from the 15th Avenue and 169th Street interchange one and six hundredths (1.06) miles north to the U.S. 20 interchange.

(17) U.S. 20 from the State Road 912 interchange three and seventeen hundredths (3.17) miles east to U.S. 12.
As added by P.L.2-1991, SEC.8. Amended by P.L.12-1991, SEC.4; P.L.123-1993, SEC.1; P.L.124-1993, SEC.1; P.L.119-1995, SEC.2; P.L.45-1999, SEC.1; P.L.79-2000, SEC.3; P.L.147-2002, SEC.2.

IC 9-20-5-4.5

Repealed

(Repealed by P.L.123-1993, SEC.2.)

IC 9-20-5-5

Maximum size and weight limitations; extra heavy duty highways

Sec. 5. The maximum size and weight limits for vehicles operated with a special weight permit on an extra heavy duty highway are as follows:

(1) A vehicle may not have a maximum wheel weight, unladen or with load, in excess of eight hundred (800) pounds per inch width of tire, measured between the flanges of the rim.

(2) A single axle weight may not exceed eighteen thousand (18,000) pounds.

(3) An axle in an axle combination may not exceed thirteen thousand (13,000) pounds per axle, with the exception of one (1)

tandem group that may weigh sixteen thousand (16,000) pounds per axle or a total of thirty-two thousand (32,000) pounds.

(4) The total gross weight, with load, of any vehicle or combination of vehicles may not exceed one hundred thirty-four thousand (134,000) pounds.

(5) Axle spacings may not be less than three (3) feet, six (6) inches, between each axle in an axle combination.

(6) Axle spacings may not be less than eight (8) feet between each axle or axle combination.

As added by P.L.2-1991, SEC.8.

IC 9-20-5-6

Safety procedures; implementation

Sec. 6. The Indiana department of transportation shall implement procedures that, in cooperation with the state police department and local police departments, enhance the safety of citizens along and near extra heavy duty highways listed in section 4 of this chapter.

As added by P.L.2-1991, SEC.8.

IC 9-20-5-7

Special weight permits; extra heavy duty highways

Sec. 7. The owner or operator of a vehicle or combination of vehicles having a total gross weight in excess of eighty thousand (80,000) pounds but less than one hundred thirty-four thousand (134,000) pounds must:

(1) obtain a special weight registration permit;

(2) register annually and pay annually a registration fee to the department of state revenue; and

(3) install an approved automated vehicle identifier in each vehicle operating with a special weight permit;

to travel on an extra heavy duty highway.

As added by P.L.2-1991, SEC.8. Amended by P.L.122-1993, SEC.2; P.L.129-2001, SEC.30.

IC 9-20-5-8

Conditions under which permits not to be issued

Sec. 8. The Indiana department of transportation may not issue a permit under this chapter for the operation of a vehicle if any of the following conditions apply:

(1) The owner or operator of the vehicle has not complied with IC 8-2.1-24.

(2) The owner or operator of the vehicle has not provided the Indiana department of transportation with the owner's or operator's Social Security number or federal identification number.

(3) The owner or operator of the vehicle has not registered the vehicle with the bureau, if the vehicle is required to be registered under IC 9-18.

As added by P.L.122-1993, SEC.3. Amended by P.L.110-1995, SEC.30.

Appendix B – Typical Trucks

B.1 Introduction

This section contains photographs taken in the field of different trucks observed on the extra heavy duty highway. Included are pictures of the typical Class 9 truck and Class 13, “Michigan Train” truck. The pictures were taken at various times and show a wide variety of truck configurations.

B.2 Pictures

Figure B.1 – Typical Class 9 Truck

Figure B.2 – 10 Axle Michigan Train Truck Traveling Westbound Along the Structure

Figure B.3 – 9 Axle Michigan Train Truck Traveling Eastbound Along the Structure

Figure B.4 – 11 Axle Michigan Train Truck Traveling Eastbound Along the Structure

Figure B.5 – Michigan Train Truck Traveling Westbound Along the Structure

Appendix C – Instrumentation Details

C.1 Introduction

This section provides detailed dimensions for the cross sectional gage locations as it varies from interior span to end span. The gage locations vary slightly due to the different beam sizes in each span. The interior span is composed of a W27X102 steel section, while the end span is composed of a W27X84 steel section. The depth of the diaphragm however does not change with respect to the top of the beam flanges thus the depth of the gages from the top of beam do not change.

C.2 Cross Sectional Gage Locations

Figure C.1 – End Span Gage Locations

Figure C.2 – Interior Span Gage Locations

Appendix D – FHWA Vehicle Types

D.1 Introduction

This section provides the Federal Highway Administration's vehicle types. These vehicle types describe the vehicle classification used in the Weigh-In-Motion system. The most common truck type is of Class 9. The "Michigan Train" truck of concern in this study is of Class 13.

D.2 FHWA Vehicle Types

The classification scheme is separated into categories depending on whether the vehicle carries passengers or commodities. Non-passenger vehicles are further subdivided by number of axles and number of units, including both power and trailer units. Note that the addition of a light trailer to a vehicle does not change the classification of the vehicle.

FHWA VEHICLE CLASSES WITH DEFINITIONS

Type Name and Description

1. ***Motorcycles*** (Optional) -- All two or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handlebars rather than steering wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles, and three-wheel motorcycles. This vehicle type may be reported at the option of the State.
2. ***Passenger Cars*** -- All sedans, coupes, and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.

3. ***Other Two-Axle, Four-Tire Single Unit Vehicles*** -- All two-axle, four-tire, vehicles, other than passenger cars. Included in this classification are pickups, panels, vans, and other vehicles such as campers, motor homes, ambulances, hearses, carryalls, and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification. *Because automatic vehicle classifiers have difficulty distinguishing class 3 from class 2, these two classes may be combined into class 2.*

4. ***Buses*** -- All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger- carrying vehicles. Modified buses should be considered to be a truck and should be appropriately classified.

NOTE: In reporting information on trucks the following criteria should be used:

- a. Truck tractor units traveling without a trailer will be considered single-unit trucks.
- b. A truck tractor unit pulling other such units in a "saddle mount" configuration will be considered one single-unit truck and will be defined only by the axles on the pulling unit.
- c. Vehicles are defined by the number of axles in contact with the road. Therefore, "floating" axles are counted only when in the down position.
- d. The term "trailer" includes both semi- and full trailers.

5. ***Two-Axle, Six-Tire, Single-Unit Trucks*** -- All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with two axles and dual rear wheels.

6. ***Three-Axle Single-Unit Trucks*** -- All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with three axles.

7. ***Four or More Axle Single-Unit Trucks*** -- All trucks on a single frame with four or more axles.

8. ***Four or Fewer Axle Single-Trailer Trucks*** -- All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.

9. ***Five-Axle Single-Trailer Trucks*** -- All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.

10. ***Six or More Axle Single-Trailer Trucks*** -- All vehicles with six or more axles consisting of two units, one of which is a tractor or straight truck power unit.

11. ***Five or fewer Axle Multi-Trailer Trucks*** -- All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.

12. ***Six-Axle Multi-Trailer Trucks*** -- All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.

13. ***Seven or More Axle Multi-Trailer Trucks*** -- All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.